# CONCEPTUAL DESIGN OF EARTHQUAKE-RESISTANT CONCRETE BUILDINGS

# PRINCIPLES & APPLICATION RULES FOR CONCEPTUAL SEISMIC DESIGN OF CONCRETE BUILDINGS - THEIR RATIONALE

# Importance of conceptual design

- Structural layout can limit deviations of actual, strongly inelastic displacement response to “design seismic action” from that calculated (for member dimensioning) through simplified analysis for presumed elastic response.
- Structural layout is a prime factor for seismic performance/ vulnerability. Observation of damage in strong earthquakes:

  All other design conditions being the same (design code, computational methods & tools, professional skill or design effort), irregular/geometrically complex structures perform on average worse than simple/regular ones.
- Reliable computer codes for elastic analysis of structures in 3D:

  Give designers false confidence to their ability to produce a safe seismic design for very complex/irregular structural layouts.
- Impossible to make up later for poor conceptual design choices (by using sophisticated analysis or extra attention to detailing).
- Hard to achieve optimal structural layout after architectural design has been completed/finalized:

  Good conceptual design: easier if structural engineer interacts w/ architect since early stages of architectural design.

# Fundamental attributes of good structural layout (in buildings):

- Clear structural system.
- Simplicity & uniformity of structural layout.
- Symmetry & regularity in plan.
- Significant torsional stiffness about vertical axis.
- Geometry, mass & lateral stiffness: regular in elevation.
- Lateral resistance: regular in elevation.
- Redundancy of the structural system.
- Continuity of force path, w/o local concentrations of force or deformation demands.
- Effective horizontal connection of vertical elements at all floor levels.
- Minimal total mass.
- No adverse effects of non-structural masonry infills.

# 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 on other beams,

  and/or

  - (essentially) rectangular shear walls,

  arranged in two orthogonal horizontal directions.
- Clear (expected) inelastic response mechanism (location of plastic hinges), w/o excessive reliance on mechanistic application of strong column/weak beam rule:
  - avoid (significant) reduction of cross-section of vertical elements from one storey to the next,
  - select from the outset big column cross-sections

# Simplicity & uniformity in structural layout

- At every storey the seismic force/deformation demands will be uniformly distributed to all members of the same type, w/o concentration of deformation demands to a single location and early failure,

  **<u>if</u>**, in each one of the two orthogonal horizontal directions, the structural system consists of:
  - few identical, regularly arranged shear walls, or
  - identical, regularly spaced plane frames w/ bays of same length & member cross-sections

    (if the two exterior columns of such a frame have ~half effective cross-sectional stiffness, $(EI)_{eff}$, & flexural resistance, $M_R$, compared to interior columns → seismic bending moments & chord rotations ~same at all beam ends of a storey).
- But: No redundancy → all plastic hinges will develop simultaneously; little overstrength after formation of 1$^{st}$ plastic hinge; little opportunity to redistribute forces.

# 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).
- Asymmetry 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), or
  - centre of resistance (CR, important in the inelastic response).
- One of EC8 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:

$$x_{CS} = \frac{\sum (xEI_y)}{\sum (EI_y)}; \qquad y_{CS} = \frac{\sum (yEI_x)}{\sum (EI_x)}$$

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

- Another EC8 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/ period > that 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.

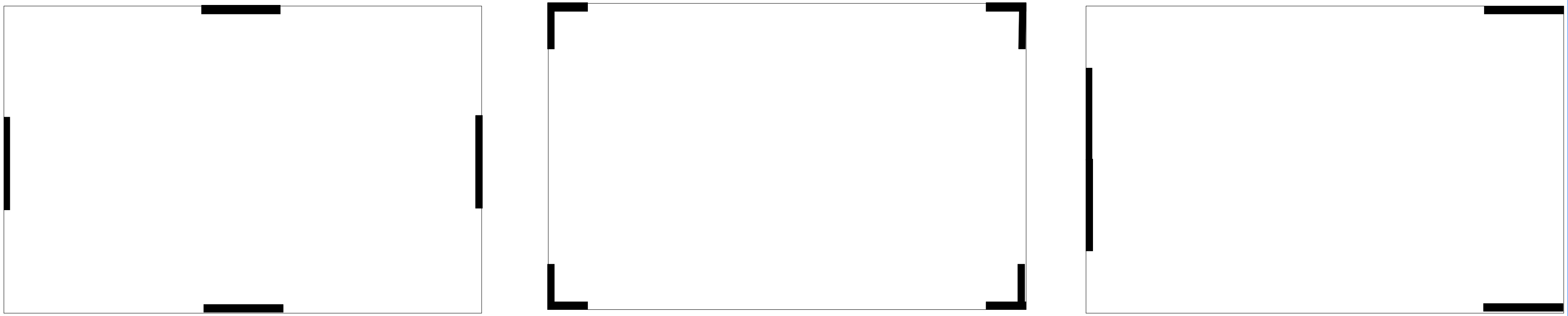
- Avoided through Eurocode 8 criterion for regularity in plan:
  - "torsional radii" $r_x$ (better $r_{mx}$: $r_{mx} = \sqrt{r_x^2 + e_x^2}$) & $r_y$ ($r_{my}$: $r_{my} = \sqrt{r_y^2 + e_y^2}$) >
  - radius of gyration of floor mass in plan $l_s$ = √ ratio of:
    - polar moment of inertia in plan of 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 <u>at the perimeter</u>**



(α) (β) (γ)

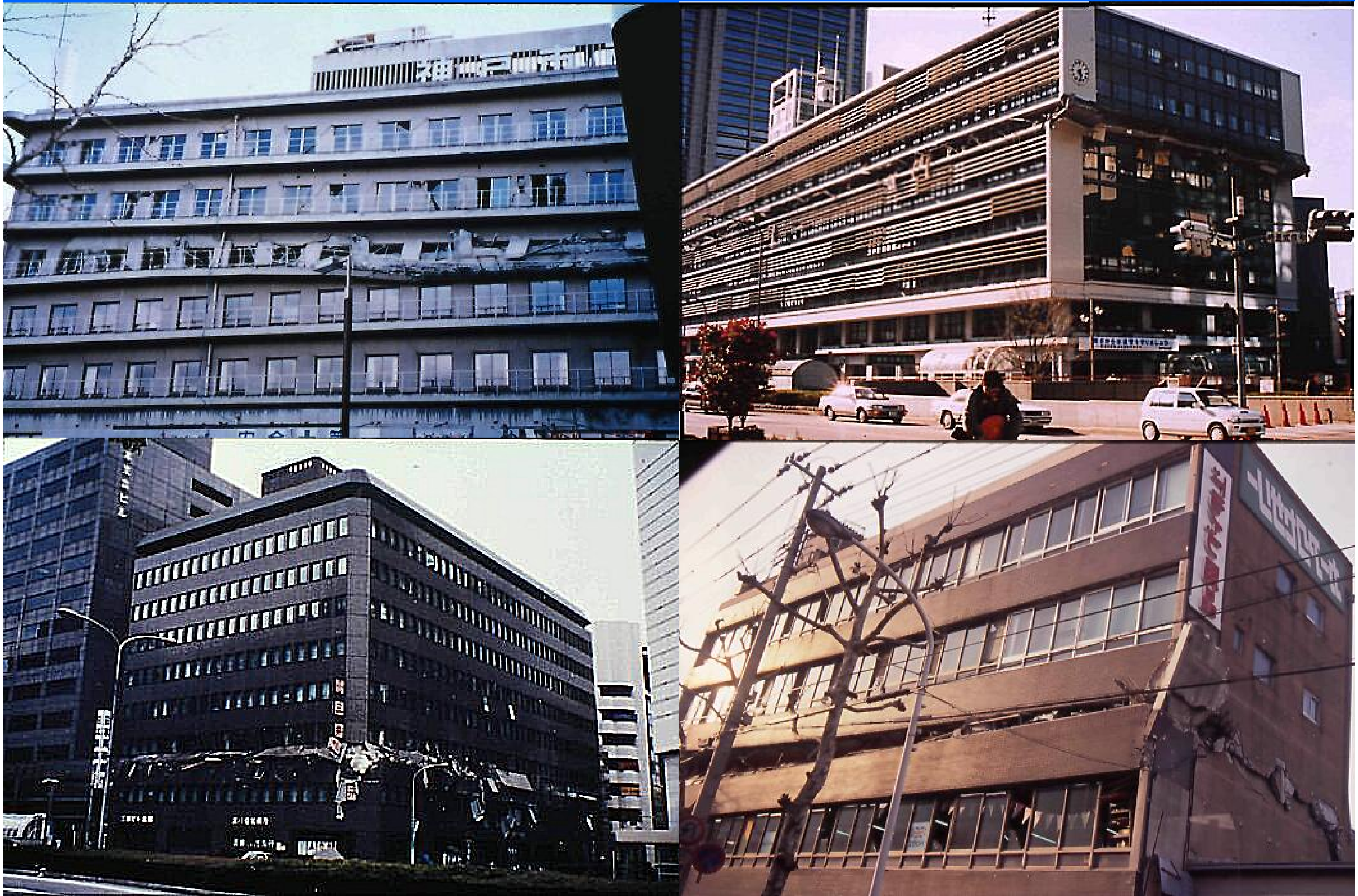
Arrangements of shear walls in plan:

(a) preferable;
(b) drawbacks due to restraint of floors & difficulties of foundation at the corners;
(c) sensitive to failure of individual walls

# Geometry, mass & lateral stiffness: regular in elevation

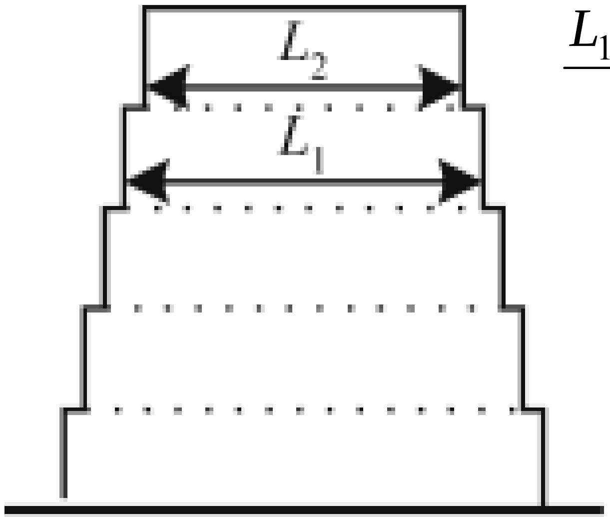
Collapse of upper storeys w/ reduced plan dimensions or stiffness
left: Kalamata (GR) 1986;
right: Kocaeli (TR) 1999.

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

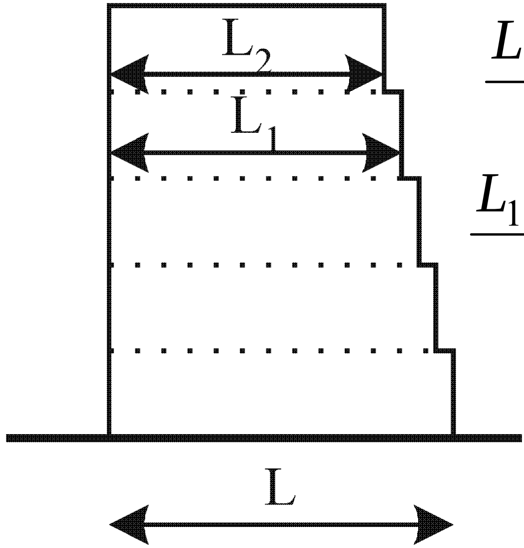


Intermediate story collapses due to abrupt changes in vertical elements (Kobe 1995)

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

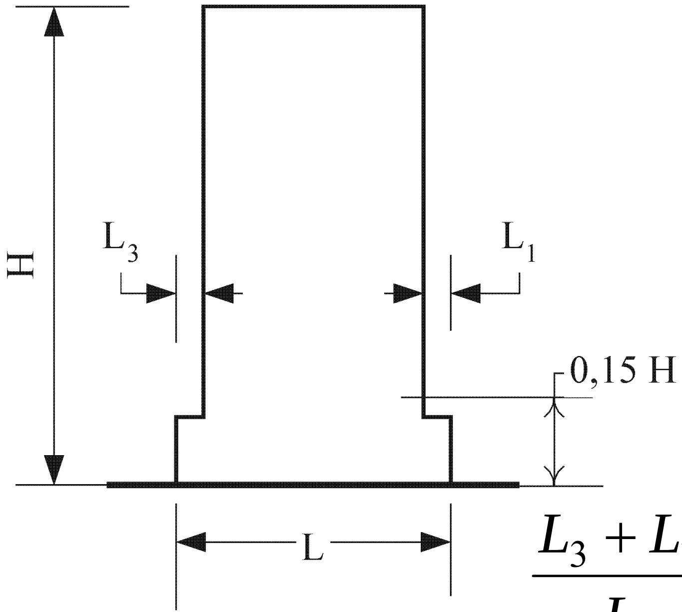



$$\frac{L_1 - L_2}{L_1} \leq 0{,}20$$

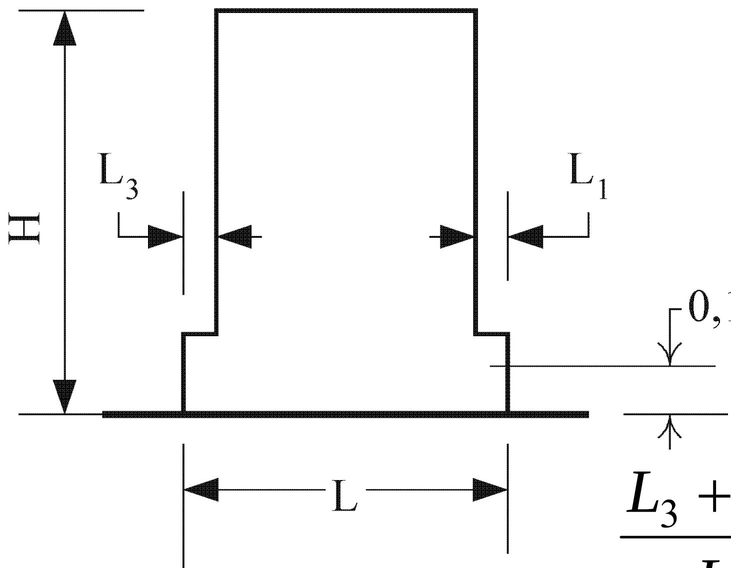



$$\frac{L - L_2}{L} \leq 0{,}30$$

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




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




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

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

# Lateral resistance regular in elevation

Objective:

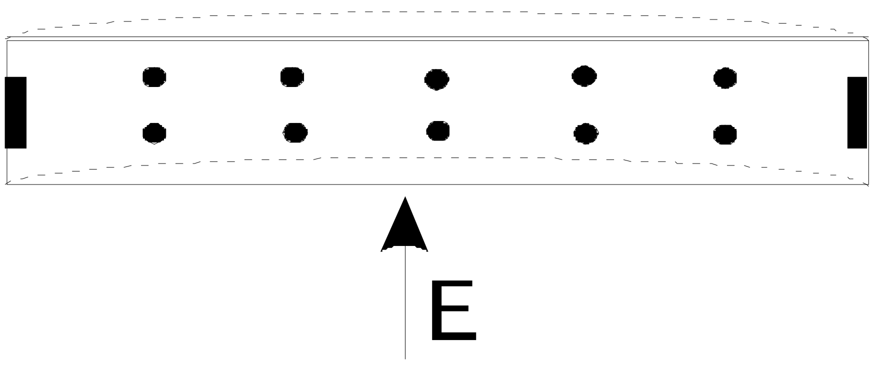
- Avoid soft-storey mechanism:
  - Avoid beam flexural overstrength w.r.to moments from analysis for design seismic action:

    Select beam depth so that:
    - $M_d$ from analysis for gravity load combination ~equal to
    - $M_d$ from analysis for seismic load combination.
  - Make sure that over all beam-column joints at every storey:

$$\Sigma(\Sigma M_{Rb}) < \Sigma(\Sigma M_{Rc})$$

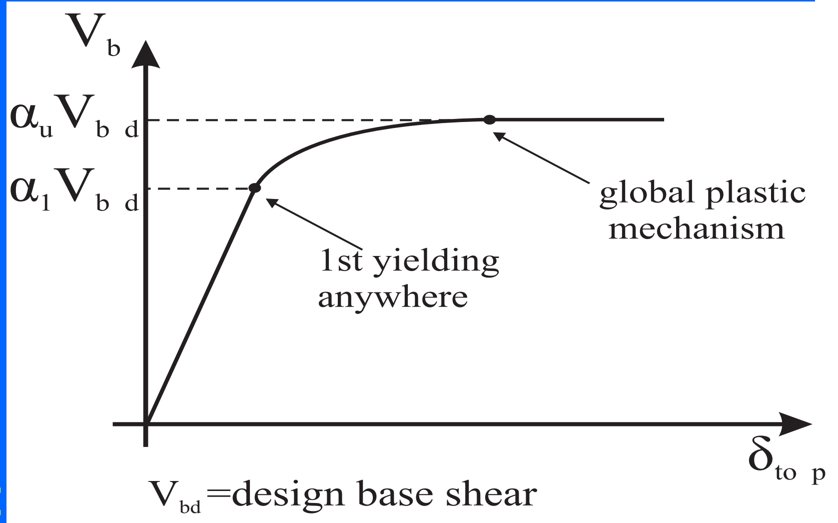
# Redundancy of structural system

- 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, relative to the results of design w/ rigid diaphragm

**Eurocode 8:** Bonus to system redundancy: $q_o$ proportional to $\alpha_u/\alpha_1$ :




**US codes:** Penalty to non-redundant systems:
Divide force reduction factor $R$ by factor $\rho$ ≤1.5 & ≥1 that decreases w/ ratio of:
- max. (among all vert. members in story) seismic shear in single vertical member, to
- total storey shear.

# Continuity of force path, w/o local concentrations of stresses & deformation demands

- Need smooth/continuous path of forces, from the masses where they are generated by the inertia, to the foundation.
- Cast-in-situ RC 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 loads to the lateral-load-resisting system & be adequately connected to it.
- Continuity of lateral-load-resisting system itself may be disrupted, by:
  - strongly eccentric beam-to-column connections,
  - beams supported indirectly (i.e., on other beams or girders),
  - beam axis offset w.r.to that in an adjacent span,
  - column axis offset w.r.to that of an adjacent storey,
  - columns, or walls, supported on a beam or a girder, instead of continuing to the ground,
  - walls supported on two columns, instead of continuouing to the ground
- 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.

## Continuity of force path, w/o local concentrations of stresses & deformation demands (cont'd)

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

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

# Continuity of force path, w/o local concentration of stress & deformation demands (cont'd)





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

# Effective horizontal connection of vertical elements at all floor levels

- Vertical elements of lateral-force resisting system should be connected together, via a combination of floor diaphragms & beams:
  - at all horizontal levels where significant masses are concentrated, and
  - at the foundation level,
  - for the effective transfer of inertia forces from the floor masses to the vertical elements, and
  - to tie-together the system as a whole.

## Effective horizontal connection of vertical elements at all floor levels (cont'd)

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

## Effective horizontal connection of vertical elements at all floor levels (cont'd)

- Solid concrete slabs w/ thickness ≥ 120mm & ≥ min. slab reinforcement at top & bottom in both horiz. directions: sufficient if:
  - lateral stiffness has similar distribution in plan at all storeys,
  - at every floor the slab is at a single horizontal level (no step-wise arrangements),
  - the slab continuity in plan is not impaired by large openings.
- If one or more important vertical elements are discontinued vertically, the diaphragm has to transfer horizontally also shear forces from certain locations in plan to others:
  - e.g. at the basement top slab: from all interior vertical elements to the basement perimeter wall.
- Diaphragms:
  - between strong & stiff vertical elements (walls) far apart from each other, or
  - in buildings with a L-, T-, U-, H-shaped plan, etc., w/o seismic joints between individual rectangular parts, or
  - w/ large openings for patios, etc.

  → develop large in-plane flexural stresses: need strong beams along the edge of the diaphragm, especially at re-entrant corners

# Minimal total mass

- The peak elastic base shear & the peak elastic or inelastic displacement demand are proportional to:
  - the total mass of the building, ***M***, if the fundamental period, ***T***, is in the constant spectral pseudo-acceleration range of the response spectrum, or
  - to $\sqrt{M}$, if ***T*** is in the constant spectral pseudo-velocity range.
- Reduction of ***M*** should be pursued through:
  - light finishings, claddings & veneers in the building,
  - thickness of concrete slabs = minimum required for serviceability, durability, fire rating & strength under gravity loads & for their role as diaphragms under seismic loading,
  - lightweight partitions & exterior walls (but not at the expense of damage limitation requirements for seismic loading).

# No adverse effects of non-structural infills

## Overall effect of masonry infills

- Field experience & numerical/experimental research 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 may be a $2^{nd}$ line of defence & a source of significant overstrength.

## No adverse effects of non-structural infills (cont'd)

## Current position of EC8 on masonry infills

- Eurocode 8 does not encourage designers to profit from the beneficial effects of masonry infills by reducing 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.

## No adverse effects of non-structural infills (cont'd)

## Possible adverse effects of masonry infills

- Infills that are too strong & stiff relative to the concrete structure itself

  → may overrule its seismic design and the efforts of the designer & intent of codes to control the inelastic response by spreading the inelastic deformation demands throughout the entire 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 one part of the structure.
- Adverse local effects on structural frame

  → pre-emptive brittle failures.

