# **Second Generation of Eurocode 8**

# Webinar 1-2.2: EN 1998-1-2 Reinforced concrete buildings

**Humberto Varum** 

Credits: José Melo

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# Eurocode 8 – Work of PT2 under the Mandate M/515 (phase 2)

Sub-task Ref.: 3

Sub-task name: Concrete buildings: Ductility Classes and flat slab systems Brief description, background and reasons for the work:

#### **Objectives:**

Updating of section with specific rules for **Concrete Buildings** 

- revising the design rules for the three Ductility Classes, in view of simplifying the design process
- enhance "Ease of use"
- new rules for buildings with flat slab systems (for analysis, design and detailing)

#### A part from:

Comments and suggestions from the **systematic review** by **CEN members**, **WGs**, **TGs**, **Technical Reviewer** and

Including state-of-the-art concepts and rules commonly accepted



### Eurocode 8 – Reorganization of the concrete buildings section

- EN 1998-1:2004 (Chapter 5 Specific rules for concrete buildings) 58 pages design and detailing requirements are organized in sections per ductility classes (DCL, DCM, DCH)
- prEN 1998-1-2:2022 (Chapter 10 Specific rules for concrete buildings) 42 pages (including a new section flat slabs,...) - design and detailing requirements are organized in sections per type of element (beams, columns, beamcolumn joints, ductile walls, large walls, flat slabs)





# **Ductility classes**

- prEN 1998-1-1:2022 defines 3 ductility classes (DC1, DC2 and DC3)
- The new ductility classes are:
  - DC1: the overstrength capacity is taken into account, while the inelastic deformation capacity and energy dissipation capacity are disregarded.
  - DC2: the local overstrength capacity, the local deformation capacity and the local energy dissipation capacity are taken into account. Global plastic mechanisms are controlled.
  - DC3: the ability of the structure to form a global plastic mechanism at SD limit state and its local overstrength capacity, local deformation capacity and local energy dissipation capacity are taken into account.
- For **RC structures**:



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# Eurocode 8 - Design of structures for earthquake resistance Part 1-2: Buildings (prEN 1998-1-2:2022)

- 0. Introduction
- 1. Scope
- 2. Normative references
- 3. Terms, definitions and symbols
- 4. Basis of design
- 5. Modelling and structural analysis
- 6. Verifications of structural members to limit states
- 7. Ancillary elements
- 8. Base isolated buildings
- 9. Buildings with energy dissipation systems

# 10. Concrete buildings

- 11. Steel buildings
- 12. Composite steel-concrete buildings
- 13. Timber buildings
- 14. Masonry buildings
- 15. Aluminium buildings



# Eurocode 8 - Design of structures for earthquake resistance Part 1-2: Buildings (prEN 1998-1-2:2022)

<ul> <li>0. Introduction</li> <li>1. Scope</li> <li>2. Normative references</li> <li>3. Terms, definitions and symbols</li> <li>4. Basis of design</li> <li>5. Modelling and structural analysis</li> <li>6. Verifications of structural members to limit</li> <li>7. Ancillary elements</li> <li>8. Base isolated buildings</li> <li>9. Buildings with energy dissipation systems</li> <li>10. Concrete buildings</li> <li>11. Steel buildings</li> <li>12. Composite steel-concrete buildings</li> <li>13. Timber buildings</li> <li>14. Masonry buildings</li> <li>15. Aluminium buildings</li> </ul>	<ol> <li>Specific rules for concrete buildings</li> <li>General</li> <li>Basis of design and design criteria</li> <li>Materials requirements</li> <li>Structural types, behaviour factors, limits of seismic action, limits of drift and partial factors for the displacement-based approach</li> <li>Beams</li> <li>Columns</li> <li>Columns</li> <li>Beam-column joints</li> <li>Ductile walls</li> <li>Large walls</li> <li>In Provisions for anchorages and laps</li> <li>Prestressed concrete</li> <li>Prestressed concrete structures</li> </ol>
15. Aluminium buildings	10.14 Precast concrete structures 10.15 Design and detailing of foundations



# Eurocode 8 – Main changes introduced in the concrete buildings' section

**LEGEND** (Symbol):

- Main concept/rule/requirement maintained, eventually with changes for simplification, clarification, designation/nomenclature, harmonization, ease-of-use, ...
- **Changes** introduced
- **NEW** New sub-section, Clause, Formula, Table, Figure,...

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- Structural types, behaviour factors, limits of seismic action, limits of drift, materials
- Local ductility condition
- Beams
- Columns
- Beam-column joints
- Ductile walls
- Large walls
- Flat slabs
- Anchorages and laps
- Diaphragms
- Precast concrete structures

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#### Structural types (prEN 1998-1-2:2022 10.4.1)

- **Concrete buildings** designed to be dissipative should be **classified** into one structural type according to their behaviour under horizontal seismic actions:
  - Moment resisting frame structure
  - Dual structure (moment resisting frame-equivalent or wall-equivalent)
    - Moment resisting frame-equivalent dual structures
    - Wall-equivalent dual structures
  - Wall structures (coupled or uncoupled)

 $V_{\text{Rd,MRF}}/V_{\text{Rd,total}} \ge 65\%$   $V_{\text{Rd,MRF}}/V_{\text{Rd,total}} = 35\% \text{ to } 65\%$   $V_{\text{Rd,MRF}}/V_{\text{Rd,total}} = 50\% \text{ to } 65\%$   $V_{\text{Rd,MRF}}/V_{\text{Rd,total}} = 35\% \text{ to } 50\%$   $V_{\text{Rd,MRF}}/V_{\text{Rd,total}} \ge 65\%$ 

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### Structural types (prEN 1998-1-2:2022 10.4.1)

- **Coupled walls structures:** ductile wall structure in which at least 50% of the total shear resistance is provided by walls; comprising two or more single ductile walls, connected by a regular pattern of ductile beams ("coupling beams"), where at least 25% of the total overturning moment at the base is supported by frame action of the vertical walls ("coupled walls") to the coupling beams
- **Large walls structures:** wall structures with at least two large walls in the horizontal direction of interest, which collectively support at least 20% of the total gravity load and have a fundamental fixed base period  $T_1$  not greater than  $T_c$ . It is sufficient to have only one wall meeting these conditions in one of the two directions, provided that: (a) the basic value of the behaviour factor, q, in that direction is divided by a factor of 1,5 over the value in Table 10.1 and (b) there are at least two walls meeting these conditions in the orthogonal direction
- Flat slab structure: those composed of flat slabs and columns considered as primary seismic members which contribute to the resistance to lateral loads by a slab-column mechanism
- **Inverted pendulum structures:** those in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building member
- Concrete buildings may be classified to one structural type in one horizontal direction and to another in the other direction

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## **Behaviour factor definition**

**Behaviour factor**, **q**: used for design purposes to reduce the forces obtained from a linear analysis, to account for the overstrength as well as for the non-linear response of a structure, associated with the material, the structural system and the design procedures

EN 1998-1:2004

$$q = q_{\rm o} k_{\rm w} \ge 1$$

prEN 1998-1-2:2022 (10.4.2)  $\rightarrow$  **q** =  $q_R \cdot q_S \cdot q_D$ 

- $q_0$  basic value of the behaviour factor  $k_w$  factor reflecting the **prevailing failure mode** in structural systems with **walls**
- $q_{\rm R}$  accounting for overstrength due to the **redistribution** of seismic action effects  $q_{\rm S}$  accounting for **overstrength** due to **all other sources**
- $q_{\rm D}$  accounting for the deformation capacity and energy dissipation capacity

 $\mathbf{k}_{w}$  – the concept is kept, but only applies for large walls

### Behaviour factor for DCL → DC1 (10.4.2.1)

For **DC1**, a behaviour factor **q** equal to **1**,**5** may be used, regardless of the regularity.

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					prEN 1998-1-2:2022 (1	0.4.2)					
Behaviour factors		~	,	c+	ructural type	a	q	lь	<b>q</b> = q <sub>R</sub>	$q_{s} q_{D}$	
						Υ <sub>R</sub>	DC2	DC3	DC2	DC3	1
EN 1998-1:2004				Moment resisting	multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual	1,3			2,5	3,9	
Structural type	DCW			moment resisting	structures		1,3	2,0			
Frame system, dual system,	3,0 a/ <sub>u</sub> / a	¥ 1		frame-equivalent dual structures	resisting frames	1,2			2,3	3,6	-
	3.0	_			one-storey moment resisting frames	1,1			2,1	3,3	
Torsionally flexible system	2,0			Moment resisting frame or moment							
Inverted pendulum system	1,5			equivalent dual structures with		1,1	1,2	1,4	2,0	2,3	
Frames or frame-equivalent dual systems		$\begin{array}{c} \alpha_{\rm u} \\ \alpha_{\rm 1} \end{array}$		interacting masonry							
One-storey buildings		1,1	]		wall-equivalent dual structures	1,2	1,3		2,3	3,6	
Multistorey, one-bay frames		1,2	1	wall-equivalent	coupled walls structures	1,2	1,4	2,0	2,5	3,6	
Multistorey, multi-bay frames or frame-			1	dual structures	uncoupled walls structures	1,0	1,3		2,0	3,0	,
equivalent dual structures:		1,3			large walls structures		-	-	3,0	k <sub>w</sub>	
Wall- or wall-equivalent dual systems				Flat slab structures		1,1	1,2		2,0		NEV
wall systems with only two uncoupled wal horizontal direction	ls per	1,0		Torsionally flexible bub be taken as the minin	<b>uildings</b> , the value of the behaviou mum of both horizontal directions	ır fact where	or q sho the	ould	<b>≠</b>		
Other uncoupled wall systems		1,1		primary bracings are	different, multiplied by <b>0,8</b> , but no	t sma	ller thar	nq <sub>s</sub> .			
wall-equivalent dual, or coupled wall syste	ems	1,2		For inverted pendulu	m structures, 1,5 should be used fo	or the	behavi	our fact	or q 🍣		

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### Limits of seismic action for design to DC1, DC2 and DC3 (prEN 1998-1-2:2022 10.4.3)

prEN 1998-1-1:2022 5.2.1(2)	prEN 1998-1-2:2022 10.4.3					
S <sub>δ,475</sub> (m/s²)	FrameDualstructuresstructures		Wall structures	Flat slab structures		
< 2,5	DC1/DC2/DC3	DC1/DC2/DC3	DC1/DC2/DC3 DC1/DC2			
2,5 - 5,0	DC2/DC3	DC2/DC3	DC1/DC2/DC3	DC2		
> 5,0	DC3	DC2/DC3	DC2/DC3			

