Modelling and Analysis

(Chapter 4 of EC8-1)

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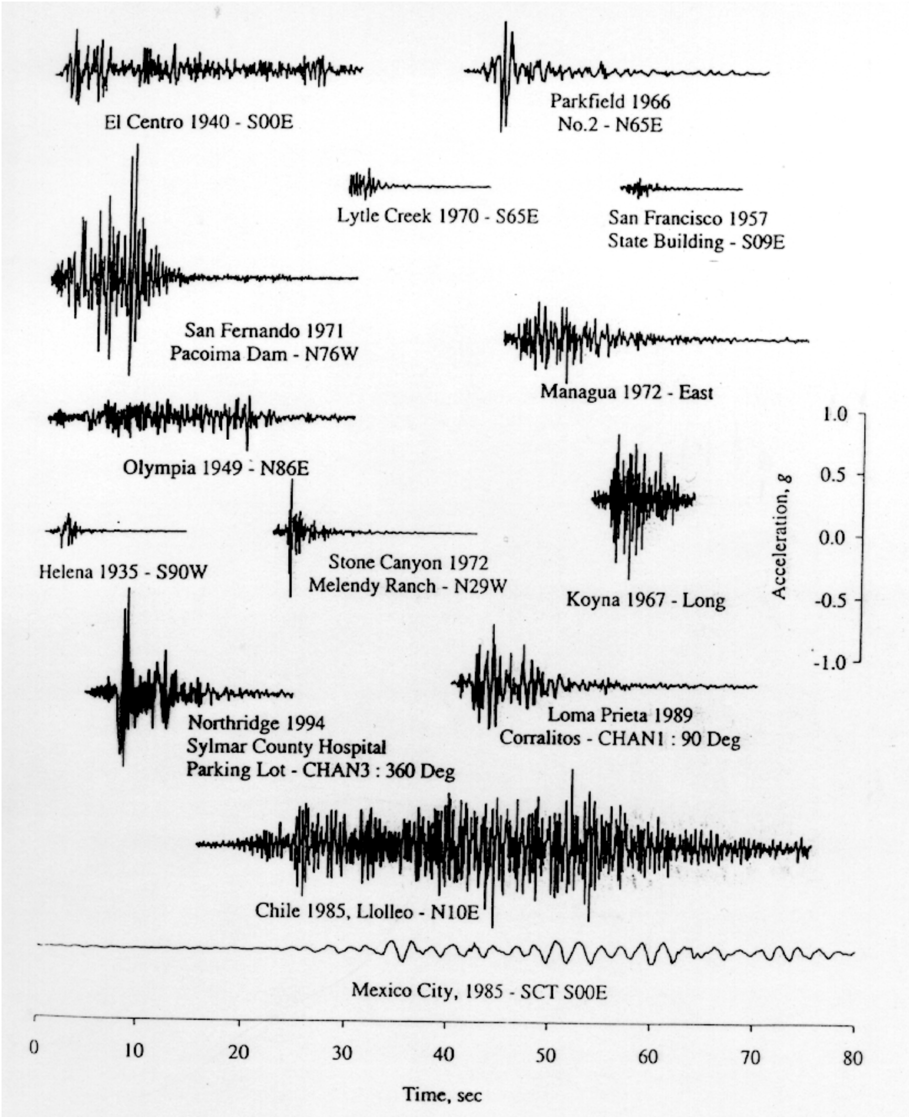
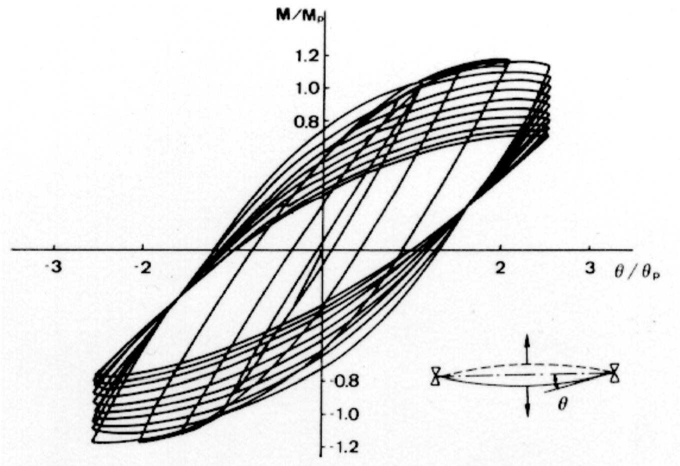
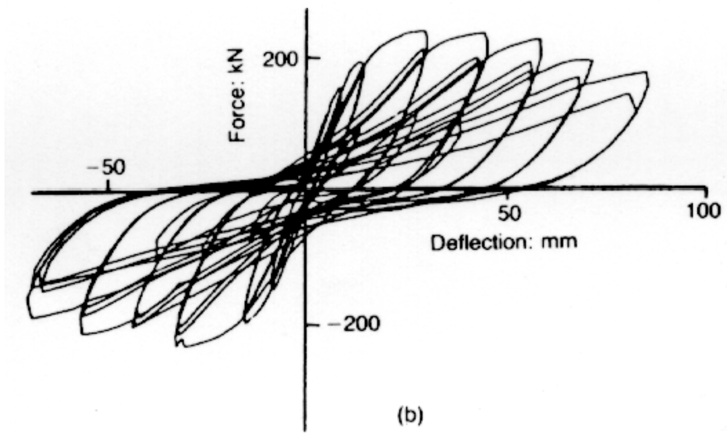
Accelerograms



