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

M.N. Fardis
University of Patras (GR)

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 2nd 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.

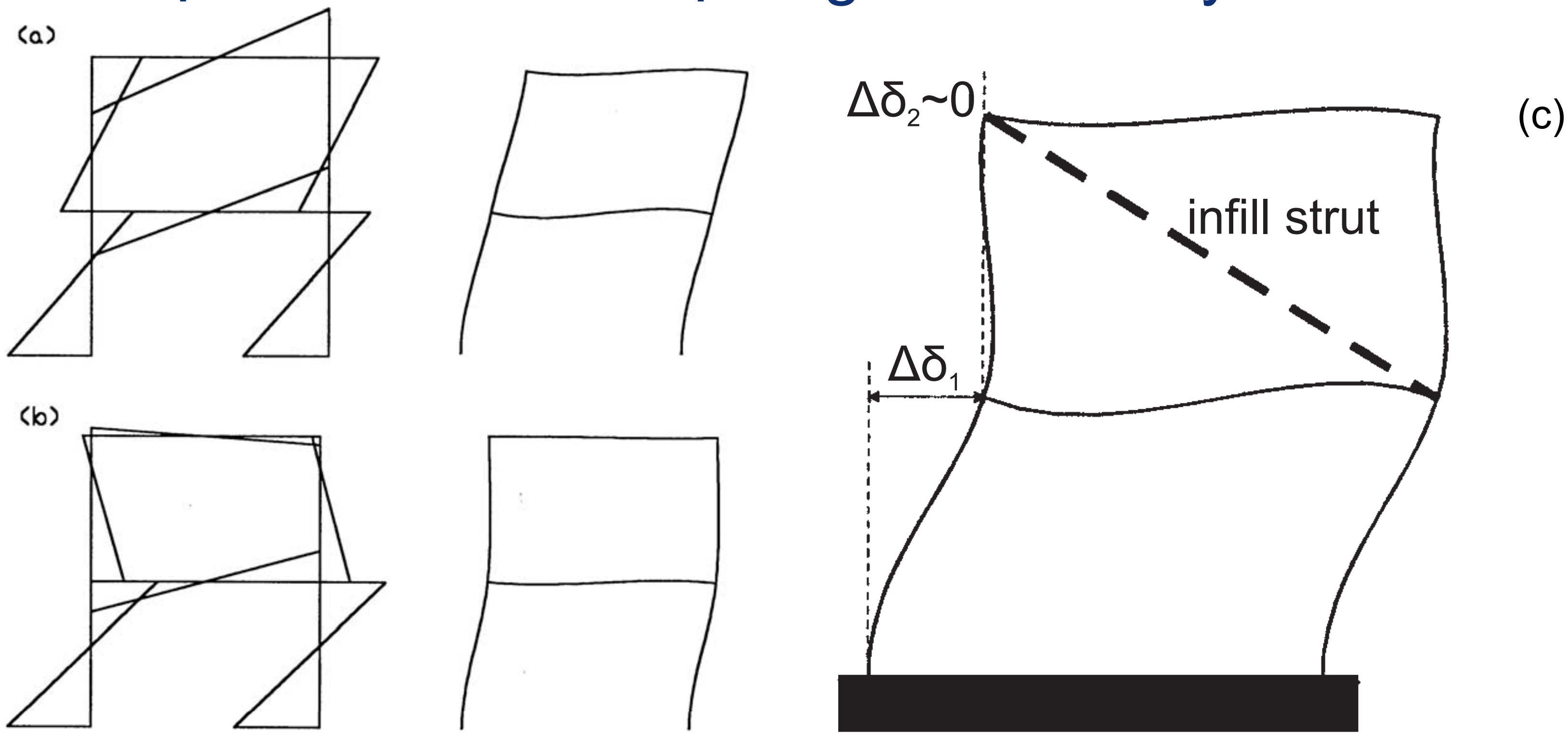
Possible adverse effects of masonry infills *(cont'd)*

- 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 2nd 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:

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

- ΔV_{Rw}: total reduction of resistance of masonry walls in storey concerned w.r.to storey above,
- ΔV_{Ed}: sum of seismic shear forces in all vertical primary seismic members of the storey (storey design shear).
- If η < 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.

