Design of buildings for earthquake resistance, according to Eurocode 8-Part 1

(concrete & masonry buildings)

STRUCTURE OF EN 1998-1:2004

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2 Performance Requirements and Compliance Criteria
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Fundamental features of good structural layout

- Clear structural system.
- Simplicity & uniformity in geometry of structural system.
- Symmetry & regularity in plan.
- Significant torsional stiffness about vertical axis.
- Geometry, mass & lateral stiffness: regular in elevation.
- Redundancy of structural system.
- Effective horizontal connection of vertical elements at all floor levels.

Clear structural system

- System of:
  - plane frames continuous in plan, from one side of the plan to the opposite, w/o offsets or interruption in plan, or indirect supports of beams,

  and/or

  - (essentially) rectangular shear walls,

arranged in two orthogonal horizontal directions.

Symmetry - regularity in plan

- Lateral stiffness & mass ~symmetric w.r.to two orthogonal horizontal axes (full symmetry → response to translational horizontal components of seismic action will not include any torsion w.r.to the vertical axis).
- Lack of symmetry in plan often measured via "static eccentricity", *e*, between:
  - centre of mass of storey (centroid of overlying masses, CM) and
  - centre of stiffness (CS, important during the elastic response).

- One of Eurocode 8 criteria for regularity in plan: $e_x \leq 0.3\, r_x; \qquad e_y \leq 0.3\, r_y$
  - "torsional radius" r_x (r_y) = √ratio of:
    - torsional stiffness of storey w.r.to CS, to
    - storey lateral stiffness in *y* (*x*) direction, orthogonal to *x* (*y*).
- CS, CR & r_x, r_y: unique & independent of lateral loading only in single-storey buildings:

$$x_{CS} = \frac{\sum(xEI_y)}{\sum(EI_y)}; \qquad y_{CS} = \frac{\sum(yEI_x)}{\sum(EI_x)} \qquad r_x = \sqrt{\frac{\sum(x^2 EI_y + y^2 EI_x)}{\sum(EI_y)}}; \qquad r_y = \sqrt{\frac{\sum(x^2 EI_y + y^2 EI_x)}{\sum(EI_x)}}$$

- Another Eurocode 8 criterion for regularity in plan: compact outline in plan, enveloped by convex polygonal line. Re-entrant corners in plan don't leave area up to convex polygonal envelope >5% of area inside outline.
- T-, U-, H-, L-shaped etc. plan: floors may not behave as rigid diaphragms, but deform in horizontal plane (increased uncertainty of response).

Symmetry - regularity in plan (cont'd)

Torsional response → difference in seismic displacements between opposite sides in plan; larger local deformation demands on side experiencing the larger displacement ("flexible side").

Collapse of building due to its torsional response about a stiff shaft at the corner (Athens, 1999 earthquake).

High torsional stiffness w.r.to vertical axis

- (~)Purely torsional natural mode w.r.to vertical axis w/ T > T of lowest (~)purely translational natural mode →

  accidental torsional vibrations w.r.to vertical axis by transfer of vibration energy from the response in the lowest translational mode to the torsional one → significant & unpredictable horizontal displacements at the perimeter.

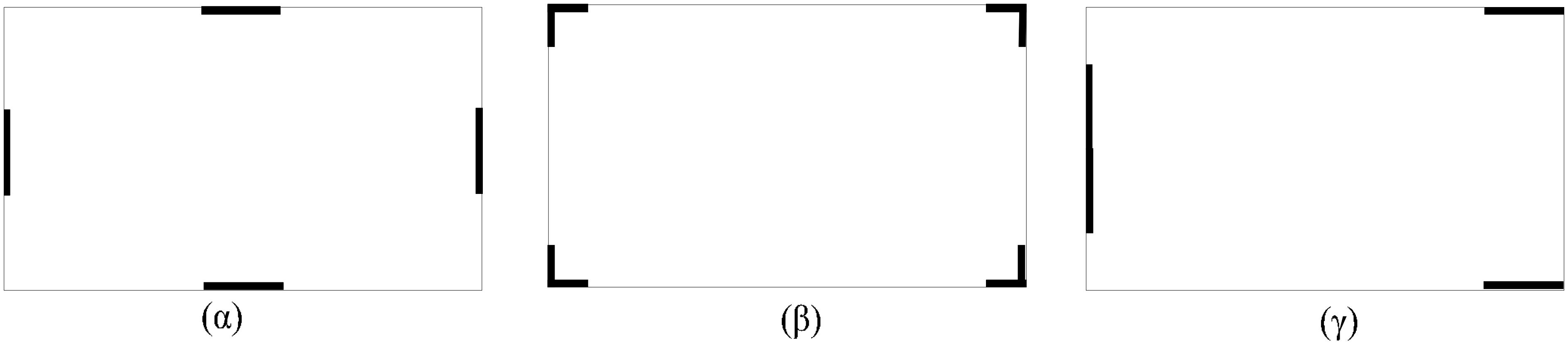
- Avoided through Eurocode 8 criterion for regularity in plan:
  - "torsional radii" r_x (better r_{mx}: $r_{mx} = \sqrt{r_x^2 + e_x^2}$) & r_y (r_{my}: $r_{my} = \sqrt{r_y^2 + e_y^2}$) >
  - radius of gyration of floor mass in plan l_s = √ ratio of:
    - polar moment of inertia in plan of total mass of floors above w.r.to floor CM, to
    - total mass of floors above

    For rectangular floor area: $l_s = \sqrt{(l^2 + b^2)/12}$

$$r_x \geq l_s; \qquad r_y \geq l_s$$

High torsional stiffness w.r.to vertical axis (cont'd)

Means of providing torsional stiffness about a vertical axis: Shear walls or strong frames at the perimeter



Arrangements of shear walls in plan:

(a) preferable;

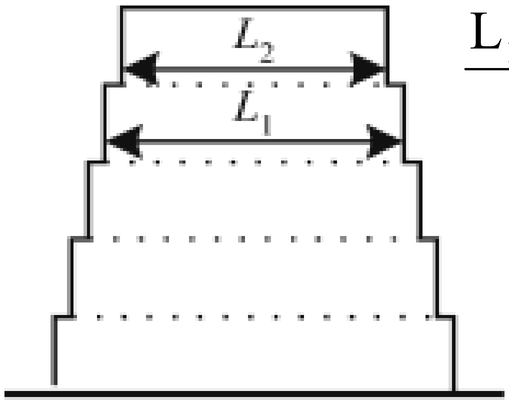
(b) drawbacks due to restraint of floors & difficulties of foundation at the corners;

(c) sensitive to failure of individual walls

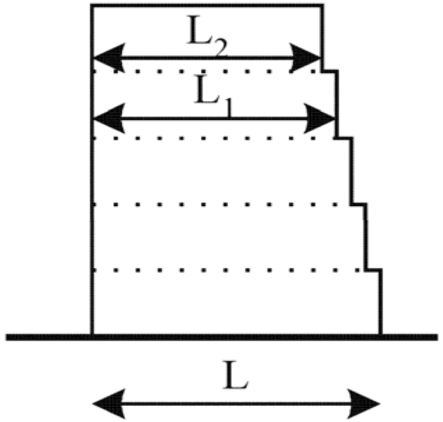
Geometry, mass, stiffness: regular in elevation

Collapse of upper or intermediate storeys w/ reduced plan dimensions or stiffness
Top left: Kalamata (GR) 1986; top right: Kocaeli (TR) 1999.
Bottom: Kobe (JP) 1995.