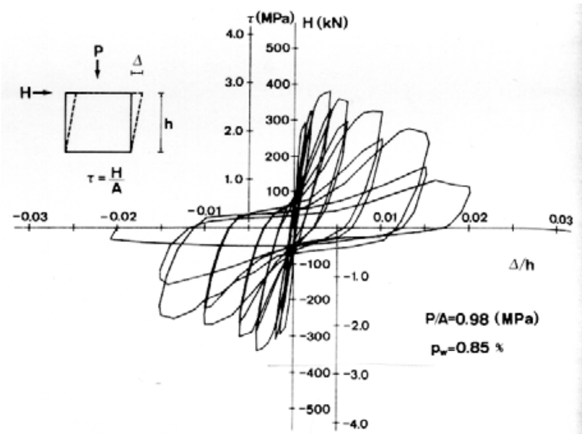
Hysteretic behaviour



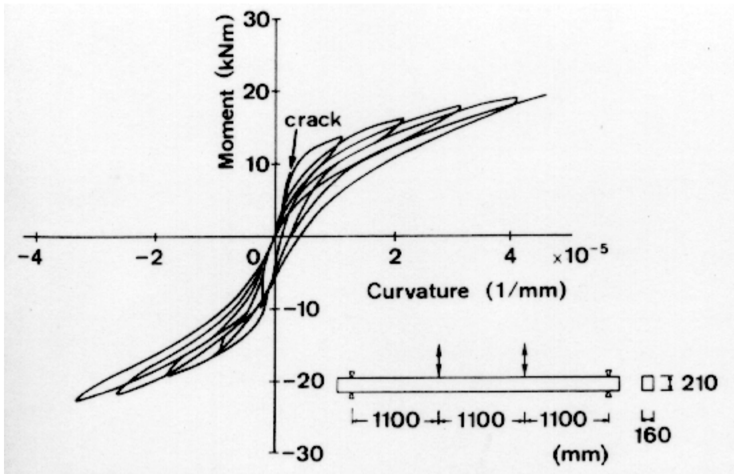
STEEL



REINFORCED CONCRETE

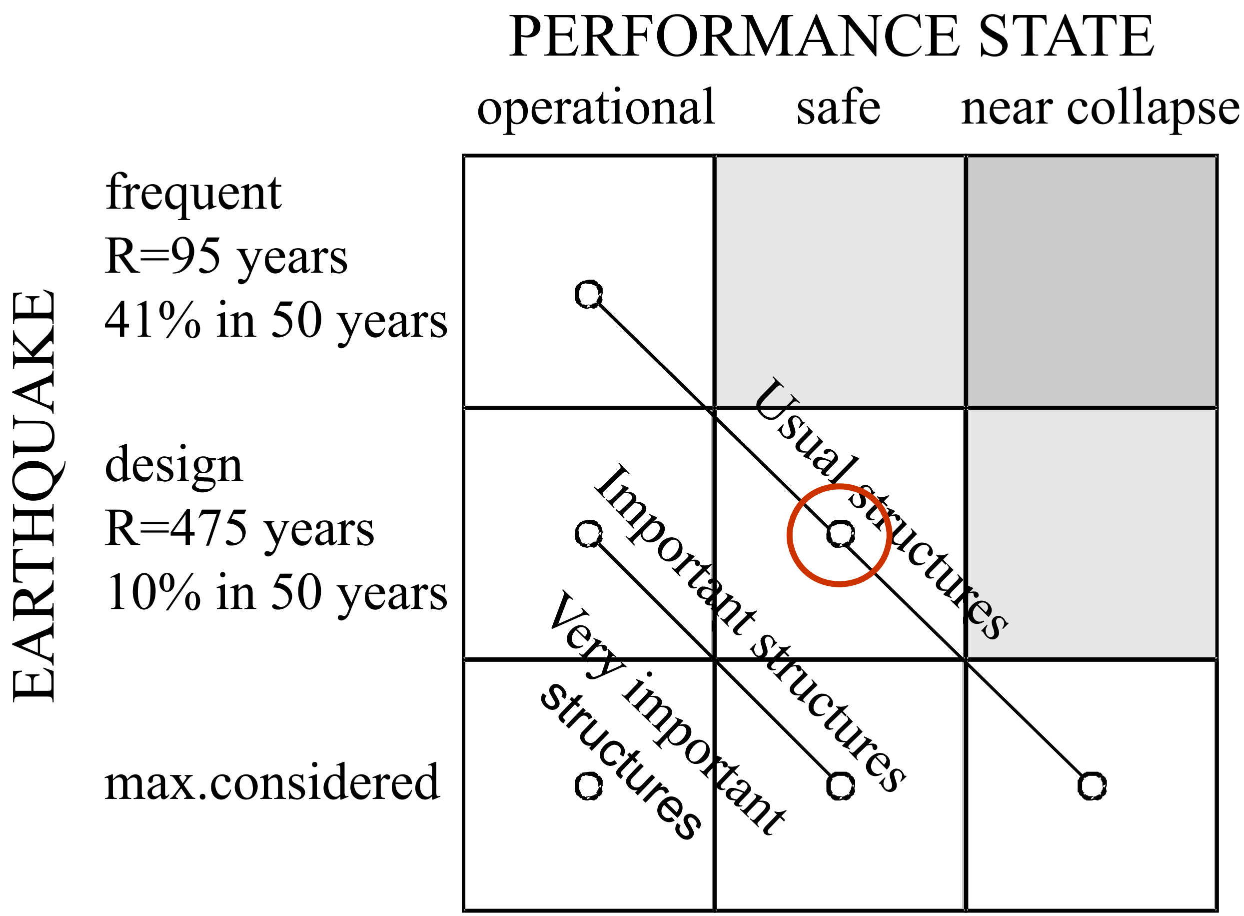


MASONRY

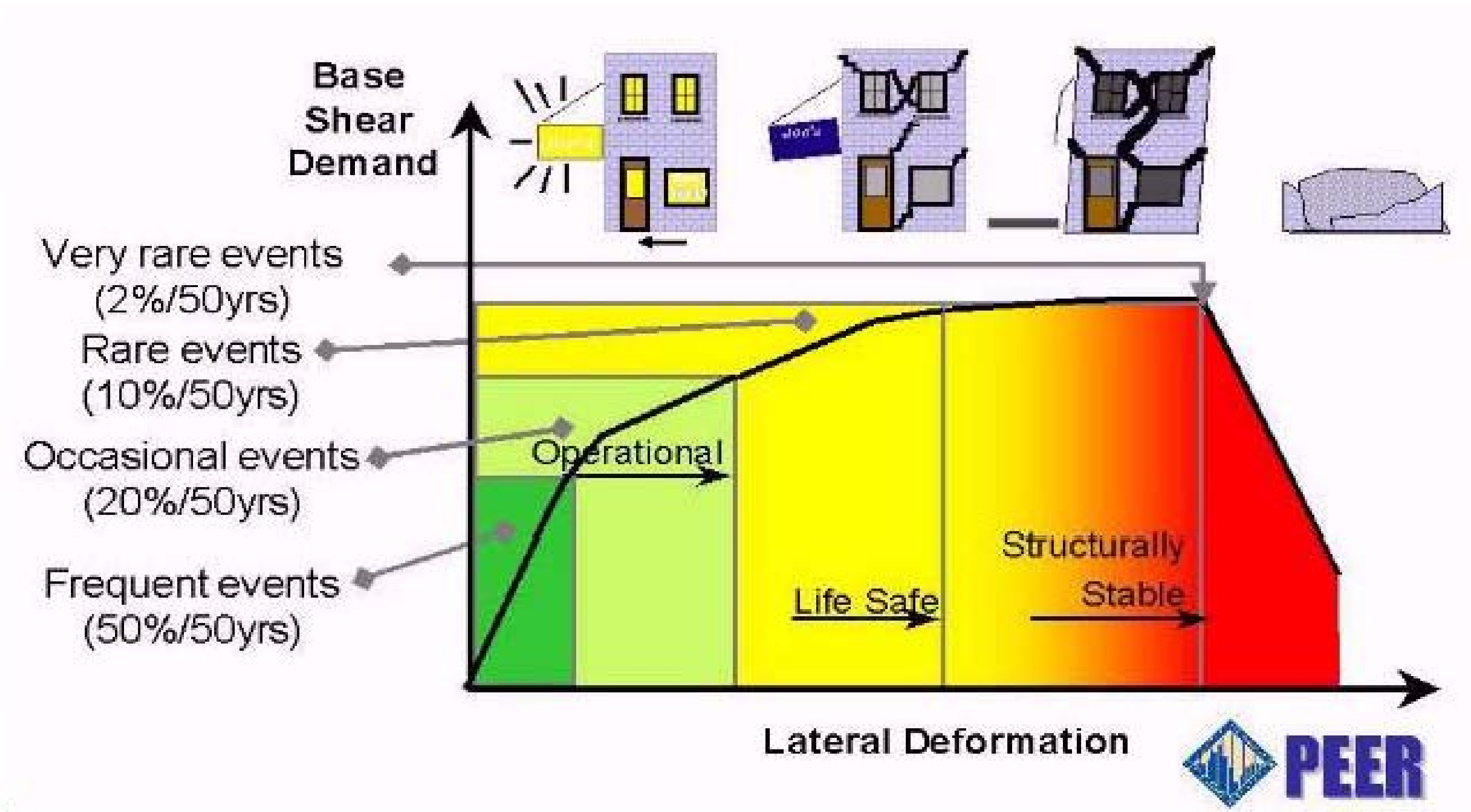


PRESTRESSED CONCRETE

"Philosophy" of seismic design



Performance states



Everything should be made as simple as possible, but not simpler

Albert Einstein

Scope

- EC8-1, Chapter 4, Overview and comments
  - 4.2 Characteristics of earthquake resistant buildings
  - 4.3 Structural analysis
  - 4.4 Safety verifications
- Test building
  - Modelling
  - Analysis
- Code designed versus old buildings

Basic principles of conceptual design

- Structural simplicity
- Uniformity, symmetry and redundancy
- Bi-directional resistance and stiffness
- Torsional resistance and stiffness
- Diaphragmatic behaviour at storey level
- Adequate foundation