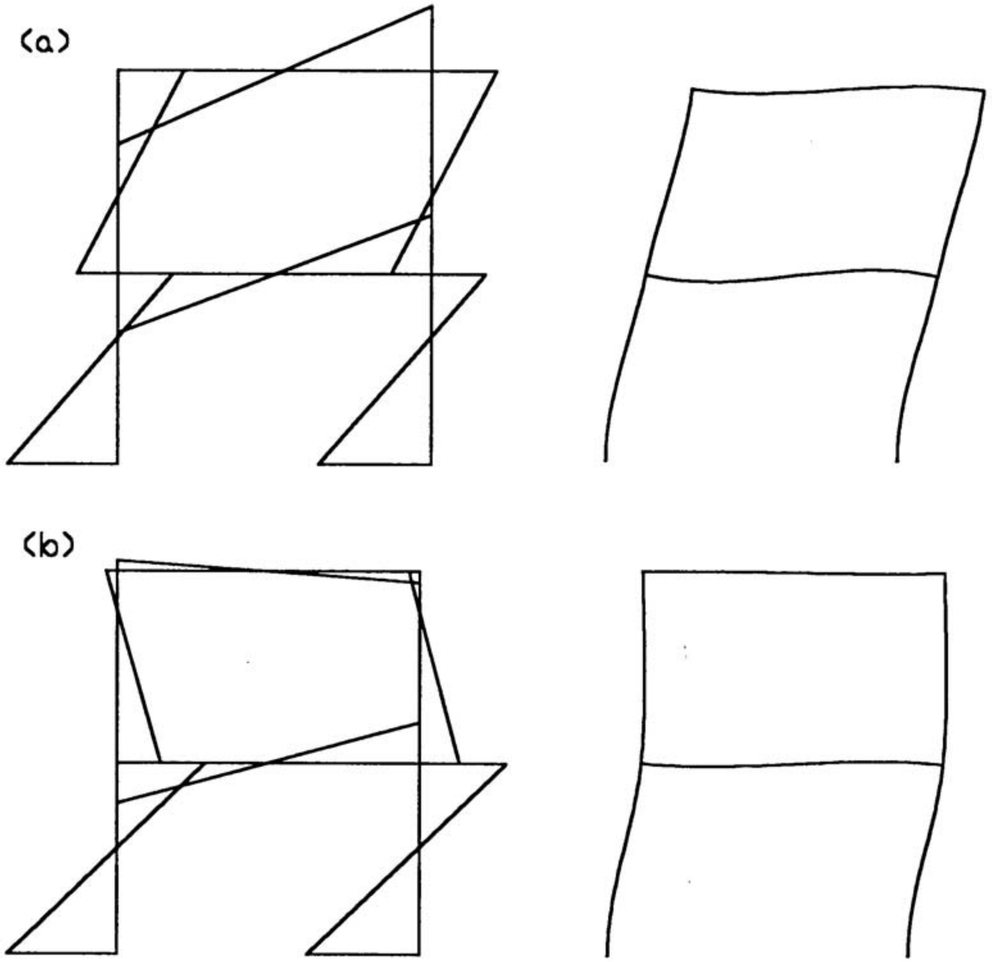
## No adverse effects of non-structural infills (cont'd)

- Best way to protect concrete building from adverse effects of irregular masonry infilling:

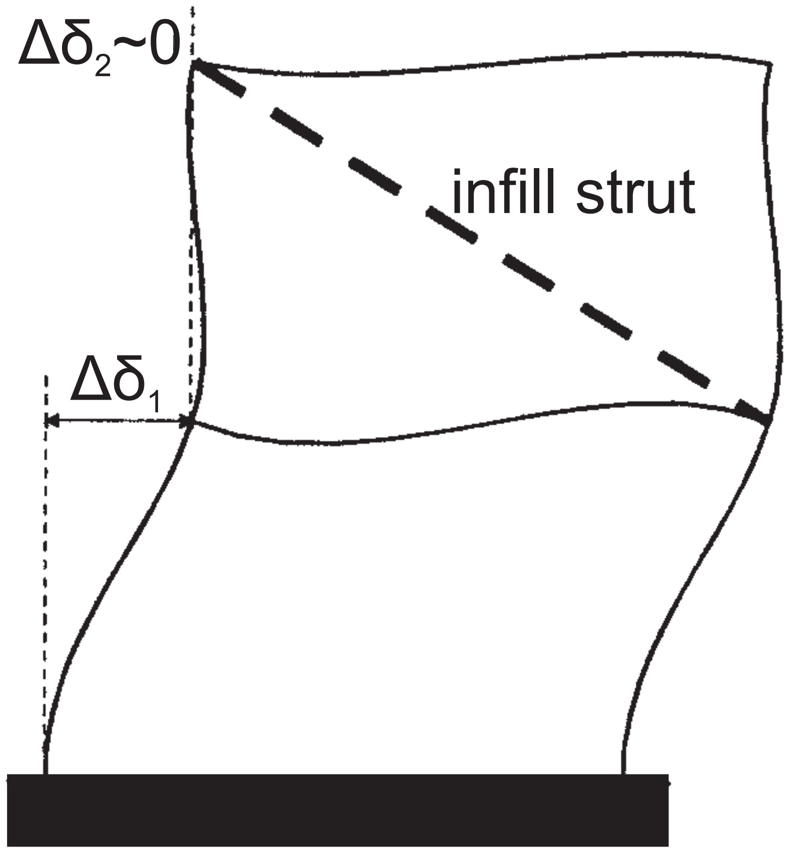
  shear walls sufficiently strong/stiff to overshadow the infilling.
- Eurocode 8:
  - Shear walls that resist at least 50% of the seismic base shear are considered sufficient for waiving the special requirements for buildings with infills.

# No adverse effects of non-structural infills (cont'd)

## Worst possible effect: Open ground storey→ soft-storey




(c)




2-storey frame: Protection of elements in infilled storey from large moments & deformations - overloading of ground storey columns:

(a) bending moments & deformation in frame w/o infills;

(b), (c) bending moments & deformation in frame w/ stiff infills in 2nd storey.

# No adverse effects of non-structural infills (cont'd)
## 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

## No adverse effects of non-structural infills (cont'd)

# EC8 design for infill heightwise irregularity

- Eurocode 8: design columns of storey where infills are reduced relative to overlying storey, to remain elastic till the infills in the storey above reach their ultimate force resistance:
  - A deficit in infill shear strength in a storey is compensated by increased resistance of the frame (vertical) members there:
  - In DC H frame or frame-equivalent dual buildings, seismic internal forces 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,
    - $\Sigma V_{Ed}$ : sum of seismic shear forces in all vertical primary seismic members of storey (storey design shear force).
  - If $\eta$ < 1.1, magnification of seismic action effects may be omitted.

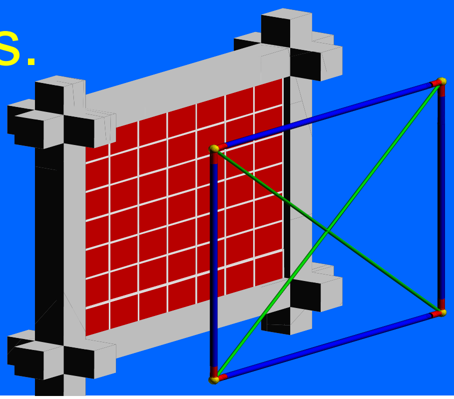
## No adverse effects of non-structural infills (cont'd)
## Asymmetry of infills in plan

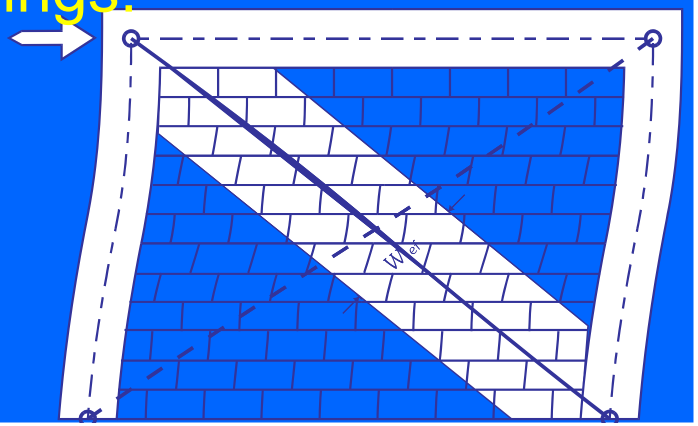
- Asymmetric distribution of infills in plan → torsional response to the translational horizontal components of the seismic action:
  - Members on the side with fewer infills ("flexible" side) have larger deformation demands & fail first.
- The increase in global lateral strength & stiffness due to the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