NEW

#### $S_{\delta} = \delta F_{\alpha} F_{\rm T} S_{\alpha,475}$

**S**<sub>δ</sub> - Seismic action index (function of the consequence class, site amplification, topography amplification, and spectral acceleration for the **RP 475years**)



Limits of drift (SD limit state) (prEN 1998-1-2:2022 10.4.4)

- For **all types of structural systems**, the interstorey drift at **SD** limit state should be limited to:  $d_{r,SD} \leq 0.02 h_s$  where  $d_{r,SD}$  is the design interstorey drift and  $h_s$  is the storey heigh
- For MRF with interaction infills other limits are imposed (pren 1998-1-2:2022 7.4.2.1)

NEW



### **Material requirements**

EN 1998-1:2004 5.3.2 and 5.4.1.1

Primary seismic elements	DCL	DCM		
Concrete	_	≥C 16/20		
Reinforcing steel	Class B or C	Class B or C & Ribbed bars in critical regions		

prEN 1998-1-2:2022 10.3.1

Primary seismic elements	DC1	DC2 and DC3	
Concrete	≥C16	≥ C20	ŧ
Reinforcing steel	Class B or C	Class B or C in primary Seismic elements <b>Ribbed bars</b> in critical regions	*

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Local ductility condition (prEN 1998-1-2:2022 10.2.3)

- (1) For the **required overall ductility** of the **structure** to be achieved, the **potential regions for plastic hinge** formation (defined for each type of building member) should satisfy the conditions, in addition to the material properties provisions in 10.3.1 and 10.3.3:
  - a) A sufficient **ductility** and **plastic rotation capacity** should be provided in **all critical regions** of primary seismic members
  - b) Local buckling of compressed steel bars in potential plastic hinge regions of primary seismic members should be prevented by applying relevant provisions:
    - SD limit state verifications and detailing for **beams** (cl.10.5.4)
    - SD limit state verifications and detailing for **columns** (cl.10.6.3)
    - Verification of beam-columns joints (cl.10.7)
    - SD limit state verifications and detailing for **ductile walls** (cl.10.8.3)
    - SD limit state verifications and detailing for large walls (cl.10.9.3)



Local ductility condition (prEN 1998-1-2:2022 10.2.3)

(2) To satisfy (1)a), the product of the global displacement ductility factor, taken equal to the product of the components of the behaviour factor q<sub>R</sub> and q<sub>D</sub> and the chord rotation at yielding of each member end, determined according to prEN 1998-1-1:2022, 7.2.2.1.1, should not exceed the chord rotation for the SD Limit State given in prEN 1998-1-1:2022, Formula (6.32).



Note:  $\delta$  is the deformation parameter corresponding to the relevant plastic mechanism. For **frames**,  $\delta$  is the chord rotation  $\theta$ .

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Local ductility condition (prEN 1998-1-2:2022 10.2.3) (prEN 1998-1-1:2022 7.2.2.1.1)



(1) The chord rotation,  $\theta_y$ , of the shear span,  $L_v$ , at yielding of the end section, may be evaluated using, as appropriate, a) to c):

a) For rectangular beams and columns:

 $\theta_{\rm y} = \phi_{\rm y} \frac{L_{\rm V} + a_1}{3} + \frac{\phi_{\rm y} d_{\rm bL} f_{\rm y}}{8\sqrt{f_{\rm c}}} + 0,001 \ 9 \left(1 + \frac{h}{1.6 \ L_{\rm V}}\right)$ 

b) For walls of any cross-sectional shape and members with hollow rectangular section:  $\theta_{y} = \phi_{y} \frac{L_{V} + a_{1}}{3} + \frac{\phi_{y} d_{bL} f_{y}}{8\sqrt{f_{c}}} + 0,0011 \left(1 + \frac{h}{3L_{V}}\right)$ 

c) For circular columns:

$$\theta_{\rm y} = \phi_{\rm y} \frac{L_{\rm V} + a_1}{3} + \frac{\phi_{\rm y} d_{\rm bL} f_{\rm y}}{8\sqrt{f_{\rm c}}} + 0,002 \ 5\left(1 - \min\left(1; \frac{L_{\rm V}}{8 \ D}\right)\right)$$

**Note:**  $L_V = M/V =$  moment/shear at the end section, i.e. the point of contraflexure



Local ductility condition (prEN 1998-1-2:2022 10.2.3) (prEN 1998-1-1:2022 6.7.2)

## Verifications to **SD limit state**

For the verification of SD, the **resistance of ductile mechanisms** should be taken as given by:

 $\theta \downarrow SD = 1/\gamma \downarrow Rd, SD, \theta \cdot (\theta \downarrow y + \alpha \downarrow SD, \theta \cdot \theta \downarrow u \uparrow pl)$ 

 $\alpha_{SD,\theta}$  is the portion of the plastic part of the ultimate deformation  $\delta_{u}^{pl} = \delta_{u} - \delta_{y}$  that corresponds to the attainment of SD. The value of  $\alpha_{SD,\theta}$  is 0,5, unless a different value is given in the relevant part of EN 1998

- $\delta_{\rm v}$  is the local deformation of the member at **yield**
- $\delta_{\rm u}$  is the **ultimate local deformation** of the member in the ductile post-elastic mechanism

 $\gamma_{\text{Rd,SD},\theta}$  is a partial factor on resistance at SD; values of  $\gamma_{\text{Rd,SD},\theta}$  are given in Table 10.3 in prEN 1998-1-2:2022 10.4.5 ( $\gamma_{\text{Rd,SD},\theta}$  =1,26~1,35)

Note:  $\delta$  is the deformation parameter corresponding to the relevant plastic mechanism. For frames,  $\delta$  is the chord rotation  $\theta$ .

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Local ductility condition (prEN 1998-1-2:2022 10.2.3)



 $q \downarrow R \cdot q \downarrow D \cdot \theta \downarrow y \leq \theta \downarrow SD$ 

 $\theta \downarrow SD = 1/\gamma \downarrow Rd, SD, \theta \cdot (\theta \downarrow \gamma + \alpha \downarrow SD, \theta \cdot \theta \downarrow u \uparrow pl$ 

#### (1/2) **Ultimate chord rotation of a critical zone at the end of a concrete element** (prEN 1998-1-1:2022 7.2.2)

(2) If the compression zone is rectangular and at right angles to the web of the element, the **plastic part of the ultimate chord rotation** may be calculated by:

 $\theta \downarrow u \uparrow pl = \kappa \downarrow conform \cdot \kappa \downarrow axial \cdot \kappa \downarrow reinf \cdot \kappa \downarrow concrete \cdot \kappa \downarrow shearspan \cdot$ 

 $\theta^{pl}_{\mu}$  is the basic value of plastic chord rotation capacity of an element, assuming:

i) the element is detailed for ductility

ii) a concrete strength equal to 25 MPa

iii)  $L_V/h = (M/V)/h = 2,5$  (shear-span-to-depth ratio) at the section of maximum moment

iv) zero axial force

v) symmetric reinforcement concentrated at opposite ends of the section

with these assumptions,  $\theta^{\text{pl}}_{\ \text{u}}$  should be taken equal to:

= 0,039 rad if the element is a **beam** or a **column** with section consisting of rectangular parts

= 0,023 rad if the element is a **rectangular wall** 

= 0,027 rad if the element is a **wall with a barbelled**, **T-**, **I-**, **H-**, **C-** or hollow rectangular (box) section

Local ductility condition (prEN 1998-1-2:2022 10.2.3)



 $q \downarrow R \cdot q \downarrow D \cdot \theta \downarrow y \leq \theta \downarrow SD$ 

 $\theta \downarrow SD = 1/\gamma \downarrow Rd, SD, \theta \cdot (\theta \downarrow \gamma + \alpha \downarrow SD, \theta \cdot \theta \downarrow u \uparrow pl$ 

#### (2/2) **Ultimate chord rotation of a critical zone at the end of a concrete element** (prEN 1998-1-1:2022 7.2.2)

(2) If the compression zone is rectangular and at right angles to the web of the element, the plastic part of the ultimate chord rotation may be calculated by: θlu1pl=κlconform·κlaxial · κlreinf · κlconcrete · κlshearspan · κlconfinement ·θlu01pl =0,9 for a structure conforming to DC3

= 0,8 for a structure conforming to DC1

 $\kappa_{\text{axial}} = 0, 2^{\nu}$  is the correction factor for an axial force different than 0, where  $\nu$  the normalised axial force

 $\kappa \downarrow reinf = [\max(0,0,1;\omega \uparrow')/\max(0,001;\omega \downarrow tot - \omega'] \uparrow 0,25 \text{ is the correction factor for symmetrical reinforcement, with } \omega_{tot} = \rho_{tot}f_y/f_c \text{ the mechanical ratio of all longitudinal bars, with } \rho_{tot} = \sum A_s/A_c, \text{ and } \omega' = \rho' f_y/f_c \text{ the mechanical ratio of the bars under compression, with } \rho' = A'_s/A_c$   $\kappa \downarrow concrete = [min(2;f \downarrow c (MPa)/25)] \uparrow 0,1 \text{ is the correction factor for concrete strength different than 25 MPa}$   $\kappa \downarrow shearspan = [1/2,5 min(9;L \downarrow V/h)] \uparrow 0,35 \text{ is the correction factor for shear span ratio different than 2,5}$ 

 $\kappa \downarrow shearspan = 24 \uparrow (\alpha \cdot \rho \downarrow sw \cdot f \downarrow yw / f \downarrow c)$  is the correction factor taking into account the confinement of concrete due transverse bars

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European <u>Co</u>mmission Beams - Geometrical and other provisions (prEN 1998-1-2:2022 10.5.1)

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- (1) Prismatic concrete members should be designed as beams when subjected mainly to transverse loads and to a **normalised design axial force** in the seismic **a** design situation  $\nu_d = N_{Ed}/(A_c f_{cd})$  **not greater than 0,1**.
- (2) The eccentricity of the primary seismic beam axis relative to that of the column into which it frames should not be greater than b<sub>max</sub>/3, where b<sub>max</sub> is the largest ≠ cross-sectional dimension of the column normal to the longitudinal axis of the beam
  EN 1998-1:2004 5.4.1.2.1 b<sub>max</sub>/4 for DCM

(3) The width b of the primary seismic beam should not be greater than the minimum of  $b_{max}$ +h and  $2b_{max}$ , where h is the beam depth and  $b_{max}$  is as defined in (2).



Beams - Geometrical and other provisions (prEN 1998-1-2:2022 10.5.1)

(4) The effective flange width of beams cast monolithically with the slab may be taken in the model for the analysis equal to the constant value specified in prEN ≠ 1992-1-1:2022, 7.2.3(4), for the whole span.