L'Aquila 2009

L'Aquila 2009

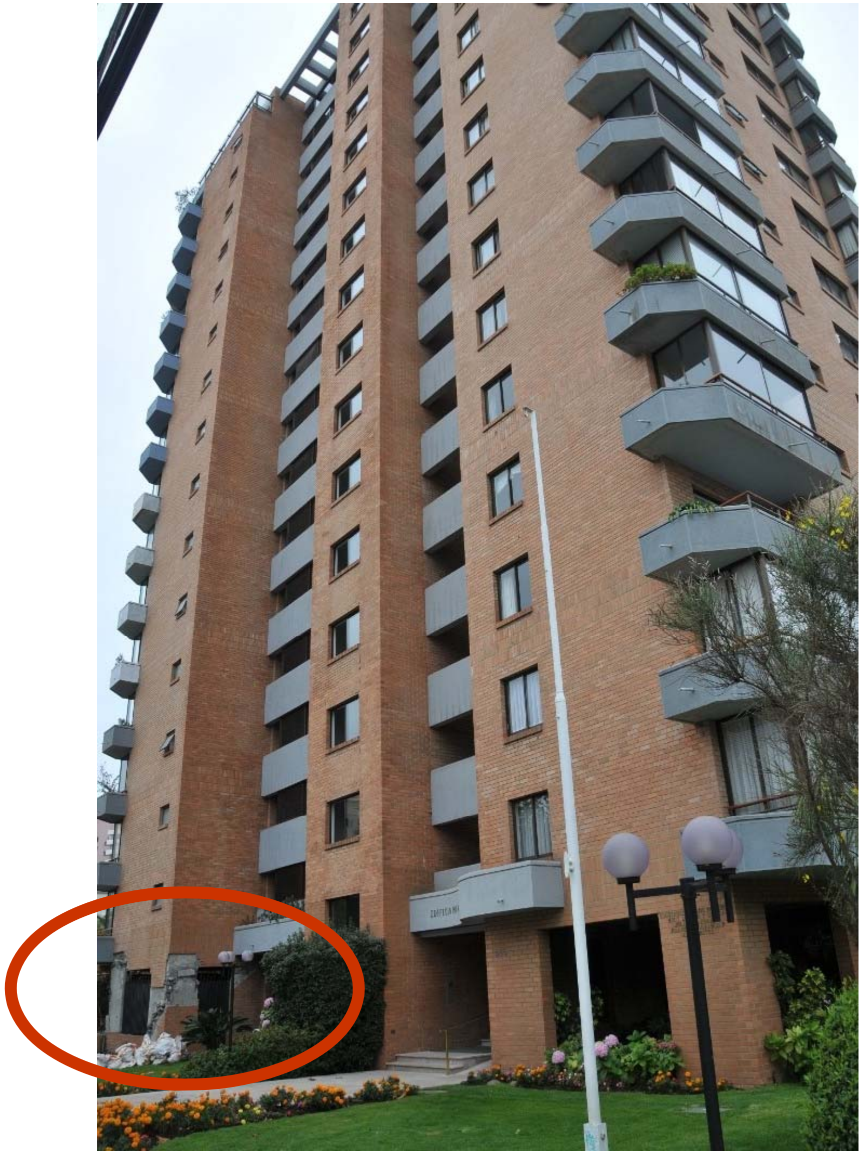
L’Aquila 2009

Kobe 1995

Izmit 1999

Kobe 1995

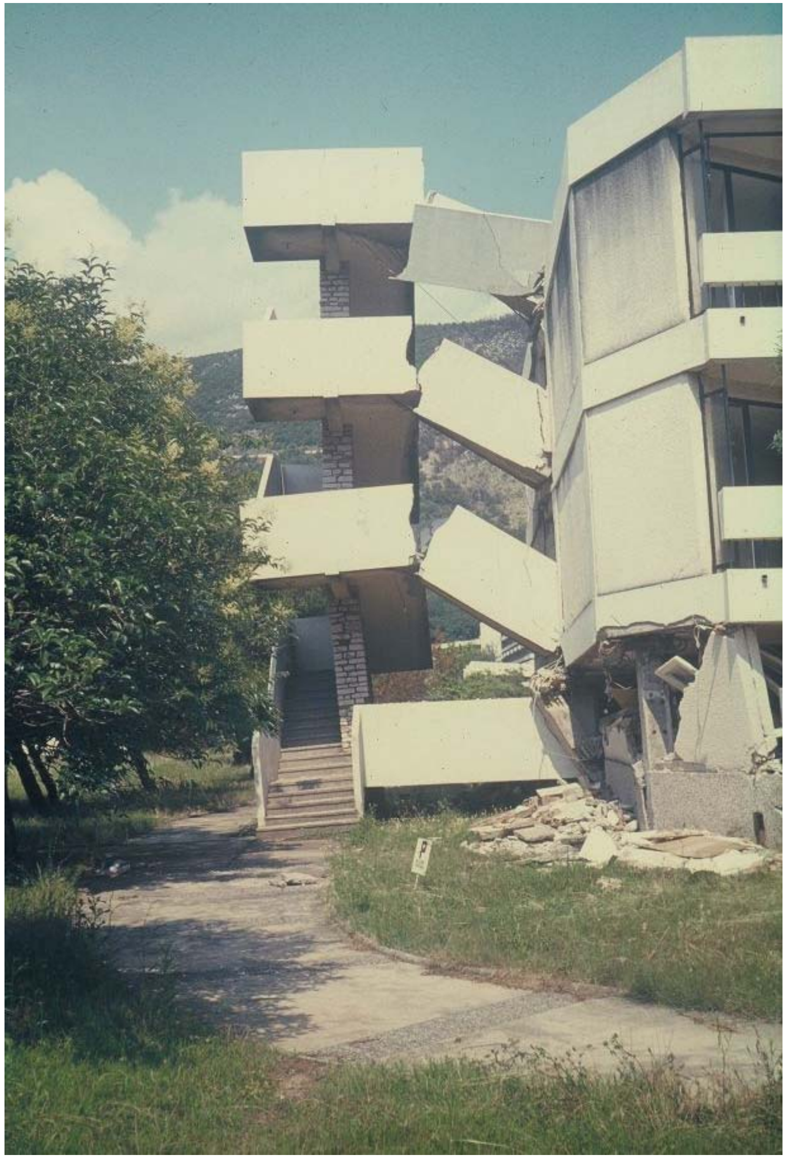
Chile 2010



Kobe 2010

L'Aquila 2009

Montenegro 1979



Kobe 1995

Montenegro 1979

Montenegro 1979

Primary seismic members

- Members considered as part of the structural system that resists the seismic action, modelled in the analysis for the seismic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998

Secondary seismic members

- Members which are not considered as part of the seismic action resisting system and whose strength and stiffness against seismic actions is neglected

Structural (ir)regularity

Table 4.1: Consequences of structural regularity on seismic analysis and design

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

[a] If the condition of **4.3.3.2.1(2)**a) is also met.

[b] Under the specific conditions given in **4.3.3.1(8)** a separate planar model may be used in each horizontal direction, in accordance with **4.3.3.1(8)**.

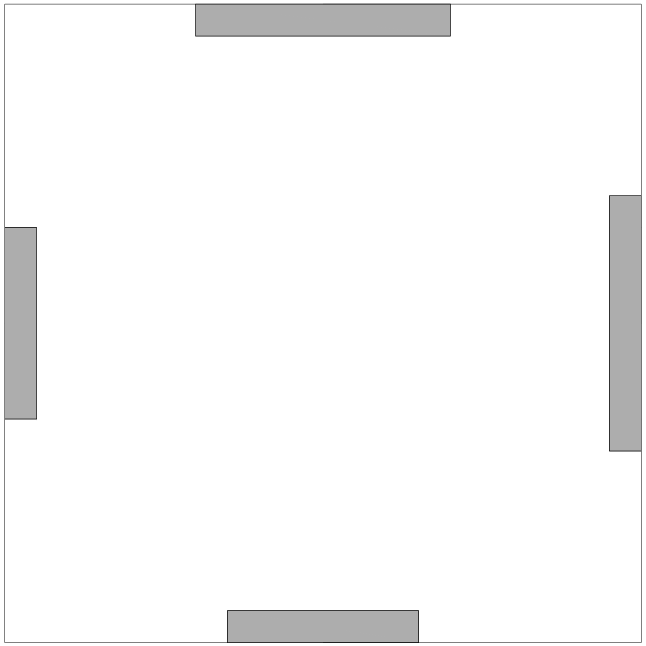
Regularity

- Regularity in plan
  - Symmetry
  - Compact plan configuration
  - Adequate in-plan stiffness of the floors
  - Small in-plan slenderness
  - Adequate torsional stiffness
- Regularity in elevation
  - No interruption of lateral load resisting systems in elevation
  - No abrupt changes of stiffness, mass and overstrength
  - Limitations of setbacks

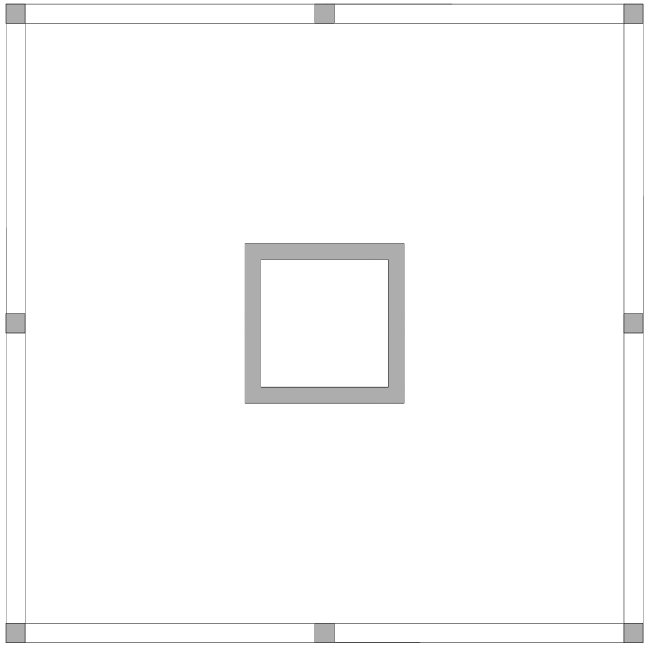
Torsional flexibility

$$T_\varphi \geq T_x \text{ and/or } T_\varphi \geq T_y$$

$$r_x \geq l_s \text{ and/or } r_y \geq l_s$$



Torsionally stiff



Torsionally flexible

Importance classes

Table 4.3 Importance classes for buildings

Importance class	Buildings
I $\gamma_I = 0.8$	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II $\gamma_I = 1.0$	Ordinary buildings, not belonging in the other categories.
III $\gamma_I = 1.2$	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV $\gamma_I = 1.4$	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Importance factor

Importance factor	Return period T (years)
0.8	230
1.0	475
1.2	780
1.3	1000
1.4	1250

(based on data for Slovenia)

Combination of loads (EC0)

$$\sum_{j\geq 1} G_{k,j} \;"+"\; P \;"+"\; A_{E,d} \;"+"\; \sum_{i\geq 1} \Psi_{2,i} Q_{k,i}$$

- Permanent loads “G”
- Prestressing loads “P”
- Seismic loads “A”
- Variable – live loads “Q” (factor Ψ_2 in EC1)

Determination of masses

$$\sum G_{kj} \quad "+" \quad \sum \psi_{Ei} \cdot Q_{ki}$$

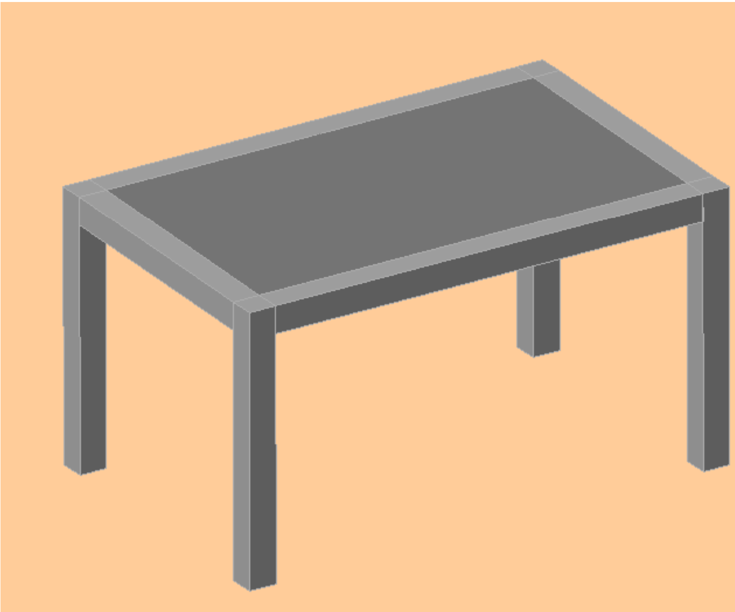
$$\psi_{\mathrm{Ei}} = \varphi \cdot \psi_{\mathrm{2i}}$$

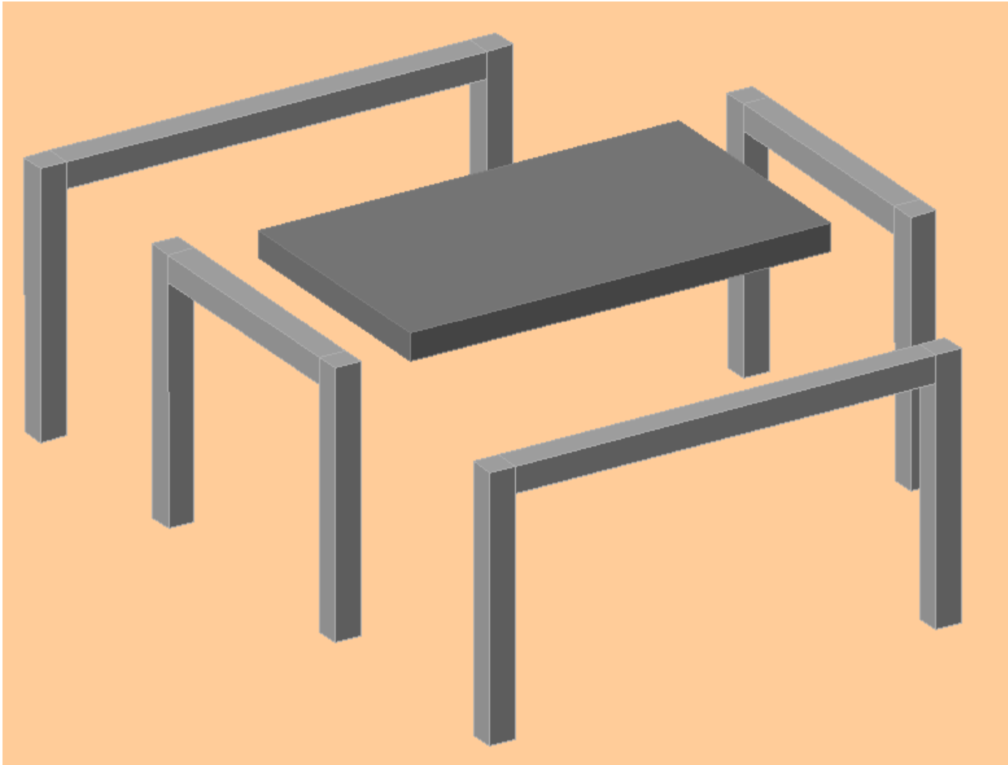
Table 4.2: Values of φ for calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C*	Roof	1,0
	Storeys with correlated occupancies	0,8
	Independently occupied storeys	0,5
Categories D-F* and Archives		1,0

* Categories as defined in EN 1991-1-1:2002.

