

**Local effects due to infills  
Design and detailing of  
secondary seismic elements  
Provisions for concrete diaphragms**

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# MASONRY-INFILLED FRAMES

# Overall effect of masonry infills on seismic performance

- Field experience & numerical/experimental research do show that:
  - masonry infills attached to the structural frame, in general have a beneficial effect on seismic performance, especially if the building structure has little engineered earthquake resistance.
- If effectively confined by the surrounding frame, regularly distributed infill panels:
  - reduce, through their in-plane shear stiffness, storey drift demands & deformations in structural members
  - increase, via their in-plane shear strength, storey lateral force resistance,
  - contribute, through their hysteresis, to the global energy dissipation.
- In buildings designed for earthquake resistance, non-structural masonry infills can serve as a 2<sup>nd</sup> line of defence & a source of significant overstrength.

## Position of EC8 on masonry infills

- Eurocode 8 does not encourage designers to profit from the beneficial effects of masonry infills to reduce the seismic action effects for which the structure is designed.
- Eurocode 8 warns against the adverse effects of infills & requires prevention measures for them.
- If there is structural connection between the masonry infill & the surrounding frame (by shear connectors, or other ties, belts or posts), the building is considered/designed as a confined masonry building, not as a concrete structure with masonry infills.

# Possible adverse effects of masonry infills

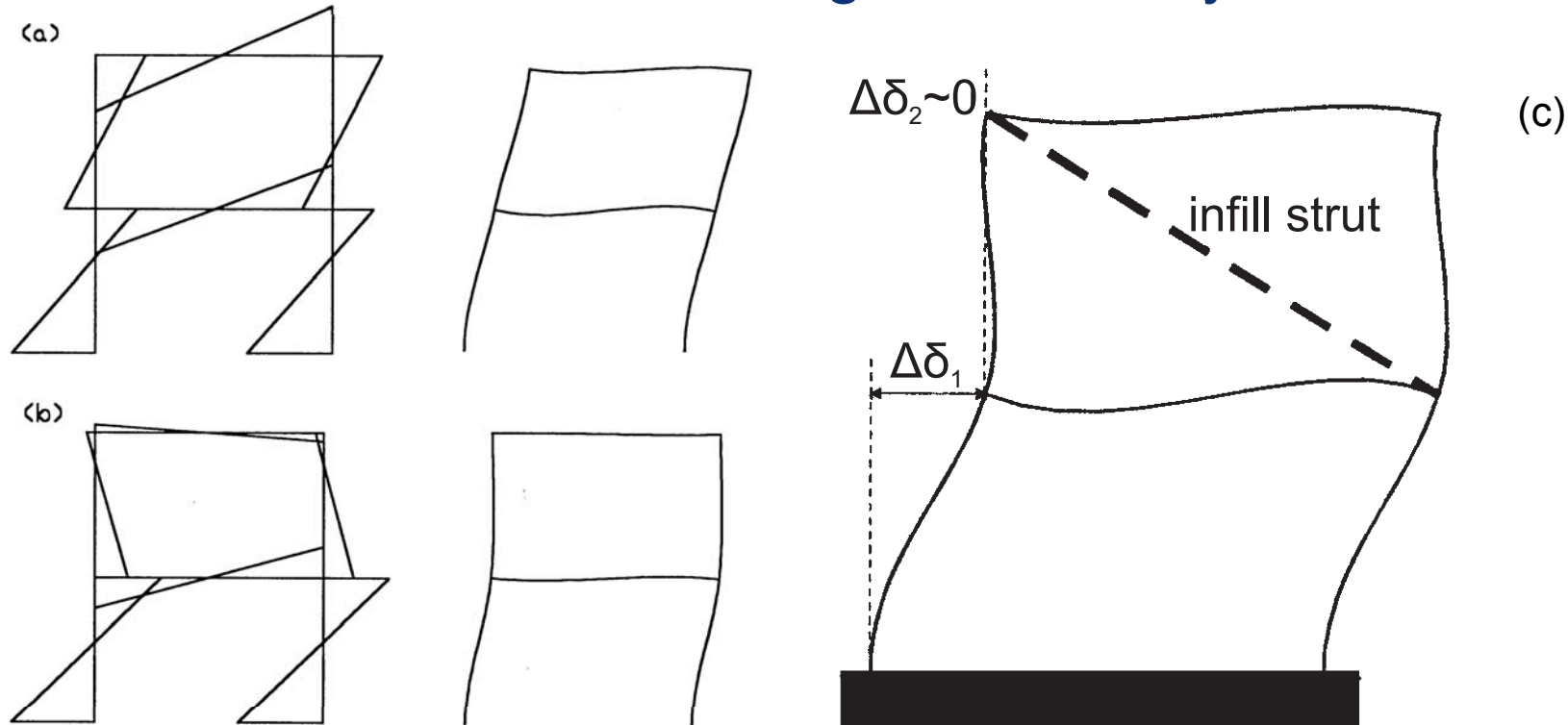
- Infills that are too strong & stiff relative to the concrete structure itself
  - may override its seismic design, including the efforts of the designer & the intent of EC8 to control inelastic response by spreading inelastic deformation demands throughout the structure (e.g. when ground storey infills fail → soft storey).
- Infills non-uniformly distributed in plan or in elevation:
  - concentration of inelastic deformation demands in part of the structure.
- Adverse local effects on structural frame
  - pre-emptive brittle failures.

