Seismic Design of New R.C. Structures

Prof. Stephanos E. Dritsos

University of Patras, Greece.

Pisa, March 2015

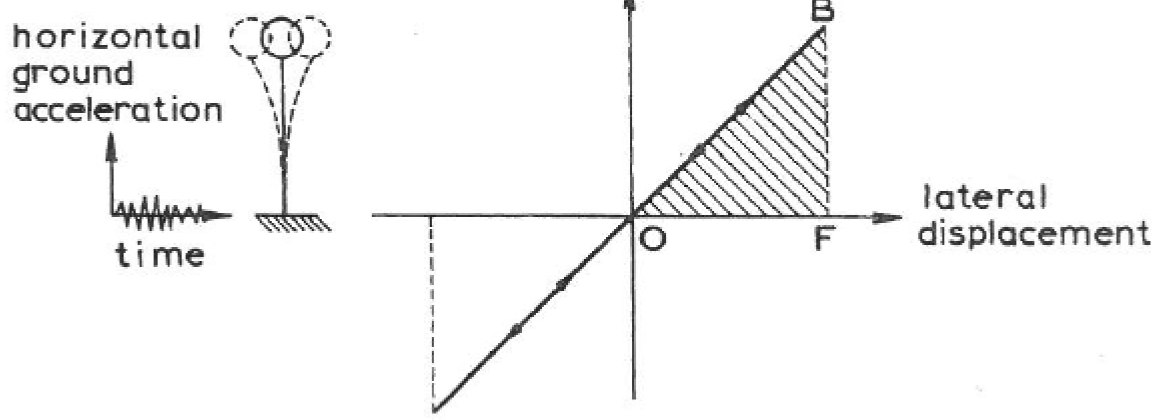
Seismic Design Philosophy

<u>Main Concepts</u>

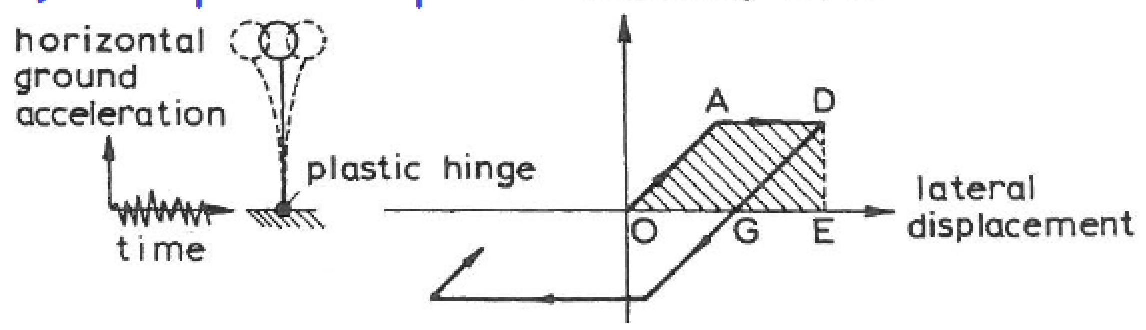
- Energy dissipation
- Ductility
- Capacity design
- Learning from Earthquakes

Energy Dissipation

1) Elastic Response

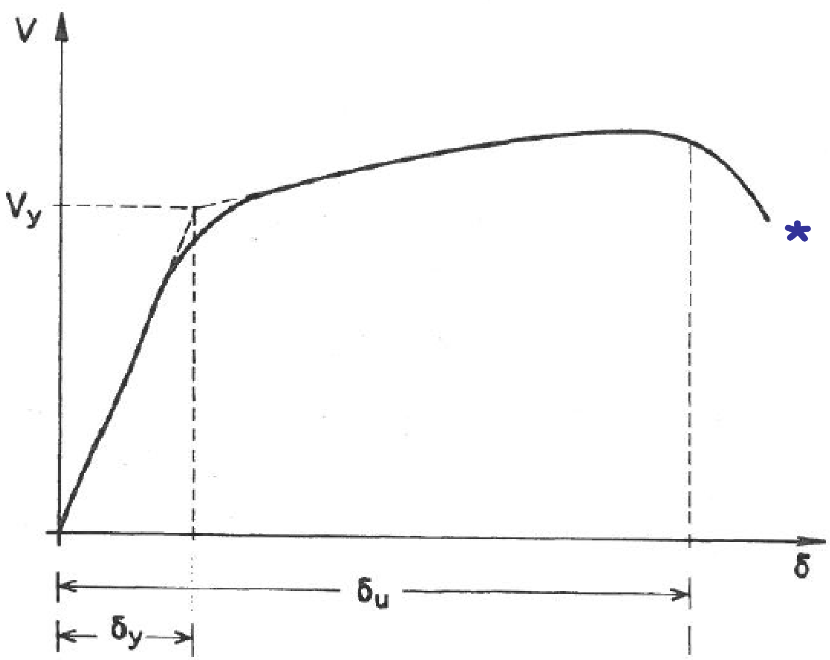


2) Elastoplastic Response



Ductility and Ductility Factors

- **Ductility** is the ability of the system to undergo **plastic deformation**. The structural system deforms before collapse without a substantial loss of strength but with a significant energy dissipation.
- The system can be designed with smaller restoring forces, exploiting its ability to undergo plastic deformation.



- **Ductility factor (δ_u/δ_y):** Ratio of the ultimate deformation at failure δ_u to the yield deformation δ_y.

* δ_u is defined for design purposes as the deformation for which the material or the structural element loses a predefined percentage of its maximum strength.

Ductility Factors

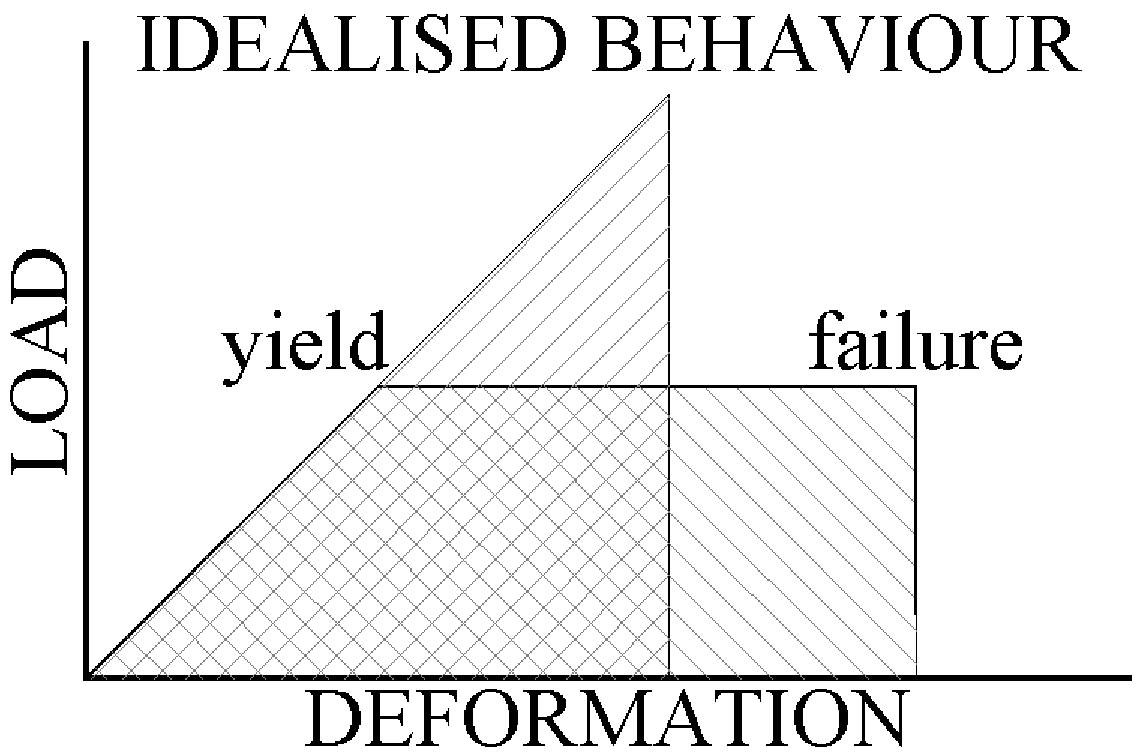
- **In terms of displacements:** $$\mu_\delta = \frac{\delta_u}{\delta_y}$$ δ_u: ultimate deformation at failure
  δ_y: yield deformation

- **In terms of rotations: (for members)** $$\mu_\theta = \frac{\theta_u}{\theta_y}$$ θ_u: ultimate rotation at failure
  θ_y: yield rotation

- **In terms of curvatures: (for members)** $$\mu_\varphi = \frac{\varphi_u}{\varphi_y}$$ φ_u: ultimate curvature at failure
  φ_y: yield curvature

Behaviour q Factor

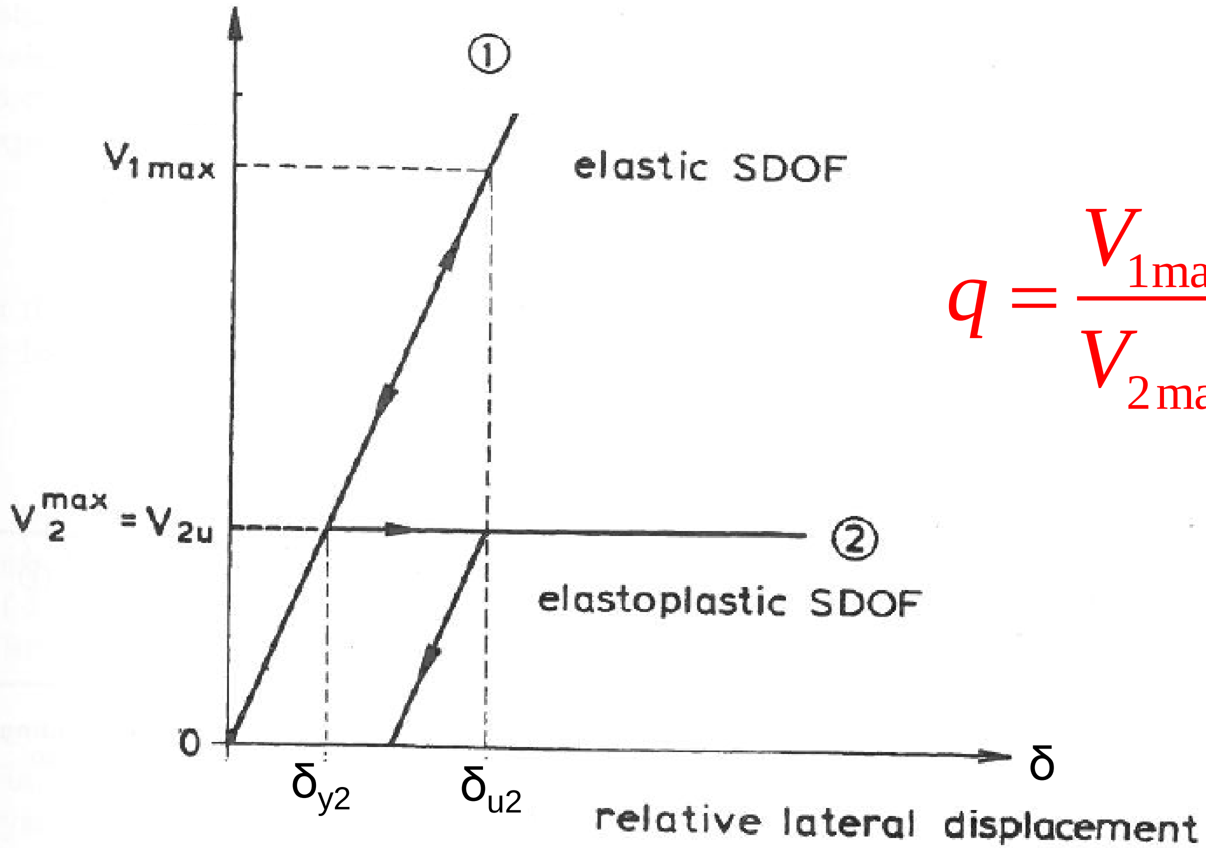
The q factor corresponds to the reduction in the level of seismic forces due to nonlinear behaviour as compared with the expected elastic force levels.