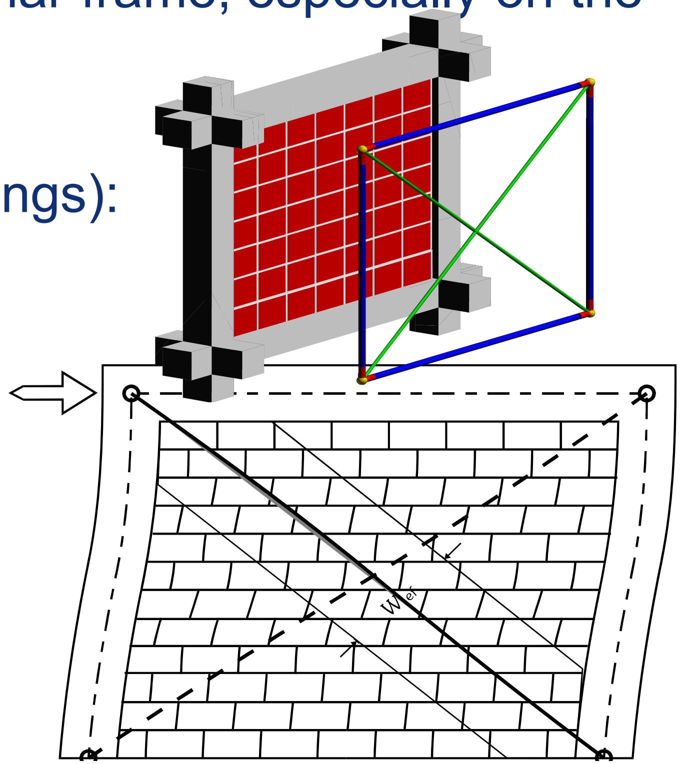
Asymmetry of infills in plan

- 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

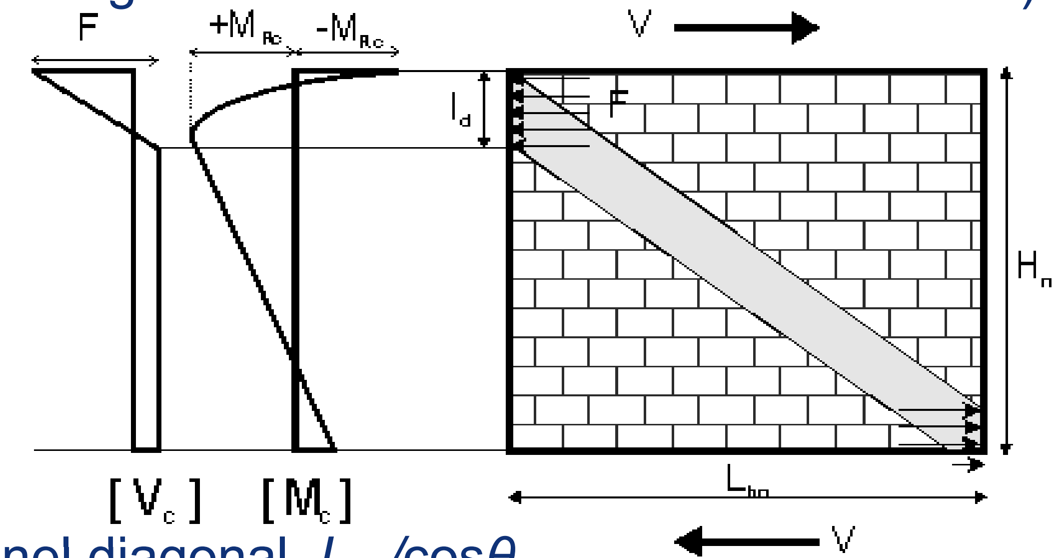
EC8 design against local effect of strong infills

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.):