# No adverse effects of non-structural infills (cont'd)

## EC8 design against infill asymmetry in plan

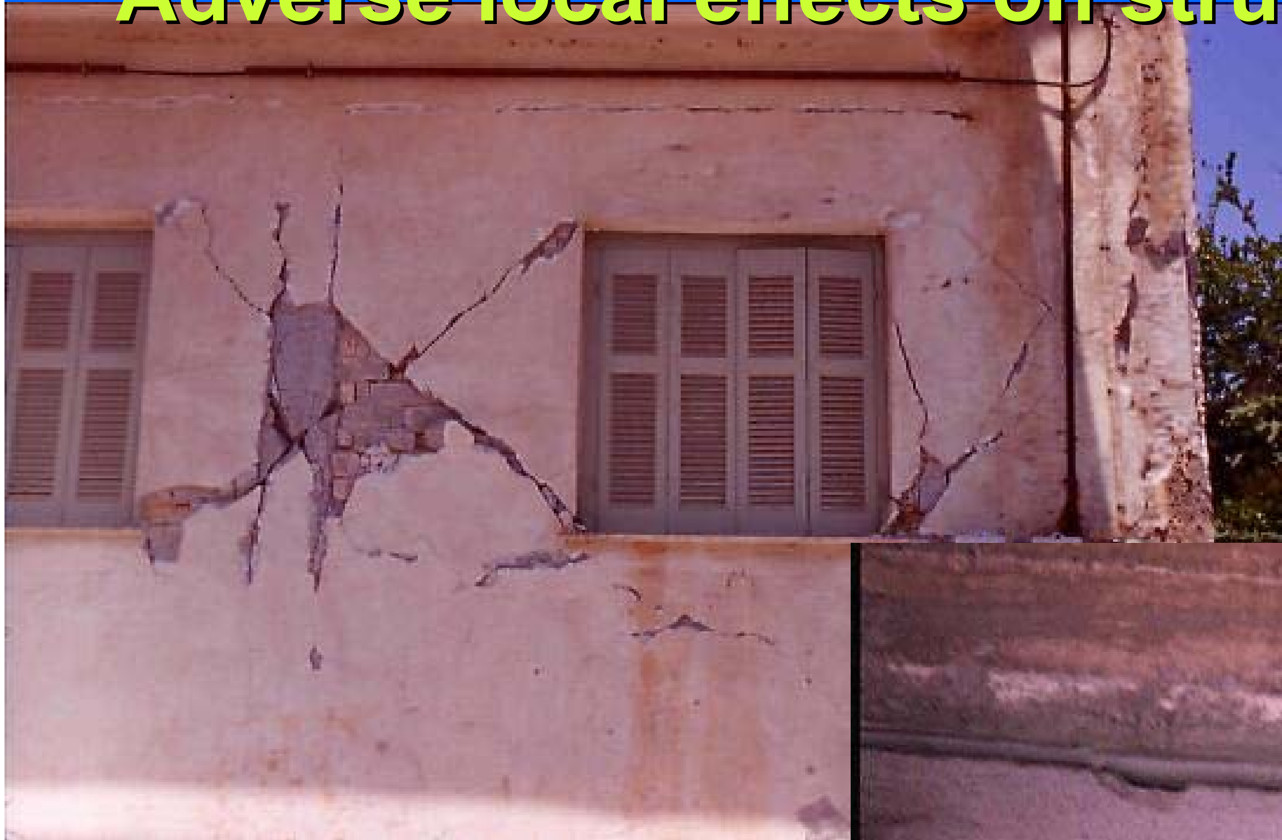
- Eurocode 8: doubles accidental eccentricity (from 5 to 10%) in the analysis, if infills are planwise irregular.
- Doubling of accidental eccentricity: is not enough for "severely irregular" arrangement of infills in plan →
  - analysis of 3D structural model explicitly including the infills,
  - sensitivity analysis of the effect of stiffness & position of infills (neglect one out of 3-4 infill panels per planar frame, especially on flexible sides).
- <u>But:</u>
  - No guidance is given for in-plane modelling of infills.
  - Simplest modelling of panel w/o openings:
    - two diagonal struts.
  - Effect of openings?





# No adverse effects of non-structural infills (cont'd)

## Adverse local effects on structural frame



Shear failure of weak columns due to interaction with strong infills

# No adverse effects of non-structural infills (cont'd)

# EC8 design against local effect of strong infills

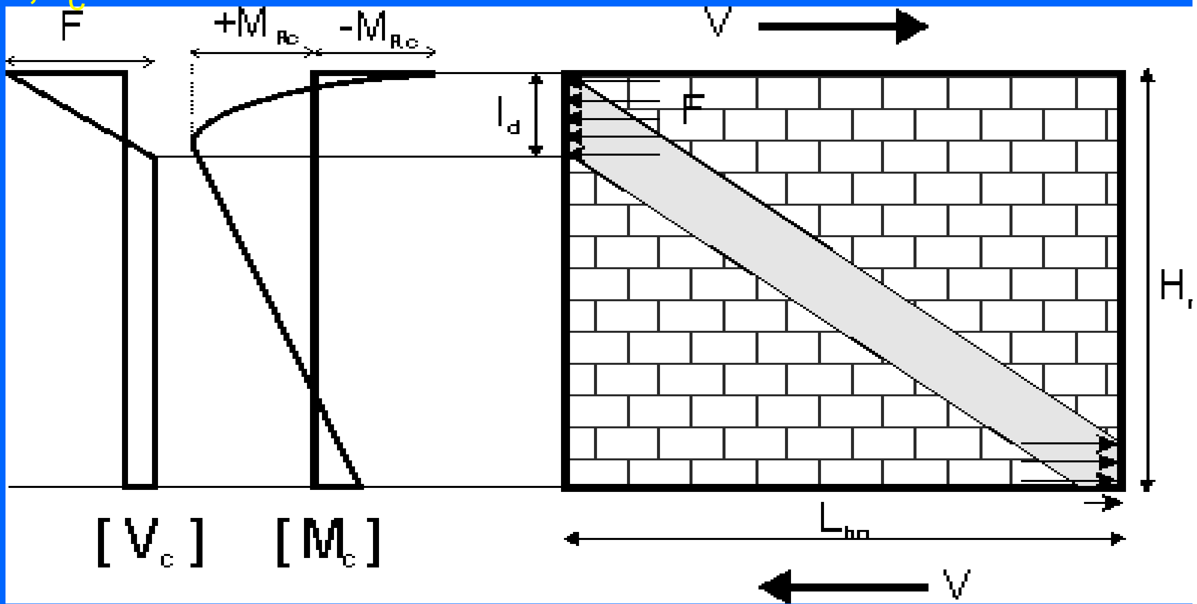
## Shear loading of column by infill strut force:

- Eurocode 8: verify in shear the length $l_c = w_{inf}/\cos\theta$, at top & bottom of column where diagonal strut force of infill may be applied, for the smaller of the two design shear forces:
  - Horizontal component of infill strut force, equal to the horizontal shear strength of the panel (shear strength of bed joints times horizontal cross-sectional area of panel); or
  - Capacity design shear: 2x(design value of column flexural capacity, $M_{Rd,c}$) divided by contact length, $l_c$

Width of strut:

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

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




Eurocode 8: fraction (~15%) of panel diagonal, $L_{bn}/\cos\theta$

# No adverse effects of non-structural infills (cont'd)

## Adverse local effects on structural frame

### Shear failures of squat (captive) columns



## No adverse effects of non-structural infills (cont'd)

# EC8 rules for squat "captive" columns

- Capacity-design calculation of design shear force, w/:
  - clear length of column, $l_{cl}$ = length of column not in contact to the infills &
  - plastic hinging assumed to take place at the section of the column at the end of contact w/ the infill wall.
- Transverse reinforcement required to resist the design shear force is placed not just along the clear length of column, $l_{cl}$, but also along a part of the column in contact to the infill (over a length equal to the column depth, $h_c$, within the plane of the infill).
- The full length of the column is taken as a "critical region" & stirrups follow rules for "critical regions".