Ductility and Behaviour Factor q

$$Definition \quad q = \frac{V_{el.}}{V_{inel}}$$

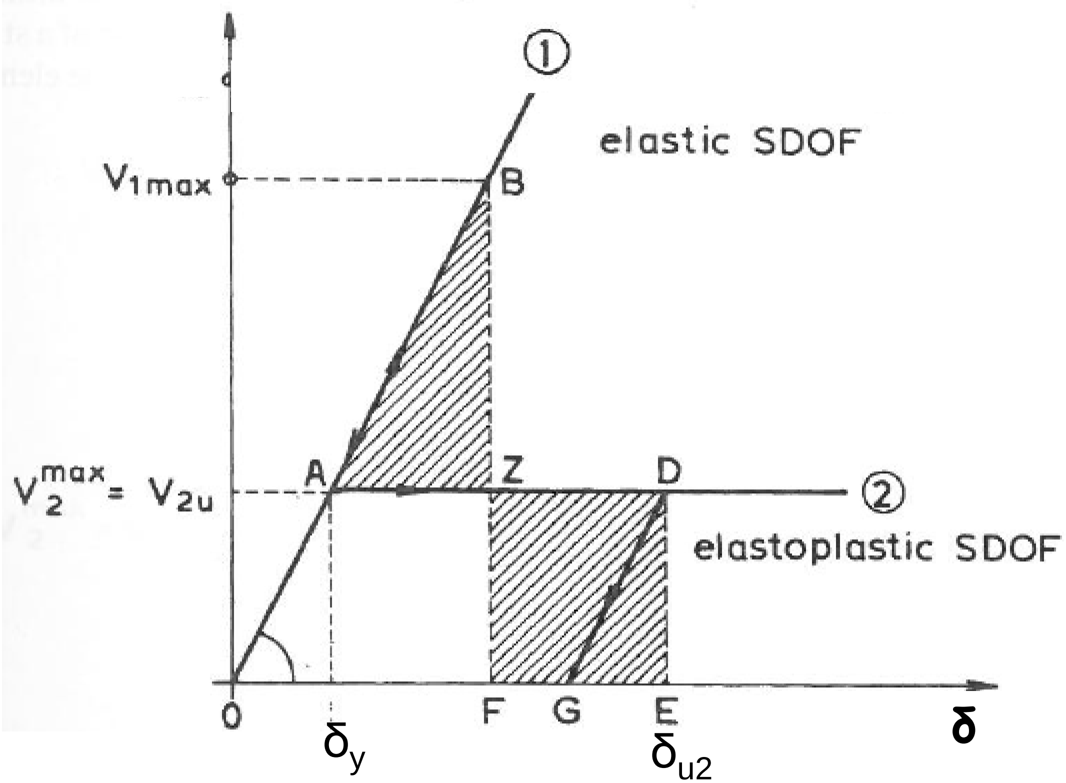
Flexible Structures T ≥ Tc : Rule of equal dispacement



$$q = \frac{V_{1\max}}{V_{2\max}} = \frac{\delta_{u2}}{\delta_{y2}} = \mu_\delta$$

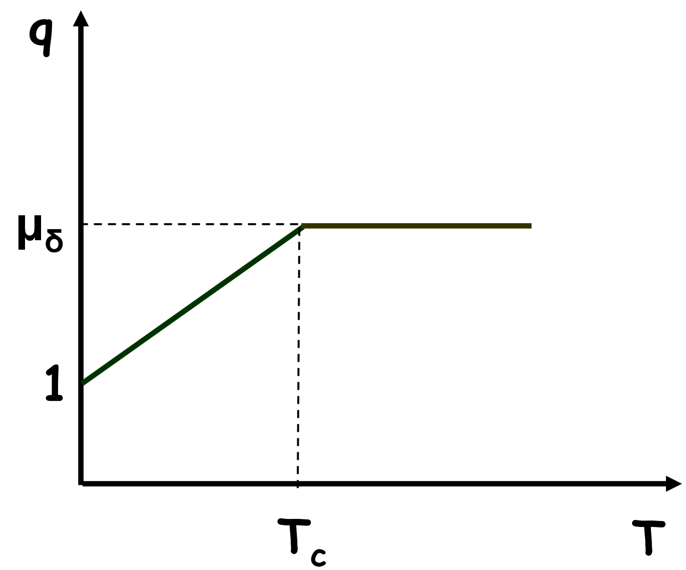
Ductility and Behaviour Factor q

Stiff Structures T≤Tc



for T=0 → q=1

for $T=T_c \rightarrow q=\mu_\delta$



Rule of equal dissipating energy

$$q = \frac{V_{el}}{V_{inel}} = \frac{V_{1\max}}{V_{2\max}} = (2\mu_\delta - 1)^{1/2}$$

$$q = 1 + (\mu_\delta - 1)\frac{T}{T_c}$$ **(Eurocode 8)**

Design spectrum for linear analysis

- The capacity of structural systems to resist seismic actions in the non-linear range permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

- The energy dissipation capacity of the structure is taken into account mainly through the ductile behavior of its elements by performing a linear analysis based on a reduced response spectrum, called design spectrum. This reduction is accomplished by introducing the behavior factor q.

Design spectrum for linear analysis (Eurocode 8)

- For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \le T \le T_C : \ S_d(T) = a_g \cdot S \cdot \frac{2,5}{q}$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases}$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases}$$

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I . a_{gR}$); γ_I=importance factor
T_B is the lower limit of the period of the constant spectral acceleration branch;
T_C is the upper limit of the period of the constant spectral acceleration branch;
T_D is the value defining the beginning of the constant displacement response range
of the spectrum;
S is the soil factor
$S_d(T)$ is the design spectrum;
q is the behaviour factor;
β is the lower bound factor for the horizontal design spectrum, recommended β=0,2

Importance Classes (Eurocode 8)

Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Behaviour Factor (Eurocode 8)

- The upper limit value of the behavior factor q, introduced to account for energy dissipation capacity, shall be derived for each design direction as follows: $q = q_o \cdot k_w \geq 1{,}5$

Where q_o is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation;

k_w is the factor reflecting the prevailing failure mode in structural systems with walls: $k_u = (1 + a_o)/3 \leq 1 \ and \geq 0{,}5 \quad a_o = \text{Prevailing wall aspect ratio} = \Sigma h_{wi} / \Sigma \ell_{wi}$

- **Low Ductility Class (DCL):** Seismic design for low ductility , following EC2 without any additional requirements other than those of § 5.3.2, is **recomended only for low seismicity cases** (see §3.2.1(4)).

Behaviour Factor (Eurocode 8)

A behaviour factor **q of up to 1,5** may be used in deriving the seismic actions, regardless of the structural system and the regularity in elevation.

- **Medium (DCM) and High Ductility Class (DCH):**

Basic value of the behaviour factor, q_o, for systems regular in elevation

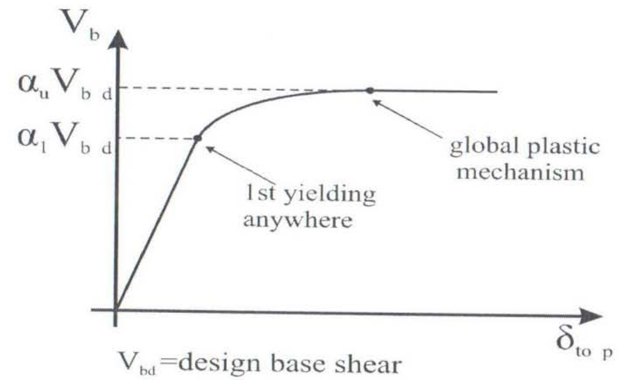
STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4{,}0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

For buildings which are **not regular in elevation**, the value of q_o should be reduced by 20%

α_u/α_1 in behaviour factor of buildings designed for ductility: due to system redundancy & overstrenght

Normally:
α_u & α_1 from base shear - top displacement curve from pushover analysis.

- α_u: seismic action at development of global mechanism;
- α_1: seismic action at 1st flexural yielding anywhere.



- $\alpha_u/\alpha_1 \leq 1.5$;
- default values given between 1 to 1.3 for buildings regular in plan:
  - = 1.0 for wall systems w/ just 2 uncoupled walls per horiz. direction;
  - = 1.1 for:
    one-storey frame or frame-equivalent dual systems, and
    wall systems w/ > 2 uncoupled walls per direction;
  - = 1.2 for:
    one-bay multi-storey frame or frame-equivalent dual systems,
    wall-equivalent dual systems & coupled wall systems;
  - = 1.3 for:
    multi-storey multi-bay frame or frame-equivalent dual systems.
- for buildings irregular in plan:
  default value = average of default value of buildings regular in plan and 1.0

Structural Regularity (Eurocode 8)

- For seismic design, building structures in all modern codes are separated in two categories: a) regular buildings
  b) non-regular buildings
- This distinction has implications for the following aspects of the seismic design:
  - the structural model, which can be either a simplified planar model or a spatial model ;
  - the method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one;
  - the value of the behavior factor q, which shall be decreased for buildings non-regular in elevation

Structural Regularity (Eurocode 8)

Consequences of structural regularity on seismic analysis and design

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear-elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force[a]	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial[b]	Lateral force[a]	Reference value
No	No	Spatial	Modal	Decreased value

Criteria for Regularity in Elevation (Eurocode 8)

- All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.

- Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

- When setbacks are present, special additional provisions apply.

STRUCTURE OF EN1998-1:2004

1 General
2 Performance Requirements and Compliance Criteria
3 Ground Conditions and Seismic Action
4 <u>Design of Buildings</u>
5 Specific Rules for Concrete Buildings
6 Specific Rules for Steel Buildings
7 Specific Rules for Steel-Concrete Composite Buildings
8 Specific Rules for Timber Buildings
9 Specific Rules for Masonry Buildings
10 Base Isolation

How q is achieved?

- Specific requirements in detailing (e.g. confining actions by well anchored stirrups)
- Avoid brittle failures
- Avoid soft storey mechanism
- Avoid short columns
- Provide seismic joints to protect from earthquake induced pounding from adjacent structures
- …..