# Beams - Specific rules for beams supporting discontinued vertical members (prEN 1998-1-2:2022 10.5.2)

- (1) Structural walls should not be supported on beams, unless the beams are designed as transfer zones in the seismic design situation according to 6.2.11 and 10.15.
- (2) For a primary seismic beam which support **discontinued columns**, conditions a) and b) should be fulfilled:
  - a) there should be no eccentricity of the column axis relative to that of the beam
  - b) the beam should be supported by at least two direct supports, such as walls or columns directly founded, **unless the beam is part of the transfer structure** designed according to 6.2.11 and 10.15
- (3) 10.5.4.2(2) should be applied.

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### Beams - Design action effects (prEN 1998-1-2:2022 10.5.3)

- (1) For DC2 and DC3, in primary seismic beams of all structural types, with clear length I<sub>cl</sub>, the design values of shear forces V<sub>i,d</sub> should be determined on the basis of the equilibrium of the beam under the effects of actions in a) and b):
  - a) the transverse load acting on it in the seismic design situation
  - b) end moments  $M_{i,d}$  (with i = 1, 2 denoting the **ends of the beam**), corresponding to plastic hinge formation at the ends of the beam or, if they form there first (as in the exceptions in 10.2.4(2) for frame or frame-equivalent structures), in the vertical members connected to the joints into which the beam ends frame, for positive and negative directions of seismic demand:

$$M_{i,d} = \begin{cases} \gamma_{Rd} M_{Rd,b,i} & \text{if } \Sigma M_{Rd,b} < \Sigma M_{Rd,c} \\ \gamma_{Rd} M_{Rd,b,i} (\Sigma M_{Rc} / \Sigma M_{Rb}) & \text{if } \Sigma M_{Rd,b} > \Sigma M_{Rd,c} \end{cases}$$



EN 1998-1:2004 5.4.2.2

**1,0** for **DCM** 

#### Beams - Design action effects (prEN 1998-1-2:2022 10.5.3)

- $M_{i,d} = \begin{cases} \gamma_{\text{Rd}} M_{\text{Rd,b,i}} & \text{if } \Sigma M_{\text{Rd,b}} < \Sigma M_{\text{Rd,c}} \\ \gamma_{\text{Rd}} M_{\text{Rd,b,i}} (\Sigma M_{\text{Rc}} / \Sigma M_{\text{Rb}}) & \text{if } \Sigma M_{\text{Rd,b}} > \Sigma M_{\text{Rd,c}} \end{cases}$
- $\gamma_{Rd}$  is a **factor** accounting for possible **overstrength** due to steel strain hardening and confinement of the concrete of the compression zone, which may be taken equal to 1,1 for DC2 and 1,15 for DC3
- $M_{\rm Rd,b,i}$  is the design value of the beam resisting moment at end *i* in the sense of bending moment considered, taking into account the slab reinforcement within an effective width defined in 10.2.4(3)
- ΣM<sub>Rd,c</sub> is the sum of the design values of the resisting moments of the columns framing into the joint (see 6.2.7(2)), corresponding to the column axial force(s) in the seismic design situation for the considered sense of beam bending
- $\Sigma M_{Rd,b}$  is the sum of the design values of resisting moments of the beams framing into the joint (see 6.2.7(2))



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### Beams - Design action effects (prEN 1998-1-2:2022 10.5.3)

(2) At a **beam end i supported by another beam**, instead of framing into a vertical member, the beam end moment  $M_{i,d}$  there may be taken equal to the moment at the beam end  $\approx$  section obtained from the analysis in the seismic design situation.



## Beams – SD limit state verifications and detailing Resistance in bending and shear (prEN 1998-1-2:2022 10.5.4.1)

- The bending resistance should be calculated in accordance with prEN 1992-1-1:2022, 8.1.
- (2) The **shear resistance** should be calculated in accordance with prEN 1992-1-1:2022, 8.2, as modified in prEN 1998-1-1:2022, 7.2.3.

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# Beams – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.5.4.2)

- (1) The regions of a primary seismic beam up to a distance I<sub>cr</sub> = h (where h denotes the beam depth) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, should be considered as critical regions.
- (2) In primary seismic beams supporting discontinued (cut-off) vertical members, the regions up to a distance of 2h on each side of the supported vertical member should be considered as being critical regions.



# Beams – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.5.4.2)

(3) Along the entire length of a primary seismic beam, the **reinforcement ratio** of the tension zone,  $\rho_{I}$ , should not be smaller than the **minimum** value  $\rho_{I,min}$  given by:

 $\rho_{\rm l,min} = 0.5 (f_{\rm ctm}/f_{\rm yk})$ 

Alternatively to the explicit calculation of  $\rho_{l,min}$ , the values in Table may be used.

		Steel grade			
	Concrete grade	B400	B500		
	C20-C25	0,35%	0,25%		
	C30-C45	0,50%	0,35%		
	C50-C90	0,60%	0,45%		

NEW



# Beams – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.5.4.2)

- (4) At the compression zone of critical regions of primary seismic beams, reinforcement not smaller than half of the reinforcement provided at the tension zone should be placed.
- (5) The **tension reinforcement ratio** should not exceed a **maximum** value  $\rho_{l,max}$  in Table 10.5, where  $\rho_l$ ' is the reinforcement ratio in the compression zone. For beams with reinforcing steel grade B450, the maximum tension reinforcement ratio may be obtained by linear interpolation of the values in Table 10.4.

	7	prEN 1998-1-2:2022		10.5.4.2	
		D	22	D	СЗ
		Steel	grade	Steel	grade
EN 1998-1:2004 5.4.3.1.2	Concrete grade	B400	B500	B400	B500
$\rho \downarrow max = \rho \uparrow' + 0,0018 / \mu \downarrow \phi$	C20-C25	$\rho_{\rm l}'$ + 0,7%	$ ho_{ m l}$ ' + 0,5%	$ ho_{ m l}$ ' + 0,5%	$\rho_{\rm l}'$ + 0,3%
$\varepsilon \downarrow sy, d \cdot f \downarrow cd / f \downarrow yd$	C30-C45	$ ho_{ m l}$ ' + 1,2%	$ ho_{ m l}$ ' + 0,8%	$ ho_{ m l}$ ' + 1,0%	$\rho_{\rm l}'$ + 0,6%
	C50-C90	$\rho_{l}$ + 1,7%	$ ho_{ m l}$ ' + 1,2%	$ ho_{ m l}$ ' + 1,5%	$ ho_{ m l}$ ' + 1,0%



< 50mm

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# Beams – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.5.4.2)

- (6) Within the critical regions of primary seismic beams, hoops should satisfy the conditions in a) to d):
  - a) The **diameter**  $d_{\rm bw}$  of the hoops should not be smaller than 6mm
  - b) The **spacing** s **of hoops** should not exceed:
    - for **DC2**: s = min{h/4; 30 $d_{bw}$ ; 12 $d_{bL,min}$ };
    - for **DC3**: s = min{h/4; 24 $d_{bw}$ ; 8 $d_{bL,min}$ }
  - c) the distance of the **first hoop** to the beam end section should not be greater than 50mm
  - d) 10.11.1(2) should be satisfied

 $d_{\rm bL,min}$  is the minimum longitudinal bar diameter h is the beam depth



< 50mm

NEW

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### Columns - Geometrical and other provisions (prEN 1998-1-2:2022 10.6.1)

- (1) Prismatic concrete members with a **depth to width ratio**  $h_c/b_c$  **not greater than 4** should be designed as columns when supporting gravity loads by axial compression or when subjected to a normalised design compression axial force in the seismic design situation  $v_d = N_{Ed}/(A_c f_{cd})$  greater or equal to 0,1.
- (2) The **minimum cross-sectional dimension** of primary seismic columns should not be smaller than:
  - a) If  $\theta \le 0.05$ : 200mm (where  $\theta$  is the interstorey drift sensitivity coefficient).
  - b) If  $\theta > 0.05$ : the maximum of:
    - one tenth of the longer distance between the point of contraflexure of the deflected shape and the ends of the column, for bending within a plane parallel to the column dimension considered;
    - 200mm for DC2 or 250mm for DC3.



### Columns - Design action effects (prEN 1998-1-2:2022 10.6.2)

 In primary seismic columns of DC2 and DC3 the design column axial force should be obtained from:

$$N_{Ed,i} = N_{Ed,G,i}"+" \Omega N_{Ed,E,i}$$

where:

N<sub>Ed,G,i</sub> is the column axial force fraction due to the **gravity loads** in the seismic design situation;

N<sub>Ed,E,i</sub> is the column axial force fraction due to the **seismic action** in the seismic design situation;

 $\Omega$  is the magnification factor of the column axial force, which may be taken equal to 2,0; "+" means combined with + or – sign.



(2) For DC2 and DC3, in primary seismic columns, the design shear forces  $V_{i,d}$  should be determined on the basis of equilibrium of the column under end moments  $M_{i,d}$  (i = 1, 2 denoting the end sections), corresponding to plastic hinge formation for positive and negative directions of the response:

$$M_{i,d} = \begin{cases} \gamma_{Rd} M_{Rd,c,i} (\sum M_{Rdb} / \sum M_{Rdc}) & if \ \sum M_{Rd,b} < \sum M_{Rd,c} \\ \gamma_{Rd} M_{Rd,c,i} & if \ \sum M_{Rd,b} > \sum M_{Rd,c} \end{cases}$$

where:

 $\boldsymbol{\gamma}_{Rd}$  accounts for overstrength due to strain hardening and confinement and may be taken equal to 1,1

 $M_{Rd,c,i}$  is the design value of the column resisting moment at end i

 $\Sigma M_{Rd,c}$  and  $\Sigma M_{Rd,b}$  are defined in 10.5.3.1(1)b) and should correspond to the column axial force(s) in the seismic design situation

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### Columns – SD limit state verifications and detailing Resistance in bending and shear (prEN 1998-1-2:2022 10.6.3.1)

- (1) In primary seismic columns, the **normalised axial force**  $v_d$  based in Formula (10.5) should not exceed:  $N_{\rm Ed,i} = N_{\rm Ed,G,i}$ "+"  $\Omega N_{\rm Ed,E,i}$ 
  - for DC2: 0,65 NEW

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- for DC3: 0,55 (in EN 1998-1:2004  $\rightarrow$  DCM: 0,65)



- (1) The total **longitudinal reinforcement ratio**  $\rho_1$  should not be less than 1% and not greater  $\approx$  than 4%.
- (2) The diameter of the longitudinal bars should not be smaller than 12mm.
- (3) In symmetrical cross sections, symmetrical reinforcement should be provided ( $\rho_1 = \rho_1'$ ).
- (4) At least one intermediate bar should be provided between corner bars along each column side.
- (5) The region up to a distance I<sub>cr</sub> from an end section of a primary seismic column should be considered a critical region:

 $l_{\rm cr} = max\{b_{\rm max}; l_{\rm cl}/6; 0,45 m\}$ 

where:

 $b_{\max}$  is the largest cross-sectional dimension of the column;

- $I_{\rm cl}$  is the clear length of the column.
- (6) If I<sub>cl</sub> / b<sub>max</sub> < 3, the entire height of the primary seismic column should be taken as critical region.