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

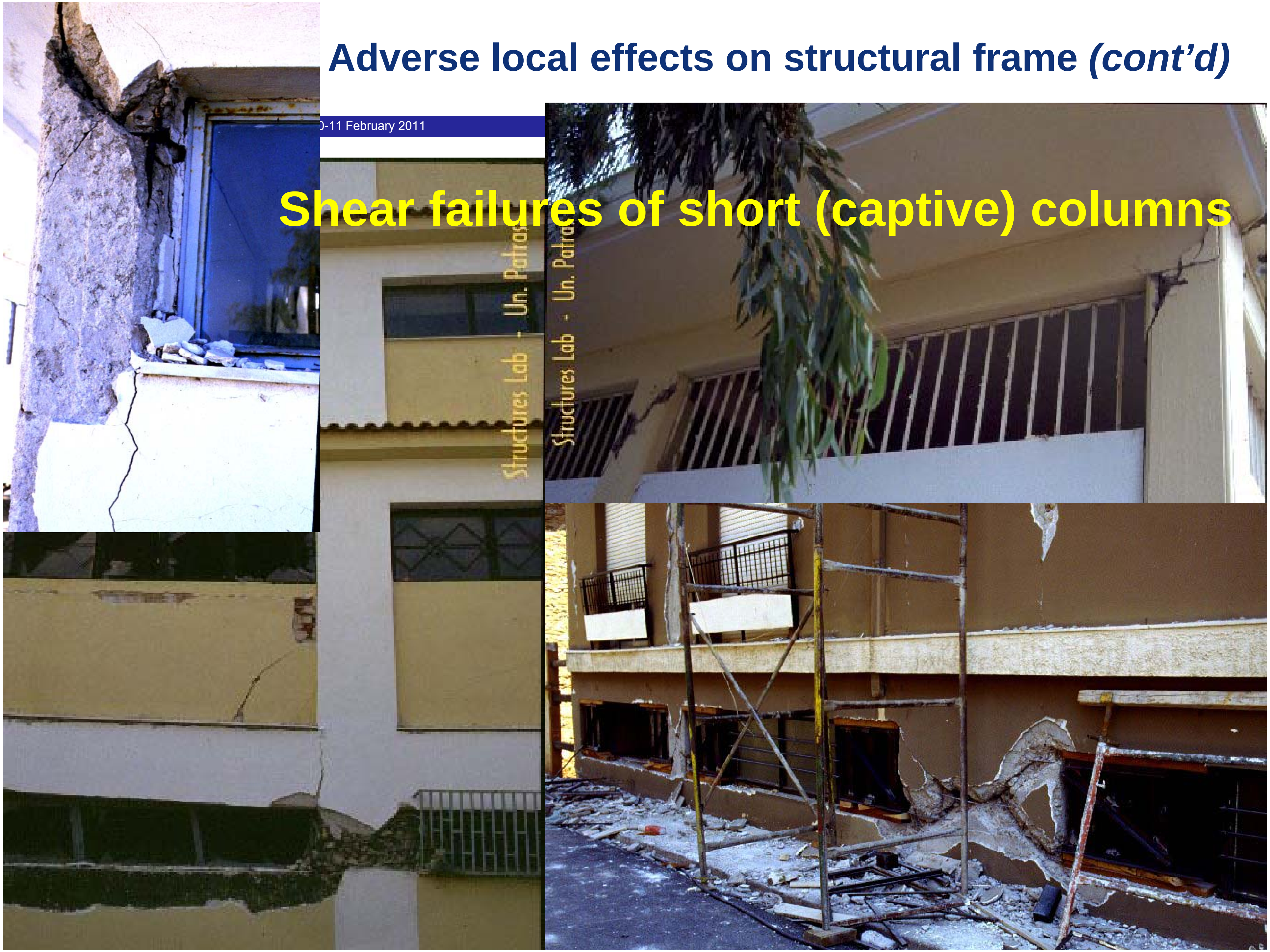
$$\lambda = \left(\frac{E_w b_w \sin 2\theta}{4 E_c I_c H_n} \right)^{\frac{1}{4}}$$