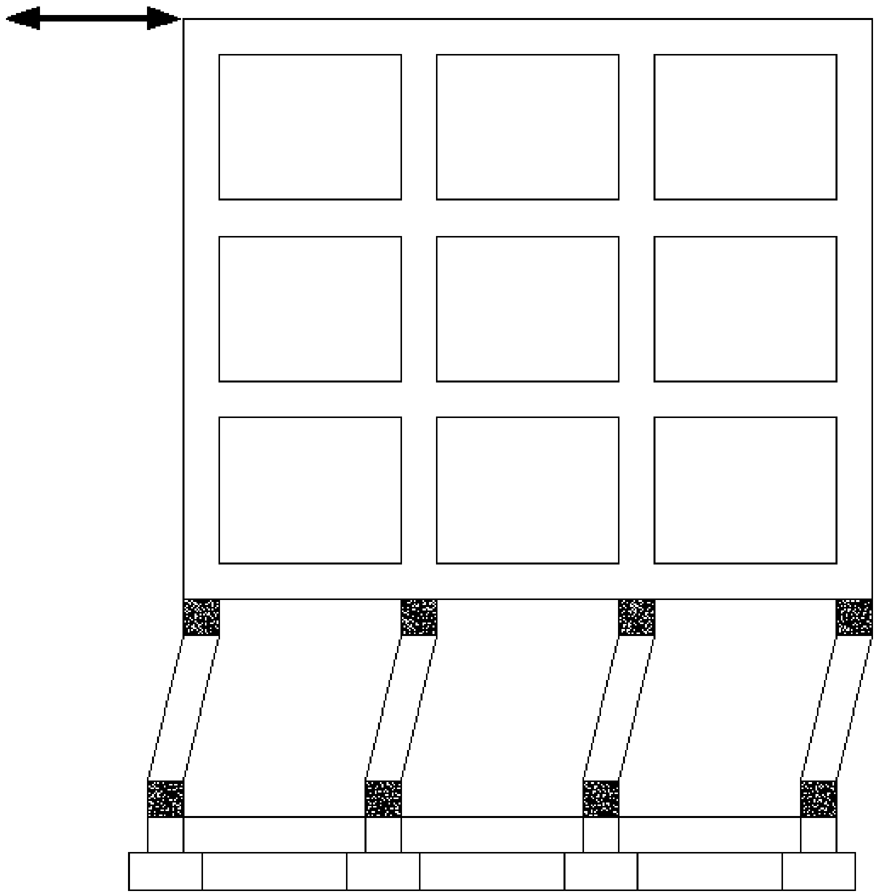
Material limitations for primary seismic elements“

Ductility Class	DC L (Low)	DC M (Medium)	DC H (High)
Concrete grade	No limit	≥ C16/20	≥ C16/20
Steel class per EN 1992-1-1, Table C1	B or C	B or C	only C
longitudinal bars		only ribbed	only ribbed
Steel overstrength:	No limit	No limit	$f_{yk,0.95} \leq 1.25 f_{yk}$

Recommended: Use same values as for persistent & transient design situations (i.e. concrete: γ_c=1.5, steel: γ_s=1.15);

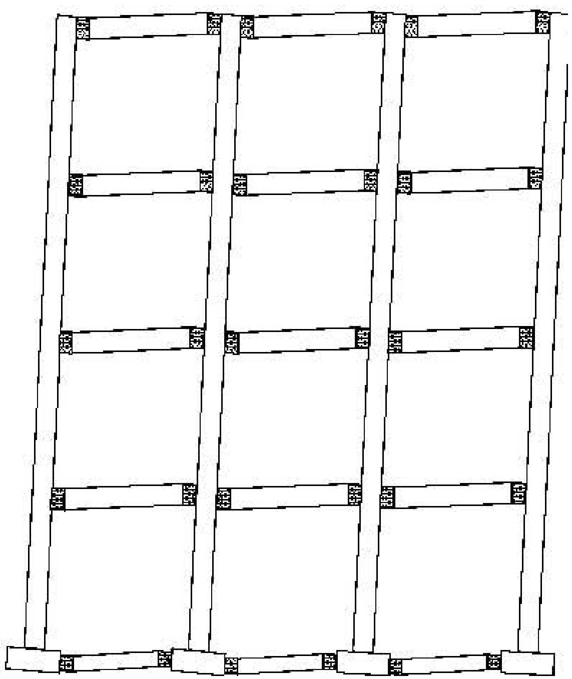
Capacity Design

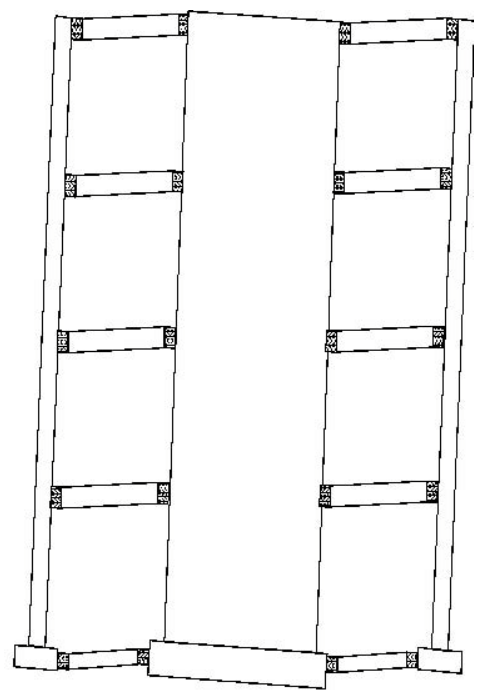
Avoid weak column/strong beam frames



Capacity Design

Provide strong column/weak beam frames or wall equivalent dual frames, with beam sway mechanisms, trying to involve plastic hinging at all beam ends



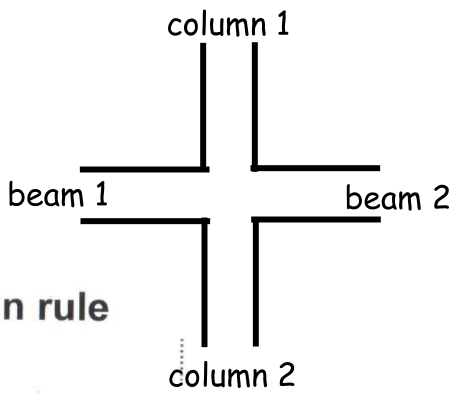


Capacity Design (Eurocode 8)

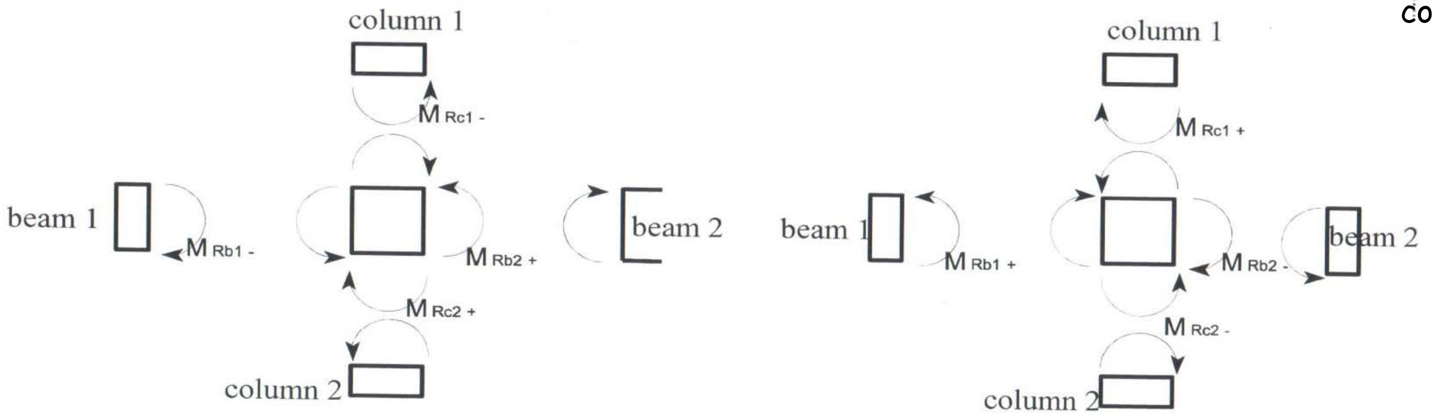
<u>Strong column/weak beam</u> capacity design rule in frames or frame-equivalent dual systems (frames resist >50% of seismic base shear) above two storeys (except at top storey joints):

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb}$$

- Overstrength factor γ_{Rd} on beam strengths $\gamma_{Rd} = 1.3$



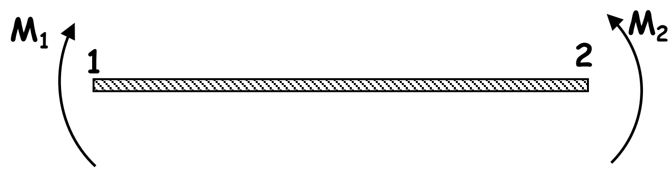
Beam & column flexural capacities at a joint in Capacity Design rule



Exceptions: see EC8 §5.2.3.3 (2)

Shear Capacity Design (Eurocode 8)

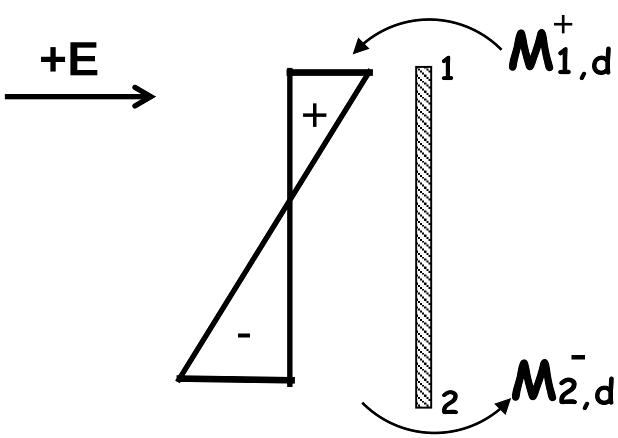
Avoid Brittle failure



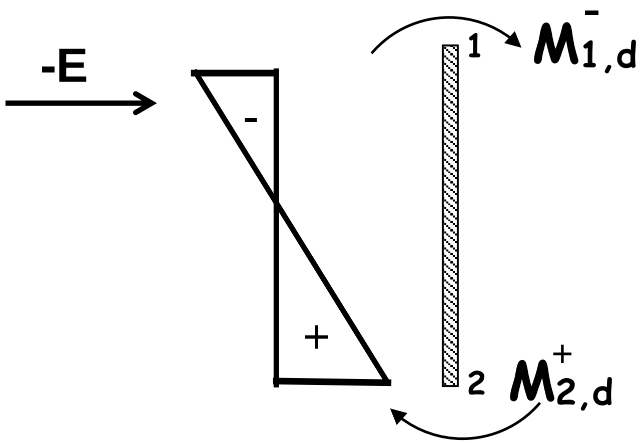
$$V = \frac{\Delta M}{\Delta x}$$

$$V = \frac{M_2 - M_1}{\ell_{12}}$$

Column moment distribution



or



$$V_{\max,c} = \frac{M^{+}_{1,d} + M^{-}_{2,d}}{\ell_{12}}, \qquad \ell_{12} = \ell_{clear}$$

$$V_{\max,c} = -\frac{M^{+}_{2,d} + M^{-}_{1,d}}{\ell_{12}}$$

Shear Capacity Design of Columns (Eurocode 8)

$M_{1,d}^{+}$ →

- $\gamma_{Rd} \cdot M_{Rc,1}^{+}$, when $\Sigma M_{Rb} > \Sigma M_{Rc}$, weak columns
- $\gamma_{Rd} \cdot M_{Rc,1}^{+} \cdot (\Sigma M_{Rd,b} \backslash \Sigma M_{Rd,c})$, when $\Sigma M_{Rb} < \Sigma M_{Rc}$, weak beams: (moment developed in the column when beams fail)

Similarly

$M_{1,d}^{-}$ →

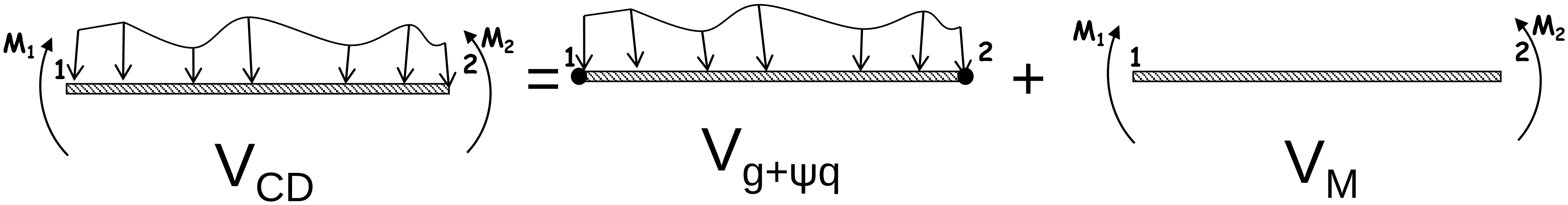
- $\gamma_{Rd} \cdot M_{Rc,1}^{-}$, when $\Sigma M_{Rb} < \Sigma M_{Rc}$
- $\gamma_{Rd} \cdot M_{Rc,1}^{-} \cdot (\Sigma M_{Rd,b} \backslash \Sigma M_{Rd,c})$