# Frame, wall, or dual systems for concrete buildings

# Seismic behaviour & conceptual design of frame systems

- Frames resist the seismic storey shears, $V_{storey}$, through bending moments in the columns:
  - The algebraic difference of the moment between top & bottom of a column, gives the column's contribution to $V_{storey}$.
- The seismic overturning moment in the building gives axial forces N in the columns ($N > 0$ at one side of the plan, $N < 0$ at the opposite).
- In ~regular frames, column inflection points ~close to storey mid-height: the shear span at the column end with the maximum bending moment among the two ends, ~ 1/2 to 2/3 of the column clear length:
  - Column shear ratio, M/Vh, normally > 2.5: if column dimensioned so that it doesn't fail in shear before it yields in flexure →

    Column inelastic behaviour & ultimate deformation governed by bending (by normal action effects, M & N) → ductile.

# Seismic behaviour & conceptual design of frame systems (cont'd)

- Ideal geometry of (plane) frame:
  - same span length in all bays,
  - constant cross-section of beams & interior columns throughout each storey,
  - (effective) rigidity of two exterior columns ~50% of interior ones.
- Then, elastic seismic bending moments:
  - the same at ends of all beams in a storey,
  - the same at all interior column tops or bottoms of the storey;
  - in the two exterior columns they will be 50% of those in interior ones.
- Very long beam spans:
  - top reinforcement at supports governed by gravity loads → unfavourable for:
    - capacity design moments of columns at joints,
    - capacity design shears in beams or columns.
  - flexural overstrength in beams w.r.to seismic demands → uncertainty about inelastic response & plastic mechanism.
- Very short beam spans:
  - very high seismic shears from the analysis & from capacity design
  - almost full reversal of sign of beam shears → sliding shear failure at the beam ends; beam needs diagonal reinforcement (hard to place!)
  - low shear span ratio (often below 2.5) → unless beam is diagonally reinforced, its deformation capacity is controlled by shear and, hence, is very low.
- Optimum beam span for earthquake resistance of frame :
  - For common storey heights & ordinary gravity loads: 4 – 5 m

# Pros of frames

- Beams & columns: inherently ductile, easy to detail for ductility.
- High redundancy; multiple load paths.
- (If frame has concentric connections & regular geometry) Less uncertainty about the seismic response:
  - the seismic performance of frames & their members has been thoroughly studied experimentally & analytically;
  - frames are easier to model & analyze for design purposes.
- Certain features make frames cost-effective for earthquake resistance:
  - Beams & columns are needed anyway for gravity loads → Why not use them for earthquake resistance as well?
  - Columns have ~same strength & stiffness against both horizontal components of seismic action;
  - It is easy to design the foundation of smaller vertical elements (columns) than of larger ones; each foundation element takes a small fraction of the seismic base shear.

## Seismic behaviour & conceptual design of frame systems (cont'd)

# Cons of frames

- Inherently flexible:
  - poor protection from damage under more frequent earthquakes;
  - larger sensitivity to presence & irregularity of infills.
- Tendency to form soft-storey, due to column counter-flexure within a storey.
- Some elements of uncertainty:
  - Effect of eccentric connections & of irregular geometry?
  - Effective slab width in tension? ( → if large, more likely to have plastic hinges in columns)
  - Behaviour of columns in biaxial bending w/ varying axial force?
  - Possibility of column plastic hinging under biaxial moment demands from beams connected to the column in two horizontal directions.
- Certain features make frames less cost-effective:
  - Large member sizes for damage limitation → vertical reinforcement may be controlled by minimum requirements.

# Seismic behaviour & conceptual design of wall systems

- Conventional (code) definition of a wall:
  - ≠ column in that the cross-section is elongated ($l_w/b_w>4$):
    - A wall resists lateral forces primarily in one horizontal direction;
    - The wall is designed for such unidirectional resistance by assigning:
      - flexural resistance to the opposite ends of the section (“flanges”, or “tension & compression chords”): vertical reinforcement is concentrated & concrete confinement is limited there, and
      - shear resistance to the “web” in-between the ends (horizontal reinforcement & development of the diagonal compression field),

    i.e., as in beams

## Seismic behaviour & conceptual design of wall systems (cont'd)

- Walls resist directly both:
  - the seismic storey shears, and
  - the seismic overturning moments (resisted by bending moments - not by N's - in individual walls)

  → Wall's bending moment is large & its shear span $L_s$ (M/V-ratio) is long.

- Wall shear span, $L_s$:
  - If beams are very flexible, wall is ~vertical cantilever subjected to horizontal forces at storey levels: $L_s$ ~2/3 of wall height, $H_{tot}$.
  - For usual beam sizes:
    - $L_s$ ~0.5$H_{tot}$, if wall length $l_w$ is fairly large,
    - $L_s$ ~1.5 x storey height, if $l_w$ is short (~4$b_w$).

## Seismic behaviour & conceptual design of wall systems (cont'd)

# Wall cost-effectiveness for earthquake resistance

(for given $V_{storey}$ & concrete volume, i.e. $\Sigma bh$ of vert. members)

- Member shear resistance, $V_w$, $V_c$ or $V_{Rmax}$: is proportional to $h$ $\longrightarrow$
  - Storey shear resistance depends only on $\Sigma h$ of vertical elements.
- Member rigidity, EI, is proportional to $h^3$ $\longrightarrow$
  - More cost-effective: to lump $\Sigma bh$ to few members with large $h$.
- For the vertical reinforcement:
  - $\mu$ = M/(bh²f$_c$) = (L$_s$/h)·V/(bhf$_c$) = (L$_s$/h)·υ $\longrightarrow$
  - in order to save vertical steel (i.e $\mu$) reduce $L_s/h$, i.e (as $L_s$~fixed):

Increase $h$ (=$l_w$) By how much? As much as flexure-controlled behaviour permits:

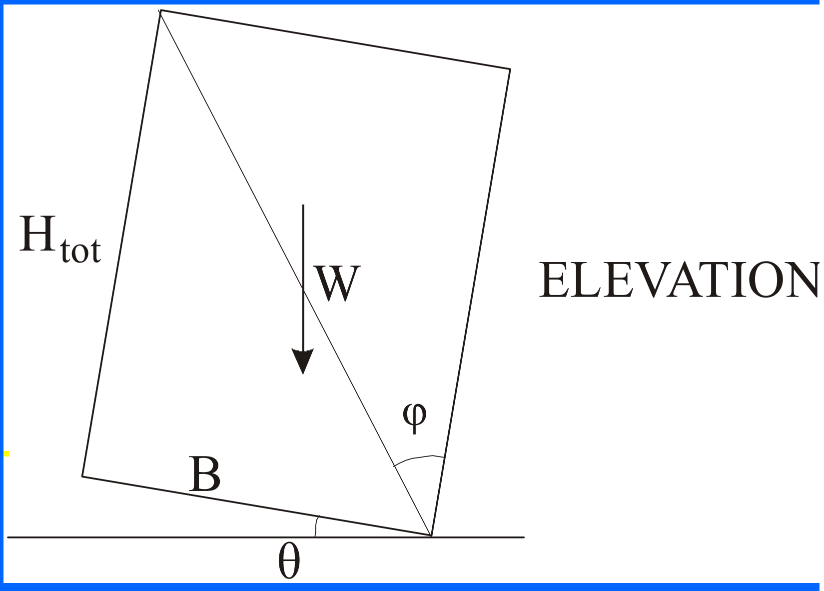
e.g. $L_s/h=3$ $\longrightarrow$ (for $L_s \sim 0.5H_{tot}$ & $H_{storey} \sim 3$m) $l_w = h \approx H_{tot}/6 \approx n_{storey}/2$