- Eurocode 8: fraction (~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



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

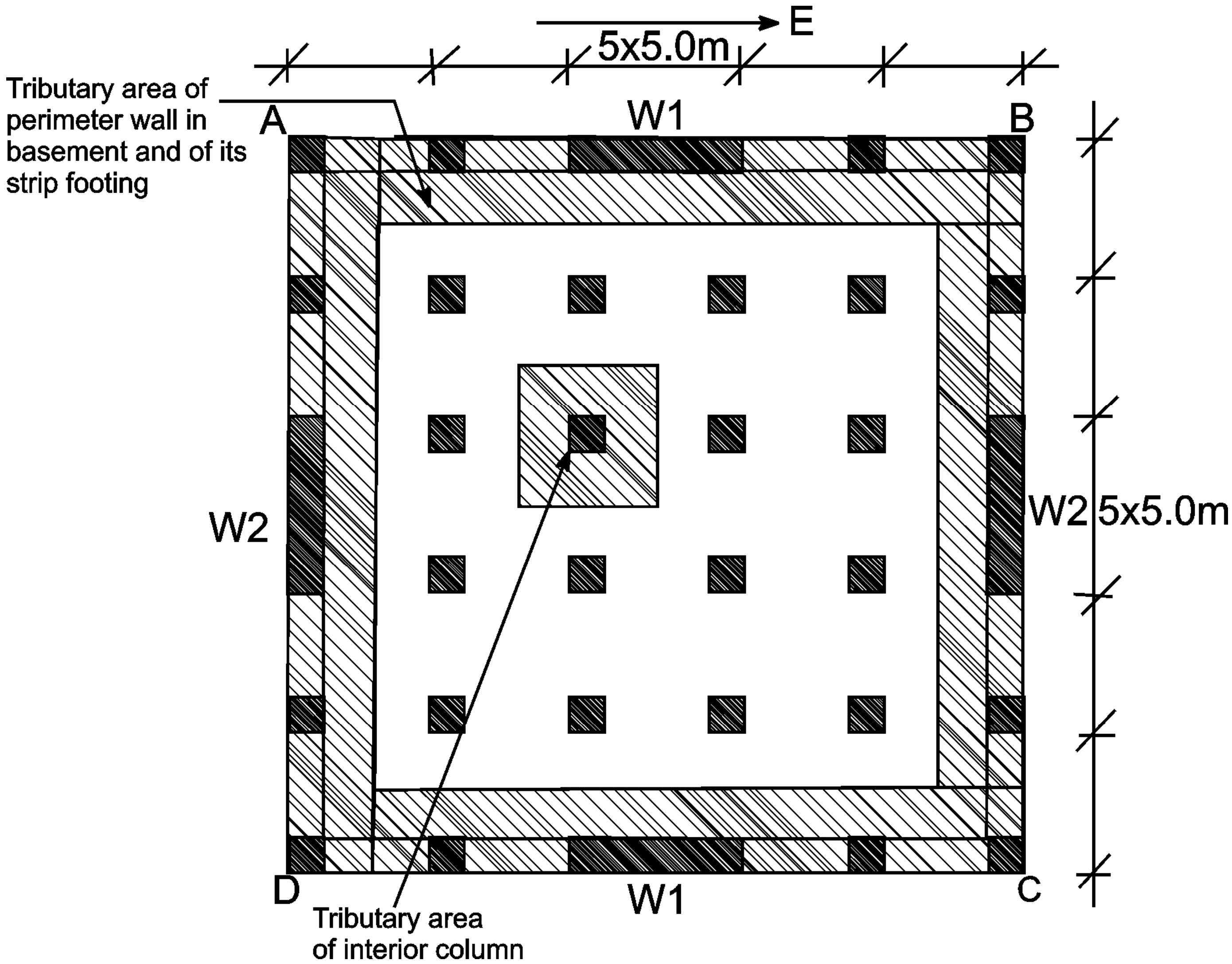
- Contribution of "secondary seismic elements" to resistance & stiffness for seismic actions is discounted in design (& in linear analysis model, too).
- The designer is free to assign elements to this class of elements,if:
  - Their total contribution to lateral stiffness ≤ 15% of that of the other ("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 ≤ 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 ≤ 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=139kN,
  - End moments: 240kNm and 127kNm at top & bottom.
- Maximum M in any interior column:
  - 372kNm, at the ground storey.
  - (V=141kN, 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: ρ=0.342%:

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

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:

ρ_w= (2+√2)x50.25/(165x600)=0.00173.

- Shear resistance for shear compression per EC2:

$V_{Rd,max}$=0.3x(1-35/250)x0.6x0.9x0.565x(35000/1.5)sin2δ=1269kN>$V_{Ed,c;1}$, if cotδ=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:
- $V_{Rd,s}$=0.6x0.9x0.565x0.00173x(500000/1.15)x2.5+205x(0.6-0.084x0.565)/2.65 = 616.5kN>$V_{Ed,c;n}$.
- Ground storey:
- $V_{Rd,s}$= 0.6x0.9x0.565x0.00173x(500000/1.15)x2.5+1435x(0.6-0.249x0.565)/2.65 =822.5kN>$V_{Ed,c;1}$.