Pseudo 3D model



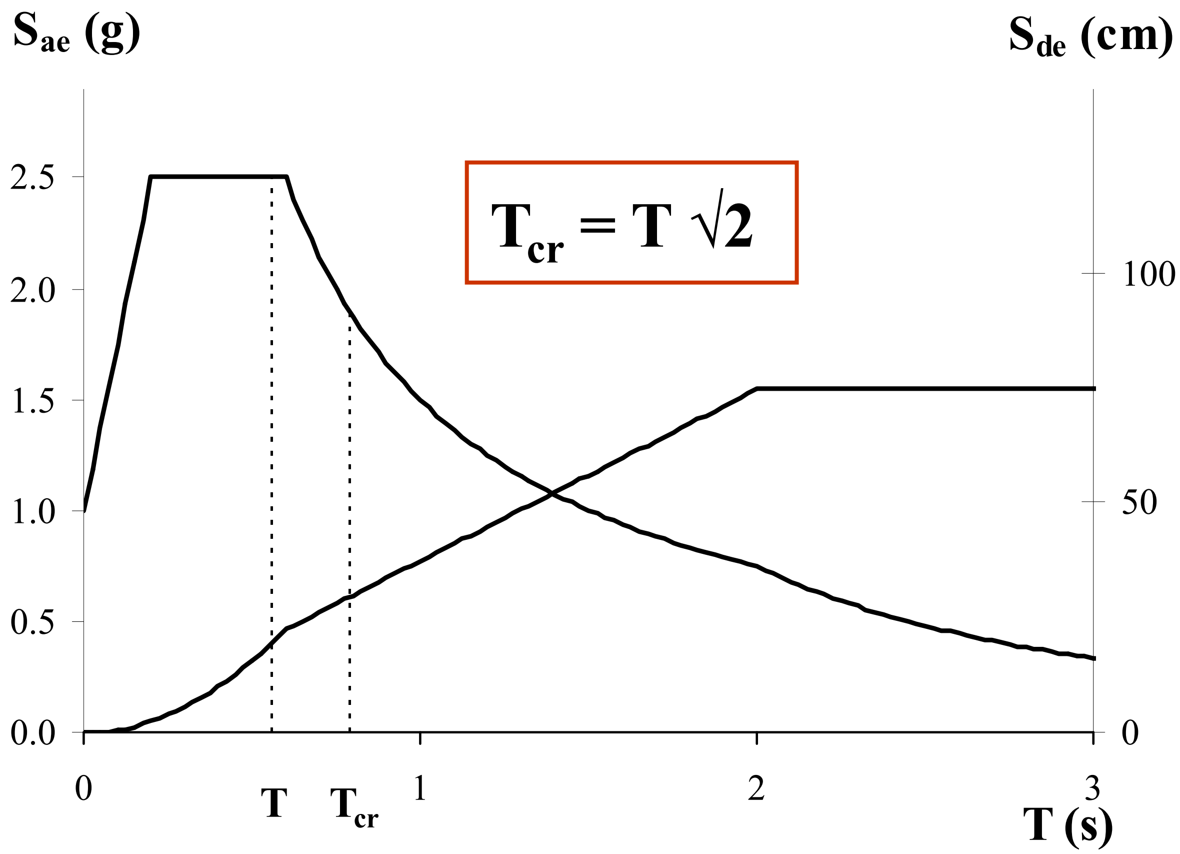


Cracked sections

- In **concrete** buildings, in **composite steel-concrete** buildings and in **masonry** buildings the stiffness of the load bearing elements should take into account the **effect of cracking** (Secant stiffness to the initiation of yielding of the reinforcement).

- The elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to **one-half** of the corresponding **stiffness** of the uncracked elements.

Cracked sections



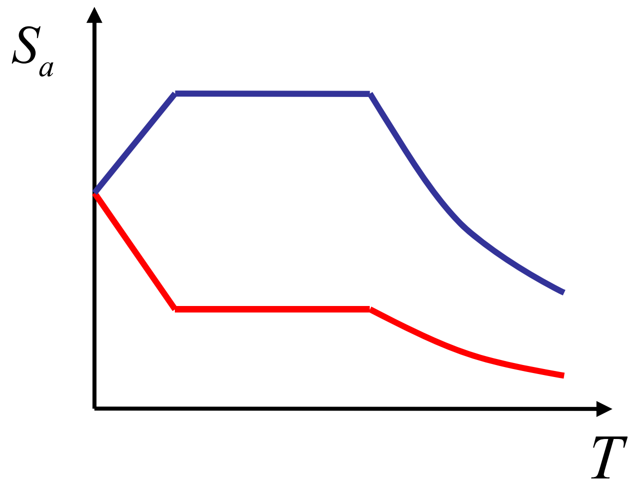
Accidental eccentricity

$$e_{ai} = \pm\, 0.05\, L_i$$

L_i is the floor-dimension perpendicular to the direction of the seismic action

Methods of analysis

	STATIC [a]	DYNAMIC
LINEAR [b]	Lateral force method	Modal response spectrum analysis
NONLINEAR	Nonlinear static (pushover) analysis	Nonlinear response-history analysis