Geometry, mass & lateral stiffness: regular in elevation (cont'd)

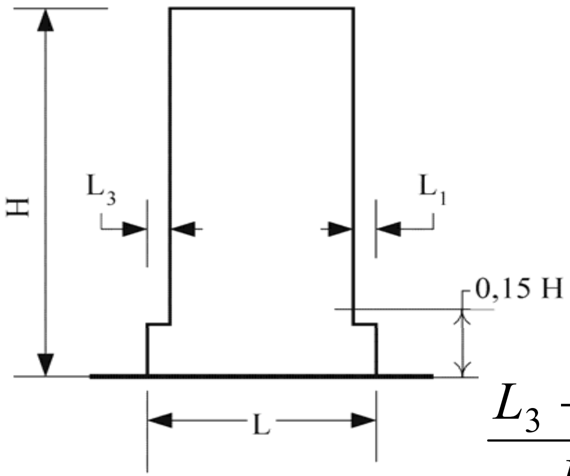


$$\frac{L_1 - L_2}{L_1} \leq 0{,}20$$

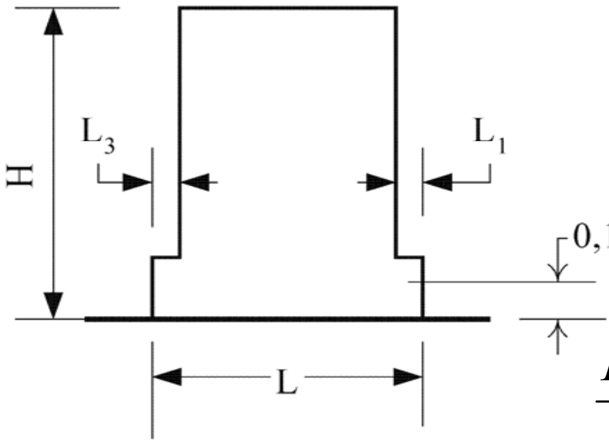


$$\frac{L - L_2}{L} \leq 0{,}30$$

$$\frac{L_1 - L_2}{L_1} \leq 0{,}10$$



$$\frac{L_3 + L_1}{L} \leq 0{,}50$$

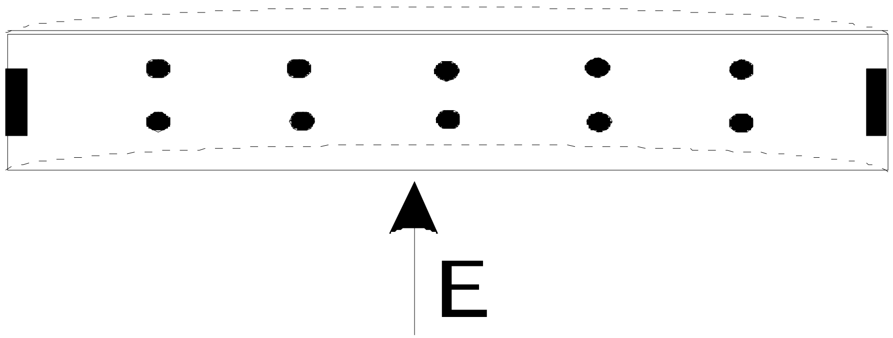


$$\frac{L_3 + L_1}{L} \leq 0{,}20$$

Eurocode 8 criteria for regularity in elevation in buildings w/ setbacks

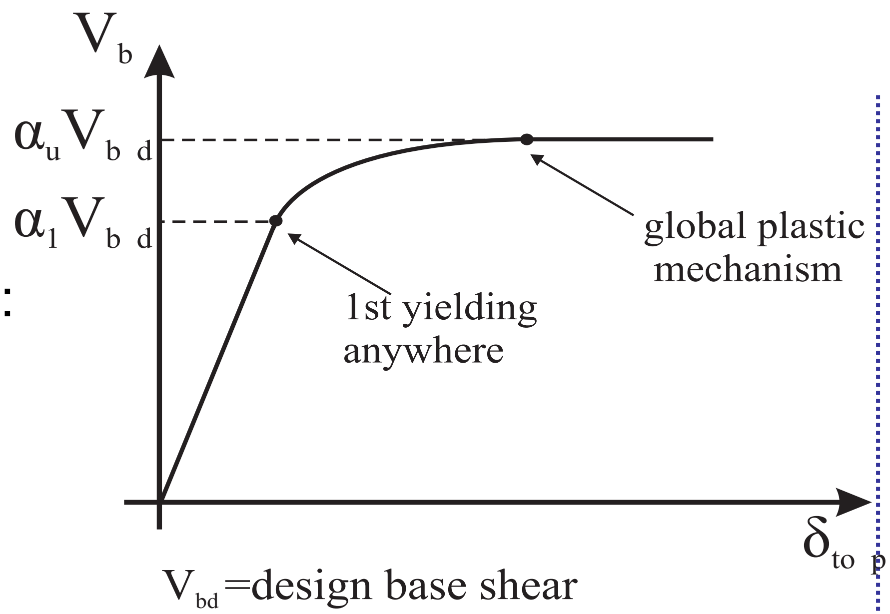
Redundancy of structural system

- Provide large number of lateral-load resisting elements & alternative paths for earthquake resistance.
- Avoid systems w/ few large walls per horizontal direction, especially in buildings long in plan:



In-plane bending of long floor diaphragms in building with two strong walls at the 2 ends → intermediate columns overloaded, compared to results of design w/ rigid diaphragm

Eurocode 8: Bonus to system redundancy:
q_o proportional to α_u/α_1 :



Continuity of floor diaphragms

- Need smooth/continuous path of forces, from the masses where they are generated due to inertia, to the foundation.
- Cast-in-situ reinforced concrete is the ideal structural material for earthquake resistant construction, compared to prefabricated elements joined together at the site: the joints between such elements are points of discontinuity.
- Floor diaphragms should have sufficient strength to transfer the inertia forces to the lateral-load-resisting system & be adequately connected to it.
- Large openings in floor slabs, due to internal patios, wide shafts or stairways, etc. may disrupt continuity of force path, especially if such openings are next to large shear walls near or at the perimeter.
- Vertical elements of lateral-force resisting system should be connected together, via combination of floor diaphragms & beams:
  – at all horizontal levels where significant masses are concentrated, and
  – at foundation level.

Continuity of floor diaphragms (cont'd)

Floors of precast concrete segments joined together & w/ structural frame via few-cm-thick lightly reinforced cast-in-situ topping, or waffle slabs w/ thin lightly reinforced top slab: Insufficient.

Collapse of precast concrete industrial building, w/ floors poorly connected to lateral-load-resisting system (Athens, 1999).

Collapse of buildings w/ precast concrete floors inadequately connected to the walls (Spitak, Armenia, 1988).