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.

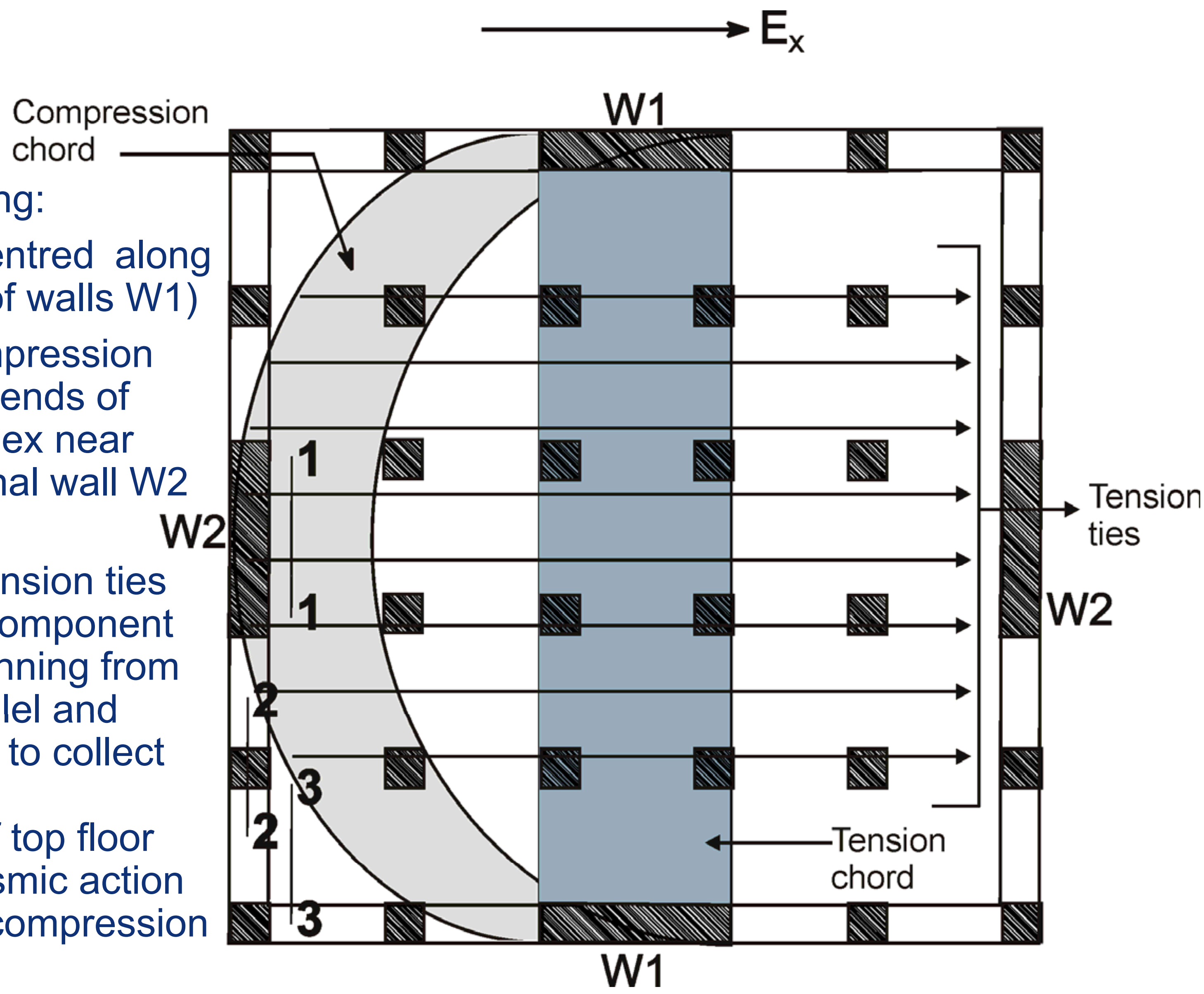
Strut-and-Tie model of diaphragm for check of top floor slab

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=1.728kN/m^2 of top floor due to design seismic action and transfer it to compression chord.