Also similarly for $M_{2,d}^{+}$, $M_{2,d}^{-}$

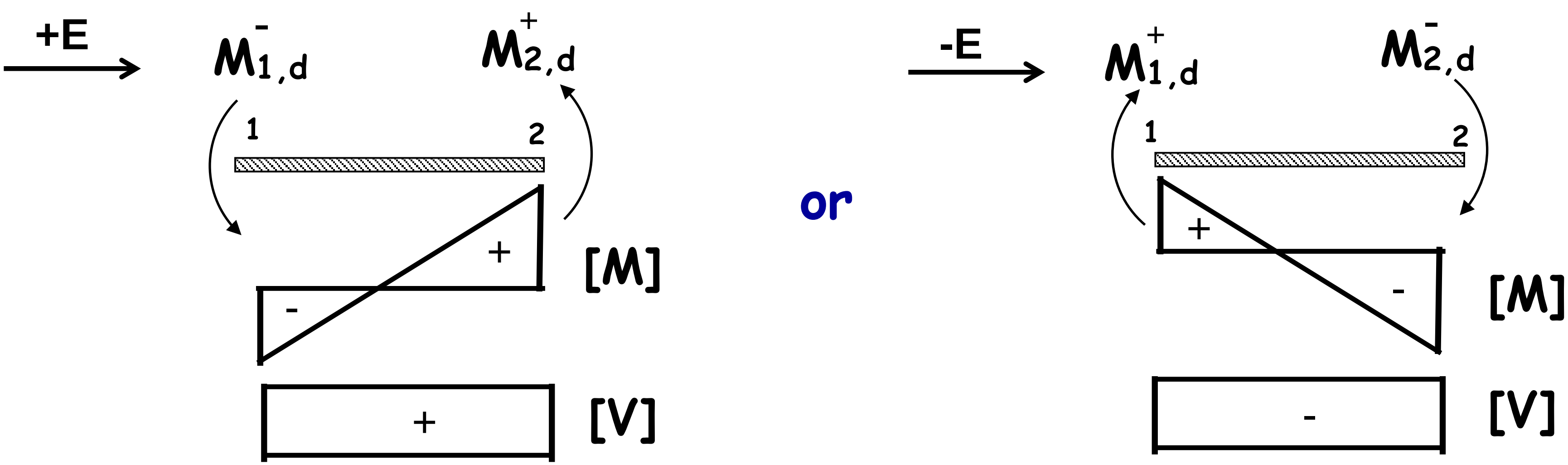
ΣM_{Rb} , ΣM_{Rc} for the corresponding direction of seismic action (+E or -E)

- In DC H $\gamma_{Rd}=1.3$
- In DC M $\gamma_{Rd}=1.1$

Shear Capacity Design of Beams (Eurocode 8)



Determination of V_M



$$V_{M\max,b} = \frac{M^{+}_{2,d} + M^{-}_{1,d}}{\ell_{12}} \qquad V_{M\max,b} = -\frac{M^{+}_{1,d} + M^{-}_{2,d}}{\ell_{12}}$$

- In DC H $\gamma_{Rd}=1.2$
- In DC M $\gamma_{Rd}=1.0$

Shear Capacity Design of Beams (Eurocode 8)

$M^{+}_{1,d}$:

- $\gamma_{Rd} \cdot M_{Rb,1}^{+}$, when $\Sigma M_{Rb} < \Sigma M_{Rc}$, weak beams
- $\gamma_{Rd} \cdot M_{Rb,1}^{+} \cdot (\Sigma M_{Rd,c} \backslash \Sigma M_{Rd,b})$, when $\Sigma M_{Rb} > \Sigma M_{Rc}$, weak columns: (moment developed in the column when beams fail)

Similarly

$M^{-}_{1,d}$:

- $\gamma_{Rd} \cdot M_{Rb,1}^{-}$, when $\Sigma M_{Rb} < \Sigma M_{Rc}$
- $\gamma_{Rd} \cdot M_{Rb,1}^{-} \cdot (\Sigma M_{Rd,c} \backslash \Sigma M_{Rd,b})$

Also similarly for $M_{2,d}^{+}$, $M_{2,d}^{-}$

ΣM_{Rb} , ΣM_{Rc} for the corresponding direction of seismic action (+E or -E)

- In DC H γ_{Rd}=1.3
- In DC M γ_{Rd}=1.1

Local Ductility Conditions

- ☑ **Relation between q and μ_δ**

  $\mu_\delta = q \; if \;\; T_1 \geq T_c \;, \;\; \mu_\delta = 1 + (q-1) T_c / T_1 \;\; if \;\;\; T_1 < T_c;$

- ☑ **Relation between μ_δ and μ_φ**

  $\mu_\delta = 1 + 3(\mu_\phi - 1) L_{pl} / L_s \,(1 - 0.5 \, L_{pl} / L_s);$ where L_{pl}:plastic hinge length, L_s: shear span

- ☑ **Relation of L_{pl} & L_s for typical RC beams, columns & walls**

  $(considering: \;\; \varepsilon^*_{cu} = 0.0035 + 0.1 a \omega_w)$

  $L_{pl} \approx 0.3 L_s \;\; and \;\; for \;\; safety \;\; factor \;\; 2: L_{pl} \approx 0.15 L_s \;\; Then: \; \mu_\phi \approx 2\mu_\delta - 1$

- ☑ For $T_1 \geq T_c$ $\qquad \mu_\phi = 2\mu_\delta - 1 = 2q - 1$

  For $T_1 \leq T_c$ $\qquad \mu_\phi = 2\mu_\delta - 1 = 2[1 + (q-1)\dfrac{T_c}{T_1}] - 1 = 1 + 2(q-1)\dfrac{T_c}{T_1}$

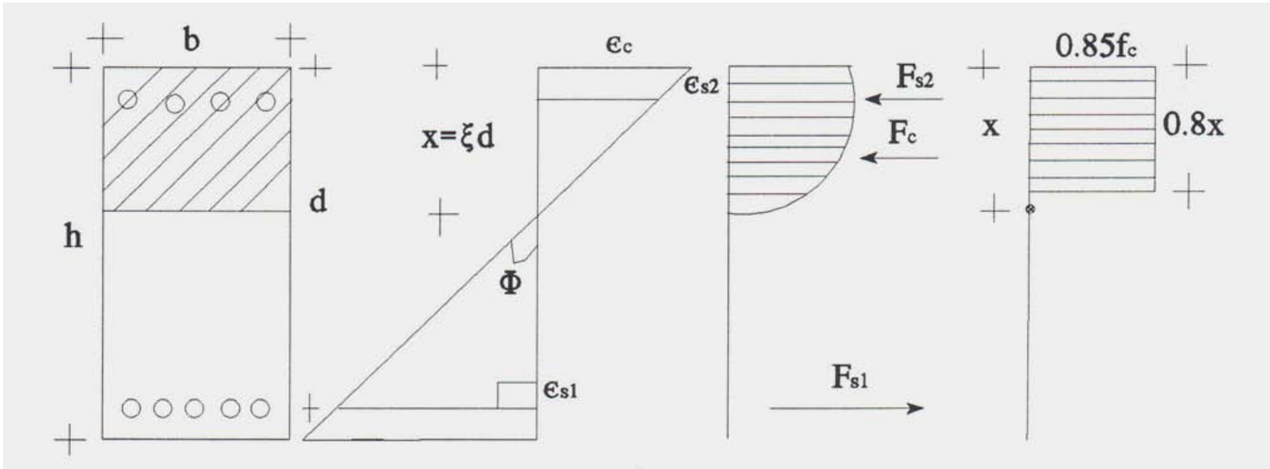
  In EC8 q_o is used instead of q concervatively to include irregular buildings ($q<q_o$) Therefore:

  $$\mu_\phi = 2q_o - 1 \; if \;\; T_1 \geq T_c$$

  $$\mu_\phi = 1 + 2(q_o - 1)\frac{T_c}{T_1} \;\; if \;\; T_1 < T_c$$

- ☑ **Note:** For Steel class B μ_φ demand increases by 50%

Ductility Estimation for Beams



$$F_c = (0.85f_c)(0.80x_u)b = 0.68 f_c x_u b$$

$$F_{s1} = A_{s1} f_y = \rho_1 bdf_y \quad \rho_1 = A_{s1}/bd$$

$$F_{s2} = A_{s2} f_y = \rho_2 bdf_y \quad \rho_2 = A_{s2}/bd$$

$$N = F_{s1} - F_{s2} - F_c = 0$$

$$\rho_1 bdf_y - \rho_2 bdf_y - 0.68 f_c x_u b = 0$$

$$\rightarrow x_u = \frac{(\rho_1 - \rho_2) f_y d}{0.68 f_c}$$

$$\phi = \frac{\epsilon_c}{x} = \frac{\epsilon_{s1}}{d-x} = \frac{\epsilon_c + \epsilon_{s1}}{d}$$

ϕ[redacted] $= \dfrac{\epsilon_{cu}}{x_u} = \dfrac{0.68 f_c}{(\rho_1 - \rho_2) f_y d} \epsilon_{cu}$

ϕ[redacted] $= \dfrac{\epsilon_y}{d - x_y} = \dfrac{f_y / E_s}{d(1 - \xi_y)}$

[redacted] $\phi = \dfrac{0.68 f_c \epsilon_{cu} E_s}{f_y^2 (\rho_1 - \rho_2)} (1 - \xi_y)$

Ductility Estimation for Beams

Ductility increases when:

$\varepsilon_{cu} \uparrow$ $f_c \uparrow$ $\longrightarrow$ confinement

$\rho_2 \uparrow$ $\rho_1 \downarrow$ $\longrightarrow$ Compressive reinforcement neccessary

While for tension reinforcement: the less the best

Ductility Estimation for Columns

$$F_c + F_{s2} - F_{c1} = N$$

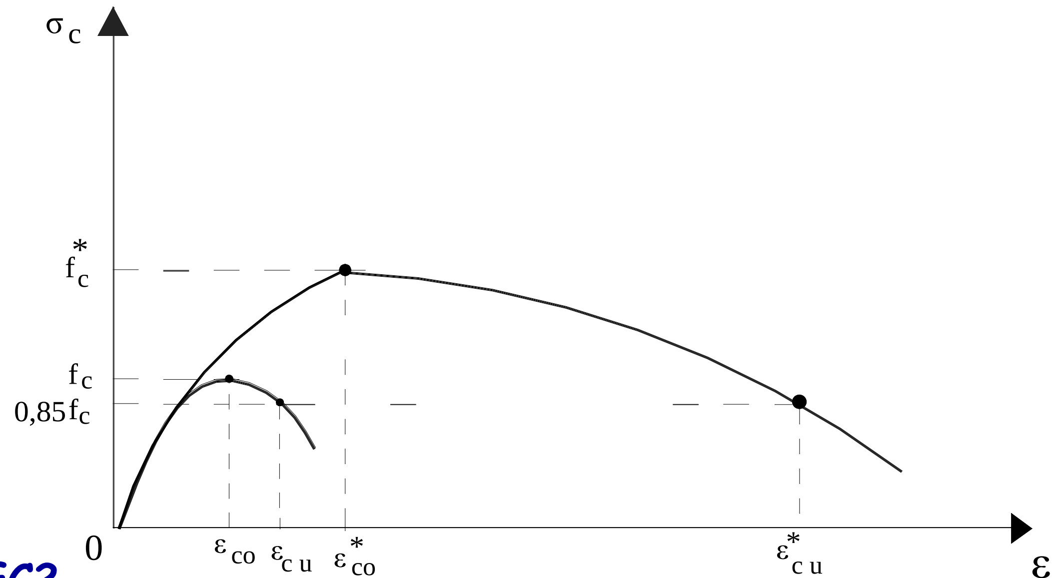
$$v = N / bdf_c$$

$$\mu_\phi = \frac{\phi_u}{\phi_y} = 1.2\frac{E_s}{f_y}[\frac{0.6}{v + (\rho_1 - k\rho_2)(f_y / f_c)} - 1]\varepsilon_{cu}$$