EC8 DESIGN CONCEPTS FOR SAFETY UNDER DESIGN SEISMIC ACTION

1. Design for energy dissipation (normally through ductility): q>1.5
   - Global ductility:
     - Structure forced to remain straight in elevation through shear walls, bracing system or strong columns ($\Sigma M_{Rc} > 1.3 \Sigma M_{Rb}$ in frames):
   - Local ductility:
     - Plastic hinges detailed for ductility capacity derived from q-factor;
     - Brittle failures prevented by overdesign/capacity design
   - Capacity design of foundations & foundation elements:
     - On the basis of overstrength of ductile elements of superstructure.

   (Or: Foundation elements - including piles - designed & detailed for ductility)
2. Design w/o energy dissipation & ductility: $q \leq 1.5$ for overstrength; design only according to EC2 - EC7 (Ductility Class "Low"– DCL) Only:
   - for Low Seismicity (NDP; recommended: PGA on rock $\leq$0.08g)
   - for superstructure of base-isolated buildings.

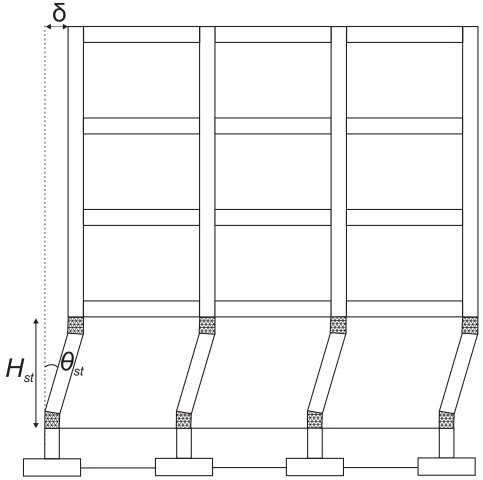
Force-based design for energy-dissipation & ductility, to meet no-(life-threatening-)collapse requirement under Design Seismic action:

- Structure allowed to develop significant inelastic deformations under design seismic action, provided that integrity of members & of the whole is not endangered.
- Basis of force-based design for ductility:
  - inelastic response spectrum of SDoF system having elastic-perfectly plastic *F*-*δ* curve, in monotonic loading.
- For given period, *T*, of elastic SDoF system, inelastic spectrum relates:
  - ratio $q = F_{el}/F_y$ of peak force, F_{el}, that would develop if the SDoF system was linear-elastic, to its yield force, F_y, ("behaviour factor")

  to

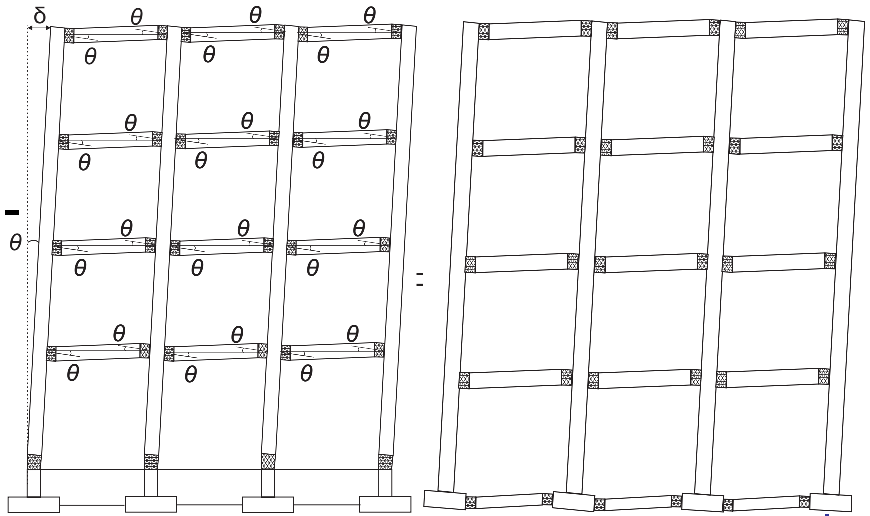
  - maximum displacement demand of the inelastic SDOF system, δ_{max}, expressed as ratio to the yield displacement, δ_y : displacement ductility factor, $\mu_\delta = \delta_{max}/\delta_y$

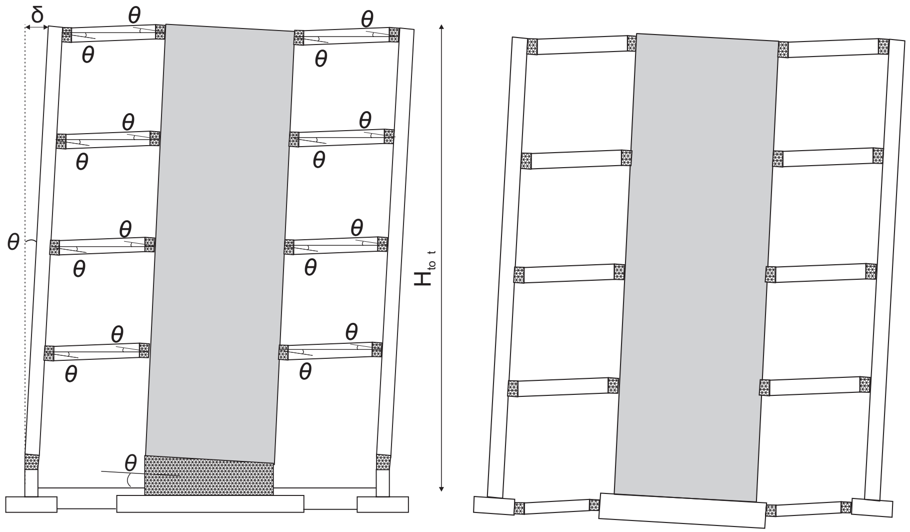
Control of inelastic seismic response: Soft-storey mechanism avoided



- Soft-storey collapse mechanism to be avoided via proper structural layout

–Strong-column/weak beam frame w/ beam-sway mechanism, involving:
plastic hinging at all beam ends, and either plastic hinging at column bottoms, or rotations at the foundation.





–Wall-equivalent dual frame, with beam-sway mechanism, involving:
plastic hinging at all beam ends, and either plastic hinging at wall & column base or rotations at the foundation.

Control of inelastic seismic response via capacity design

- Not all locations or parts in a structure are capable of ductile behaviour & energy dissipation.
- “Capacity design” provides the necessary hierarchy of strengths between adjacent structural members or regions & between different mechanisms of load transfer within the same member, to ensure that inelastic deformations will take place only in those members, regions and mechanisms capable of ductile behaviour & energy dissipation. The rest stay in the elastic range.
- The regions of members entrusted for hysteretic energy dissipation are called in Eurocode 8 “dissipative zones”. They are designed and detailed to provide the required ductility & energy-dissipation capacity.
- Before their design & detailing for the required ductility & energy-dissipation capacity, “dissipative zones” are dimensioned to provide a design value of ULS force resistance, R_d, at least equal to the design value of the action effect due to the seismic design situation, E_d, from the analysis:

$$E_d \leq R_d$$

- Normally linear analysis is used for the design seismic action (by dividing the elastic response spectrum by the behaviour factor, q)

EC8-PART 1: FOR ALL MATERIALS:

- For Dissipative Structures (except masonry):
  - Two Ductility Classes (DC):
    - ➢DC H (High).
    - ➢DC M (Medium).
  - Differences in:
    - ➢q-values (usually q > 4 for DCH, 1.5 <q <4 for DCM)
    - ➢Local ductility requirements

      (ductility of materials or section, member detailing, capacity design against brittle failure modes)

EC8-PART 1: FOR ALL MATERIALS:

- "Secondary seismic elements":
  - Their contribution to resistance & stiffness for seismic actions neglected in design (& in linear analysis model, too);
  - Required to remain elastic under deformations due to design seismic action.
  - Designer free to assign elements to the class of "secondary seismic elements", provided that:
    - ➢Their total contribution to lateral stiffness ≤ 15%;
    - ➢Regularity classification does not change.