## Possible adverse effects of masonry infills (*cont'd*)

- Best way to protect concrete building from adverse effects of irregular masonry infilling:  
shear walls sufficiently strong/stiff to overshadow any effects of the infilling.
- Eurocode 8:  
Shear walls that resist at least 50% of the seismic base shear (wall-equivalent dual or wall buildings): sufficient for waiving the special requirements for buildings with infills (except those for the local effects on columns).

## Possible adverse effects of masonry infills (cont'd)

Worst possible effect: Open ground storey → soft-storey



2-storey frame: Elements in infilled storey shielded from large moments & deformations. But ground storey columns are overloaded. See:  
(a) bending moments & deformation in frame w/o infills;  
(b), (c) bending moments & deformation in frame w/ stiff infills in 2<sup>nd</sup> storey.



# Open ground storey



(a)



(b)

Collapse of ground storey due to reduction of infills:  
(a) Olive View Hospital, San Fernando, Ca, 1971; (b) Aegio (GR) 1995



# EC8 design for infill irregularity in elevation

Eurocode 8: Design columns of storey where infills are reduced relative to overlying storey, to remain elastic till infills in storey above reach their ultimate force resistance:

- Deficit in infill shear strength in a storey is compensated by an increase in resistance of the frame's (vertical) members there:
- In DC H frame or frame-equivalent dual buildings, seismic M, V, N in the columns from the analysis for the design seismic action are multiplied by:

$$\eta = \left( 1 + \Delta V_{Rw} / \Sigma V_{Ed} \right) \leq q$$

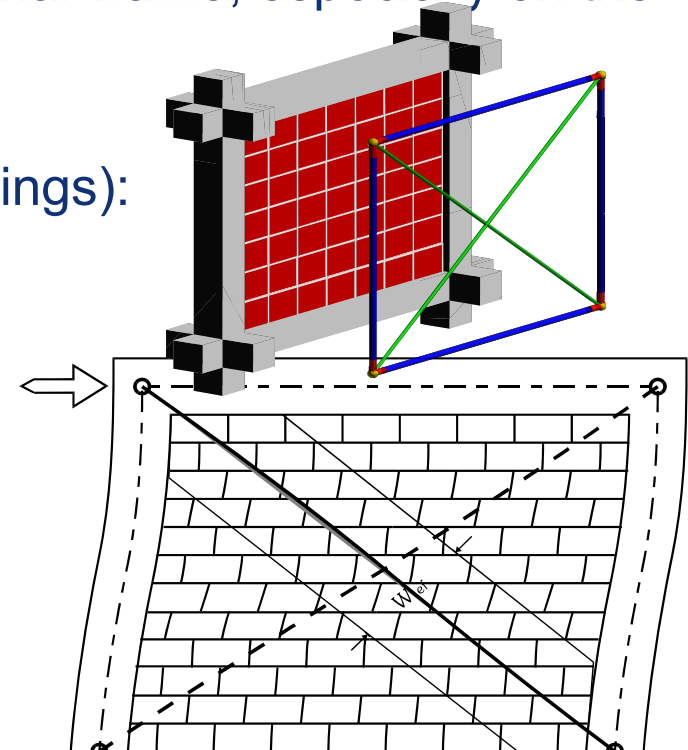
- $\Delta V_{Rw}$ : total reduction of resistance of masonry walls in storey concerned w.r.to storey above,
- $\Delta V_{Ed}$ : sum of seismic shear forces in all vertical primary seismic members of the storey (storey design shear).
- If  $\eta < 1.1$ : magnification of seismic action effects may be omitted.
- Although not required for DC M frame or frame-equivalent dual buildings, the above are (weakly) recommended for them as well.