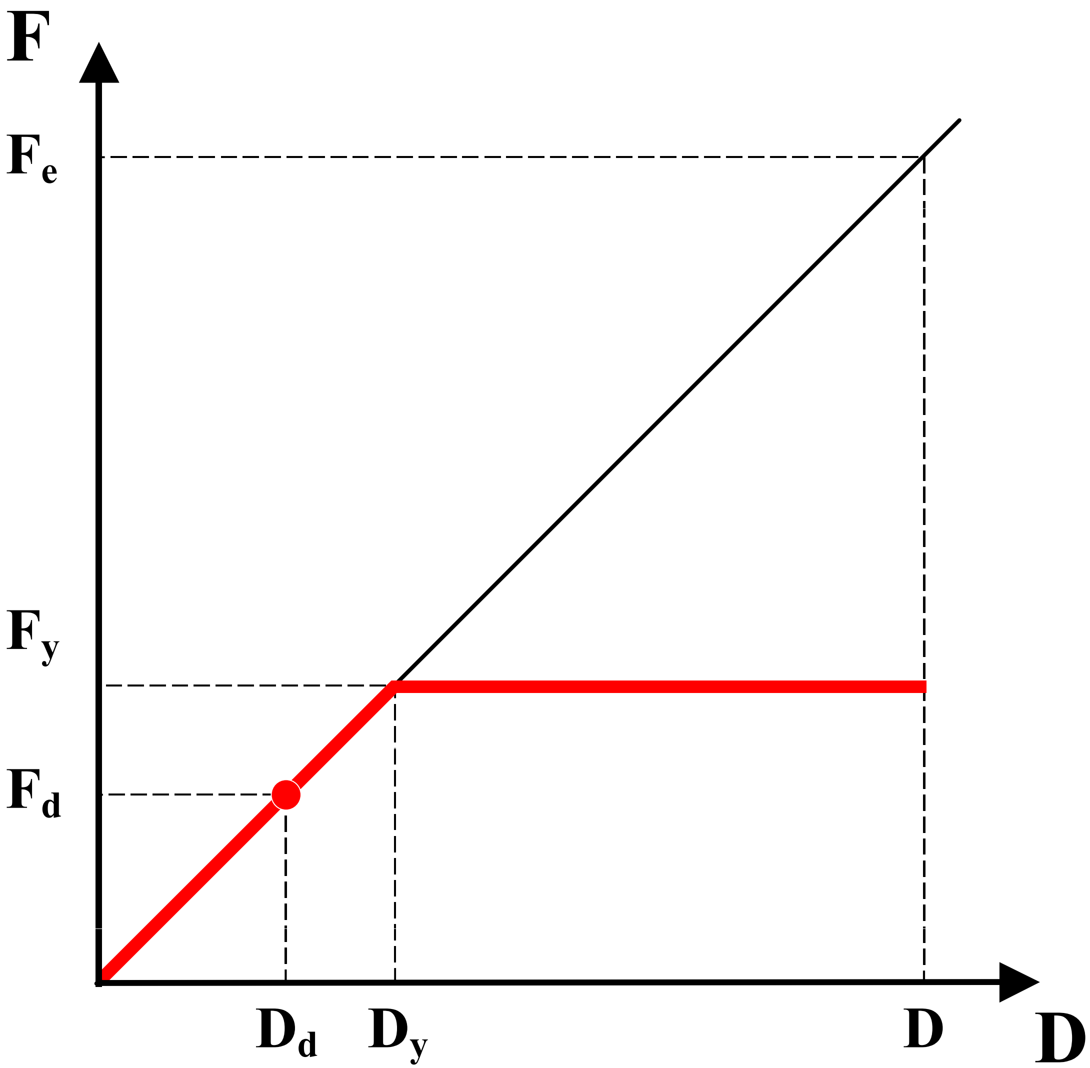
[a] combined with response spectrum

[b] combined with behaviour factor

Behaviour factor

- Factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures

Behaviour factor - background



$$R_\mu = \frac{F_e}{F_y} = \frac{D}{D_y} = \mu$$

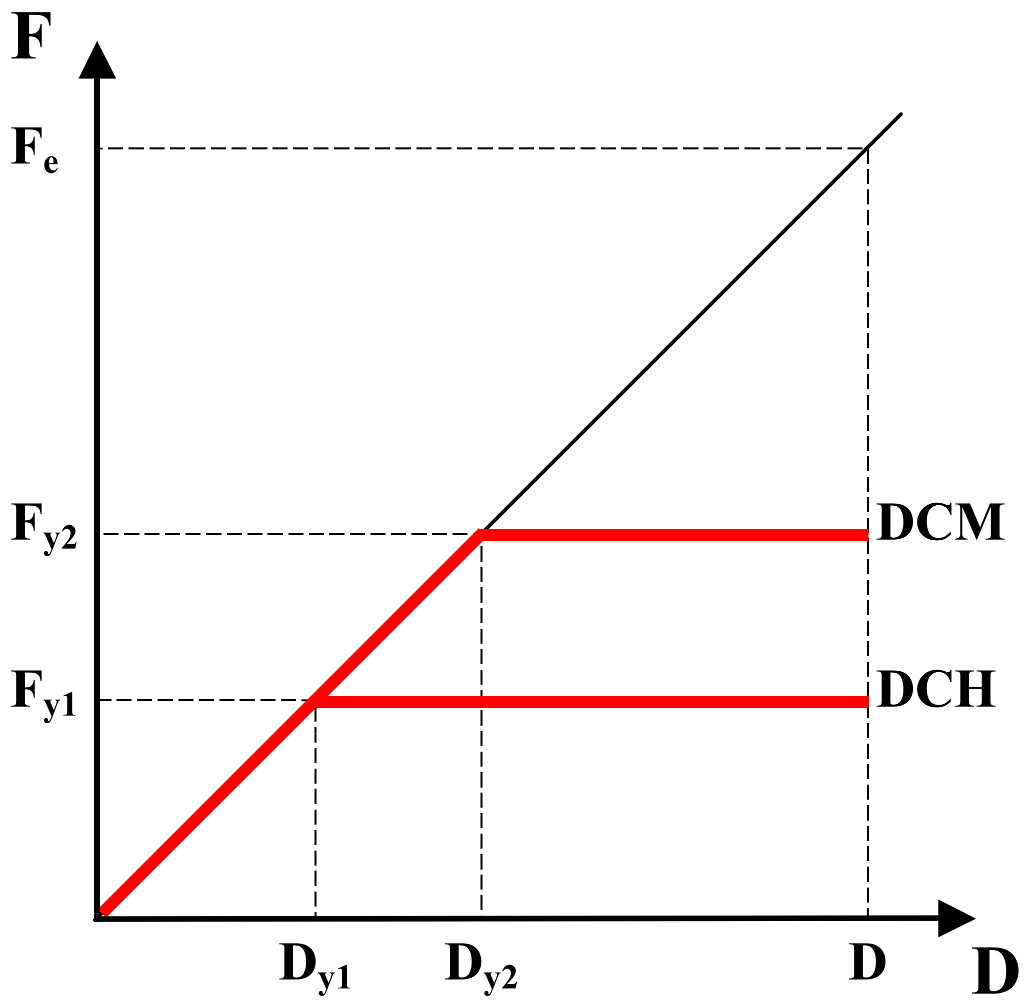
$$R_s = \frac{F_y}{F_d}$$

$$R = \frac{F_e}{F_d} = R_\mu \cdot R_s$$

Eurocode 8

$$R \equiv q \quad , \quad R_s > \frac{\alpha_u}{\alpha_1}$$

Ductility classes



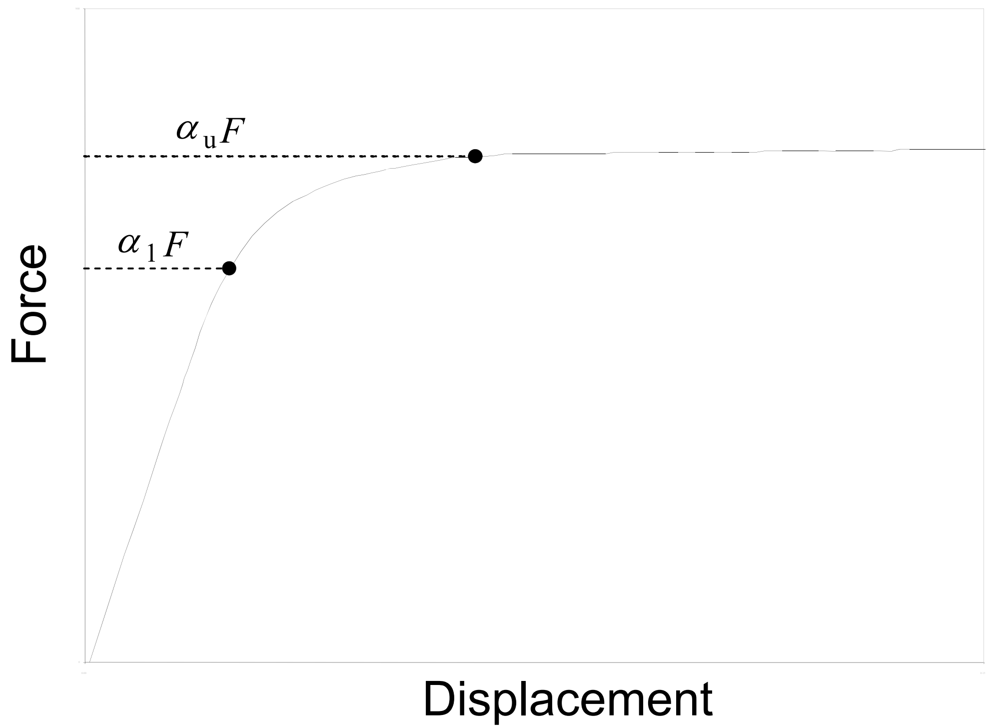
Behaviour factor

Table 5.1: Basic value of the behaviour factor, q_o, for systems regular in elevation

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

(3) For buildings which are not regular in elevation, the value of q_o should be reduced by 20% (see **4.2.3.1(7)** and Table 4.1).

Overstrength



Overstrength factor = α_u / α_1

Montenegro 1979

Kobe 1995

Overstrength factor

- Wall- or wall-equivalent dual systems
  - wall systems with only two uncoupled walls per horizontal direction: $\alpha_u / \alpha_1 = 1,0$
  - other uncoupled wall systems: $\alpha_u / \alpha_1 =1,1$
  - wall-equivalent dual, or coupled wall systems: $\alpha_u / \alpha_1 = 1,2$
- Irregular in plan: reduced values
- Pushover analysis: increased values

Lateral force method

- Regular structures with small influence of higher modes
  - $T_1 \leq 4\,T_C$ in $T_1 \leq 2.0$ s

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

Lateral force method

- **Approximate formulas for the period T_1**

- **Distribution of horizontal forces**

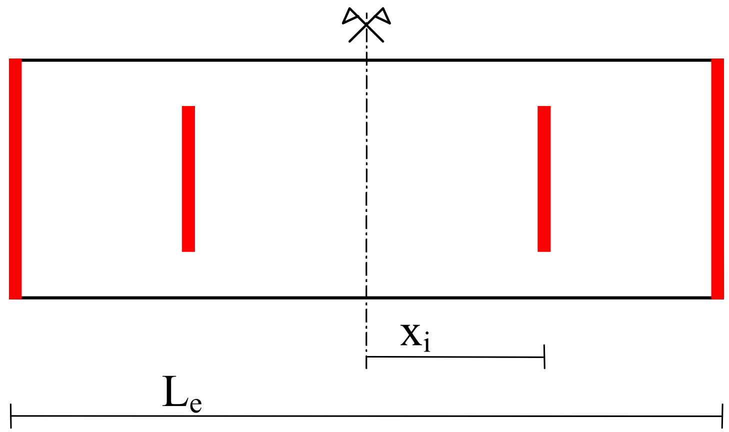
$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma\, s_j \cdot m_j}$$

or

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma\, z_j \cdot m_j}$$

- **Accidental eccentricity**

$$\delta = 1 + 1{,}2\ (x/L_e)$$



Approximate formulas for T_1

Rayleigh

$$T_1 = 2\pi \sqrt{\frac{\sum_j u_j^2 \, m_j}{\sum_j u_j \, p_j}}$$

Empirical formula

$$T_1 = C_t H^{3/4}$$

Modal response spectrum analysis

Displacements:

$$\mathbf{U}_{i\max} = \mathbf{\Phi}_i \, \Gamma_i \, S_{Di} = \mathbf{\Phi}_i \, \Gamma_i \, \frac{T_i^2}{4\pi^2} S_{Ai}$$

Forces:

$$\mathbf{F}_i = \mathbf{M} \, \mathbf{\Phi}_i \, \Gamma_i \, S_{ai}$$

$$\Gamma_i = \frac{L_i}{M_i}$$

$$L_i = \mathbf{\Phi}_i^T \, \mathbf{M} \, \mathbf{s}$$

$$M_i = \mathbf{\Phi}_i^T \, \mathbf{M} \, \mathbf{\Phi}_i$$

Number of modes

- The response of all modes of vibration contributing significantly to the global response shall be taken into account
  - the sum of the effective modal masses amounts to at least 90% of the total mass of the structure
  - all modes with effective modal masses greater than 5% of the total mass are taken into account