CONCRETE & MASONRY BUILDINGS

- Yield-point stiffness in analysis (50% of uncracked section EI):
  - Reduction in design seismic forces vis-a-vis use of full section EI
  - Increase of displacements for drift-control & P-Δ effects (governs sizes of frame members).

Implementation of EC8 seismic design philosophy

- Damage limitation (storey drift ratio < 0.5-1%) under the damage limitation earthquake (~50% of "design seismic action"), using 50% of uncracked gross section stiffness.
- Member verification for the Ultimate Limit State (ULS) in bending under the "design seismic action", with elastic spectrum reduced by the behaviour factor q.
- In frames or frame-equivalent dual systems: Fulfilment of strong column/weak beam capacity design rule, with overstrength factor of 1.3 on beam strengths.
- Capacity design of members and joints in shear.
- Detailing of plastic hinge regions, on the basis of the value of the curvature ductility factor that corresponds to the q-factor value.

EC8-PART 1: DAMAGE LIMITATION CHECK

- Seismic action for “damage limitation”: NDP.
- Recommended for ordinary buildings: 10%/10yrs (95yr EQ);
- ~50% of “design seismic action” (475yr EQ).
- Interstorey drift ratio calculated for “damage limitation” action via “equal displacement rule” (elastic response):
  – <0.5% for brittle nonstructural elements attached to structure;
  – <0.75% for ductile nonstructural elements attached to structure;
  – < 1% for nonstructural elements not present or not interfering w/ structural response (: damage limitation for structure).
- Concrete (& masonry):
  – Elastic stiffness = 50% of uncracked gross-section stiffness.
- In concrete, steel or composite frames:

  damage limitation check governs member sizes.

ULS Verification of dissipative zones

- The regions of members entrusted for hysteretic energy dissipation - called in Eurocode 8 “dissipative zones” - are designed & detailed to provide the required ductility & energy-dissipation capacity.
- Before their design & detailing for the required ductility & energy-dissipation capacity, “dissipative zones” are dimensioned to provide a design value of ULS force resistance, R_d, at least equal to the design value of the action effect due to the seismic design situation, E_d, from the analysis:

$$E_d \leq R_d$$

- Normally linear analysis is used for the design seismic action (by dividing the elastic response spectrum by the behaviour factor, q)

NDP-partial factors for materials in ULS:

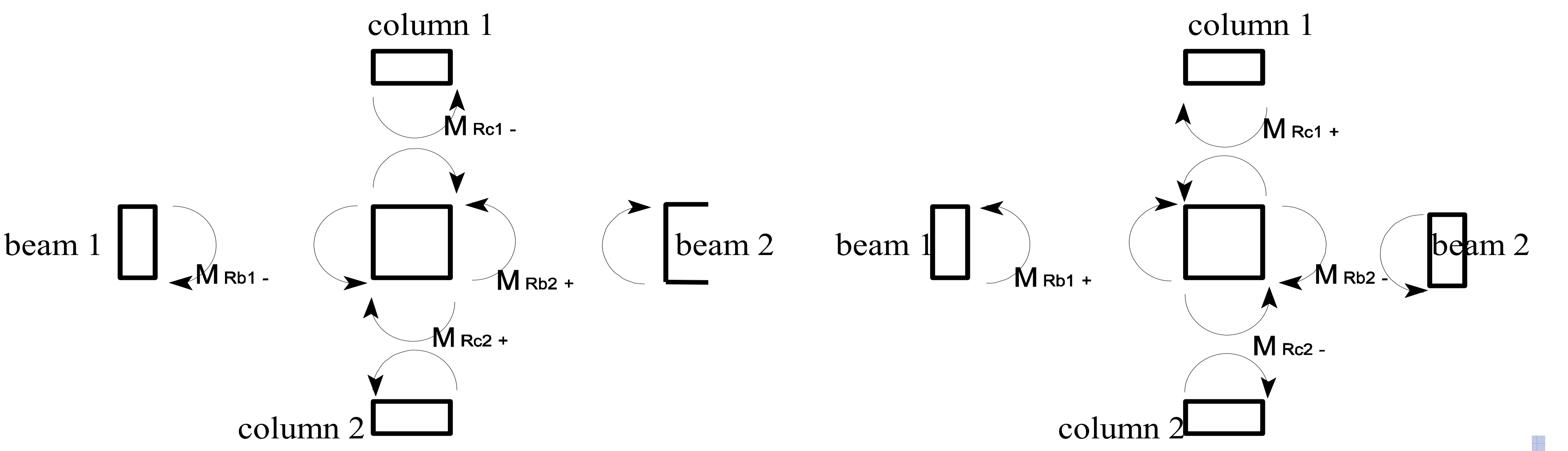
- Recommended:
- Use same values as for persistent & transient design situations (i.e. in concrete buildings: γ_c=1.5, γ_s=1.15);

Strong column/weak beam 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



Seismic design of the foundation

- Objective: The ground and the foundation system should not reach its ULS before the superstructure, i.e. remain elastic while inelasticity develops in the superstructure.
- Means:
  - The ground and the foundation system are designed for their ULS under seismic action effects from the analysis derived for q=1.5, i.e. lower than the q-value used for the design of the superstructure; or
  - The ground and the foundation system are designed for their ULS under seismic action effects from the analysis multiplied by $\gamma_{Rd}(R_{di}/E_{di}) \leq q$, where R_{di} force capacity in the dissipative zone or element controlling the seismic action effect of interest, E_{di} the seismic action effect there from the elastic analysis and γ_{Rd}=1.2
    - For individual spread footings of walls or columns of moment-resisting frames, R_{di}/E_{di} is the minimum value of M_{Rd}/M_{Ed} in the two orthogonal principal directions at the lowest cross-section of the vertical element where a plastic hinge can form in the seismic design situation;
    - For individual spread footings of columns of concentric braced frames, R_{di}/E_{di} is the minimum value of $N_{pl.Rd}/N_{Ed}$ among all diagonals which are in tension in the particular seismic design situation; for eccentric braced frames, R_{di}/E_{di} is the minimum value of $V_{pl.Rd}/V_{Ed}$ and $M_{pl.Rd}/M_{Ed}$ among all seismic links of the frame;
    - For common foundations of more than one elements, $\gamma_{Rd}(R_{di}/E_{di})$ =1.4.