Verification of tension ties *(cont'd)*

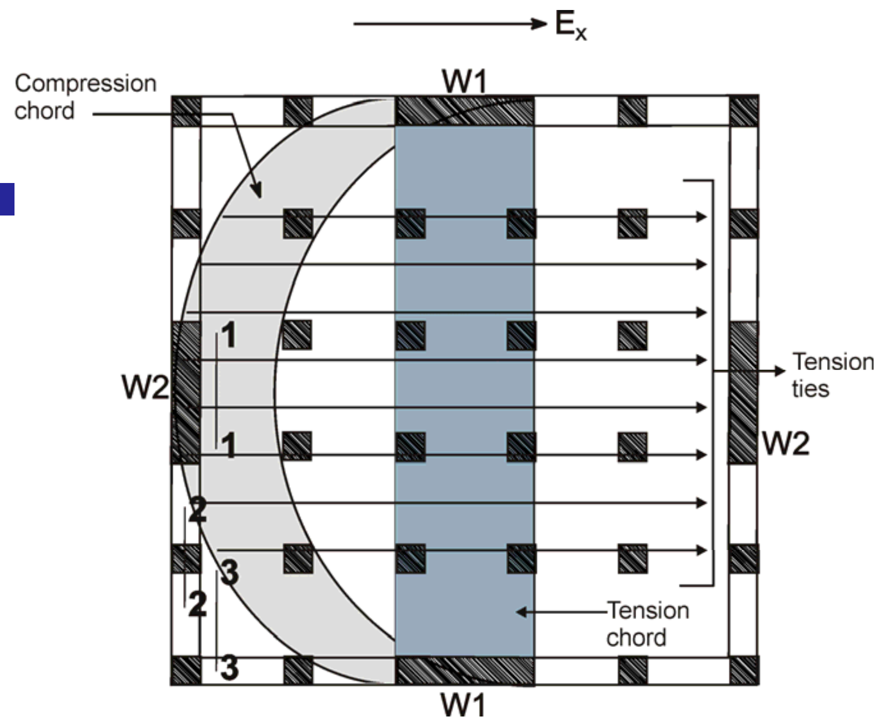
- Longest tension ties collect in-plane load q_E =1.728kN/m^2 along the full plan dimension, L_x. For their ULS verification in tension, any vertical section through the flat slab normal to hor. direction X should have reinforcement area at least $\gamma_d q_E L_x/f_{yd}$=1.1x1.728x25/(0.5/1.15)=110mm^2/m over and above what is required for moment resistance of the flat slab for the moment due to the quasi-permanent floor gravity load, $M_{g+\psi 2q}$.

  (γ_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: ΔA_s=max[$A_{s,min}$; $M_d/(zf_{yd})$]-$M_{g+\psi 2q}/(zf_{yd})$, where z=0.11m 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 ΔA_s: where M_d is minimum.

Verification of tension ties



- Minimum M_d along longest tension ties:

  Sagging moment at mid-distance between W2 and 1st row of interior columns parallel to W2 (Section 1-1)

  – Surplus ΔA_s=(1-8.2/14.7)x10.2/(0.11x0.5/1.15)=94.4mm^2/m <110mm^2/m.

  – Increase reinforcement of flat slab within its middle strips between W2 and the 1st parallel row of interior columns, and between any rows of interior columns, to ≥110+213x8.2/14.7=229mm^2/m.

- Potentially critical: tension ties heading towards the edge column next to W2 (Section 2-2): ΔA_s=(1-8.2/14.7)x47.3/(0.11x0.5/1.15) =438mm^2/m >110mm^2/m

- Between edge columns and 1st parallel row of interior columns (Section 3-3): ΔA_s=(1-8.2/14.7)x37.85/(0.11x0.5/1.15)=350mm^2/m >110mm^2/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^2 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: $A_{s,t\text{-}chord} = \gamma_d q_E L_y^2/(4f_{yd}) = 1.1 \times 1.728 \times 25^2/(4 \times 0.5/1.15) =$ 683mm^2 , i.e. 683/5=136.5mm^2/m in the 5m-width of tension chord.

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

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

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