Effective masses

$$m_i^* = \frac{L_i^2}{M_i}$$

$$L_i = \mathbf{\Phi}_i^T \; \mathbf{M}\,\mathbf{s} \qquad\qquad M_i = \mathbf{\Phi}_i^T \; \mathbf{M}\,\mathbf{\Phi}_i$$

$$\sum_{i=1}^{n} m_i^* = \sum_{j=1}^{m} m_j = M$$

Combination of modal responses

$$E_E = \sqrt{\Sigma E_{Ei}^2} \qquad \textbf{(SRSS)}$$

if $T_j \leq 0.9\, T_i$

Otherwise more accurate procedure, such as

CQC

Accidental eccentricity

Accidental torsional effects

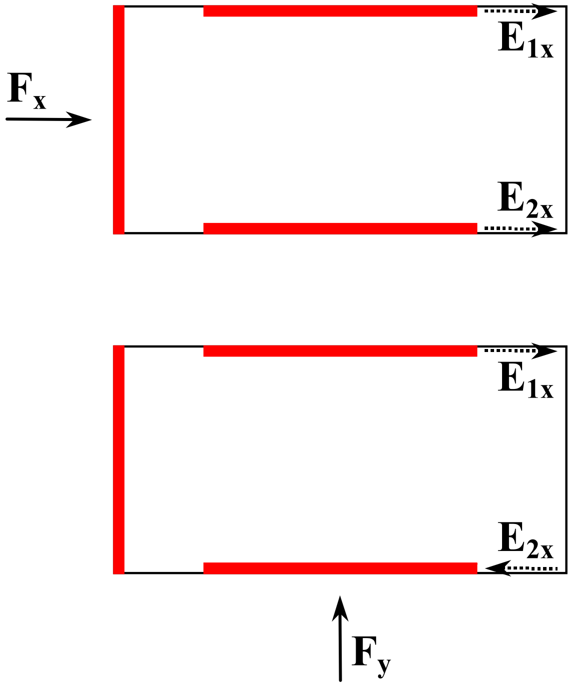
$$M_{X,i} = F_{X,i} \cdot 0.05 L_{Y,i} \,, \quad M_{Y,i} = F_{Y,i} \cdot 0.05 L_{X,i}$$

Pushover analysis

- N2 method (basic)
  - Target displacement: Annex B (informative)
- Extended N2
  - Higher mode effects in plan and elevation
  - Complies with the EC8-3 requirement “4.4.4.5 Procedure for estimation of torsional and higher mode effects”

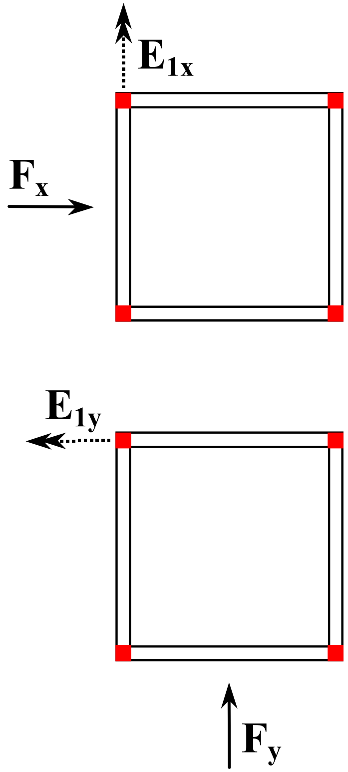
Combination of effects of components

SRSS



E_{Edx} "+" $0{,}30E_{Edy}$

$0{,}30E_{Edx}$ "+" E_{Edy}



Vertical seismic action

If a_{vg} is greater than 0,25 g (2,5 m/s2) the vertical component of the seismic action should be taken into account

- for horizontal or nearly horizontal structural members spanning 20 m or more
- for horizontal or nearly horizontal cantilever components longer than 5 m
- for horizontal or nearly horizontal pre-stressed components
- for beams supporting columns
- in base-isolated structures

Displacement calculation

$$d_s = q_d \, d_e$$

d_s displacement induced by the design seismic action

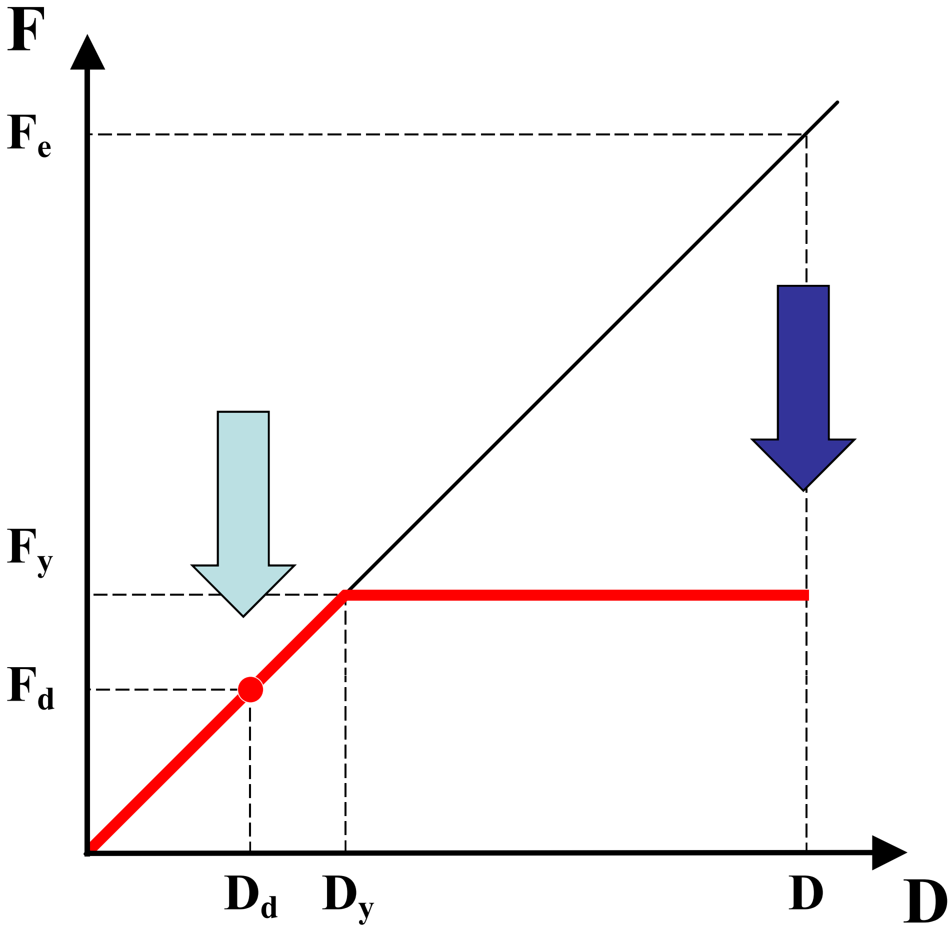
q_d behaviour factor for displacements (q_d = q, unless otherwise specified)

d_e displacement determined by a linear analysis based on the design response spectrum

Upper limit: value from the elastic displacement spectrum

Torsional effect are taken into account

Actual displacements



$q = F_e/F_d$

$D = q\ D_d$

Non-structural element

- Architectural, mechanical or electrical element, system and component which, whether due to lack of strength or to the way it is connected to the structure, is not considered in the seismic design as load carrying element

Non-structural elements

- For non-structural elements of great importance or of a particularly dangerous nature
  - Floor-response spectra
- For other non-structural elements
  - Simplified procedure

Non-structural elements

Simplified analysis

$$F_a = (S_a \cdot W_a \cdot \gamma_a) / q_a$$

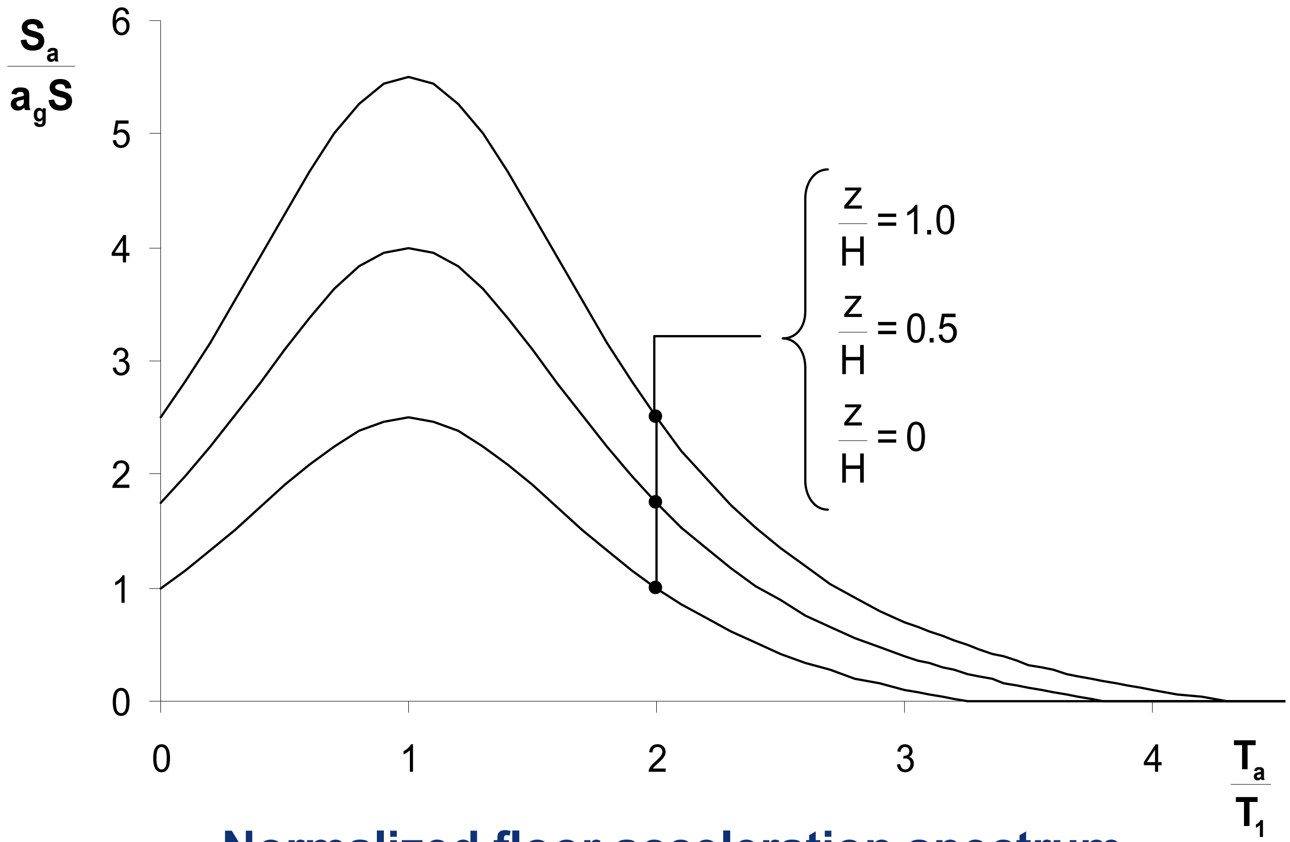
$$S_a = \alpha \cdot S \cdot [3(1 + z/H) / (1 + (1 - T_a/T_1)^2) - 0,5]$$