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- (7) In a critical region of a primary seismic column, hoops and cross-ties, of at least 6mm in diameter or d<sub>bL,min</sub>/4, whichever is greater, should be provided with a pattern such that the cross-section benefits from confinement. 10.11.1(2) should be satisfied. The **spacing**, *s*, of **hoops and cross-ties** should not exceed:
  - for DC2: s  $\leq$  min{ $b_{0C}/2$ ; 200mm; 9 $d_{bL,min}$ }

- for DC3:  $s \le min\{b_{0C}/2; 175mm; 8d_{bL,min}\}$ 

where:

b<sub>oc</sub> is the smallest dimension of the concrete core (to the centreline of the hoops)

 $d_{\rm bL,min}$  is the minimum diameter of the longitudinal bars





- (8) The distance between consecutive longitudinal bars engaged by hoops or cross-ties should not exceed 250mm for DC2 and 200mm for DC3, taking into account prEN 1992-1-1:2022, 12.6. NEW
- (9) A minimum value of mechanical volumetric ratio of confining hoops within the critical regions should be provided within all critical regions in the primary seismic columns equal to:  $\omega_{\rm wd} = \frac{volume \ of \ confining \ hoops}{volume \ of \ concrete \ core} \ f_{\rm ywd} / f_{\rm cd}$ 
  - for DC2: 0,05;
  - for DC3: 0,08.
- (10) The mechanical volumetric ratio of confining hoops of columns with rectangular cross sections with dissimilar amount of reinforcement in the two directions should be taken from:

$$\omega_{\rm wd} = 2 \min\{\rho_{\rm w,x}, \rho_{\rm w,y}\} f_{\rm ywd}/f_{\rm cd}$$

where  $\rho_{w,x}$  and  $\rho_{w,y}$  are the volumetric ratios of confining hoop legs in the perpendicular directions x and y.

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#### Beam-columns joints (prEN 1998-1-2:2022 10.7)

- (1) For DC1, the horizontal confinement reinforcement in beam-column joints should not be smaller than that of the columns framing into the joint, with the exception in (3).
  NEW
- (2) For DC2 and DC3, the horizontal confinement reinforcement in beam-column joints NEW |≈ should not be smaller than that specified in 10.6.3.2(7) to (10) for the critical regions of columns, with the exception in (3). 10.11.1(2) should be satisfied.
- (3) If beams frame into all four sides of the joint and their width is at least three-quarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that specified in (1) and (2), but should not exceed 150mm.
- (4) For DC1, DC2 and DC3, at least one intermediate vertical bar should be provided NEW |≈ between column corner bars at each side of the primary beam-column joint.

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#### Beam-columns joints (prEN 1998-1-2:2022 10.7)

For **DC3**, unless (6) is used, the **verification of the joint** should be done according to prEN **1998-1-1**:2022, 7.2.4, with some adaptations/modifications

(prEN 1998-1-1:2022 7.2.4)

(1) The shear force acting on the core concrete of the joints,  $V_{Edj}$ , obtained in each seismic design situation by either a) or b), should not exceed the shear resistance given by (2) to (6):

a) 
$$V_{\rm Edj,ext} = \gamma_{\rm Rd} A_{\rm s1} f_{\rm yk} - V_{\rm c}$$

b) 
$$V_{\text{Edj,int}} = \gamma_{\text{Rd}} (A_{s1} + A_{s2}) f_{\text{yk}} - V_{\text{c}}$$

(2) ... (6) – Procedure to determine the joint shear resistance

The model considers the stress fields in the concrete and the reinforcement as continuous and uniform within the joint's volume, uses first principles of mechanics (principal stresses and strains, constitutive relations, equilibrium) and searches for the inclination of the compression field direction and of the principal compressive strain in the joint which maximizes its shear resistance. (Fardis, 2021)



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### Beam-columns joints (prEN 1998-1-1:2022 7.2.4)

(2) ... (6) – Procedure to determine the joint shear resistance

Shear forces acting on the joints depend on:

Beam longitudinal reinforcement generating shear Yield strength of the beam longitudinal reinforcement Shear force in the column above the joint

**Shear resistance** of the joint depends on:

Column axial load

Concrete and reinforcing steel mechanical properties

Cross-sections of the beams and columns

Amount of reinforcement (vertical and horizontal) in the joint

#### Verifications:

The **shear forces acting** on the joints should **not exceed**:

- the shear resistance of the joint assumed unreinforced, at first cracking, to horizontal shear force
- the shear resistance of the reinforced joint after cracking



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#### Beam-columns joints (prEN 1998-1-1:2022 7.2.4)

#### (2) ... (6) – Procedure to determine the joint shear resistance

To obtain the shear resistance of a reinforced joint after cracking  $\rightarrow$  An iterative process is adopted that depends on: (i) Minimum joint shear resistance; (ii) Contribution of the horizontal reinforcement; (iii) Contribution of the diagonal concrete compression field; (iv) Contribution of the vertical reinforcement



Simplified proposal is included for the estimation of the inclination of the compression field ( $\theta$ ) – which gives a safe-sided estimative of the joint shear resistance.

**NOTE: Other simplifications** can be made in the calculation of the **joint shear resistance** (**neglecting** the contribution of: (i) **joint reinforcement** and (ii) **axial compression** in the connected members).

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#### Beam-columns joints (prEN 1998-1-2:2022 10.7)

- (6) Alternatively to (5), the values in Table 10.6 may be taken as minimum horizontal shear reinforcement ratio in joints ( $\rho_{sh} = A_{sh}/(b_{j,ef} h b)$ ), when a) to f) are satisfied: NEW
  - a) Axial load ratio:  $v \ge 0,1$  for interior joints; and  $0,1 \le v \le 0,25$  for exterior joints
  - b)  $b_b \leq b_c$
  - c)  $0,20m \le b_b \le 0,40m$
  - d)  $0,30m \le h_{\rm b} \le 0,60m$
  - e)  $0,30m \le b_c \le 0,60m$
  - f)  $0,30m \le h_c \le 0,60m$

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#### Beam-columns joints (prEN 1998-1-2:2022 10.7)

#### Table 10.6 – Minimum horizontal reinforcement ratio in beam-column joints ( $\rho_{sh}$ )

	Concrete grade	Top ( $\rho_{s1}$ ) and bottom ( $\rho_{s2}$ ) beam longitudinal reinforcement ratio of beams generating shear in the joints	Steel grade			
			B400		B500	
			$0,75 \le h_c/h_b < 1,25$	$1,25 \le h_c/h_b \le 2,0$	$0,75 \le h_c/h_b < 1,25$	$1,25 \le h_c/h_b \le 2,0$
Interior beam- column joints	C20-C25	$\rho_{s1} + \rho_{s2} \le 0.8\%$	0,45%	0,40%	0,55%	0,45%
		$0.8\% < \rho_{s1} + \rho_{s2} \le 1.2\%$	0,90%	0,80%	1,00%	0,85%
	C30-C45	$\rho_{s1} + \rho_{s2} \le 0.8\%$	0,40%	0,40%	0,45%	0,45%
		$0.8\% < \rho_{s1} + \rho_{s2} \le 1.2\%$	0,80%	0,65%	0,85%	0,70%
Exterior beam- column joints	C50-C90	$\rho_{s1} + \rho_{s2} \le 0.8\%$	0,40%	0,40%	0,45%	0,45%
		$0.8\% < \rho_{s1} + \rho_{s2} \le 1.2\%$	0,55%	0,45%	0,70%	0,55%
	C20-C25	$\max\{\rho_{s1}; \rho_{s2}\} \le 0.6\%$	0,45%	0,35%	0,50%	0,45%
		0,6% < max{ p <sub>s1</sub> ; p <sub>s2</sub> } ≤ 0,8%	0,70%	0,60%	0,70%	0,65%
	C30-C45	$\max\{\rho_{s1}; \rho_{s2}\} \le 0.6\%$	0,40%	0,40%	0,40%	0,40%
		$0,6\% < \max\{\rho_{s1}; \rho_{s2}\} \le 0,8\%$	0,60%	0,50%	0,65%	0,55%

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#### Ductile walls - Geometrical and other constraints (prEN 1998-1-2:2022 10.8.1)

- (1) Concrete walls are defined as the vertical members which have an **elongated cross**section with a length to thickness ratio  $I_w/b_w$  greater than 4.
- (2) Concrete ductile walls should be **fixed at the base** so that the relative rotation of their base with respect to the rest of the structure is prevented and should be **designed and**  $\approx$  **detailed** to **dissipate energy in a flexural plastic hinge zone** free of large openings or large perforations just above their base **and** at levels where the **change in wall length** <sub>NEW</sub>  $I_{\rm w}$  is greater than 30%.
- (3) The **thickness of the web**,  $b_{w0}$ , should satisfy:  $b_{w0} \ge max\{0,15 m; h_{s,cl}/20; l_w/40\}$ where  $h_{s,cl}$  is the clear storey height.

Boundary element Web  $\neq$   $l_w$ 

(4) **Openings** not regularly arranged to form coupled walls should be avoided in primary seismic walls, unless their effect is either insignificant or accounted for in the analysis, **NEW** dimensioning and detailing.

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- (1) Seismic action effects may be redistributed between primary seismic walls up to 30%, provided that the total resistance is not reduced. Shear forces should be redistributed along with bending moments, so that in the individual walls the ratio of bending moment to shear force is not changed by more than 20%. If the variation of the axial force is large, as e.g. in coupled walls, bending moments and shear forces should be redistributed to the wall(s) which are under low compression or in net tension, to those which are under greater compression.
- (2) In coupled walls, as defined in 10.4(1)f), seismic action effects may be redistributed between coupling beams of different storeys up to 20%, provided that the seismic axial ≈ force at the base of each individual wall is not affected.



(3) The **flexural resistance of the wall** at its base should not be lower than the design moment  $M'_{Edw,base}$  from the analysis. Elsewhere along the height of the wall, the **design moment**  $M_{Edw}$  should be given by a **vertically displaced** (tension shift) **envelope of the bending moment diagram** from the analysis, with the **value at the base** section **replaced** by the **flexural overstrength moment**,  $M_0 = \gamma_{Rd} M_{Rdw,base}$ , with  $\gamma_{Rd} = 1,2$ , where  $M_{Rdw,base}$  is the *flexural resistance* of the wall at the base section including the vertical web reinforcement.