Ductility is reduced when axial load increases

EC8 limits: $v_d \ngtr 0.65 \ for \ DCM \ and \ v_d \ngtr 0.55 \ for \ DCH$

Confined Concrete Model



According to EC2

$$f_c^* = \beta\, f_c \qquad \varepsilon_{co}^* = \beta^2\, \varepsilon_{co}$$

Adopting $\beta \approx 1 + 3,7\left(\frac{p}{f_c}\right)^{0.86}$

(Newman K. & Newnan J.B. 1971)

$$\beta \approx \min\left(1 + 5\frac{p}{f_c},\quad 1.125 + 2.5\frac{p}{f_c}\right)$$

$$\varepsilon_{cu}^* = 0.0035 + 0.2\frac{p}{f_c}$$

When hoops are used $\frac{p}{f_c} \approx 0.5\, a\, \omega_w$

ω_w= Mechanical volumertic ratio of hoops

$$\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}}$$

a=Confinement effectiveness factor, $a = a_s\, a_n$

Therefore

$$\beta = \min\left(1 + 2.5 a\omega_w,\quad 1.125 + 1.25 a\omega_w\right)$$

$$\varepsilon_{cu}^* = 0.0035 + 0.1 a\omega_w$$

Detailing of primary beams for local ductility

EN 1998-1:2004 (E) § 5.4.3.1.2

For Tension Reinforcement

$$\rho_{\min} = 0{,}5\left(\frac{f_{\mathrm{ctm}}}{f_{\mathrm{yk}}}\right)$$

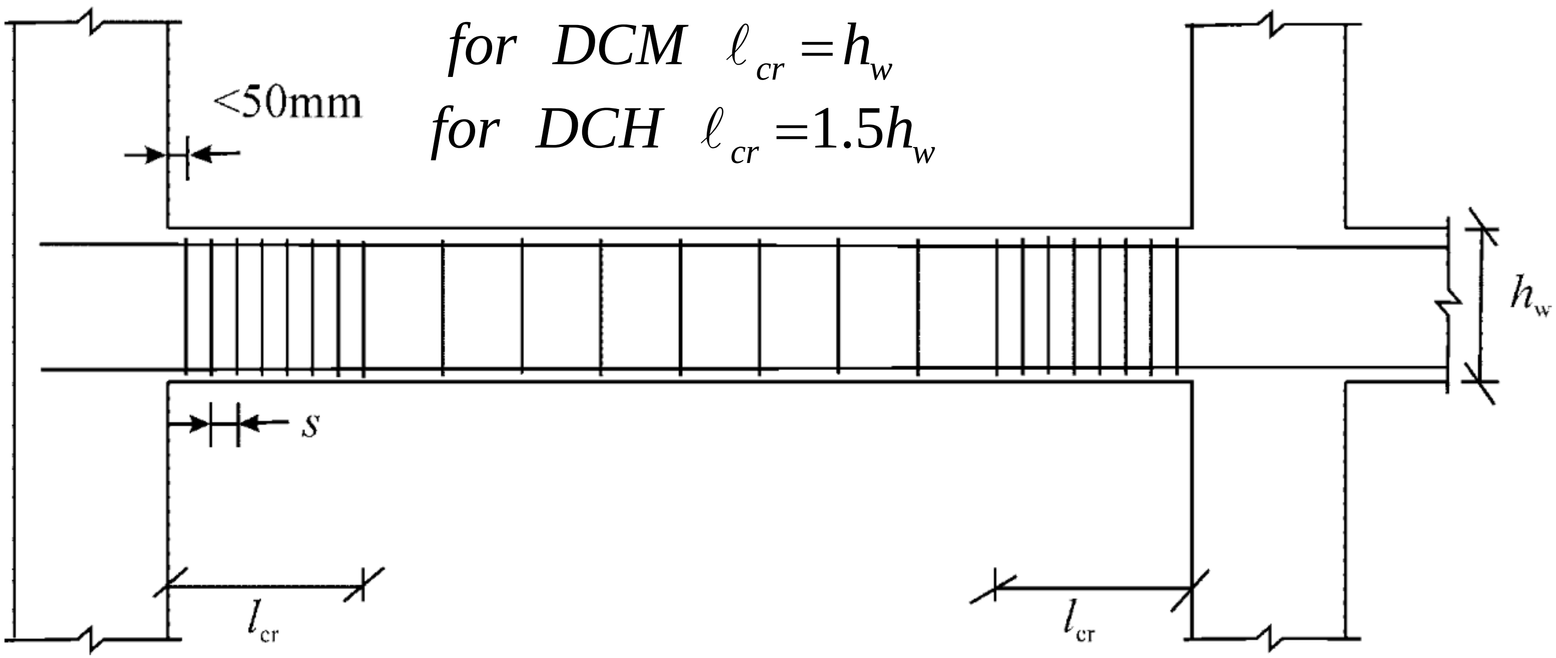
$$\rho_{\max} = \rho' + \frac{0{,}0018}{\mu_{\varphi}\varepsilon_{\mathrm{sy,d}}} \cdot \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd}}}$$

For Comresssion Reinforcement

$$\rho_2 = \rho_2^{req} + 0.5\rho$$

- **More detailing rules for DCH**

Detailing of primary beams for local ductility



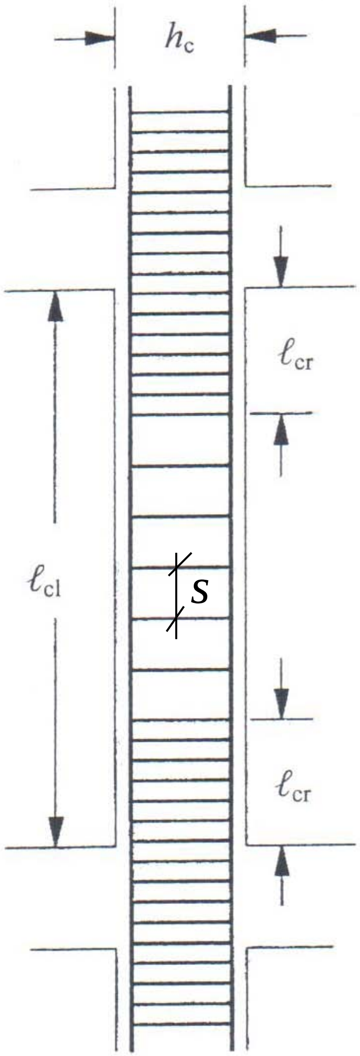
Within ℓ_{cr} transverse reinforcement in critical regions of beams:

☑ $d_{bw} \geq 6mm$

☑ $s \leq$
- $h_w / 4$
- $24 d_{bw}$
- $8 d_{bL}$ (DCM) or $6 d_{bL}$ (DCH)
- $225mm$ (DCM) or $175mm$ (DCH)

Detailing of primary seismic columns for local ductility

EN 1998-1:2004 (E) § 5.4.3.2.2



$$\text{DCM} : \ell_{cr} = \max\begin{Bmatrix} h_c \\ \ell_{cl}/6 \\ 0.45\text{ m} \end{Bmatrix}$$

$$\text{DCH} : \ell_{cr} = \max\begin{Bmatrix} 1.5\,h_c \\ \ell_{cl}/6 \\ 0.60\text{ m} \end{Bmatrix}$$

$$For\ \ell_{cr}/h_c < 3.0 \rightarrow \ell_{cr} = \ell_{cl}$$

Everywhere

$$d_{bw} \geq \begin{cases} 6\,mm \\ \frac{1}{4} d_{bL,\max} \end{cases}$$

- more restrictions for DCH critical regions

In critical regions

for DCM

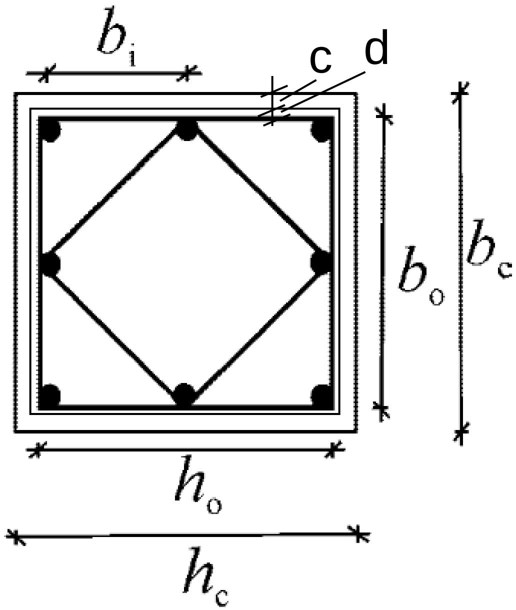
$$s \leq \begin{cases} b_o/2 \\ 8\,d_{bL,\min} \\ 175\,mm \end{cases}$$

for DCH

$$s \leq \begin{cases} b_o/3 \\ 6\,d_{bL,\min} \\ 125\,mm \end{cases}$$

Detailing of primary seismic columns for local ductility

EN 1998-1:2004 (E) § 5.4.3.2.2



Normilised Axial Load

$$v_d \leq 0.65 \quad for \quad DCM$$
$$v_d \leq 0.55 \quad for \quad DCH$$

$$b_o = b_c - 2(c + \frac{d_w}{2})$$
$$b_i \leq 200\,mm \;\; for\ DCM$$
$$b_i \leq 150\,mm \;\; for\ DCH$$

$$\rho_{tot} = \frac{A_{stot}}{bu}$$

$$\min \rho_{tot} = 1\%$$
$$\max \rho_{tot} = 4\%$$

At least 3 bars in every slide

Detailing of primary seismic columns for local ductility for DCM & DCH in critical region at column base

EN 1998-1:2004 (E) § 5.4.3.2.2

$$\alpha\omega_{\mathrm{wd}} \geq 30\mu_{\varphi} v_{\mathrm{d}} \cdot \varepsilon_{\mathrm{sy,d}} \cdot \frac{b_{\mathrm{c}}}{b_{\mathrm{o}}} - 0{,}035$$

$\omega_w \geq 0.08$ *for DCM* $\omega_w \geq 0.12$ *for DCH*

$$\left[\omega_{\mathrm{wd}} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{\mathrm{yd}}}{f_{\mathrm{cd}}}\right]$$

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$,

For rectangular cross-sections:

$$\alpha_{\mathrm{n}} = 1 - \sum_{\mathrm{n}} b_{\mathrm{i}}^2 / 6 b_{\mathrm{o}} h_{\mathrm{o}}$$

$$\alpha_{\mathrm{s}} = (1 - s/2b_{\mathrm{o}})(1 - s/2h_{\mathrm{o}})$$

Beam-Column Joints

- **DCM**

- Horizontal hoops as in critical region of columns

- At least one intermediate column bar at each joint slide

- **DCH**

  Specific rules in § $5.5.33$

Types of Dissipative Walls

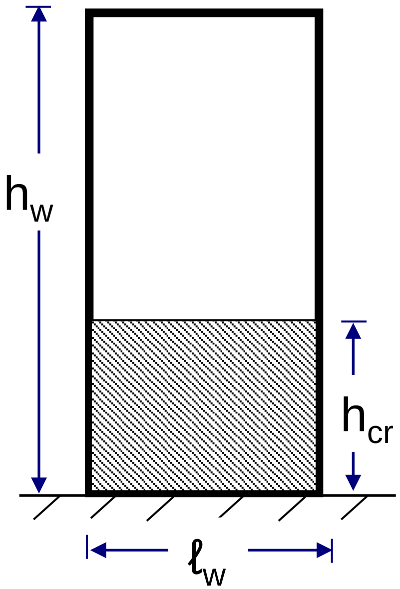
Ductile wall:

- Fixed at base, to prevent rotation there w.r.to rest of structural system.
- Designed & detailed to dissipate energy only in flexural plastic hinge just above the base.