## Asymmetry of infills in plan

- Asymmetric distribution of infills in plan → torsional response to translational horizontal components of seismic action:
  - Members on the side with the fewer infills (“flexible” side) are subjected to larger deformation demands & fail first.
- The increase in global lateral strength & stiffness due to the infills makes up for an uneven distribution of interstorey drift demands in plan:
  - The maximum member deformation demands for planwise irregular infilling do not exceed peak demands anywhere in plan, in a similar structure w/o infills.

# EC8 design against infill planwise asymmetry

- Eurocode 8: doubles accidental eccentricity (from 5 to 10%) in the analysis, if the infills are planwise irregular.
- Doubling of accidental eccentricity: is not enough for “severely irregular” arrangement of infills in plan →
  - Need analysis of a 3D structural model explicitly including the infills,
  - Need sensitivity analysis of the effect of stiffness & position of infills (disregarding one out of 3-4 infill panels per planar frame, especially on the flexible sides).
- In-plane modelling of infills.
  - Simplest modelling of solid panel (without openings):
    - Two diagonal struts.
  - Effect of openings: Reduction factors?
- The above are required for DC H frame or frame-equivalent dual buildings and (weakly) recommended for DC M ones.



# Adverse local effects on structural frame



Shear failure of weak columns due to interaction with strong infills



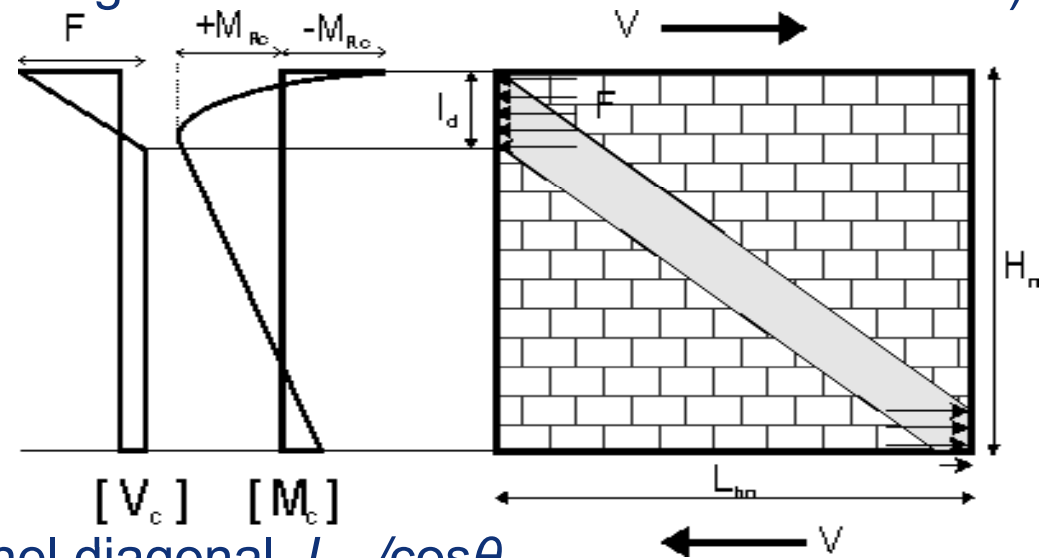
# EC8 design against local effect of strong infills

Shear loading of the column by the infill strut force:

- Eurocode 8 for all columns: verify in shear a length  $l_c = w_{inf} / \cos\theta$ , near the top & the bottom of the column over which the diagonal strut force of infill may be applied, for the smaller of the two design shear forces:
  - The horizontal component of the infill strut force, taken equal to the horizontal shear strength of the panel (shear strength of the bed joints times the horizontal cross-sectional area of panel); or
  - Capacity design shear:  $2 M_{Rd,c}$  (: design value of column moment resistance) /  $l_c$  (: contact length)
- Width of the strut (e.g.):

$$w_{inf} = \frac{0.175 L_{bn}}{\cos\theta (\lambda H)^{0.4}}$$

$$\lambda = \left( \frac{E_w b_w \sin 2\theta}{4 E_c I_c H_n} \right)^{\frac{1}{4}}$$