## Seismic behaviour & conceptual design of wall systems (cont'd)

# It is difficult to provide a fixed foundation to a wall

- Large $l_w(=h)$ →
  - large moment at the base
  - (for given axial load) low normalized axial force $v=N/(bhf_c)$~0.05.
- Footing of usual size w/ tie-beams of usual size are insufficient:
  - Max normalized moment $\mu = M/(bh^2f_{cd})$ that can be transferred to the soil:
  - $\mu$ ~0.5$v$, i.e. ~wall cracking moment! →

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




~Large wall on large footing:

Rocking → radiation damping in the soil.

Rotation of rocking wall:

$\theta \sim S_v^2/Bg << \varphi = \arctan(B/H_{tot})$ →

Very stable nonlinear-elastic behaviour; but hard to consider in analysis or design

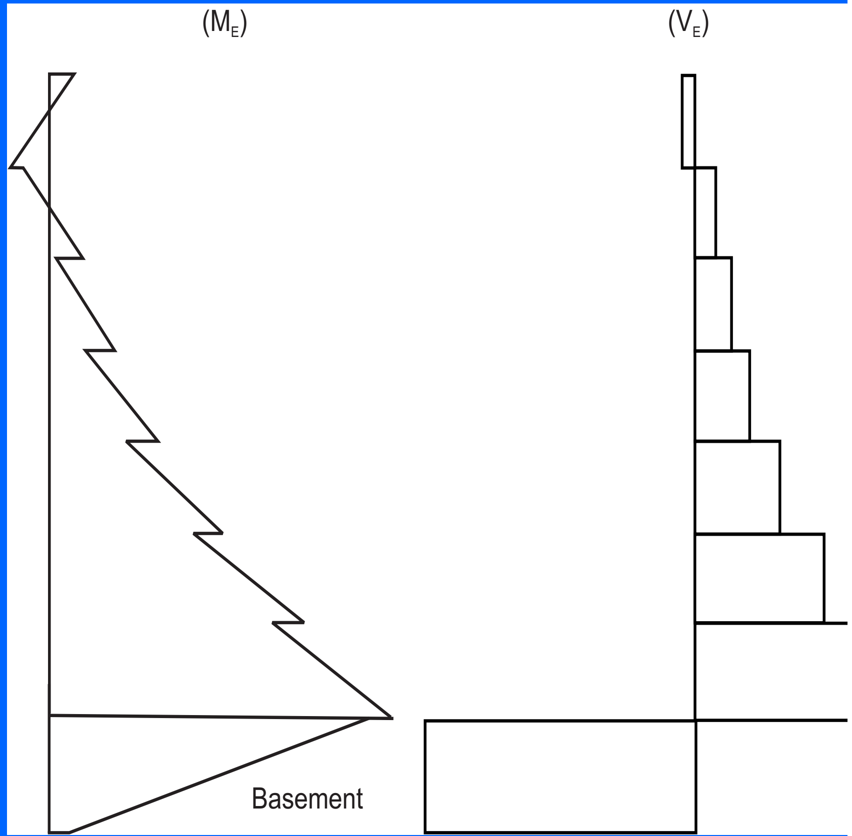
# Seismic behaviour & conceptual design of wall systems (cont'd)

## The wall foundation problem (cont'd)

- To form plastic hinge at wall base → Need fixity there:
  - Very large & heavy footing; adds own weight to N & does not uplift; or
  - Fixity of the wall in a "box type" foundation system:

1. Wall-like deep foundation beams all along perimeter of the foundation (possibly supplemented w/ interior ones across full length of foundation system) = main foundation elements transferring seismic action effects to the ground.

   In buildings w/ basement: perimeter foundation beams may also serve as basement walls.
2. Slab - designed to act as a rigid diaphragm - at the level of the top flange of perimeter foundation beams (e.g. basement roof).
3. Foundation slab, or two-way tie-beams or foundation beams, at the level of the bottom of the perimeter foundation beams.



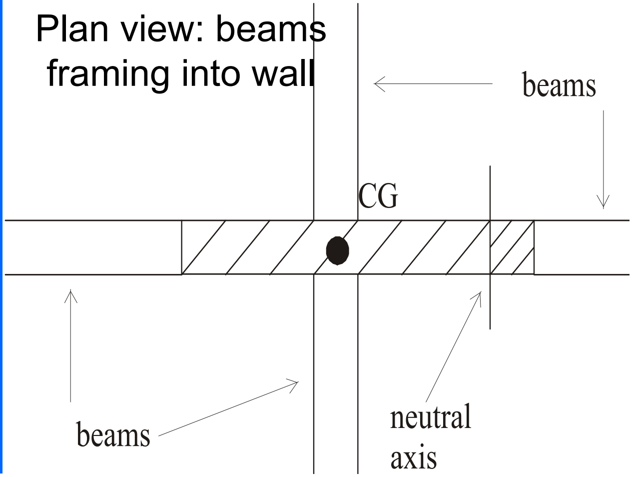

Fixity of interior walls is provided by couple of horizontal forces between 2 & 3 → High reverse shear in the part of the wall within the basement

## Seismic behaviour & conceptual design of wall systems (cont'd)

# Geometric effects in large walls, due to rocking or plastic hinging

- Rotation of uplifting/rocking wall takes place about a point close to the toe of its footing.
- Rotation at wall plastic hinge at the base takes place about a neutral axis close to the edge of the wall section.
  - The ends of beams framing into the wall move mostly upwards →
- In both cases centroid of wall section is raised during rotation:
  - The Centre of Gravity (CG) of masses supported by wall raised too → (temporary) harmless increase in potential energy, instead of damaging deformation energy;

    beam moments & shears: stabilize the wall.
- Wall responds as a "stack" of rigid blocks, uplifting at the base & at hor. sections that crack & yield (storey bottom). The favourable effects are indirectly taken into account in design → q-factor



# Two types of walls in Eurocode 8

- "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 horiz. dimension $l_w \geq 4$m, is expected to develop during design EQ little 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 the base, or connectivity with transverse walls preventing plastic hinge rotation at the base, the wall cannot be designed for energy dissipation in a plastic hinge at the base.