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Seismic Design Philosophy for RC buildings according to Eurocode 8

- **Ductility Classes (DC)**
  - Design based on **energy dissipation** and **ductility**:
    - **DC** (**M**) **Medium** q=3 x system overstrength factor (≈1.3).
    - **DC** (**H**) **High** q= 4-4.5 x system overstrength factor (≈1.3).
- The aim of the design is to control the **inelastic seismic response**:
  - Structural **layout** & relative **sizing** of members ensures beam-sway mechanism.
  - Plastic hinge regions (beam ends, base of columns) are **detailed** to sustain inelastic deformation demands related to **behaviour factor q**:
  - $\mu_\delta = q$ if $T > T_c$
  - $\mu_\delta = 1 + (q-1)T_c/T$ if $T \leq T_c$

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	$f_{yk,0.95} \leq 1.25 f_{yk}$

Basic value, q_o, of behaviour factor for regular in elevation concrete buildings

Lateral-load resisting structural system	**DC M**	**DC H**
Inverted pendulum system*	1.5	2
Torsionally flexible structural system**	2	3
Uncoupled wall system (> 65% of seismic base shear resisted by walls; more than half by uncoupled walls) not belonging in one of the categories above	3	$4\alpha_u/\alpha_1$
Any structural system other than those above	$3\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$

* : at least 50% of total mass in upper-third of the height, or with energy dissipation at base of a single element (except one-storey frames w/ all columns connected at the top via beams in both horizontal directions in plan & with max. value of normalized axial load ν_d in combination(s) of the design seismic action with the concurrent gravity loads ≤ 0.3).

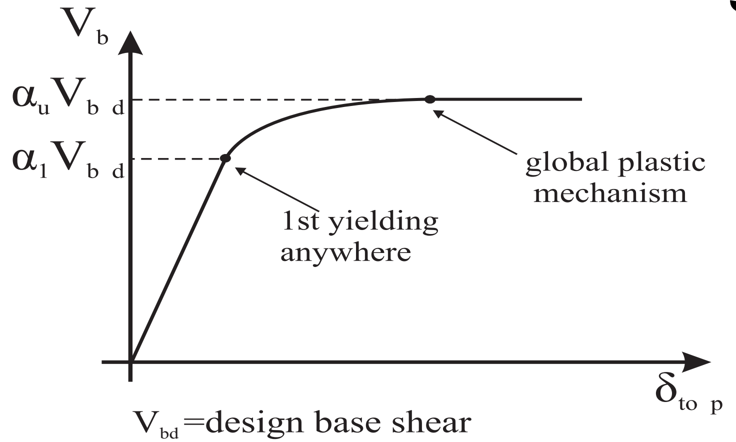
** : at any floor: radius of gyration of floor mass > torsional radius in one or both main horizontal directions (sensitive to torsional response about vertical axis).

- Buildings irregular in elevation: behaviour factor ***q* = 0.8*q*$_o$;**
- Wall or wall-equivalent dual systems: q multiplied (further) by $(1+a_o)/3 \le 1$, (a_o: prevailing wall aspect ratio = $\Sigma H_i/\Sigma l_{wi}$).

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

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

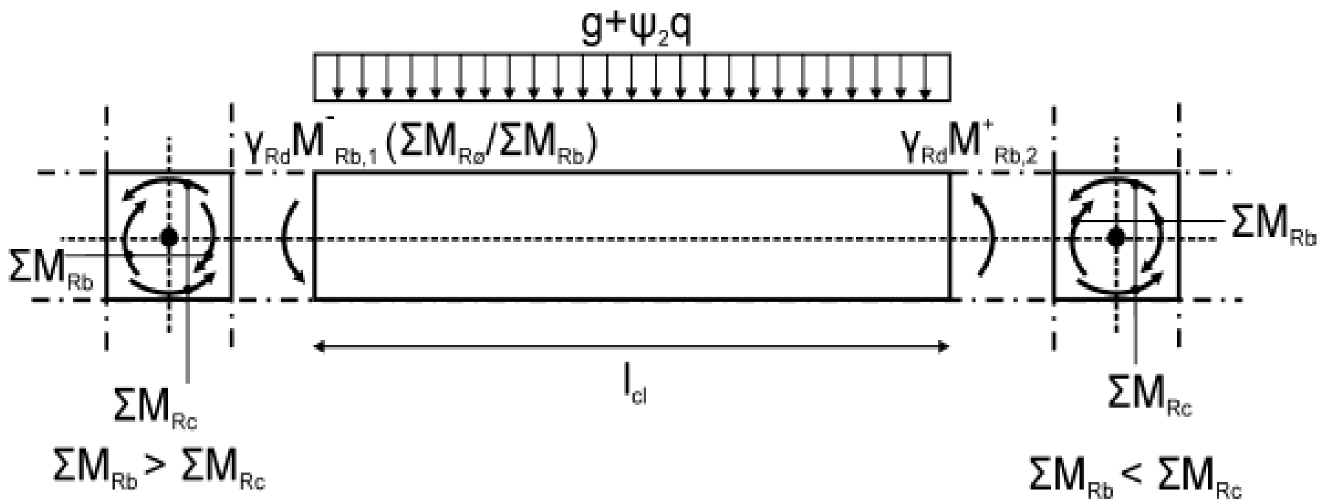
- α_u: seismic action at development of global mechanism;
- α_1: seismic action at 1st flexural yielding anywhere.



- $\alpha_u/\alpha_1 \leq 1.5$;
- 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

Capacity design of members, against pre-emptive shear failure

I. Beams



$$\max V_{i,d}(x) = \frac{\gamma_{Rd}\left[M_{Rd,bi}^{-}\min\left(1;\frac{\sum M_{Rd,c}}{\sum M_{Rd,b}}\right)_i + M_{Rd,bj}^{+}\min\left(1;\frac{\sum M_{Rd,c}}{\sum M_{Rd,b}}\right)_j\right]}{l_{cl}} + V_{g+\psi q,o}(x)$$

$$\min V_{i,d}(x) = -\frac{\gamma_{Rd}\left[M_{Rd,bi}^{+}\min\left(1;\frac{\sum M_{Rd,c}}{\sum M_{Rd,b}}\right)_i + M_{Rd,bj}^{-}\min\left(1;\frac{\sum M_{Rd,c}}{\sum M_{Rd,b}}\right)_j\right]}{l_{cl}} + V_{g+\psi q,o}(x)$$

- **in DC H γ_{Rd}=1.2** - reversal of V accounted for, depending on: $\zeta_i = \frac{\min V_{i,d}(x_i)}{\max V_{i,d}(x_i)}$
  in DC M γ_{Rd}=1.0,