- Eurocode 8: fraction ( $\sim 15\%$ ) of panel diagonal,  $L_{bn} / \cos\theta$
- Columns in contact with infill all along only one side: Full clear height = critical region



## Adverse local effects on structural frame (*cont'd*)

### Shear failures of short (captive) columns



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# EC8 design of squat “captive” columns

- Capacity-design calculation of design shear force, w/
  - clear length of the column,  $l_{cl}$  = length of the column not in contact to the infills &
  - plastic hinging assumed to take place at the column section at the termination of the contact with the infill wall.
- Transverse reinforcement required to resist the design shear force is placed not just along the clear length of the column,  $l_{cl}$ , but also into the column part which is in contact to the infills (over length equal to the column depth,  $h_c$ , within plane of infill).
- Entire length of the column is taken as critical region, with stirrups detailed as in column “critical” regions.
- Use diagonal reinforcement over length of column not in contact to the infill (to resist M & V), if it is less than 1.5 times the column depth.



# **SECONDARY (SEISMIC) ELEMENTS IN EC8**

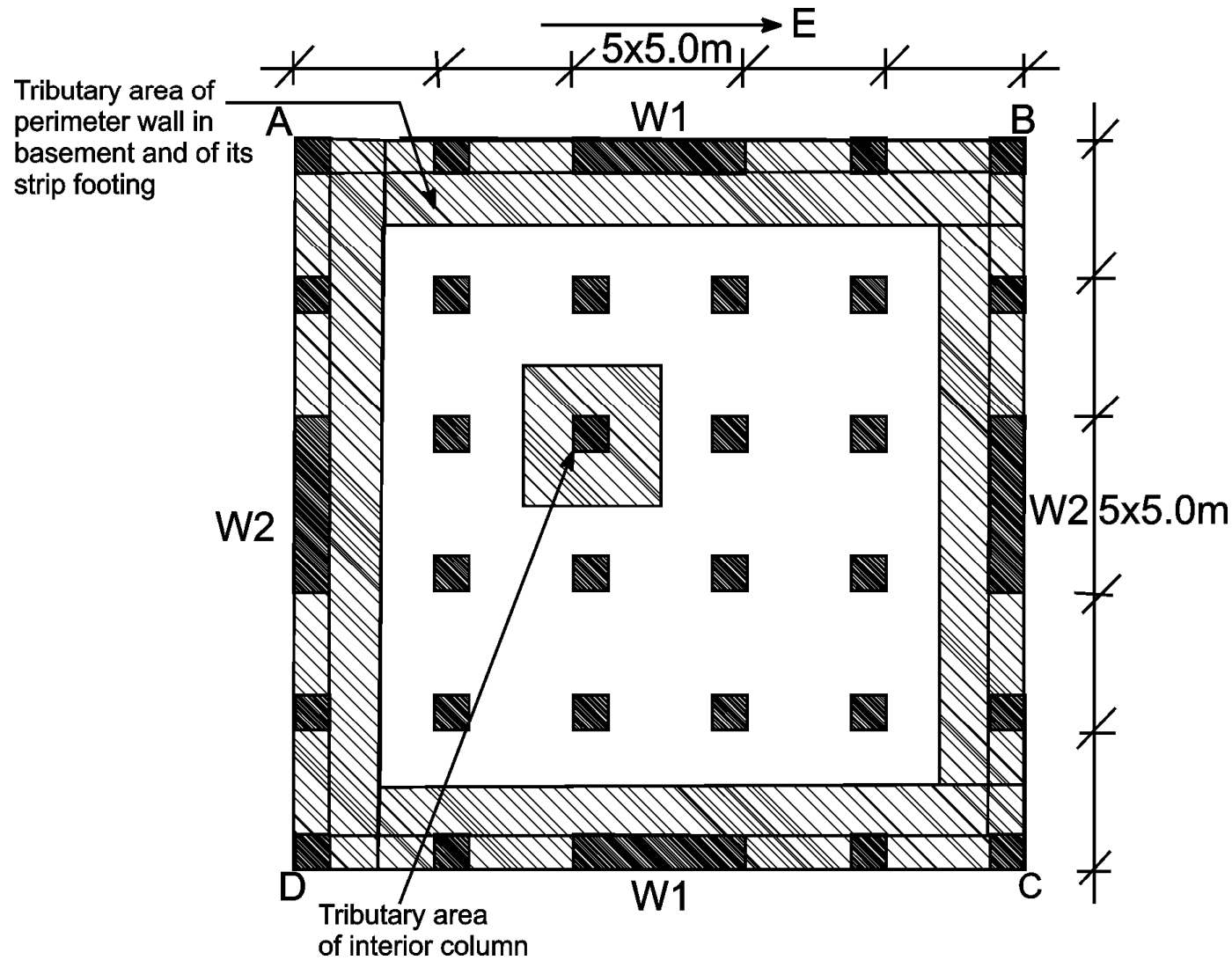
# Secondary seismic elements

- Contribution of “secondary seismic elements” to resistance & stiffness for seismic actions is discounted in design (& in linear analysis model, too).
- The designer is free to assign elements to this class of elements, if:
  - Their total contribution to lateral stiffness  $\leq 15\%$  of that of the other (“primary seismic”) elements;
  - The building’s regularity classification does not change.
- “Secondary seismic elements”:
  - not subject to the geometric etc. restrictions of EC8
  - not ULS-designed for any seismic force demands,
  - not detailed for ductility, etc.
- **But:** they are required to remain elastic under the deformations imposed by the design seismic action (:  $q$ -times their deformations from an elastic analysis with the contribution of secondary elements neglected):
  - Requirement hard to meet.
- Therefore, consider as “secondary seismic” only those elements which cannot be made to meet EC8 rules (e.g., if they are outside EC8’s scope: prestressed girders, flat-slab frames, etc.)

# Design procedure if some elements are “Secondary seismic”

1. Carry out linear analysis for the design seismic action using two models :
  - **Model SP:** including the contribution of all elements (“secondary” or “primary seismic”) to lateral stiffness;