# Large, lightly reinforced wall (systems) in EC8

- Large walls (even when lightly reinforced):
  - preclude (collapse due to) soft-storey mechanism,
  - minimize nonstructural damage,
  - have shown satisfactory performance in strong EQs, partly due to geometric effects (uplift of CGs, radiation damping, etc).
  - cannot (be designed to) form plastic hinge at the base.
- For large walls, code minimum reinforcement of ductile walls →
  - very high cost;
  - flexural overstrength that cannot be transmitted to

# Seismic behaviour & conceptual design of wall systems (cont'd)

## EXAMPLES OF LARGE WALLS

# Large, lightly reinforced wall systems in EC8

- *Vertical steel* tailored to demands due to M & N from analysis:
  - cracking & yielding at construction joint at floor levels encouraged → more opportunities for rigid body rotation of individual wall storeys → stronger geometric effects (raise of CGs);
  - little excess (minimum) reinforcement, to minimize flexural overstrength.
- Shear verification for V from analysis times (1+q)/2 ~2:
  - If so-amplified shear demand is less than the (design) shear resistance w/o shear reinforcement

# Pros of wall systems

- Inherently stiff:
  - Excellent damage limitation under more frequent events;
  - Insensitive to presence & irregularity of infills.
- Soft-storey formation physically impossible, due to absence of wall counter-flexure within a storey (excellent collapse prevention).
- Geometric effects of large walls are favourable for the response.
- All things considered: wall systems are more cost-effective for earthquake-resistance.

Seismic behaviour & conceptual design of wall systems (cont'd)

# Cons of wall systems

- Walls: inherently less ductile (sensitive to shear effects) & harder to detail for ductility.
- Limited redundancy / load paths.
- Large uncertainty of seismic response:
  - seismic performance of walls & wall systems less studied experimentally & analytically (practical difficulties);
  - Effect of rocking & of rotations about wall neutral axis: cannot be accounted for;
  - More complex to model, analyze and design (esp. walls of non-rectangular section: L-, T-, U-, H-, etc.).
- Constrain architectural design (especially at façades).
- Too costly to use walls alone for gravity loads: need some beams & columns anyway.
- Hard to design foundation, especially for isolated footings.

# Dual systems of frames and walls

- Combine pros & cons of frame & wall systems in a most cost-effective way.
- Walls offer better protection from non-structural damage in frequent, moderate earthquakes.
- Frames are the $2^{nd}$ line of defense (back-up system) in a strong earthquake, after the - inherently less ductile - walls fail.

# Dual systems of frames and walls (cont'd)

- Heightwise pattern of interstorey drifts:
  - In frames: follows the pattern of storey seismic shears →
    ↓ from base to the roof.
  - Walls fixed at the base: ~vertical cantilevers →
    interstorey drifts ↑ from base to the roof.
- In dual system, floor diaphragms impose on frames & walls ~common floor displacements →
  - at lower floors the walls restrain the frame, take ~100% of floor inertia load
  - near the roof, the frame resists the full floor inertia loads & also holds back the walls (which – if unrestrained – would have large deflection at the top).
- Walls may be thought of as being subjected to:
  - the full inertia loads of all floors, and
  - a concentrated force at roof, reverse to the peak seismic response & the floor inertia loads → reverse bending in upper wall storeys.
- Frames may be considered as being subjected to:
  - concentrated force at top, same sense as floor inertia loads, etc. → constant seismic shear in columns of all storeys → do not reduce column sizes; possibly more vert. reinforcement in upper storeys.

## Dual systems of frames and walls (cont'd)

- Dual systems: higher uncertainty of seismic response, should be taken into account in conceptual design, e.g.:
  - wall(s) rocking at the base are more flexible →

    part of the storey shears are transferred from the wall(s) to the frame:
    - unsafe for the frames; to avoid, we should prevent rocking by providing fixity of all walls at base, through box-type foundation system.
  - diaphragms transfer horizontal forces from the frames to the walls or vice-versa →

    should be thicker & stronger within their plane, than in pure frame or wall systems.

# Conceptual design of shallow (spread) foundation systems for earthquake-resistance

# Conceptual design of shallow (spread) foundation systems for earthquake-resistance

- "Shallow" foundations (in Eurocode 7 "spread" foundations):
  - isolated footings (pads),
  - tie-beams,
  - foundation-beams,
  - rafts.
- Deep foundations (piles, caisons, shafts, etc.): uncommon in buildings, not treated at all here.
- Despite its importance, the foundation receives little attention in design practice:
  - its conceptual design is done last,
  - layout conforms w/ choices in conceptual design of

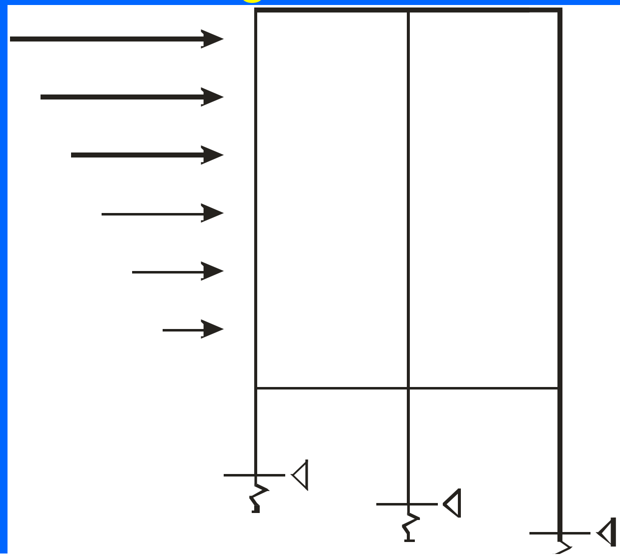
# Bearing capacity failures at Adapazari, TR, in 1999 Kocaeli earthquake

## Conceptual design of shallow foundation systems for earthquake-resistance (cont'd)

# Foundation of entire building at the same level

- All foundation elements should be connected horizontally, to provide a system that introduces to the base of the structure the same ground motion throughout the plan → they should be essentially at the same horizontal level.
- If the foundation of the entire building is at the same horizontal level, modelling of support conditions for the analysis is easier:
  - all nodes at foundation-ground interface are constrained horizontally;
  - if the nodes of the foundation-ground interface are at different levels → analysis results for the effect of overturning moment are fictitious.

Usual modelling of support conditions in building w/ foundation elements at different levels:



# Conceptual design of shallow foundation systems for earthquake-resistance (cont'd)

## Selection of foundation system

- Isolated footings with tie-beams:
  - If moderate or high seismicity: they are not very effective against large eccentricity caused by combination of moment(s) & vertical reaction for design seismic action plus concurrent gravity loads.
  - Realistic modeling of soil compliance: essential for reliable evaluation of action effects in ground, foundation system & superstructure; but due to uplift (nonlinearity), footing rotation compliance is hard to reliably model.
- Two-way foundation-beams throughout plan :
  - Much more (cost-)effective for earthquake resistance than footings w/ two-way tie-beams (especially at the perimeter, where axial loads = low)
  - Modelling of soil compliance by (elastic) subgrade-reaction modulus approach: normally sufficient, as no significant uplift is expected.
- Best of all: “box-type” foundation system:
  1. Wall-like deep foundation beams all along the perimeter.
  2. Rigid diaphragm at the top level of perimeter foundation beams.
  3. Foundation slab, or two-way tie- or foundation beams at bottom level

## Conceptual design of shallow foundation systems for earthquake-resistance (cont'd)

# "Box-type" foundation system

1. Wall-like deep foundation beams all along perimeter of the foundation (possibly supplemented w/ interior ones across full length of foundation system): main foundation elements transferring seismic action effects to the ground, plus:
2. slab at the level of top flange of perimeter foundation beams (e.g. basement roof), w/ thickness & reinforcement as for a rigid diaphragm, plus:
3. foundation slab, or two-way tie- or foundation beams, at the level of the bottom of perimeter foundation beams.

- It works as a rigid body:

# Conceptual design of shallow foundation systems for earthquake-resistance (cont'd)

Surface faulting in Awaji island during the Kobe (1995) earthquake, going through part of a building's foundation