Large lightly-reinforced wall (only for DC M):

- Wall with horizontal dimension $l_w \geq$ 4m, expected to develop limited cracking or inelastic behaviour, but to transform seismic energy to potential energy (uplift of masses) & energy dissipated in the soil by rigid-body rocking, etc.
- Due to its dimensions, or lack-of-fixity at base wall cannot be designed for energy dissipation in plastic hinge at the base.

Ductile Walls



$$h_{cr} = \max \begin{cases} \ell_w \\ h_w / 6 \end{cases}$$

$$h_{cr} \leq \begin{cases} 2\ell_w \\ h_s \quad for \quad n \leq 6\,storeys \\ 2h_s \quad for \quad n \geq 7\,storeys \end{cases}$$

$$h_s = clear \;\; storey \;\; height$$

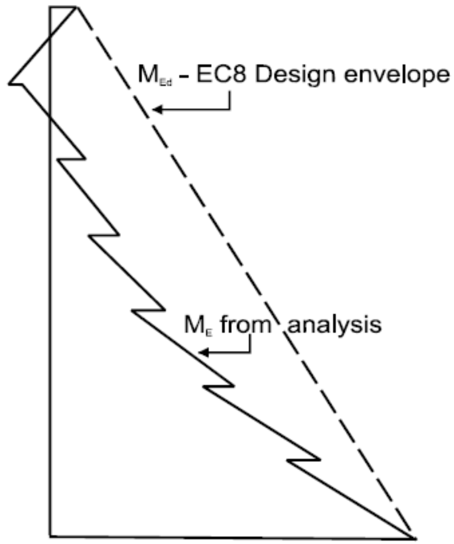
$$\mu_\phi \quad after \;\; q_o' = q_o M_{Ed} / M_{Rd}$$

$$M_{Ed} / M_{Rd} \quad at \;\; the \;\; base$$

Normilised axial load $for \quad DCM \quad v_d \not> 0.40 \;\; and \;\; for \; DCH \;\; v_d \not> 0.35$

No strong column/weak beam capacity design required in wall or wall-equivallent dual systems (<50% of seismic base shear in walls)

But:
design of ductile walls in flexure, to ensure that plastic hinge develops only at the base:



Typical moment diagram in a concrete wall from the analysis & linear envelope for its (over-)design in flexure according Eurocode 8

For Shear

Design in shear for V from analysis, times:

$$1.5 \quad for \quad DCM$$

- $For \quad DCH,$

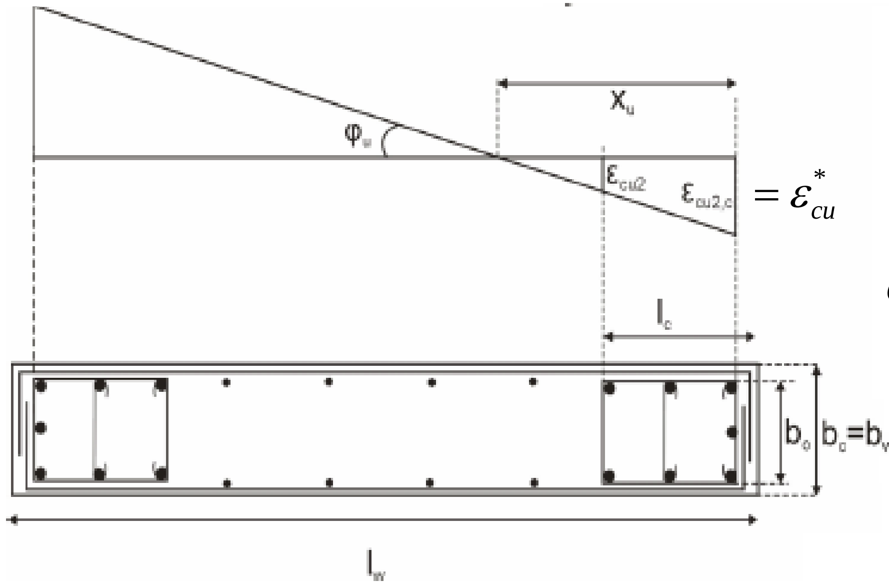
$$V_{Ed} = \varepsilon V_{Ed} \qquad \varepsilon \geq 0.5$$

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0{,}1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q$$

Design and Detailing of Ductile Walls

- Inelastic action limited to plastic hinge at base
- Wall provided with flexural overstrength above plastic hinge region

strain distribution



also $\ell_c \geq (0.15\ell_w,\ 1.5b_c)$

Confined boundary element of free-edge wall end: Longitudinal reinforcement $\rho_{tot} \geq 0.5\%$

Same restrictions as in columns e.g. $\omega_{wd} \geq 0.08\ (DCM)$ $\omega_{wd} \geq 0.12\ (DCH)$

$S_{\max},\ etc$

- In plastic hinge zone: boundary elements w/ confining reinforcement of effective mechanical volumetric ratio:

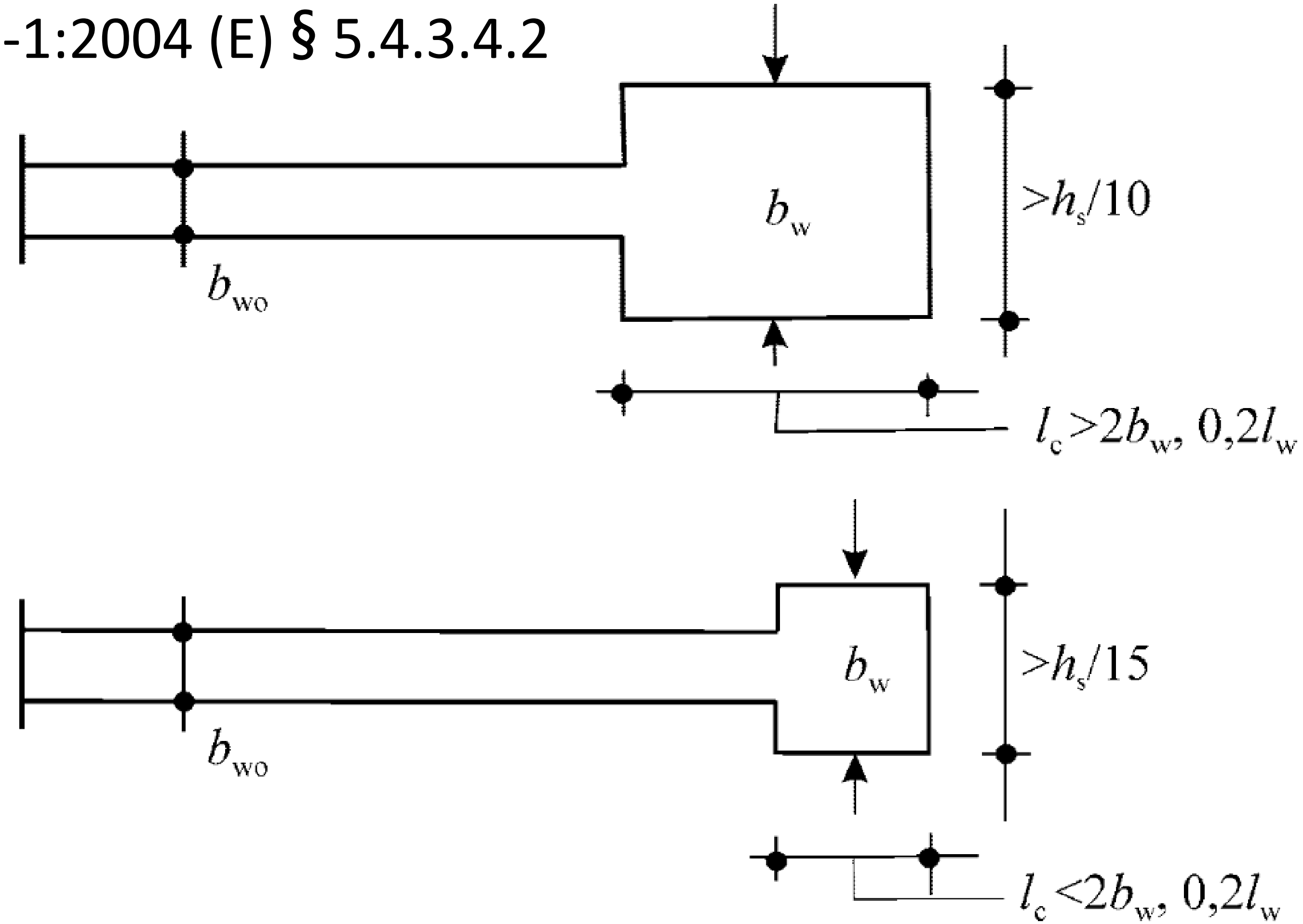
$$\alpha\omega_{wd}=30\mu_{\varphi}(v_d+\omega_v)\varepsilon_{yd}b_c/b_o-0.035$$

over part of compression zone depth: $x_u=(v_d+\omega_v)\varepsilon_{yd}b_c/b_o$ *where* $\omega_v = \rho_v\ f_{yd,v} / f_{cd}$

where strain between: $\varepsilon^*_{cu}=0.0035+0.1\alpha\omega_w$ & $\varepsilon_{cu}=0.0035$