W_a weight of the element

γ_a importance factor for the element

q_a behaviour factor for the element

Floor acceleration spectrum (simplified)



Normalized floor acceleration spectrum

(q_a = 1, γ_a = 1, z height up to the floor , H total height, T_a period of the element, T_1 period of the structure)

Additional measures for masonry infilled frames

- Provisions apply to frame or frame equivalent dual concrete systems of DCH and to steel or steel-concrete composite moment resisting frames of DCH with interacting non-engineered masonry infills
  - Recommendation: adopt also for DCM or DCL concrete, steel or composite structures with masonry infills
- Irregularities in elevation
- Irregularities in plan
- Damage limitation of infills

Friuli 1976

Montenegro 1979 Izmit 1999

Safety verifications (1) - Ultimate limit state

Resistance condition

$$E_d \leq R_d$$

E_d demand, R_d capacity

P-Δ effects need not be taken into account if

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10$$

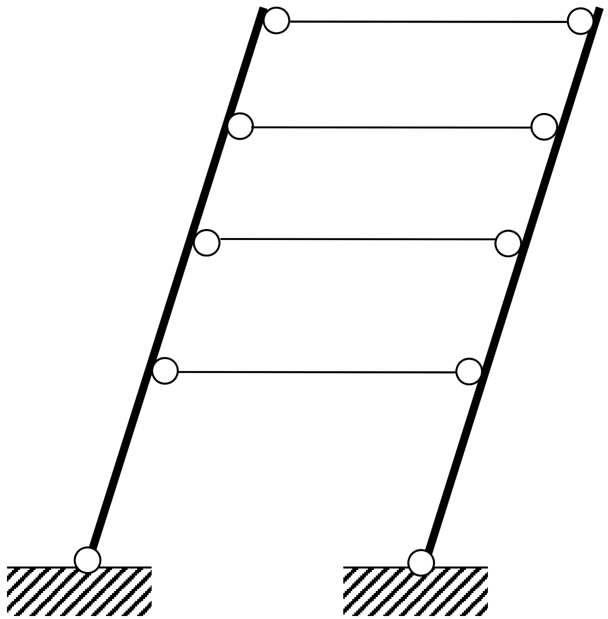
Safety verifications (2)

- Global and local ductility condition
  - Specific material related requirements shall be satisfied, including, when indicated, capacity design provisions
  - Prevention of storey mechanisms

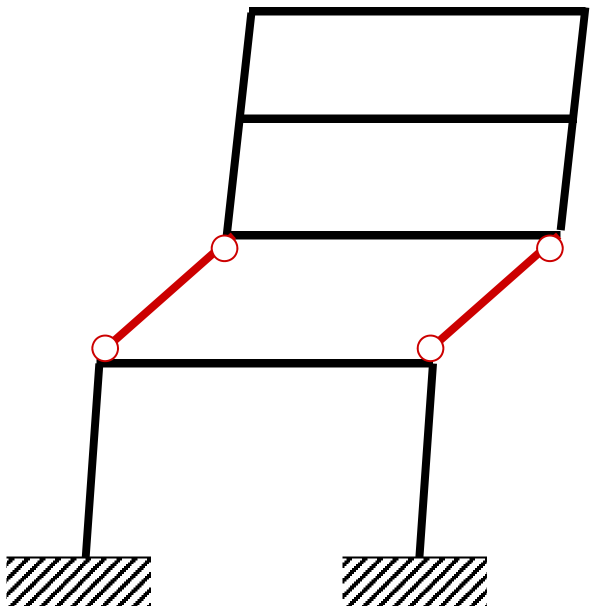
$$\sum M_{\mathrm{Rc}} \geq 1{,}3 \sum M_{\mathrm{Rb}}$$

Capacity design

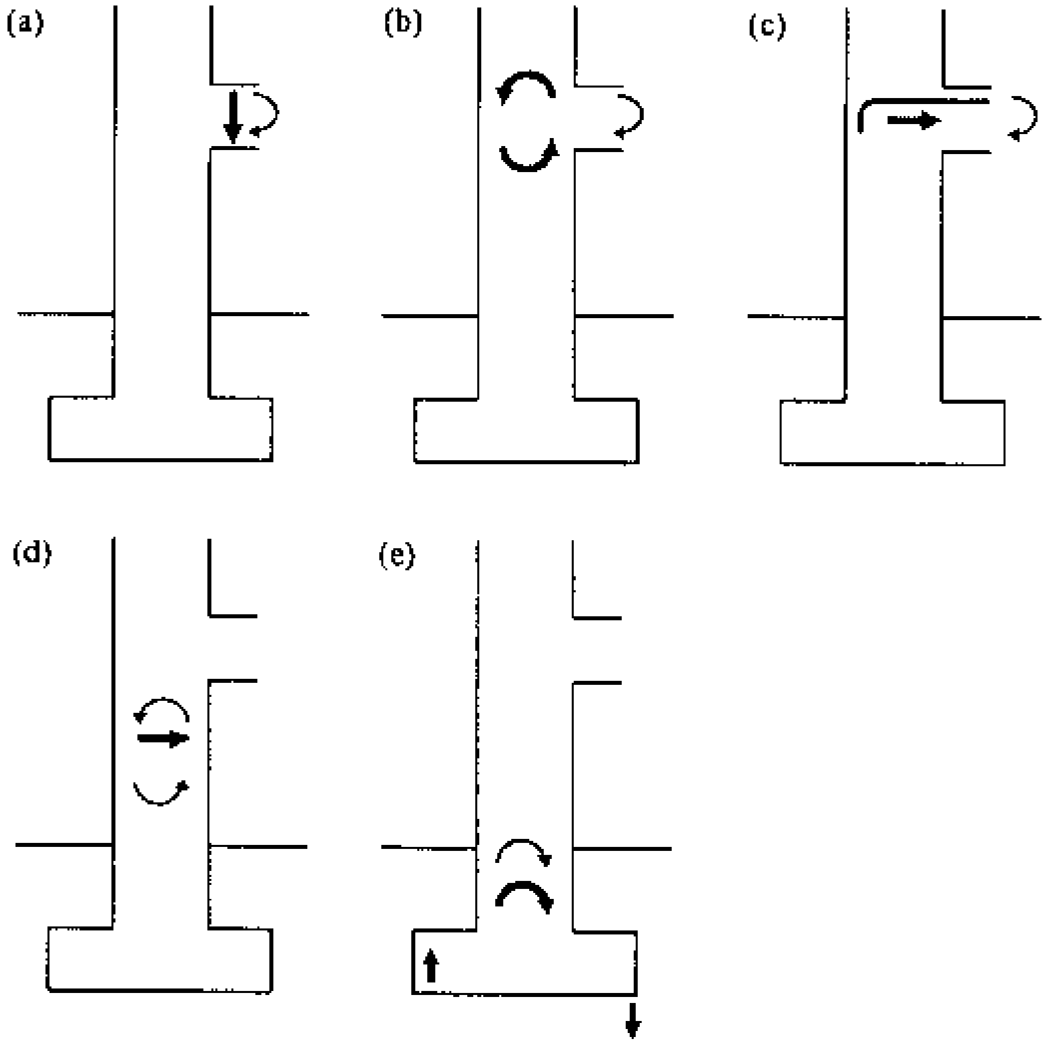
YES !



NO !



Capacity design



Kobe 1995

Kobe 1995

Kobe 1995

Safety verifications (3)

- Equilibrium condition
- Resistance of horizontal diaphragms
- Resistance of foundations
- Seismic joint condition

Damage limitation state

Limitation of interstorey drift

- non-structural elements of brittle materials

  $d_r\,\nu \leq 0.005\,h$, **$d_r \leq 0.01h$**

- ductile non-structural elements

  $d_r\,\nu \leq 0.0075\,h$

- non-structural elements do not to interfere with structural deformations, or without non-structural elements

  $d_r\,\nu \leq 0.010\,h$ **$d_r \leq 0.02h$**

$\nu = 0.4$ (importance classes III and IV)

$\nu = 0.5$ (importance classes I in II)

Return period versus **(importance) factor**

Return period T (years)	Return period T (years)
50	0.48
100	0.60
200	0.76
475	1.00
1000	1.30
10000	2.57