Within the critical height of the wall, defined in 10.8.3.2(1), the design moment should be taken equal to  $M_{Rdw,base}$ . The tension shift and the critical height of the wall should be consistent with the strut inclination taken in the SD limit state verification for shear.





(4) In ductile walls, when the force-based approach is used, the design shear force V<sub>Edw</sub> at level z should be derived from:

$$V_{\rm Edw}(z) = \varepsilon(z) \, V'_{\rm Edw,1}(z) \le q \, V'_{\rm Edw}(z)$$

V'<sub>Edw</sub>(z) is the shear force at level z calculated as the combination of shear force in all modes from the analysis;

 $V'_{Edw,1}(z)$  is the shear force at level z calculated by the analysis; if a modal response spectrum analysis is used,  $V'_{Edw,1}(z)$  is the effect due to the mode with the largest participating mass in the direction of  $V_{Edw}$ ;

q is the behaviour factor used in the design;

 $\varepsilon$  (z) is the shear magnification factor from Formula (10.12), but not smaller than 1.5 nor greater than  $q V'_{\text{Edw}}/V'_{\text{Edw},1}$ 

$$\varepsilon(z) = \sqrt{\left(\frac{\gamma_{\rm Rd}M_{\rm Rdw,base}}{M'_{\rm Edw,base}}\right)^2 + m_{\rm s}(z) \left(\frac{qS_{\rm e}(T_{\rm C})}{S_{\rm e}(T_{\rm 1})}\right)^2}$$

The **design seismic shear forces** are obtained from the first mode response. Variable shear **magnification factor** along the height of the wall. The upper limit of the shear magnification factor is related to the total shear force. (*Rejec, Isakovic & Fischinger, 2012*)

(5) For DC2, alternatively to Formula (10.12),  $\varepsilon(z)$  may be assumed equal to q.



(6) In walls in **dual structures**, the **design shear forces**  $V_{Edw,env}$  should be in accordance with:





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# Ductile walls – SD limit state verifications and detailing Resistance in bending, shear and sliding shear (prEN 1998-1-2:2022 10.8.3.1)

- Flexural resistance should be calculated according to prEN 1992-1-1:2022, 8.1 and 8.2, unless specified otherwise in (2) to (6), using the value of the axial force from the analysis ≈ in the seismic design situation.
- (2) Vertical web reinforcement should be taken into account in the moment resistance of ≈ wall sections.
- (3) Composite wall sections consisting of connected or intersecting rectangular segments (T-, L-, U-, I- or similar sections) should be taken as integral units, consisting of a web or webs parallel, or approximately parallel, to the direction of the acting seismic shear force and a flange or flanges normal, or approximately normal, to it. In the calculation of the moment resistance and in the analysis (considering the properties of the cracked crosssection), the effective flange width on each side of a web should be taken to extend from the web face by the minimum of a) and b):
  - a) the actual flange width
  - b) one-half of the distance to an adjacent web of the wall



Ductile walls – SD limit state verifications and detailing Resistance in bending, shear and sliding shear (prEN 1998-1-2:2022 10.8.3.1)

(4) In primary seismic walls, the **normalised design axial load**  $\nu_d$  in the seismic design situation **should not exceed**:

- for DC2: 0,40;
- for DC3: 0,35.

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EN 1998-1:2004 5.4.3.4.1
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(5) The shear resistance should be calculated in accordance with 10.5.4.1(2).
 → prEN 1992:2022 with the modifications in prEN 1998-1-1:2022, 7.2.3
 With minor modifications of the general new approach for shear design of DC1, DC2 and according to prEN 1992-1-1:2022, it is applicable to design for DC1, DC2 and DC3, tending to produce a more economical design.

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(1) The height of the critical region h<sub>cr</sub> above the base of the wall (defined as the top of the foundation or of a rigid basement) may be taken from:

$$h_{\rm cr} = max\{l_{\rm w}; h_{\rm w}/6\}$$
 with the limitation given by  $h_{\rm cr} \leq \begin{cases} 2l_{\rm w} \\ h_{\rm s} & \text{if } n \leq 6 \\ 2h_{\rm s} & \text{if } n \geq 7 \end{cases}$ 

 $\approx$ 



(2) 10.6.3.2(1), (2), (7), (8), (9) and (10) should be applied within boundary elements of the wall extending vertically over the height h<sub>cr</sub> of the critical region as defined in (1), and horizontally along a length l<sub>c</sub> measured from the extreme compression fibre in the confined part of the wall up to a length not smaller than 0,15l<sub>w</sub> and 1,5b<sub>w</sub>.





(3) Confined boundary element may be omitted over wall flanges with thickness  $b_f \ge h_{s,cl}/15$ and width  $I_f \ge h_{s,cl}/5$ , where  $h_{s,cl}$  denotes the clear storey height.



 (4) In the boundary elements of walls, every other longitudinal bar should be engaged by a hoop or cross-tie.



(5) The **thickness b**<sub>w</sub> of confined parts of walls (**boundary elements**) should satisfy a) to c):

a)  $b_{\rm w}$  should not be smaller than 200mm

b) if the length of the confined part does not exceed the maximum of  $2b_w$  and  $0,2l_w$ ,  $b_w$  should not be smaller than  $h_{s,cl}/15$ , where  $h_{s,cl}$  is the clear storey height

c) if the length of the confined part exceeds both  $2b_w$  and  $0,2l_w$ ,  $b_w$  should not be smaller than  $h_{\rm s,cl}/10$ 





- (6) The relevant rules of **prEN 1992-1-1:2022**, 12.7, regarding vertical, horizontal and transverse **reinforcement** in the wall, should be applied, **complemented** with **(7) to (13)**.
- (7) In the critical region of the wall, the total vertical reinforcement ratio should not be smaller than 0,25% and the horizontal reinforcement ratio at each wall face should not be smaller than 0,125%.
- (8) In the **critical region** of the wall, the **distance between consecutive vertical and horizontal bars** should not exceed 300mm for DC2 or 250mm for DC3.
- (9) In the critical region of the wall, horizontal reinforcement contributing to shear strength should be continuous, without lapping of bar outside of the boundary zones, and their distribution should be uniform.
- (10) Vertical reinforcement should be extended of at least 0,81, beyond the wall section at which it is calculated that no reinforcement for flexure is necessary. At locations where yielding of vertical reinforcement is likely to occur, anchorage lengths should be 1,25 times the values obtained from prEN 1992-1-1:2022, 11.4.2(2) and (3).
- (11) Above the critical region of the wall, the vertical reinforcement ratio should be at least 0,5% in the parts of the section where under the seismic design situation the compressive strain  $\varepsilon_c$  exceeds 0,002.



- (12) The confining reinforcement of boundary elements should be extended 300mm below the bottom of the critical region if the boundary element is not near the edge of a footing and over the NEW s and for all anchor length if the boundary element is near the edge of a footing.
- UProvisions of the set (13) The transverse reinforcement of the boundary elements in (2)
- (14) anchored by Sor bends in the confined core of the boundary elements at not less than 150mm from the end of the section. If the horizontal web reinforcement is not more than the NEW transverse reinforcement of the boundary element parallel to it, it may terminate with a straight anchorage provided that the boundary element has sufficient length to accommodate it.



- (1) Perforations and openings in the critical region should satisfy a) to e), unless (9) is satisfied:
  - a) there should not be more than two perforations
  - b) their height and width should both be smaller than  $0.05l_{\rm w}$
  - c) they should be in the central third of  $I_{\rm w}$
  - d) the distance between centres of perforations should be smaller than  $0.2l_{\rm w}$
  - e) hairpin U reinforcement or crossties should be placed at each interruption of the face layers reinforcement by the perforations and they should have the same diameter as the face layers reinforcement



(2) Around openings, ties should be provided which should not be less than:

- a) on both sides of openings greater than 1m<sup>2</sup>, vertical boundary elements extending to the storey above and consisting of not less than four 10mm diameter bars in DC2 or four 12mm diameter bars in DC3; on both sides of openings up to 1m<sup>2</sup>, vertical ties not less than two 10mm diameter bars in DC2 or two 12mm diameter bars in DC3
- b) above and below openings, horizontal ties consisting of not less than two 10mm diameter bars in DC2 or two 12mm diameter bars in DC3
- (3) Vertical bars satisfying (2) should be engaged by hoops or crossties with a diameter of not less than 6mm or one-third of the vertical bar smallest diameter,  $d_{\rm bL}$ , whichever is greater. Hoops and crossties should be at a vertical spacing s of not more than 100mm or  $8d_{\rm bL}$ , whichever is less.



- (4) The effect of the **openings** should be considered in the analysis of the structure and in the design of walls, unless (5) is satisfied.
- (5) If in a wall the openings are not aligned vertically and if, at each storey, the area of openings is less than 10% of the area of the wall at that storey, calculated as the product of the wall length  $l_w$  by the storey height  $h_s$ , the analysis of the structure may ignore the influence of the openings in that wall.
- (6) To be considered not aligned vertically, openings at each storey should be such that any vertical line intersecting them does not intersect an opening neither in the storey immediately above, nor in the one below.
- (7) If (6) applies, the action effect in the wall may be calculated considering a strut and tie model around openings, complying with prEN 1992-1-1: 2021, 8.5.



- (8) The resistance and detailing of struts and ties in (7) should satisfy prEN 1992-1-1:2022, 8.5; compression struts in which the stress due to the design seismic action  $E_d$  is greater than 0,2  $f_{cd}$  should be confined by means of closed stirrups or crossties connecting the two face layers of reinforcement.
- (9) If in a wall the openings are aligned vertically, the structure should be considered as being two walls coupled by beams with a clear length equal to the width of the openings and a depth equal to the distance from the top of an opening to the bottom of the opening at the storey above and 10.5.3 and 10.5.4 should be satisfied for the action effects  $M_{Edcb}$  and  $V_{Edcb}$  which are the moment and shear in the coupling beam calculated by the analysis.