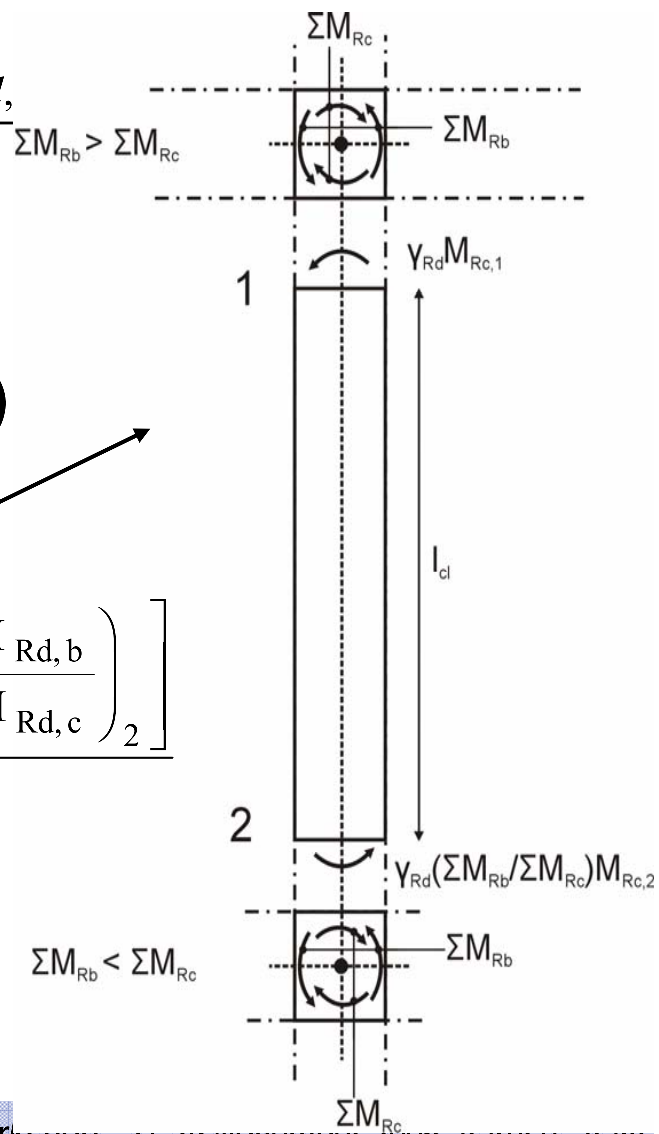
II. Columns

Capacity-design shear in column which is weaker than the beams:

$$V_{CD}^{+} = \gamma_{Rd} \frac{M_{Rd,c1}^{+} + M_{Rd,c2}^{-}}{h_{cl}} \qquad V_{CD}^{-} = \gamma_{Rd} \frac{M_{Rd,c1}^{-} + M_{Rd,}^{+}}{h_{cl}}$$

Capacity-design shear in (weak or strong) columns:

$$V_{CD,c} = \frac{\gamma_{Rd}\left[M_{Rd,c1}\min\left(1;\frac{\sum M_{Rd,b}}{\sum M_{Rd,c}}\right)_1 + M_{Rd,c2}\min\left(1;\frac{\sum M_{Rd,b}}{\sum M_{Rd,c}}\right)_2\right]}{h_{cl}}$$



- in DC H γ_{Rd}=1.3,
- in DC M γ_{Rd}=1.1

III. Walls

Eurocode 8:

Over-design in shear, by multiplying shear forces from the analysis for the design seismic action, V'_{Ed}, by factor ε:

DC M walls:

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = 1.5$$

DC H squat walls ($h_w/l_w \leq 2$):

Over-design for flexural overstrength of base w.r.to analysis

M_{Edo}: design moment at base section (from analysis),
M_{Rdo}: design flexural resistance at base section,
γ_{Rd}=1.2

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \gamma_{Rd}\left(\frac{M_{Rdo}}{M_{Edo}}\right) \leq q$$

DC H slender walls ($h_w/l_w > 2$):

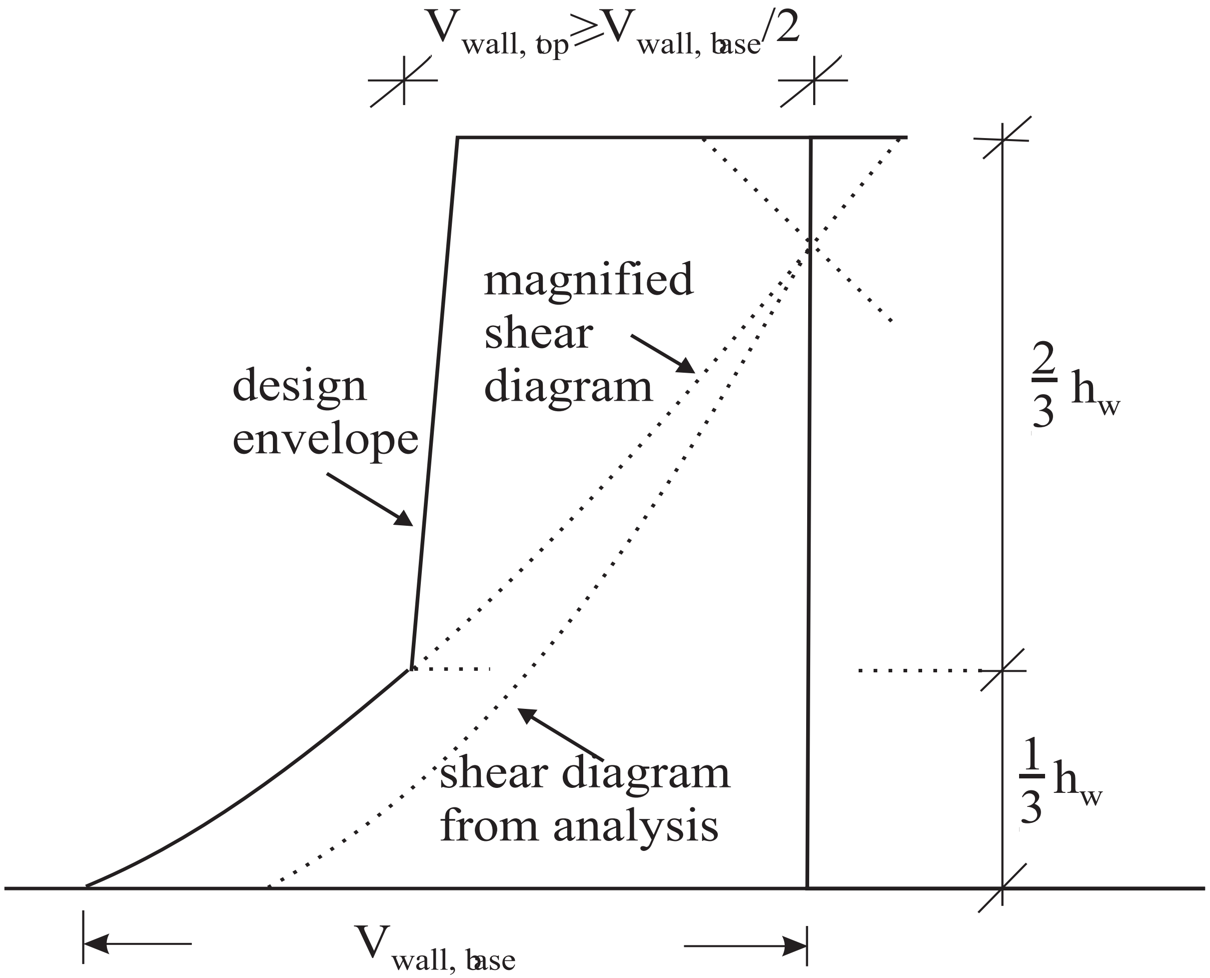
Over-design for flexural overstrength of base w.r.to analysis & for increased inelastic shears

$S_e(T)$: ordinate of elastic response spectrum

T_C: upper limit T of const. spectral acc. region
T_1: fundamental period.