  - **Model P:** neglecting the contribution of “secondary seismic elements” to lateral resistance & stiffness  
(e.g., introduce appropriate hinges at their connections to “primary” elements, so that the “secondary” ones have stiffness only against gravity loads).
2. Calculate the ratio of storey drifts of **Model P** to those of **Model SP** and check that it is  $\leq 115\%$  at every storey.
3. Estimate the deformations of “secondary” seismic elements under the design seismic action as  $q$ -times their deformations from **Model SP**, times the **P/SP-ratio** of interstorey drifts in **2** above.
4. From the deformations from **3** above and the cracked stiffness of the “secondary seismic element” (50% of uncracked gross stiffness) find their internal forces and check that they are in the elastic domain.
  - Equivalent: Dimension (check) “secondary seismic elements” for the internal forces from **Model SP**, times  $q$ , times the **P/SP-ratio** of interstorey drifts from **2** above.

# 7-storey wall building with flat-slab frames taken as “secondary seismic”



# Contribution of secondary elements to lateral stiffness $\leq 15\%$ of that of primary elements

- Elastic analysis of full structural system in 3D, including in-plane flexibility of the diaphragm and taking the flat slab as an effective beam w/ width of 2.5m at the interior of the plan or 1.25m at the perimeter:
- Total contribution of flat slab frames and of the walls in their weak direction to lateral stiffness: 13.9% of that of the walls in their strong direction.

# Deformation-induced seismic action effects in secondary seismic columns

- Elastic M & V in the secondary columns from elastic analysis of full structural system in 3D (including the flat slab frames) under design seismic action, multiplied by q and divided by the fraction of the base shear taken by the primary seismic elements, i.e., multiplied times:
  - $qV/(V-0.139V)=3.48$ :
- In a top storey column near the centre in plan:
  - $V=139\text{kN}$ ,
  - End moments: 240kNm and 127kNm at top & bottom.
- Maximum M in any interior column:
  - 372kNm, at the ground storey.
  - ( $V=141\text{kN}$ , at the ground storey).

# Check of secondary seismic columns for the deformation-induced seismic moments

Min. vertical steel ratio per EC2: 0.2%. For 8 14mm-dia. bars:  $\rho=0.342\%$ :

- Top storey axial load  $N=205\text{kN}$ , giving  $M_{\text{Rdc},n}=346\text{kNm} > M_{\text{Ec},n}$ .
- Ground storey  $N=1435\text{kN}$ ,  $M_{\text{Rdc},1}=795\text{kNm} \gg M_{\text{Ec},1}$ .