- (10) In (9), the coupling beams may be analysed in a strut and tie model under the shear action effects  $V_{Edcb}$  acting vertically at its ends.
- (11) If the resistance to shear requires diagonal reinforcement of the coupling beams, these reinforcement should consist of a minimum of four bars anchored in the walls to develop  $f_{yd}$ ; reinforcement transverse to these diagonals should be provided in compliance with 10.5.4.2(6).
- (12) The reinforcement transverse to the diagonals in (11) should be as in a) or b):
  - a) hoops enclosing separately each group of four diagonal bars satisfying 10.6.3.2(7) to 10.6.3.2(10)
  - b) stirrups enclosing the entire cross section of the coupling beam completed by crossties to satisfy 10.6.3.2(7) to (10)



# Ductile walls – SD limit state verifications and detailing Tying systems (prEN 1998-1-2:2022 10.8.3.4)

- (1) Outside of boundary elements, bars in the form of continuous steel ties, horizontal or vertical, should be provided, as given in a) to c), but not less than those specified in prEN 1992-1-1:2022, 12.9:
  - a) along all intersections of walls and along the connections of walls with flanges: not less than four vertical bars with 10mm diameter
  - b) an effectively continuous peripheral tie with at least 300mm<sup>2</sup> cross-sectional area at each floor and roof level
  - c) a horizontal tie at interior wall-floor connections with a cross sectional area of at least max {150mm<sup>2</sup>; 28L}, where L is the distance between the horizontal tie and the adjacent wall, in metres

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#### Large walls - Geometrical provisions (prEN 1998-1-2:2022 10.9.1)



Geometrical provisions Design action effects SD limit state verifications Detailing

- (1) Concrete large walls should have a horizontal dimension I<sub>w</sub> at least equal to 4,0m or to two-thirds of the height of the wall h<sub>w</sub>, whichever is less.
- (2) 10.8.1(3) should be applied to large walls.
Large walls - Design action effects (prEN 1998-1-2:2022 10.9.2)

Geometrical provisions Design action effects SD limit state verifications Detailing

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- (1) **The additional dynamic axial forces** developing in large walls due to geometrical nonlinearities or contact effects, should be taken into account in the SD limit state verification of the wall for bending with axial force.
- (2) Unless a response-history non-linear analysis takes into account the effects listed in (1), the **additional dynamic axial forces** may be taken as being **50% of the axial force due to the gravity loads** in the seismic design situation, with a plus or a minus sign, whichever is most unfavourable.
- (3) The **dynamic axial force** in (1) and (2) **may be neglected** in cases a) or b), as appropriate:
  - a) if the force-based approach is used and the behaviour factor q does not exceed 2,0
  - b) if the displacement-based approach is used and the geometrical non-linearity not accounted for, if the displacement on the force-displacement curve does not exceed 150% of the elastic limit, as defined in prEN 1998-1-1, 6.5.3





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- (4) To ensure that flexural yielding precedes attainment of the SD limit state in shear, the shear force  $V'_{Ed}$  from the analysis should be increased.
- (5) (4) may be considered satisfied if at every storey of the wall the design shear force V<sub>Ed</sub> is obtained from a) or b), as appropriate:

a) In the force-based approach, using:

$$V_{\rm Ed} = \frac{q+1}{2} V_{\rm Ed}'$$

b) In the displacement-based approach, the amplification factor should be taken as  $\gamma_{\rm Rd}$  = 1,3

(6) When using a **non-linear static analysis** of large walls structures, the elastic stiffness k\* of a bilinear idealisation as in prEN 1998-1-1:2022, 6.5.3(2), should correspond to their **NEW** uncracked stiffness.



## Large walls – SD limit state verifications and detailing Resistance in bending (prEN 1998-1-2:2022 10.9.3.1) ≈

Geometrical provisions Design action effects SD limit state verifications Detailing

- (1) The SD limit state in bending with axial force should be verified according to prEN 1992-1-1:2022, 8.1.
- (2) Normal stresses in the concrete shall be limited, to prevent out-of-plane instability of the wall.
- (3) (2) may be satisfied on the basis of the rules of prEN 1992-1-1:2022, 7.4, for second-order effects.
- (4) In the SD limit state, verification for bending with axial force taking into account the dynamic axial force defined in 10.9.2(1) and (2) may be performed in modifying the mean strain in the section, while keeping constant the curvature; the limiting strain  $\varepsilon_{\rm cu}$  may be increased to a greater value in accordance with prEN 1992-1-1:2022, 8.1.4, provided that spalling of the unconfined concrete cover is taken into account in the verification.



## Large walls – SD limit state verifications and detailing Resistance in shear and sliding shear (prEN 1998-1-2:2022 10.9.3.2) **≈**

Geometrical provisions Design action effects SD limit state verifications Detailing

- (1) Wherever the value of  $\tau_{Ed}$  calculated from  $V_{Ed}$  in 10.9.2(5) in accordance with prEN 1992-1-1:2022, 8.2.1(3) is less than the design value of shear resistance without shear reinforcement,  $\tau_{Rd,c}$  in prEN 1992-1-1:2022, 8.2, the web minimum shear reinforcement ratio  $\rho_{w,min}$  may be omitted.
- (2) Wherever the condition  $\tau_{Ed} \leq \tau_{Rd,c}$  is not fulfilled, web shear reinforcement should be calculated in accordance with prEN 1992-1-1:2022, 8.2.3, on the basis of a variable inclination truss model, or a strut-and-tie model, whichever is most appropriate for the particular geometry of the wall.
- (3) If a strut-and-tie model is used, the strut width should take into account any openings in the wall and should not exceed  $0,25l_w$  or  $4b_{w0}$ , whichever is smaller, where  $l_w$  is the wall length and  $b_{w0}$  the web thickness.

(4) prEN 1998-1-1:2022, 7.2.3(8), should be applied.



Detailina

## Large walls – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.9.3.3)

(1) The amount of vertical reinforcement placed in the wall should not unnecessarily
 exceed the amount required for the verification of the SD limit state in flexure with axial
 load and for the integrity of concrete.

- (2) The wall **vertical reinforcement** should be at least the **minimal reinforcement of flexural members** given in prEN 1992-1-1:2022, 12.2(2) and (3), or the **minimum vertical wall reinforcement** given in prEN 1992-1-1:2022, 12.7(2) and (3), whichever is greater.
- (3) Vertical bars for the verification of the SD limit state in bending with axial force should be concentrated in boundary elements at the ends of the cross-section. These boundary elements should extend in the direction of the length  $I_w$  over a length not smaller than  $b_{w0}$  or  $3b_{w0} \sigma_{cm}/f_{cd}$ , whichever is greater, where  $\sigma_{cm}$  is the mean value of the concrete stress in the compression zone in the SD limit state of bending with axial force. They should be anchored to develop  $f_{yd}$ .



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## Large walls – SD limit state verifications and detailing Detailing for local ductility (prEN 1998-1-2:2022 10.9.3.3)

(4) The vertical reinforcement in each boundary element mentioned in (3) should not be less than four bars with 12mm diameter in the lower storey of the building, or 10mm in all other storeys.

- (5) When in any storey the length  $I_w$  of the wall is reduced over that of the storey below by more than one-third of the storey height  $h_s$ , this storey should be considered a critical region and the vertical reinforcement in each boundary element of this storey should not be less than four bars with 12mm diameter and they should be anchored in the storeys above and below.
- (6) 10.8.3.3(3) and (4) and 10.8.3.4 should be applied. (Openings)
- (7) Vertical bars for the verification of the SD limit state in bending with axial force in accordance with (3) should be engaged by hoops or crossties with a diameter of not less than 6mm and one-third of the vertical bar with smallest diameter,  $d_{\rm bL}$ , whichever is greater. Hoops and crossties should be at a vertical spacing s not greater than 100mm or  $8d_{\rm bL}$ , whichever is less.

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## Flat slabs - Introduction

# This new section was **developed** based on different **codes**, guides and recent **research outputs**:

- EN 1992-1-1:2018: Eurocode 2 Design of concrete structures Part 1-1: General rules, rules for buildings, bridges and civil engineering structures
- fib Model Code for Concrete Structures 2010 (2012): Vol. 2, 377 pages
- ACI 318M-19 (2019): Building Code Requirements for Structural Concrete and Commentary
- ACI 421.2R-10 (2010): Guide to Seismic Design of Punching Shear Reinforcement in Flat Plates
- ACI 352.1R-11 (2012): Guide for Design of Slab-Column Connections in Monolithic Concrete Structures
- ASCE 7-16 (2017): Minimum Design Loads and Associated Criteria for Buildings and Other Structures
- NTC-RCDMX (2017): Reglamento de Construcciones del Distrito Federal, Gaceta Oficial de la Ciudad de México, 15 December 2017
- Hwang, S; Moehle, J (2000): Models for Laterally Loaded Slab-Column Frames ACI Structural Journal 97 (2) 345-352
- Benavent-Climent, A; Cahis, X; Catalan, A (2008): Seismic behavior of interior connections in existing waffle-flat-plate structures Engineering
  Structures 30 (2008) 2510–2516
- Drakatos, I-S; Muttoni, A; Beyer, K (2018): Mechanical model for drift-induced punching of slab-column connections without transverse reinforcement – ACI Structural journal 115 (2) 463-473

#### Approach adopted:

- Analysis of recommendations in other international codes;
- Alignment with the procedures, design rules and nomenclature adopted in Eurocode 2



- (1) 10.10 should be **applied** to **cast-in-place flat slabs** considered as **primary seismic members**, in flat slab structures.
- (2) Vertical supporting members should be arranged in a regular pattern in two orthogonal horizontal directions.
- (3) The **connection** of flat slabs to edge and corner columns should use the full section of the vertical supporting members and should have edge beams.
- (4) The thickness of each flat slab should not be smaller than 3,5% of its largest span.
- (5) In waffle flat slabs, a solid area with constant thickness should be adopted up to a distance from the column or wall support not less than 3 times the effective depth d<sub>v</sub> of the slab.
- (6) **Openings** in flat slabs should be **avoided at a distance from the** column or wall **support less than** the **shear-resisting effective depth**  $d_v$  of the slab or the drop panel. In the presence of openings at a distance from the column or wall support up to 5,5 times the shear-resisting effective depth  $d_v$  of the slab or the drop panel, the reduction of the control perimeter according with prEN 1992-1-1:2022, 8.4.2(3) and Figure 8.19, should be adopted.



Flat slabs - Basis of design (prEN 1998-1-2:2022 10.10.1)

- (7) In a model of a flat-slab frame as a moment resisting frame in 3D for linear or nonlinear analysis, "equivalent beams" which are prismatic elements connecting adjacent columns may be used; their properties should satisfy a) to e):
  - a) a theoretical span equal to the axial distance between the columns' centroids
  - b) a cross-sectional depth equal to the slab thickness
  - c) a concrete strength equal to that of the slab
  - d) a **half-width on each side** of the axis joining the columns centroids equal to the smallest of 1) to 3): 1) the distance to the slab's edge; 2) one-half the axial distance to the nearest parallel "equivalent beam"; 3) the sum of one-half the cross-sectional dimension of the column transverse to the axis of the equivalent beam, and one-fifth of the clear span between the columns
  - e) top and bottom reinforcement ratios, reinforcement grade and cover to reinforcement within the width of the "equivalent beam" equal to those in the support strip between the connected columns





Flat slabs - Basis of design (prEN 1998-1-2:2022 10.10.1)

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- (8) If the entire flat slab is **simulated with plate models**, the elastic flexural and shear stiffness properties of the flat slab used in the analysis should take into account cracking.
- (9) (8) may be satisfied by taking elastic flexural and shear stiffness of the **flat slab** equal to **one-fourth** of their **uncracked stiffness**.