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \sqrt{\left(\gamma_{Rd}\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q$$

Design shear forces in "ductile wall" of dual structural systems per Eurocode 8



To account for increase in upper storey shears due to higher mode inelastic response (after plastic hinging at the base)

DETAILING OF DISSIPATIVE ZONES FOR CURVATURE DUCTILITY FACTOR μ_φ CONSISTENT w/ q-FACTOR

- $\mu_\varphi = 2q_o - 1$ if $T_1 \geq T_c$
- $\mu_\varphi = 1 + 2(q_o - 1)T_c/T_1$ if $T_1 < T_c$
  - T_1: fundamental period of building,
  - T_c: T at upper limit of constant spectral acceleration region,
  - q_o : q-factor unreduced for irregularity in elevation (multiplied w/ M_{Ed}/M_{Rd} at wall base).

- Derivation:
  - Relation between μ_φ & L_{pl}/L_s (L_{pl}: plastic hinge length, L_s: shear span) & μ_δ (: top displacement ductility factor) in buildings staying straight due to walls or strong columns: $\mu_\delta = 1 + 3(\mu_\varphi - 1)L_{pl}/L_s(1 - 0.5L_{pl}/L_s)$;
  - Relation q-μ_δ-T :

    $\mu_\delta = q$ if $T_1 \geq T_c$, $\mu_\delta = 1 + (q-1)T_c/T_1$ if $T_1 < T_c$;
  - Relation of L_{pl} & L_s for typical RC beams, columns & walls

    (for EC2 confinement model: $\varepsilon^*_{cu} = 0.0035 + 0.1\alpha\omega_w$):

    $L_{pl} \approx 0,3L_s$ & for (safety) factor 2: $L_{pl} = 0,15L_s$. Then: $\mu_\varphi \approx 2\mu_\delta - 1$

- For steel B (ε_u: 5-7.5%, f_t/f_y: 1.08-1.15) increase μ_φ-demand by 50%

MEANS TO ACHIEVE μ_φ IN PLASTIC HINGES

- Members w/ axial load & symmetric reinforcement, $\omega=\omega'$ (columns, ductile walls):
  - Confining reinforcement (for walls: in boundary elements) with (effective) mechanical volumetric ratio:

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

    - $\nu_d=N_d/b_c h f_{cd}$; $\varepsilon_{yd}=f_{yd}/E_s$;
    - b_c: width of compression zone; b_o: width of confined core;
    - ω_ν: mechanical ratio of longitudinal web reinforcement $=\rho_\nu f_{yd,v}/f_{cd}$
  - Columns meeting strong-column/weak-beam rule ($\Sigma M_{Rc}>1.3\Sigma M_{Rb}$), provided w/ full confining reinforcement only at (building) base;
  - DC H strong columns ($\Sigma M_{Rc}>1.3\Sigma M_{Rb}$) also provided w/ confining reinforcement for 2/3 of μ_φ in all end regions above base;
- Members w/o axial load & w/ unsymmetric reinforcement (beams):
  - Max. mechanical ratio of tension steel:

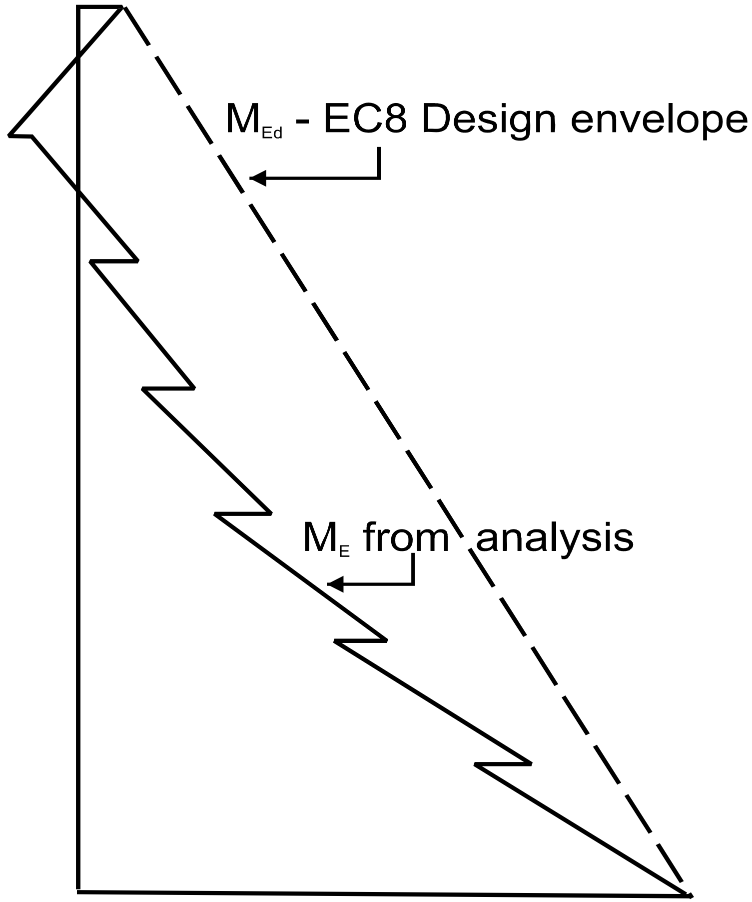
$$\omega \leq \omega' + 0.0018/\boldsymbol{\mu_\varphi}\, \varepsilon_{yd}$$

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 $l_w \geq 4$m, 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.

Strong column/weak beam capacity design not required in wall- or wall-equivalent 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

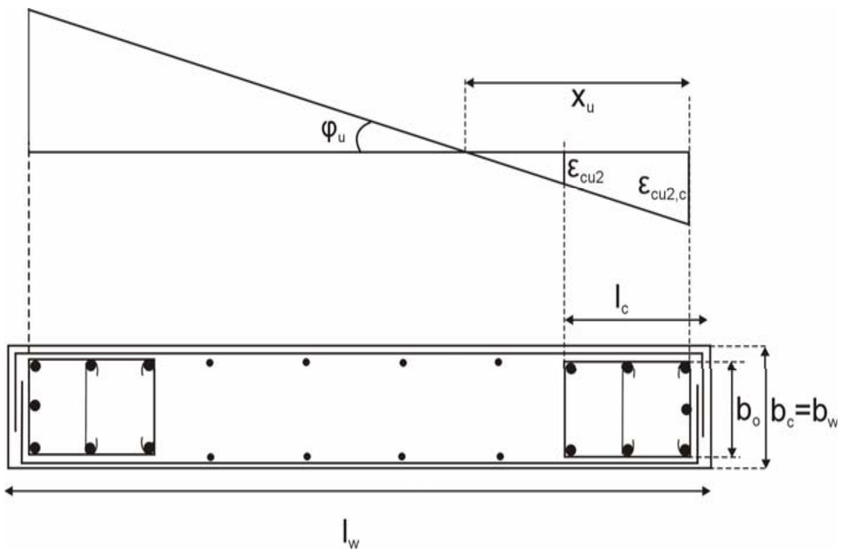
DESIGN & DETAILING OF DUCTILE WALLS

- Inelastic action limited to plastic hinge at base,

  so that cantilever relation between q & μ_φ can apply:
  - Wall provided with flexural overstrength above plastic hinge region (linear moment envelope with shift rule);
  - Design in shear for V from analysis, times:

  1.5 for DC M

  $[(1.2\, M_{Rd}/M_{Ed})^2+0.1(qS_e(T_c)/S_e(T_1))^2]^{1/2} < q$ for DC H
    - M_{Ed}: design moment at base (from analysis),
    - M_{Rd}: design flexural resistance at base,
    - $S_e(T)$: ordinate of elastic response spectrum,
    - T_c: upper limit T of const. spectral acc. region
    - T_1 fundamental period.



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

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

over part of compression zone depth: $x_u=(v_d+\omega_\nu)\varepsilon_{yd}b_c/b_o$

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

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 & 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.

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]	$\le \dfrac{6.25(1+0.8\nu_d)}{(1+0.75\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$	$\le \dfrac{7.5(1+0.8\nu_d)}{(1+0.5\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$	-
d_{bL}/h_c - bar anchored at exterior joint[3]	$\le 6.25(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$	$\le 7.5(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$	-
Transverse bars (w):			
(i) outside critical regions			
spacing $s_w\le$	$0.75d$		
$\rho_w\ge$	$0.08(f_{ck}(MPa))^{1/2}/f_{yk}(MPa)$[0]		
(ii) in critical regions:			
$d_{bw}\ge$	6mm		
spacing $s_w\le$	$6d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 175mm	$8d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 225mm	-
Shear design:			
V_{Ed}, seismic[4]	$1.2\dfrac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+\psi_2 q}$[4]	$\dfrac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+\psi_2 q}$[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 $1\le\cot\theta\le2.5$		
$V_{Rd,s}$, outside critical regions[5]	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$ [5], with $1\le\cot\theta\le2.5$		
$V_{Rd,s}$, critical regions[5]	$V_{Rd,s}=b_wz\rho_wf_{ywd}$ ($\theta=45^o$)	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$, with $1\le\cot\theta\le2.5$	
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, $d\text{-}d_1$.

(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.

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%		$0.1N_d/A_c f_{yd}$, 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 $\nu_d = N_{Ed}/A_c f_{cd}$	≤ 0.55	≤ 0.65	-
Shear design:			
V_{Ed} seismic (11)	$1.3\frac{\sum M_{Rc}^{ends}}{l_{cl}}$ (11)	$1.1\frac{\sum M_{Rc}^{ends}}{l_{cl}}$ (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: $V_{Rd,s}=b_w z\rho_w f_{ywd}\cot\theta+N_{Ed}(h-x)/l_{cl}$ (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: $\sum M_{Rb} < \sum M_{Rc}$, M_{Rc} is replaced by $M_{Rc}(\sum M_{Rb}/\sum M_{Rc})$.

(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 $\varepsilon_c > 0.0035$		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 $H_w/l_w \le 2^{(5)}$: $\varepsilon = 1.2 M_{Rdo}/M_{Edo} \le q$ if $H_w/l_w > 2^{(5),(6)}$: $\varepsilon = \sqrt{\left(1.2\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \le q$	ε=1.5	ε=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)	$V_{Ed}(z) = \left(\frac{0.75z}{H_w} - \frac{1}{4}\right)\varepsilon V_{Ed}(0) + \left(1.5 - \frac{1.5z}{H_w}\right)\varepsilon V_{Ed}\left(\frac{H_w}{3}\right)$		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)	$\rho_v f_{yvd} \ge \rho_h f_{yhd} - N_{Ed}/(0.8 l_w b_{wo})$		
Resistance to sliding shear: via bars with total area A_{si} at angle ±φ to the horizontal (10)	$V_{Rd,s} = A_{si} f_{yd}\cos\phi +$ $A_{sv}\min(0.25f_{yd},\ 1.3(f_{yd}f_{cd})^{1/2}) +$ $0.3(1-f_{ck}(MPa)/250)b_{wo}x f_{cd}$		
$\rho_{v,min}$ at construction joints (9),(11)	$0.0025,\ \dfrac{1.3f_{ctd} - \dfrac{N_{Ed}}{A_c}}{f_{yd} + 1.5\sqrt{f_{cd}f_{yd}}}$	-	

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:

$$V_{Rd,c} = \left\{ \min\left[\frac{180}{\gamma_c}(100\rho_L)^{1/3},\ 35\sqrt{1+\sqrt{\frac{0.2}{d}}}\, f_{ck}^{1/6} \right] \left(1+\sqrt{\frac{0.2}{d}}\right) f_{ck}^{1/3} + 0.15\frac{N_{Ed}}{A_c} \right\} b_w d$$

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.

STRUCTURE OF EN1998-1:2004

1 General
2 Performance Requirements and Compliance Criteria
3 Ground Conditions and Seismic Action
4 Design of Buildings
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 <u>Specific Rules for Masonry Buildings</u>
10 Base Isolation

MASONRY BUILDINGS

Mainly for regions of rather low-seismicity.

Nationally Determined Parameters (NDPs) for national flexibility:

- Allowable type of masonry units & of perpend joints
- Min. strength of masonry units & mortar;
- Max. PGA for use of unreinforced masonry w/ EC6 alone, or w/ EC8;
- q-factor values for all types of masonry buildings (ranges given, instead of single values) other than those per EC6 alone;
- Geometric limitations for shear walls:
  - min. thickness;
  - max. slenderness (height-to-thickness);
  - max height of openings relative to wall length;
- Conditions for design w/o detailed calculations (rules for "simple masonry buildings"):
  - Max. no. of storeys & min. horizontal area of walls, as function of PGA;
  - Max. aspect ratio in plan & deviation of plan from rectangular envelope;
  - Max. difference of mass & wall X-section between adjacent storeys.

MASONRY BUILDINGS (*cont'd*)

Types of masonry for EQ-resistance:

- Unreinforced masonry per EC6 alone (not recommended for PGA at site > 0.1g): *q=1.5*

- Unreinforced masonry, w/ horizontal RC belts (A_s>200mm²) at <4m centres (not recommended for PGA at site > 0.15g):
  q (NDP) = 1.5 - 2.5 (recommended: q=1.5)

- Confined masonry, w/ horizontal RC belts > 0.15x0.15 m (A_s>300mm² or 1%) at <4 m centres and similar vertical ones at <5 m centres & at wall intersections & edges of large openings:
  q (NDP) = 2 - 3 (recommended: q=2).

- Reinforced masonry, w/ ρ_h> 0.05% & ρ_v> 0.08% (plus vertical steel w/ A_s>200mm² at <5m centres & at wall intersections or free edges):
  q (NDP) =2.5 - 3 (recommended: q=2.5).