# Check of secondary seismic columns for the deformation-induced seismic shears

Max. tie spacing per EC2:

- $\max s_w = 0.6 \min\{20d_{bL}; h_c; b_c; 400\text{mm}\}$  at lap-splices of vertical bars.

8mm-dia. perimeter hoop and diamond-shaped ties mid-side vertical bars, @ 165mm centres:

$$\rho_w = (2 + \sqrt{2}) \times 50.25 / (165 \times 600) = 0.00173.$$

- Shear resistance for shear compression per EC2:

$$V_{Rd,max} = 0.3 \times (1 - 35/250) \times 0.6 \times 0.9 \times 0.565 \times (35000/1.5) \sin 2\delta = 1269 \text{ kN} > V_{Ed,1}$$

if  $\cot \delta = 2.5$

- Shear resistance due to the ties per EC2:

$$V_{Rd,s} = b_w z \rho_w f_{ywd} \cot \delta + N_{Ed} (h-x) / H_{cl}$$

with neutral axis depth,  $x = \xi d$ , at the moment resistance of the column.

- **Top storey:**

- $V_{Rd,s} = 0.6 \times 0.9 \times 0.565 \times 0.00173 \times (500000/1.15) \times 2.5 + 205 \times (0.6 - 0.084 \times 0.565) / 2.65 = 616.5 \text{ kN} > V_{Ed,n}$

- **Ground storey:**

- $V_{Rd,s} = 0.6 \times 0.9 \times 0.565 \times 0.00173 \times (500000/1.15) \times 2.5 + 1435 \times (0.6 - 0.249 \times 0.565) / 2.65 = 822.5 \text{ kN} > V_{Ed,1}$