Detailing of Ductile Walls

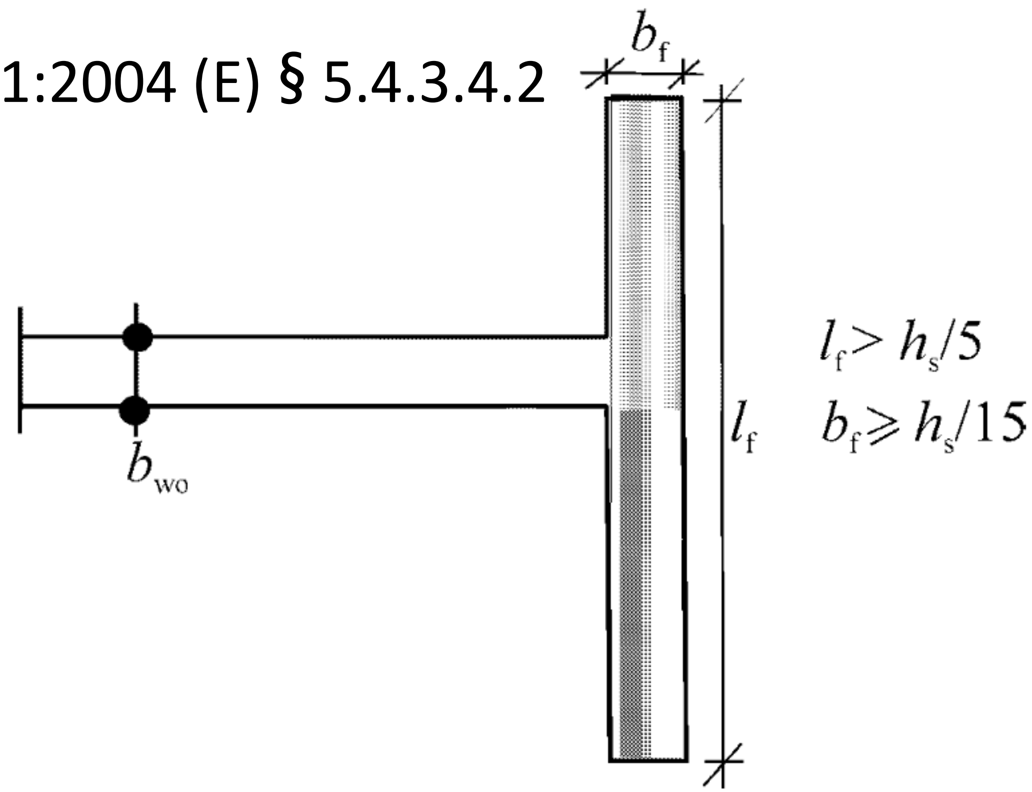
EN 1998-1:2004 (E) § 5.4.3.4.2



Minimum thickness of confined boundary elements

Detailing of Ductile Walls

EN 1998-1:2004 (E) § 5.4.3.4.2



Confined boundary element not needed at wall end with a large transverse flange

Large Lightly Reinforced Walls

- Wall system classified as one of large lightly reinforced walls if, in horizontal direction of interest:
  - at least 2 walls with l_w>4 m, supporting together >20% of gravity load above (: sufficient no. of walls / floor area & significant uplift of masses); if just one wall, q=2
  - fundamental period T_1<0.5 s for fixity at base against rotation (: wall aspect ratio low)
- Systems of large lightly reinforced walls:
  - → only DC M (q=3);
  - → special (less demanding) dimensioning & detailing.
- Rationale: For large walls, minimum reinforcement of ductile walls implies:
  - very high cost;
  - flexural overstrength that cannot be transmitted to ground.

  On the other hand, large lightly reinforced walls:
  - preclude (collapse due to) storey mechanism,
  - minimize nonstructural damage,
  - have shown satisfactory performance in strong EQs.
- If structural system does not qualify as one of large lightly reinforced walls, all its walls designed & detailed as ductile walls.

Design and Detailing of Large Lightly Reinforced Walls

- *Vertical steel* tailored to demands due to M & N from analysis
  - Little excess (minimum) reinforcement, to minimise flexural overstrength.

- Shear verification for V from analysis times (1+q)/2 ~2:
  - If so-amplified shear demand is less than (design) shear resistance w/o shear reinforcement:

    No (minimum) *horizontal reinforcement.* Reason:

    - Inclined cracking prevented (horizontal cracking & yielding due to flexure mainly at construction joints);
    - If inclined cracking occurs, crack width limited by deformation-controlled nature of response (vs. force-controlled non-seismic actions covered in EC2), even w/o min horizontal steel.

Design and Detailing of Large Lightly Reinforced Walls

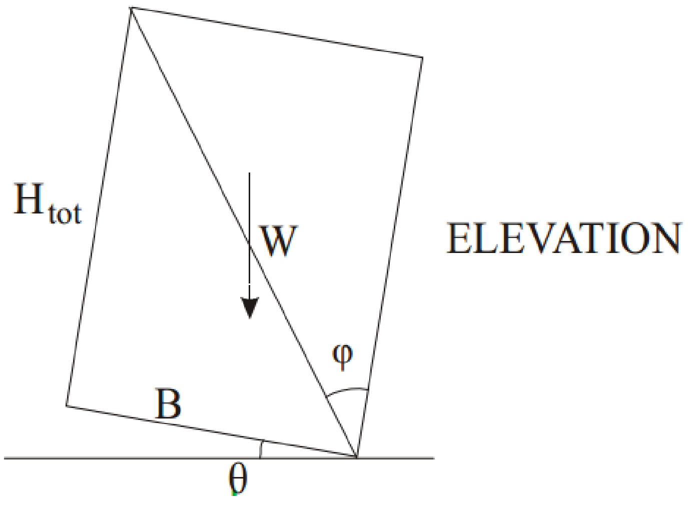
Foundation Problem

- Large l_w →

  Large moment at the base and very low normalized axial force

- Usual way of footing with tie-beams is insufficient

- Impossible to form plastic hinge at the wall base. Wall will uplift & rock as a rigid body



Large Lightly Reinforced Walls

Boundary elements

$$\ell_c \geq \max \begin{cases} b_w \\ 3 b_w \sigma_{cm} / f_{cd} \end{cases}$$

σ_{cm} = mean value of concrete compressive stress

Longitudinal Reinforcement of Boundary Elements

(a) Diameter of vertical bars (EC8- §5.4.3.5.3 (2))

lower storeys wher $\Delta\ell_w \leq h_{storey/3}$: $d_{bL} \geq 12mm$

higher storeys: $d_{bL} \geq 10mm$

(b) Stirrups (EC8- §5.4.3.5.3 (1))

In all storeys-closed stirrups

$$d_{bw} \geq \max(6mm, d_{bL}/3)$$

$$s_w \leq \min(100mm, 8d_{bL})$$

No other particular regulations for LLRCW

Secondary Seismic Members

- A limited number of structural members may be designated as secondary seismic members. The strength and stiffness of these elements against seismic actions shall be neglected.
- The total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members.
- Such elements shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation. Maximum deformations shall account for P-Δ.
- In more detail § 4.2.2., 5.2.3.6, 5.7

Specific Provisions in EC8 for:

- LOCAL EFFECTS to masonry infills see § 5.9
- CONCRETE DIAPHRAGMS see § 5.10
- PRECAST CONCRETE STRUCTURES see § 5.11

Thank you for your attention

http://www.episkeves.civil.upatras.gr

APPENDIX: Detailing & Dimensioning of seismic elements (Synopsis by M. Fardis)

Detailing & dimensioning of primary seismic beams (secondary as in DCL)

	DCH	DCM	DCL
critical region length	$1.5h_w$		h_w
Longitudinal bars (L):			
ρ_{min}, tension side	$0.5f_{ctm}/f_{yk}$		$0.26f_{ctm}/f_{yk}$, 0.13%(0)
ρ_{max}, critical regions(1)	$\rho'+0.0018f_{cd}/(\mu_\phi\varepsilon_{sy,d}f_{yd})$(1)		0.04
$A_{s,min}$, top & bottom	$2\Phi14$ ($308mm^2$)	-	
$A_{s,min}$, top-span	$A_{s,top\text{-}supports}/4$	-	
$A_{s,min}$, critical regions bottom	$0.5A_{s,top}$(2)		-
$A_{s,min}$, supports bottom	$A_{s,bottom\text{-}span}/4$(0)		
d_{bL}/h_c - bar crossing interior joint(3)			-
d_{bL}/h_c - bar anchored at exterior joint(3)			-
Transverse bars (w):			
(i) outside critical regions			
spacing $s_w\leq$	0.75d		
$\rho_w\geq$	$0.08(f_{ck}(MPa))^{1/2}/f_{yk}(MPa)$(0)		
(ii) in critical regions:			
$d_{bw}\geq$	6mm		
spacing $s_w\leq$	$6d_{bL}$, , $24d_{bw}$, 175mm	$8d_{bL}$, , $24d_{bw}$, 225mm	-
Shear design:			
V_{Ed}, seismic(4)	(4)	(4)	From the analysis for the "seismic design situation"
$V_{Rd,max}$ seismic (5)	As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)b_{wo}zf_{cd}\sin2\theta$ (5), with $1\leq\cot\theta\leq2.5$		
$V_{Rd,s}$, outside critical regions(5)	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$ (5), with $1\leq\cot\theta\leq2.5$		
$V_{Rd,s}$, critical regions(5)	$V_{Rd,s}=b_wz\rho_wf_{ywd}$ ($\theta=45°$)	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$, with $1\leq\cot\theta\leq2.5$	
If $\zeta\equiv V_{Emin}/V_{Emax}$(6) <-0.5: inclined bars at angle $\pm\alpha$ to beam axis, with cross-section A_s/direction	If $V_{Emax}/(2+\zeta)f_{ctd}b_wd>1$: $A_s=0.5V_{Emax}/f_{yd}\sin\alpha$ & stirrups for $0.5V_{Emax}$	-	

Footnotes - Table on detailing & dimensioning primary seismic beams (previous page)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) μ_ϕ is the value of the curvature ductility factor that corresponds to the basic value, q_o, of the behaviour factor used in the design

(2) The minimum area of bottom steel, $A_{s,min}$, is in addition to any compression steel that may be needed for the verification of the end section for the ULS in bending under the (absolutely) maximum negative (hogging) moment from the analysis for the "seismic design situation", M_{Ed}.

(3) h_c is the column depth in the direction of the bar, $\nu_d = N_{Ed}/A_c f_{cd}$ is the column axial load ratio, for the algebraically minimum value of the axial load in the "seismic design situation", with compression taken as positive.

(4) At a member end where the moment capacities around the joint satisfy: $\sum M_{Rb} > \sum M_{Rc}$, M_{Rb} is replaced in the calculation of the design shear force, V_{Ed}, by $M_{Rb}(\sum M_{Rc}/\sum M_{Rb})$

(5) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d-d_1$.

(6) V_{Emax}, $V_{E,min}$ are the algebraically maximum and minimum values of V_{Ed} resulting from the ± sign; V_{Emax} is the absolutely largest of the two values, and is taken positive in the calculation of ζ; the sign of V_{Emin} is determined according to whether it is the same as that of V_{Emax} or not.

EUROCODES
Background and Applications

Detailing & dimensioning of primary seismic columns (secondary as in DCL)

	DCH	DCM	DCL
Cross-section sides, h_c, $b_c \geq$	0.25m; $h_v/10$ if $\theta=P\delta/Vh>0.1$ (1)	-	
"critical region" length (1) $\geq$	$1.5\max(h_c,b_c)$, 0.6m, $l_c/5$	$\max(h_c,b_c)$, 0.6m, $l_c/5$	-
Longitudinal bars (L):			
ρ_{min}	1%		$0.1N_d/A_cf_{yd}$, 0.2% (0)
ρ_{max}	4%		4% (0)
$d_{bL} \geq$	8mm		
bars per side $\geq$	3		2
Spacing between restrained bars	$\leq$150mm	$\leq$200mm	-
distance of unrestrained to nearest restrained bar	$\leq$150mm		
Transverse bars (w):			
Outside critical regions:			
$d_{bw} \geq$	6mm, $d_{bL}/4$		
Spacing $s_w \leq$	$20d_{bL}$, $\min(h_c, b_c)$, 400mmm		
s_w in splices $\leq$	$12d_{bL}$, $0.6\min(h_c, b_c)$, 240mm		
Within critical regions: (2)			
$d_{bw} \geq$ (3)	6mm, $0.4(f_{yd}/f_{ywd})^{1/2}d_{bL}$	6mm, $d_{bL}/4$	
$s_w \leq$ (3),(4)	$6d_{bL}$, $b_o/3$, 125mm	$8d_{bL}$, $b_o/2$, 175mm	-
$\omega_{wd} \geq$ (5)	0.08	-	
$\alpha\omega_{wd} \geq$ (4),(5),(6),(7)	$30\mu_\phi{}^*\nu_d\varepsilon_{sy,d}b_c/b_o-0.035$	-	
In critical region at column base:			
$\omega_{wd} \geq$	0.12	0.08	-
$\alpha\omega_{wd} \geq$ (4),(5),(6),(8),(9)	$30\mu_\phi\nu_d\varepsilon_{sy,d}b_c/b_o-0.035$		-
Capacity design check at beam-column joints: (10)	$1.3\Sigma M_{Rb} \leq \Sigma M_{Rc}$ No moment in transverse direction of column		-
Verification for M_x-M_y-N:	Truly biaxial, or uniaxial with $(M_z/0.7, N)$, $(M_y/0.7, N)$		
Axial load ratio $\nu_d=N_{Ed}/A_cf_{cd}$	≤ 0.55	≤ 0.65	-
Shear design:			
V_{Ed} seismic (11)	(11)	(11)	From the analysis for the "seismic design situation"
$V_{Rd,max}$ seismic (12), (13)	As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)\min[1.25; (1+\nu_d); 2.5(1-\nu_d)]b_{wo}zf_{cd}\sin2\theta$, with $1\leq\cot\theta\leq2.5$		
$V_{Rd,s}$ seismic (12), (13), (14)	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta+N_{Ed}(h-x)/l_{cl}$ (13) with $1\leq\cot\theta\leq2.5$		

Footnotes - Table on detailing & dimensioning primary seismic columns (previous page)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) h_v is the distance of the inflection point to the column end further away, for bending within a plane parallel to the side of interest; l_c is the column clear length.