- (1) Flexural reinforcement should be concentrated over the supporting columns and walls and in slab strips between adjacent supports called support strips.
- (2) **Support strip width** should be taken as the maximum of a) and b):
  - a) the sum of 25% of the panel width on each side of the column or wall centreline
  - b) the dimension of the supporting vertical member perpendicular to the support strip
- (3) The slab top and bottom flexural reinforcement to resist the full hogging and sagging moments in the slab at the ULS in the seismic design situation should be placed within the support strip.



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Flat slabs - SD limit state verifications and detailing (prEN 1998-1-2:2022 10.10.2)

- (4) A **minimum flexural reinforcement** should be provided in the **support strips**, according to a) to c):
  - a) over the whole span, **top reinforcement** of not less than one-fourth of the top reinforcement at the supports
  - b) over the whole span, **bottom reinforcement** of not less than one-third of the top reinforcement at the supports
  - c) at the face of the column or wall, bottom reinforcement of not less than the mid-span bottom reinforcement





(5) In the slab strip between two support strips, called "**middle strip**", **bottom** flexural **reinforcement** over its intersection with the support strip in the orthogonal direction of not less than **half of its mid-span bottom reinforcement** should be provided.





- (6) In each direction, the slab top and bottom flexural reinforcement corresponding to the moments transferred from the slab to the supports should be placed within the effective slab width b<sub>ef</sub> defined in (7).
- (7) The effective slab width  $b_{ef}$  should be taken as the width of the column, column capital or wall plus, on each side of the support, the column or wall depth or the slab overhang length a, but not exceeding on each side 1,5 times the slab or drop panel shear-resisting effective depth  $d_v$ .
- (8) For internal slab-support connections, the slab top and bottom flexural reinforcement corresponding to the fraction of the hogging and sagging design moment in the slab transferred by flexure to the support in the seismic design situation should be placed within the effective slab width  $b_{ef}$ . This reinforcement should not be less than 50% of the reinforcement in (3).





- (9) For external slab-support connections, the slab top and bottom flexural reinforcement required to resist the total hogging and sagging design moments in the slab in the seismic design situation should be placed within the effective slab width  $b_{ef}$  and should be anchored at the support according to 10.11.2.
- (10) In addition to (3) to (9), the slab bottom reinforcement over the column or wall support width should satisfy a) and b) in each direction, ("integrity reinforcement"):



to avoid progressive collapse Based on FIB Model Code (2010)



- a) a minimum of four continuous bars with diameter not greater than  $0,12d_v$  should be placed over the support width with an anchor length from the support face not smaller than  $d_v + l_b$ , where  $d_v$  is the slab or drop panel shear-resisting effective depth at the support face, and lb the anchorage length;
- b) the vertical force resistance it provides,  $V_{Rd,int}$ , from Formula (10.17), should be greater than the vertical load transferred to the support.

$$V_{\text{Rd,int}} = \sum A_{\text{s,int}} f_{\text{yd}} k \sin \alpha_{\text{ult}} \le \frac{0.5\sqrt{f_{\text{ck}}}}{\gamma_{\text{c}}} d_{\text{v}} b_{\text{int}}$$

where:

A<sub>s,int</sub> is the integrity reinforcement's cross-sectional area

- k is the characteristic tensile strength to yield strength ratio of the reinforcement  $(f_t/f_y)_k$  defined in prEN 1992-1-1:2022, 5.2.2(1), Table 5.5
- $\alpha_{\text{ult}}$  is the angle between the integrity bars and the slab plane at failure ( $\alpha_{\text{ult}} = 20^{\circ}$  for reinforcing steel of ductility class B;  $\alpha_{\text{ult}} = 25^{\circ}$  for reinforcing steel of ductility class C)
- $b_{\text{int}}$  is the control perimeter activated by the integrity reinforcement from  $b_{\text{int}} = \sum \left( s_{\text{int}} + \frac{\pi}{2} d_{\text{v}} \right)$



- (11) Punching shear resistance should be verified in all slab-column, slab-wall connections and drop panel borders, based on the control perimeter b<sub>0,5</sub> defined in prEN 1992-1-1:2022, 8.4.2(2) and (3), where the slab or drop panel shear effective depth d<sub>v</sub> should be determined according to prEN 1992-1-1:2022, 8.4.2(1).
- (12) **Punching shear** should be verified considering the eccentricity of the shear forces, according to prEN 1992-1-1:2022, 8.4.2(6) to (8) using for all types of support the refined value of eccentricity  $e_b$  of the shear force for internal columns as given in prEN 1992-1-1:2022, 8.4.2(6), Table 8.3. The components  $e_{b,x}$  and  $e_{b,y}$  of the eccentricity should be calculated from capacity design moments, determined in each one of two orthogonal directions of the supporting column as the smaller of a) and b).
  - a)The sum of design values of moment resistances of the supporting column at the interfaces with the slab above and below its joint with the column;
  - b) The slab's flexural resistance at a support section normal to the plane of bending calculated according to 1) or 2): 1) 80% of the sum of design values of sagging and hogging moment resistances of the full width of the slab tributary to an interior joint with the column or 75% of the hogging moment resistance at edge joints in the direction of bending; 2) at interior columns, as 1.25-times the sum of design values of sagging and hogging moment resistances of the support sections of "equivalent beams" per 10.10.1(7) framing into opposite sides of the column in the plane of bending; at edge columns, the design value of hogging moment resistance of the support section of the equivalent beam per 10.10.1(7) connected to the column.



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- (13) prEN 1992-1-1:2022, 8.4, as modified in (12) should be used for the verification of flat slabs in punching shear at SD. Punching shear reinforcement may be omitted when the **maximum design punching shear stress**  $\tau_{Ed}$  in the control perimeter in the seismic design situation satisfies Formula (10.20).
- (14) At joints with columns surrounded by others in all directions, the punching shear resistance in prEN 1992-1-1:2022, 8.4.3, Formula (8.94), may be multiplied by n<sub>pm</sub> as given in Formula (10.19a).

$$\gamma_{\rm pm} = \sqrt{\frac{h}{d_{\rm v}}} \left( 1, 2 \frac{\sqrt{f_{\rm ck}}}{\rho_l f_{\rm yk}} \right)^{\frac{1}{4}} \ge 1$$

(15) Alternatively to (13) and prEN 1992-1-1:2022, 8.4, as modified in (12) to (14), the flat slab may be verified in punching shear at SD according to the modifications to prEN 1992-1-1:2022, 8.4 given in (16) and (17).



(16) The punching shear resistance in slabs without shear reinforcement should be  $\tau_{\text{Rd},c} = \frac{\sigma_{,\tau}(100\rho_{l})^{4/3}}{\gamma_{V}} \sqrt{f_{ck}} \text{ line with instances and the second sec$ calculated with Formula (10.20), instead of prEN 1992-1-1:2022:  $\tau_{\rm Rd,max} = \eta_{\rm sys}^{1/2} \tau_{\rm Rd,c}$ 



(19) Punching shear reinforcement detailing should follow prEN 1992-1-1:2022, 12.5.1(1) and
 (2), but bent-up bars or other type of inclined reinforcement should not be used.

(20) The radial spacing s of the punching shear reinforcement link legs should not exceed 0,5d<sub>v</sub>, and the first line of link legs should be placed no farther than 0,25d<sub>v</sub> from the support face.



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### Provisions for anchorages and laps - General (prEN 1998-1-2:2022 10.11.1)

(1) prEN 1992-1-1:2022, 11.4 and 11.5, for the detailing of reinforcement should be applied for DC2 and DC3, complemented with (2), 10.11.2 and 10.11.3.

- (2) For the transverse reinforcement in beams, columns, beam-column joints, walls or slabs, closed stirrups with 135° hooks and extensions of 10-diametre length should be used. Crossties should also be closed with 135° hooks and extensions of 10-diametre length.
- (3) The anchorage length of beam or column bars anchored within beam-column joints should be measured from a point on the bar at a distance  $5d_{bL}$  inside the face of the joint (see Figure 10.14a an example of a beam bar anchorage).
- (4) When post-installed bars are used, 10.11 should be applied and they should satisfy a) and b):

a) they should not be used in critical zones;

b) the anchoring system should comply with prEN 1992-1-1:2022, Annex C, C8.

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#### Provisions for anchorages and laps - Anchorage of reinforcement in beams (prEN 1998-1-2:2022 10.11.2)

- (1) The part of beam **longitudinal reinforcement** bent in beam-column joints for anchorage should always be placed **inside the corresponding column hoops**.
- (2) To **prevent bond failure**, the **diameter** *d*<sub>bL</sub> of beam **longitudinal bars** crossing beamcolumn joints **should be limited** in accordance with:
  - for interior beam-column joints:

$$\frac{d_{\rm bL}}{h_{\rm c}} \le \frac{7.5 \, f_{\rm ctm}}{\gamma_{\rm Rd} \, f_{\rm yd}} \frac{1+0.8 \, \nu_{\rm d}}{1+\frac{0.75 k_{\rm D} \, \rho'_{\rm l}}{\rho_{\rm l,max}}}$$

- for exterior beam-column joints:

$$\frac{d_{\rm bL}}{h_{\rm c}} \le \frac{7.5 \, f_{\rm ctm}}{\gamma_{\rm Rd} \, f_{\rm yd}} (1 + 0.8 \, \nu_d)$$

- $h_{\rm c}$  is the cross-sectional dimension of the column parallel to the bars
- $f_{\rm ctm}$  is the mean value of the tensile strength of concrete;
- $f_{yd}$  is the design yield strength of steel reinforcement;
- $\nu_{d} = N_{Ed}/(f_{cd} A_{c})$  is the normalised axial force in the column above the joint in the seismic design situation;
- **k**<sub>D</sub> is the **factor reflecting the ductility class** equal to **1/2 for DC2** and to 2/3 for DC3;
- $ho_1'$  is the compression steel ratio of the beam bars passing through the joint;
- $\rho_{\rm l,max}$  is the maximum allowed tension steel ratio (see 10.5.4.2(5));
- *r*<sub>Rd</sub> accounts for overstrength due to strain-hardening and may be taken equal to 1,1 for DC2 and 1,2 for DC3.