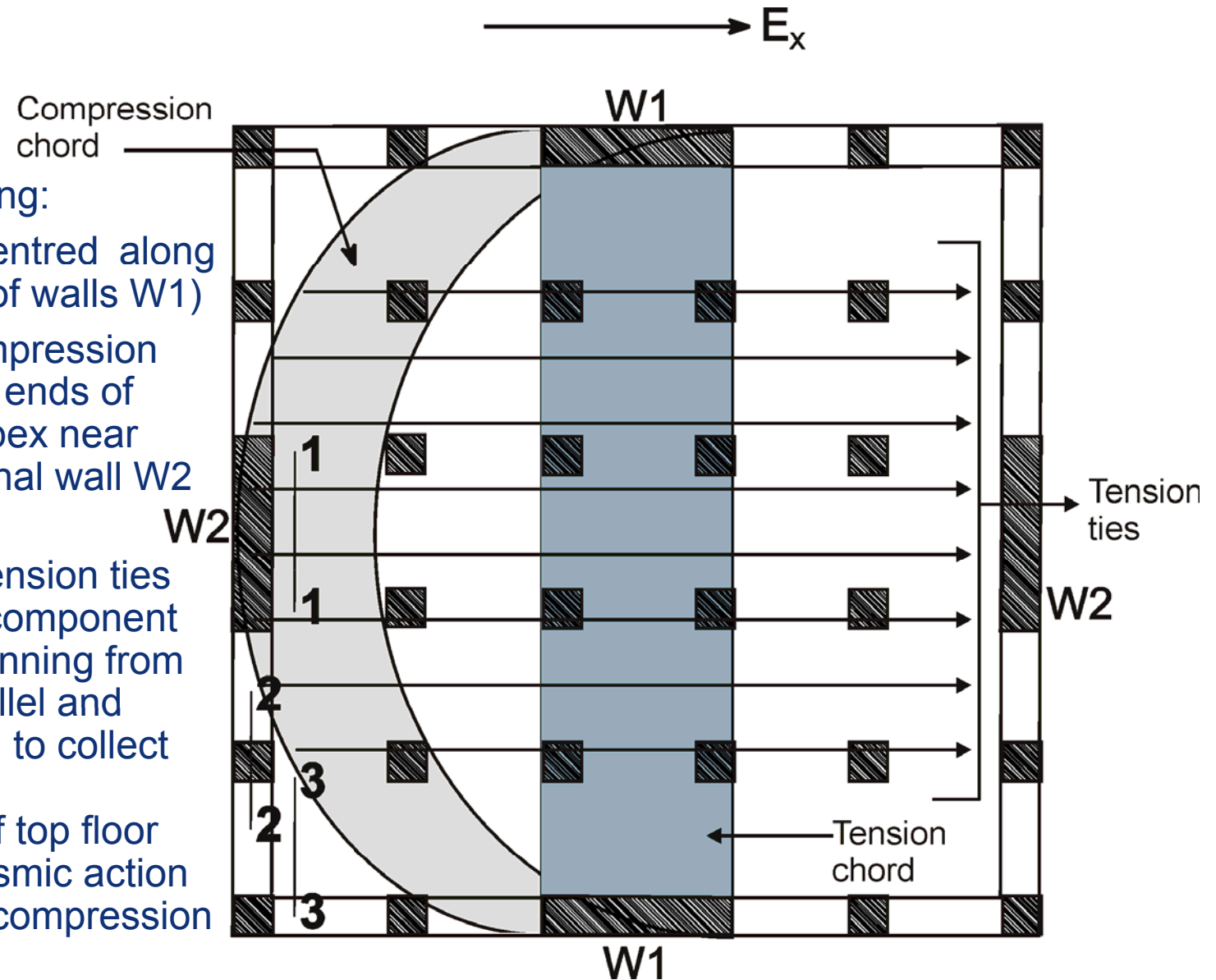
# Concrete diaphragms

- ULS verification of RC diaphragms in DCH buildings:
  - For irregular geometry or divided shapes of diaphragm in plan, recesses or re-entrances;
  - For irregular and large openings in diaphragm;
  - If irregular distribution of masses and/or stiffnesses (set-backs or off-sets);
  - In basements with walls only in part of the perimeter or only in part of the ground floor area;
  - At the interface with core and walls in core or wall structural systems.
- Model such diaphragms as deep beam or plane truss or strut-and-tie model, on elastic supports.

# Strut-and-Tie model of diaphragm for check of top floor slab

Deep beam comprising:

- a Tension chord centred along line 1 (width =  $l_w$  of walls W1)
- b Semi-circular compression chord connecting ends of tension chord, apex near centre of orthogonal wall W2 on line 2;
- c Closely spaced tension ties parallel to horiz. component seismic action, running from edge in plan parallel and opposite to line 2, to collect the in-plane load  $q_E = 1.728 \text{ kN/m}^2$  of top floor due to design seismic action and transfer it to compression chord.