(2) For DCM: If a value of q not greater than 2 is used for the design, the transverse reinforcement in critical regions of columns with axial load ratio ν_d not greater than 0.2 may just follow the rules applying to DCL columns.

(3) For DCH: In the two lower storeys of the building, the requirements on d_{bw}, s_w apply over a distance from the end section not less than 1.5 times the critical region length.

(4) Index c denotes the full concrete section and index o the confined core to the centreline of the hoops; b_o is the smaller side of this core.

(5) ω_{wd} is the ratio of the volume of confining hoops to that of the confined core to the centreline of the hoops, times f_{yd}/f_{cd}.

(6) α is the "confinement effectiveness" factor, computed as $\alpha = \alpha_s\alpha_n$; where: $\alpha_s = (1\text{-}s/2b_o)(1\text{-}s/2h_o)$ for hoops and $\alpha_s = (1\text{-}s/2b_o)$ for spirals; $\alpha_n = 1$ for circular hoops and $\alpha_n = 1-\{b_o/[(n_h-1)h_o]+h_o/[(n_b-1)b_o]\}/3$ for rectangular hoops with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the one with length h_o.

(7) For DCH: at column ends protected from plastic hinging through the capacity design check at beam-column joints, μ_ϕ^* is the value of the curvature ductility factor that corresponds to 2/3 of the basic value, q_o, of the behaviour factor used in the design; at the ends of columns where plastic hinging is not prevented because of the exemptions listed in Note (10) below, μ_ϕ^* is taken equal to μ_ϕ defined in Note (1) of the Table for the beams (see also Note (9) below); $\varepsilon_{sy,d} = f_{yd}/E_s$.

(8) Note (1) of the Table for the beams applies.

(9) For DCH: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the exceptions listed in Note (10) below.

(10) The capacity design check does not need to be fulfilled at beam-column joints: (a) of the top floor, (b) of the ground storey in two-storey buildings with axial load ratio ν_d not greater than 0.3 in all columns, (c) if shear walls resist at least 50% of the base shear parallel to the plane of the frame (wall buildings or wall-equivalent dual buildings), and (d) in one-out-of-four columns of plane frames with columns of similar size.

(11) At a member end where the moment capacities around the joint satisfy: $\Sigma M_{Rb} < \Sigma M_{Rc}$, M_{Rc} is replaced by $M_{Rc}(\Sigma M_{Rb}/\Sigma M_{Rc})$.

(12) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d\text{-}d_1$.

(13) The axial load, N_{Ed}, and its normalized value, ν_d, are taken with their most unfavourable value in the seismic design situation for the shear verification (considering both the demand, V_{Ed}, and the capacity, V_{Rd}).

(14) x is the compression zone depth at the end section in the ULS of bending with axial load.

Detailing & dimensioning of ductile walls (cont'd next page)

	DCH	DCM	DCL
Web thickness, $b_{wo} \geq$	max(150mm, $h_{storey}/20$)		-
critical region length, $h_{cr} \geq$	$\geq$ max(l_w, $H_w/6$) (1) $\leq$ min($2l_w$, h_{storey}) if wall $\leq$6 storeys $\leq$ min($2l_w$, 2 h_{storey}) if wall > 6 storeys		-
Boundary elements:			
a) in critical region:			
- length l_c from edge $\geq$	$0.15l_w$, $1.5b_w$, length over which ε_c> 0.0035		where ρ_L>2%
- thickness b_w over l_c $\geq$	200mm, $h_{st}/15$, if $l_c \leq$max($2b_w$, $l_w/5$), 200mm, $h_{st}/10$, if l_c>max($2b_w$, $l_w/5$)		-
- vertical reinforcement:			
ρ_{min} over $A_c=l_cb_w$	0.5%		$0.2\%^{(0)}$
ρ_{max} over A_c	$4\%^{(0)}$		
- confining hoops (w) (2):			
$d_{bw} \geq$	8mm	if ρ_L over $A_c=l_cb_w$ >2%:apply DCL rule for ρ_L>2%	6mm, $d_{bL}/4$
spacing $s_w \leq^{(3)}$	min($25d_{bh}$, 250mm)		min($20d_{bL}$, b_{wo} 400mm)$^{(0)}$
$\omega_{wd} \geq^{(2)}$	0.12	0.08	-
$\alpha\omega_{wd} \geq^{(3),(4)}$	$30\mu_\phi(\nu_d+\omega_v)\varepsilon_{sy,d}b_w/b_o$-0.035		-
b) storey above critical region	as is critical region, but with required $\alpha\omega_{wd}$, ω_{wd} reduced by 50%	$\rho_v \geq$0.5% wherever ε_c>0.2%; elsewhere $\rho_v \geq$0.2%	
c) over the rest of the wall:	No boundary elements. $\rho_v \geq$0.5% wherever ε_c>0.2%; elsewhere $\rho_v \geq$0.2%		-
Web:			
- vertical bars (v):			
$\rho_{v,min}$	0.2%	$0.2\%^{(0)}$	
$\rho_{v,max}$	4%		
$d_{bv} \geq$	8mm	-	
$d_{bv} \leq$	$b_{wo}/8$	-	
spacing $s_v \leq$	min($25d_{bv}$, 250mm)	Min($3b_{wo}$, 400mm)	
- horizontal bars:			
ρ_{hmin}	0.2%	max(0.1%, $0.25\rho_v$)$^{(0)}$	
$d_{bh} \geq$	8mm	-	
$d_{bh} \leq$	$b_{wo}/8$	-	
spacing $s_h \leq$	min($25d_{bh}$, 250mm)	400mm	
axial load ratio $\nu_d = N_{Ed}/A_cf_{cd}$	$\leq$0.35	$\leq$0.4	-
Design moments M_{Ed}:	If $H_w/l_w \geq$2, design moments from linear envelope of maximum moments M_{Ed} from analysis for the "seismic design situation", shifted up by the "tension shift" a_l		From analysis for "seismic design situation"

Detailing & dimensioning of ductile walls (cont'd from previous page)

	DCH		DCM	DCL
Shear design:				
Multiplicative factor ε on the shear force V'_{Ed} from the analysis for "seismic design situation":	if $H_w/l_w \le 2$ (5): if $H_w/l_w > 2$ (5), (6):	$\varepsilon = 1.2M_{Rdo}/M_{Edo} \le q$	$\varepsilon = 1.5$	$\varepsilon = 1.0$
Design shear force in walls of dual systems with $H_w/l_w > 2$, for z between $H_w/3$ and H_w: (7)				From analysis for "seismic design situation"
$V_{Rd,max}$ outside critical region	As in EC2: $V_{Rd,max} = 0.3(1-f_{ck}(MPa)/250)b_{wo}(0.8l_w)f_{cd}\sin 2\theta$, with $1 \le \cot\theta \le 2.5$			
$V_{Rd,max}$ in critical region	40% of EC2 value		As in EC2	
$V_{Rd,s}$ outside critical region	As in EC2: $V_{Rd,s} = b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$			
$V_{Rd,s}$ in critical region; web reinforcement ratios. ρ_h, ρ_v				
(i) if $\alpha_s = M_{Ed}/V_{Ed}l_w \ge 2$: $\rho_v = \rho_{v,min}$, ρ_h from $V_{Rd,s}$:	As in EC2: $V_{Rd,s} = b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$			
(ii) if $\alpha_s < 2$: ρ_h from $V_{Rd,s}$: (8)	$V_{Rd,s} = V_{Rd,c} + b_{wo}\alpha_s(0.75l_w)\rho_h f_{yhd}$		As in EC2: $V_{Rd,s} = b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$	
ρ_v from: (9)	$\rho_v f_{yvd} \ge \rho_h f_{yhd} - N_{Ed}/(0.8l_w b_{wo})$			
Resistance to sliding shear: via bars with total area A_{si} at angle $\pm\phi$ to the horizontal (10)	$V_{Rd,s} = A_{si}f_{yd}\cos\phi + A_{sv}\min(0.25f_{yd}, 1.3(f_{yd}f_{cd})^{1/2}) + 0.3(1-f_{ck}(MPa)/250)b_{wo}xf_{cd}$			
$\rho_{v,min}$ at construction joints (9),(11)			-	

Footnotes - Table on detailing & dimensioning ductile walls (previous pages)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) l_w is the long side of the rectangular wall section or rectangular part thereof; H_w is the total height of the wall; h_{storey} is the storey height.

(2) For DC M: If for the maximum value of axial force in the wall from the analysis for the "seismic design situation" the wall axial load ratio $\nu_d = N_{Ed}/A_c f_{cd}$ satisfies $\nu_d \leq 0.15$, the DCL rules may be applied for the confining reinforcement of boundary elements; the waiver applies also if this value of the wall axial load ratio is $\nu_d \leq 0.2$ but the value of q used in the design of the building is not greater than 85% of the q-value allowed when the DC M confining reinforcement is used in boundary elements.

(3) Notes (4), (5), (6) of the Table for columns apply for the confined core of boundary elements.

(4) μ_ϕ is the value of the curvature ductility factor that corresponds to the product of the basic value q_o of the behaviour factor times the value of the ratio M_{Edo}/M_{Rdo} at the base of the wall (see Note (5)); $\varepsilon_{sy,d} = f_{yd}/E_s$, ω_{vd} is the mechanical ratio of the vertical web reinforcement.

(5) M_{Edo} is the moment at the wall base from the analysis for the "seismic design situation"; M_{Rdo} is the design value of the flexural capacity at the wall base for the axial force N_{Ed} from the analysis for the same "seismic design situation".

(6) $S_e(T_1)$ is the value of the elastic spectral acceleration at the period of the fundamental mode in the horizontal direction (closest to that) of the wall shear force multiplied by ε; $S_e(T_c)$ is the spectral acceleration at the corner period T_C of the elastic spectrum.

(7) A dual structural system is one in which walls resist between 35 and 65% of the seismic base shear in the direction of the wall shear force considered; z is distance from the base of wall.

(8) For b_w and d in m, f_{ck} in MPa, ρ_L denoting the tensile reinforcement ratio, N_{Ed} in kN, $V_{Rd,c}$ (in kN) is given by:

N_{Ed} is positive for compression and its minimum value from the analysis for the "seismic design situation" is used; if the minimum value is negative (tension), $V_{Rd,c}=0$.

(9) The minimum value of the axial force from the analysis for the "seismic design situation" is used as N_{Ed} (positive for compression).

(10) A_{sv} is the total area of web vertical bars and of any additional vertical bars placed in boundary elements against shear sliding; x is the depth of the compression zone.

(11) $f_{ctd}=f_{ctk,0.05}/\gamma_c$ is the design value of the (5%-fractile of) tensile strength of concrete.