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Provisions for anchorages and laps - Anchorage of reinforcement in beams (prEN 1998-1-2:2022 10.11.2)

- (3) The **limitations** in (2) may be **disregarded** for **diagonal bars** crossing joints.
- (4) Alternatively to the explicit calculation of d<sub>bL</sub> given by Formulas (10.20) and (10.21), the values in Table may be used, for reinforcing steel of ductility class B or C, when a) and b) are satisfied:
  - a) if the **normalised design axial force** in the column is **not less** than **0,15**
  - b) at **interior** beam-column **joints**, if the ratio of beam **compression bars crossing the joint** is **less than 1%**

Interior beam- column joints	Concrete grade	Steel grade			
		B400	B500		
	C20-C25	DC2: $d_{\rm bL} \le 4,0\% h_{\rm c}$ DC3: $d_{\rm bL} \le 3,0\% h_{\rm c}$	DC2: $d_{\rm bL} \le 3,0\% h_{\rm c}$ DC3: $d_{\rm bL} \le 2,0\% h_{\rm c}$		
	C30-C45	DC2: $d_{\rm bL} \le 6,0\% h_{\rm c}$ DC3: $d_{\rm bL} \le 4,5\% h_{\rm c}$	DC2: $d_{\rm bL} \le 5,0\% h_{\rm c}$ DC3: $d_{\rm bL} \le 3,5\% h_{\rm c}$	NEW	
	C50-C90	DC2: $d_{\rm bL} \le 8,5\% h_{\rm c}$ DC3: $d_{\rm bL} \le 7,0\% h_{\rm c}$	DC2: $d_{\rm bL} \le 7,0\% h_{\rm c}$ DC3: $d_{\rm bL} \le 5,5\% h_{\rm c}$		
Exterior beam- column joints	C20-C25	DC2 or DC3: $d_{\rm bL} \le 4,5\% h_{\rm c}$	DC2 or DC3: $d_{\rm bL} \le 3,5\% h_{\rm c}$		
	C30-C45	DC2 or DC3: $d_{\rm bL} \le 6,5\% h_{\rm c}$	DC2 or DC3: $d_{\rm bL} \le 5,5\% h_{\rm c}$		
	C50-C90	DC2 or DC3: $d_{\rm bL} \le 9,5\% h_{\rm c}$	DC2 or DC3: $d_{\rm bL} \le 7,5\% h_{\rm c}$		

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#### Provisions for anchorages and laps - Anchorage of reinforcement in beams (prEN 1998-1-2:2022 10.11.2)

(5) If (2) cannot be satisfied in exterior beam-column joints because the depth, h<sub>c</sub>, of the column parallel to the bars is too shallow, one of the additional measures in a) to c) may be taken, to ensure anchorage of the longitudinal reinforcement of beams:

a) the beam or slab may be extended horizontally in the form of exterior stubs

- b) headed bars according to prEN 1992-1-1:2022, 11.4.7, may be used
- c) bends according to prEN 1992-1-1:2022, 11.4.4, and transverse reinforcement placed tightly inside the bend of the bars may be added



(6) Top or bottom bars crossing interior joints should be anchored in the members framing into the joint beyond a distance not less than I<sub>cr</sub> (length of critical region) from the face ≈ of the joint.



#### **Provisions for anchorages and laps - Laps and mechanical couplers** (prEN 1998-1-2:2022 10.11.3)

#### (1) There should not be laps in the critical regions of beams.

- (2) There should not be any lap-splicing by welding within the critical regions of structural members. For lap-splicing by welding outside of the critical regions within a distance of twice the member depth from the critical regions of primary seismic columns and beams, the design resistance of the welded splices should not be smaller than 1,25 times the design yield strength of the bars they connect.
- (3) There should not be splicing by **mechanical couplers** in the critical regions of primary seismic **columns** and **beams** and within a distance of twice the member depth from the critical regions, unless (4) is satisfied.
- (4) There may be splicing by **mechanical couplers** in the critical regions of primary seismic **walls** or within a distance of twice the member depth from the critical regions of primary seismic **columns** and **beams**, **if the devices are qualified by testing** for the seismic design situation and **if the yielding and the failure of the lapped bars are obtained before those of the coupler** in all tests.

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#### **Provisions for anchorages and laps - Laps and mechanical couplers** (prEN 1998-1-2:2022 10.11.3)

- (5) The transverse reinforcement to be provided within the lap length should be calculated in accordance with prEN 1992-1-1:2022, 11.5. In addition, a) to c) should be satisfied: ≈
  - a) if the anchored and the continuing bar are arranged in a plane parallel to the transverse reinforcement, the total area of all lapped bars,  $\Sigma a_{sL}$ , should be used in the calculation of the transverse reinforcement
  - b) if the anchored and the continuing bar are arranged within a plane normal to the transverse reinforcement, the area of transverse reinforcement should be calculated on the basis of the area of the greater lapped longitudinal bar, A<sub>sL</sub>
  - c) the spacing, s<sub>lap</sub>, of the transverse reinforcement in the lap zone (in mm) should not exceed the value given by Formula (10.26)

 $s_{\text{lap}} = \min\{h/4; 100\}$ 



#### Provisions for anchorages and laps - Laps and mechanical couplers (prEN 1998-1-2:2022 10.11.3)

(6) The required area of transverse reinforcement A<sub>st</sub> within the lap zone of the longitudinal reinforcement of columns spliced at the same location (as defined in prEN 1992-1-1:2022, 11.5), or of the longitudinal reinforcement of boundary elements in walls, may be calculated with:

## $A_{\rm st} = s \left( d_{\rm bL} / 50 \right) \left( f_{\rm yd} / f_{\rm ywd} \right)$

where

- $A_{st}$  is the area of one leg of the transverse reinforcement
- $d_{\rm bL}$  is the diameter of the lapped longitudinal bars
- s is the spacing of the transverse reinforcement
- $f_{yd}$  is the design yield strength of the longitudinal reinforcement

 $f_{ywd}$  is the design yield strength of the transverse reinforcement

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- Structural types, behaviour factors, limits of seismic action, limits of drift, materials
- Local ductility condition
- Beams
- Columns
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- Ductile walls
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- Anchorages and laps
- Diaphragms
- Precast concrete structures

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## Provisions for concrete diaphragms - Cast in place diaphragms (prEN 1998-1-2:2022 10.12.1) ≈

- A solid reinforced concrete slab may be considered as a rigid diaphragm, as defined in 5.1.3(6), if its thickness is not less than 70mm and if it is reinforced in both horizontal directions with at least the minimum reinforcement specified in prEN 1992-1-1:2022, 12.4.1(1).
- (2) A cast-in-place topping on a precast floor or roof structure may be considered as a rigid diaphragm, if a) to d) are fulfilled:
  - a) the **thickness** of the **topping** layer is not less than **40mm** for **span** between supports not longer than **8m**, or **50mm** for **longer spans**
  - b) its **mesh reinforcement** is connected to the beams or walls supporting the diaphragm
  - c) it is designed to provide alone the required diaphragm stiffness and resistance, according to 6.2.8
  - d) it is cast over a clean, rough substrate, or connected to it through shear connectors
- (3) **Design action effects should take into account overstrength** according to 6.2.8.

NEW

(4) The design resistances should be derived in accordance with prEN 1992-1-1:2022, 12.4, 12.5, 12.9.2 and 13.6.

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#### Precast concrete structures (prEN 1998-1-2:2022 10.14)

In summary:

Specific values of **behaviour factor** for the **different precast concrete structural type** 

Particular attention is given to the **beam-column joints** in **MRF** 

Specific rules/recommendations for **analysis** (modelling):

- Eccentricities between members
- Joints
- In-plane flexibility of the floors and roof precast members
- Effect of **rocking** panels and **interaction** with the main structure

	Structural type		qR	q <sub>D</sub>		$q = q_R q_S q_D$	
				DC2	DC3	DC2	DC3
	Moment resisting frame or moment resisting frame- equivalent dual structures	Multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual structures with beams with strong partial strength joints or rigid joints without cladding or with isostatic cladding	1,3	1,3	2,0	2,5	3,9
≠ ==⇒>		Multi-storey, one-bay moment resisting frames with strong partial strength joints or rigid joints without cladding or with isostatic cladding	1,2			2,3	3,6
		One-storey moment resisting frames with strong partial strength joints or rigid joints without cladding or with isostatic cladding	1,1			2,1	3,3
joints		One-storey one bay or multi-bay, moment resisting frames with beams with weak joints without cladding or with isostatic cladding and $\nu_d \le 0.3$	1,1	1,3	2,1	2,1	3,0
		Multi-storey, one bay or multi-bay, moment resisting frames with beams with weak joints without cladding or with isostatic cladding and $\nu_d \le 0.3$	1,1	NA*	2,1	NA*	2,0
		One-storey one bay or multi-bay, or multi-storey, one bay or multi-bay moment resisting frames with beams with weak joints without cladding or with isostatic cladding and $\nu_4 > 0.3$	NA**				
	Wall- or wall-equivalent dual structures	Wall-equivalent dual structures with strong partial strength joints or rigid joints with or without cladding	1,2	1,3	}	2,3	3,6
		Coupled wall structures with strong partial strength joints or rigid joints with or without cladding	1,2	1,4	2,0	2,5	3,6
		Uncoupled wall structure with only two uncoupled walls per horizontal direction, with or without cladding and other uncoupled wall structure, with or without cladding	1,0	1,3		2,0	3,0
st		Multi-storey moment resisting frame with beams simply supported or with weak partial strength connections and with integrated cladding	1,2	1,3	NA*	2,3	NA*
ho		Multi-storey moment resisting frame with beams with strong partial strength connections and with integrated cladding	1,2	1,4	NA*	2,5	NA*
IIE	Large walls structures and cell structures (factor $k_w$ given by Formula (10.1))		-	-	-	3kw	3kw
	NA*: Not Applica	ble NA**: Inverted pendulum – DC1 design only					

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Precast concrete structures (prEN 1998-1-2:2022 10.14)

- General rules applicable to all structural types and to DC1, DC2 and DC3
- Separate sections with **additional specific rules** (for the different DCs) for:
  - Precast MRF
  - Precast walls
  - Precast floors and roof diaphragms

#### For Precast Moment Resisting Frames:

Beams and columns of precast MRF may be connected by: **cast in place concrete zones**, or by **strong or weak partial strength joints** 

**Cast in place concrete joints** should conform to the **detailing** for **local ductility** and the relevant **requirements** of 10.1 to 10.13 (**cast in place structures**)

## **Second Generation of Eurocode 8**

# Thank you for your attention!

Humberto Varum (<u>hvarum@fe.up.pt</u>)

Credits: José Melo

24<sup>th</sup> January 2023