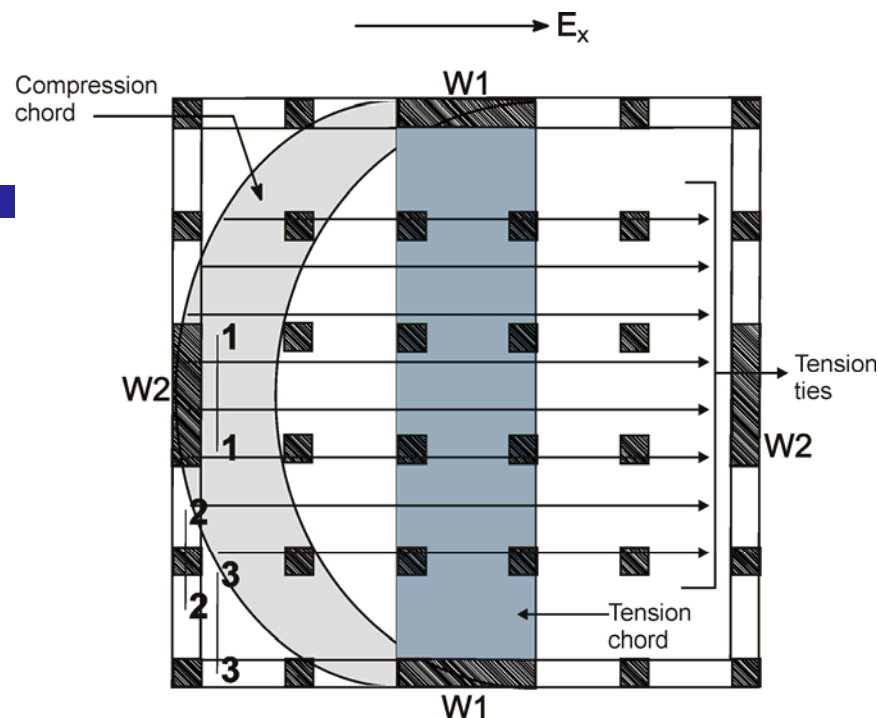


## Verification of tension ties (cont'd)

- Longest tension ties collect in-plane load  $q_E = 1.728 \text{ kN/m}^2$  along the full plan dimension,  $L_x$ . For their ULS verification in tension, any vertical section through the flat slab normal to hor. direction X should have reinforcement area at least  $\gamma_d q_E L_x / f_{yd} = 1.1 \times 1.728 \times 25 / (0.5 / 1.15) = 110 \text{ mm}^2/\text{m}$  over and above what is required for moment resistance of the flat slab for the moment due to the quasi-permanent floor gravity load,  $M_{g+\psi 2q}$ . ( $\gamma_d = 1.1$ : overstrength factor per EC8 for the design of diaphragms).
- The reinforcement of the flat slab has been dimensioned for ULS in bending for the flat slab moments under the factored gravity loads,  $M_d$ . The surplus of reinforcement area over and above what is necessary for ULS resistance under  $M_d$ :  $\Delta A_s = \max[A_{s,\min}; M_d / (z f_{yd})] - M_{g+\psi 2q} / (z f_{yd})$ , where  $z = 0.11 \text{ m}$  the internal lever arm,  $M_{g+\psi 2q} = (8.2/14.7) M_d$  and  $A_{s,\min}$  the minimum reinforcement area in the flat slab per EC2.
- Critical location for  $\Delta A_s$ : where  $M_d$  is minimum.

# Verification of tension ties

Dissemination of information for training – Lisbon 10-11 February 2011



- Minimum  $M_d$  along longest tension ties:  
Sagging moment at mid-distance between W2 and 1<sup>st</sup> row of interior columns parallel to W2 (Section 1-1)
  - Surplus  $\Delta A_s = (1 - 8.2/14.7) \times 10.2 / (0.11 \times 0.5 / 1.15) = 94.4 \text{ mm}^2/\text{m} < 110 \text{ mm}^2/\text{m}$ .
  - Increase reinforcement of flat slab within its middle strips between W2 and the 1<sup>st</sup> parallel row of interior columns, and between any rows of interior columns, to  $\geq 110 + 213 \times 8.2 / 14.7 = 229 \text{ mm}^2/\text{m}$ .
- Potentially critical: tension ties heading towards the edge column next to W2 (Section 2-2):  $\Delta A_s = (1 - 8.2/14.7) \times 47.3 / (0.11 \times 0.5 / 1.15) = 438 \text{ mm}^2/\text{m} > 110 \text{ mm}^2/\text{m}$
- Between edge columns and 1<sup>st</sup> parallel row of interior columns (Section 3-3):  $\Delta A_s = (1 - 8.2/14.7) \times 37.85 / (0.11 \times 0.5 / 1.15) = 350 \text{ mm}^2/\text{m} > 110 \text{ mm}^2/\text{m}$ .

# Check of tension chord between supports of the deep beam by walls W1.

Tension force in chord from moment equilibrium between:

- couple of internal forces in tension chord & in compression chord near W2,
- uniform in-plane load of 1.728kN/m<sup>2</sup> and force reactions to it at walls W1.

Internal lever arm in deep beam  $z \approx L_x/2$  and force in tension chord:

$$(q_E L_x L_y^2 / 8) / (L_x / 2) = q_E L_y^2 / 4.$$

Required steel area:  $A_{s,t\text{-chord}} = \gamma_d q_E L_y^2 / (4 f_{yd}) = 1.1 \times 1.728 \times 25^2 / (4 \times 0.5 / 1.15) = 683 \text{mm}^2$ , i.e.  $683/5 = 136.5 \text{mm}^2/\text{m}$  in the 5m-width of tension chord.

Minimum design moment along chord is in the middle strip, giving surplus  $\Delta A_s = (1 - 8.2/14.7) \times 10.2 / (0.11 \times 0.5 / 1.15) = 94.4 \text{mm}^2/\text{m} < 136.5 \text{mm}^2/\text{m}$ .

Increase reinforcement area between W1 and 1<sup>st</sup> parallel row of interior columns, as well as between any rows of interior columns between the two walls W1, to at least:

$$136.5 \text{mm}^2/\text{m} + M_{g+\psi 2q} / (z f_{yd}) = 136.5 + 8.2/14.7 \times 10.2 / (0.11 \times 0.5 / 1.15) = 255.5 \text{mm}^2/\text{m}$$