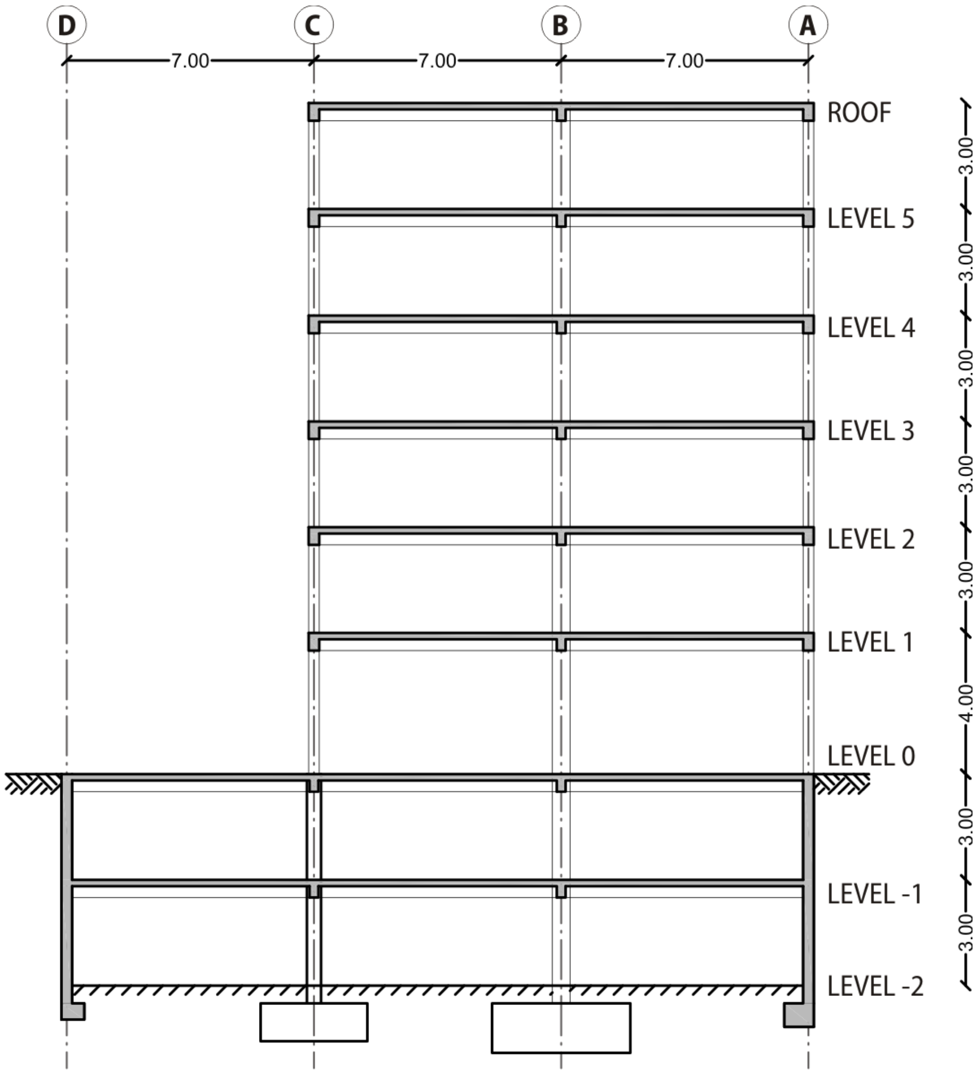
Valid for Slovenia

Test example

RC building

6 stories + 2 basements

Description of building

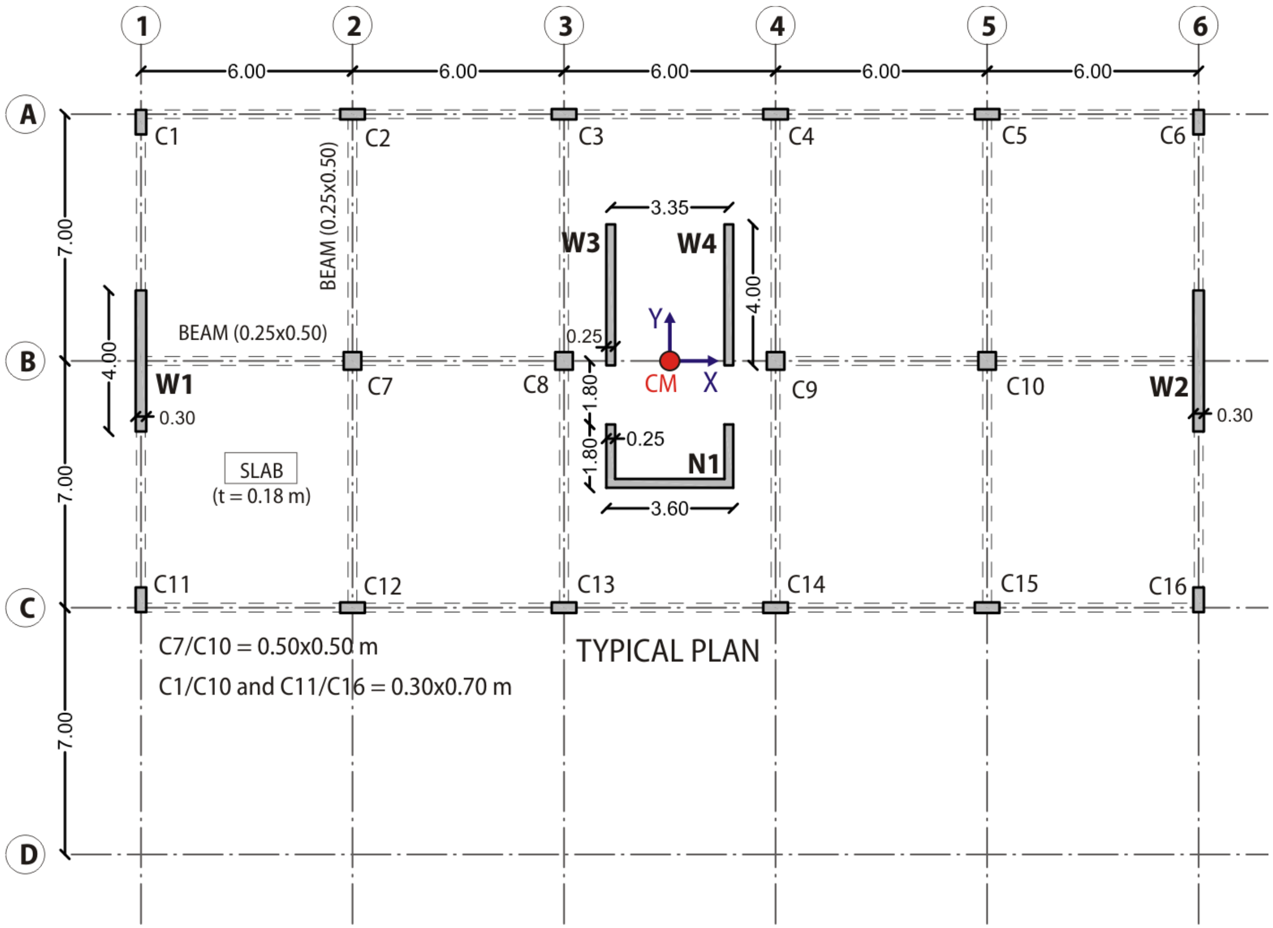


SCHEMATIC SECTION

SCHEMATIC SECTION

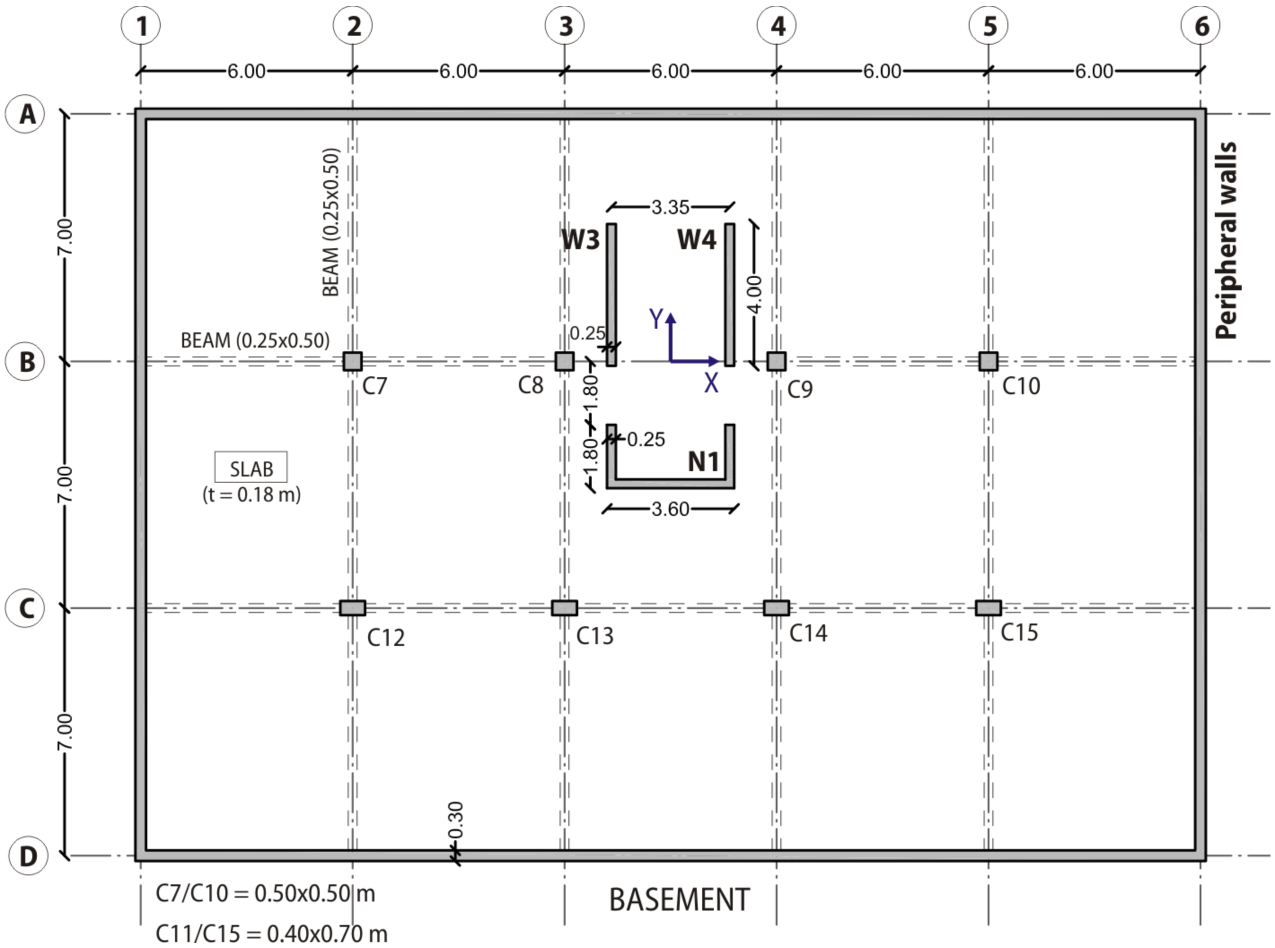
Description of building

TYPICAL PLAN



Description of building

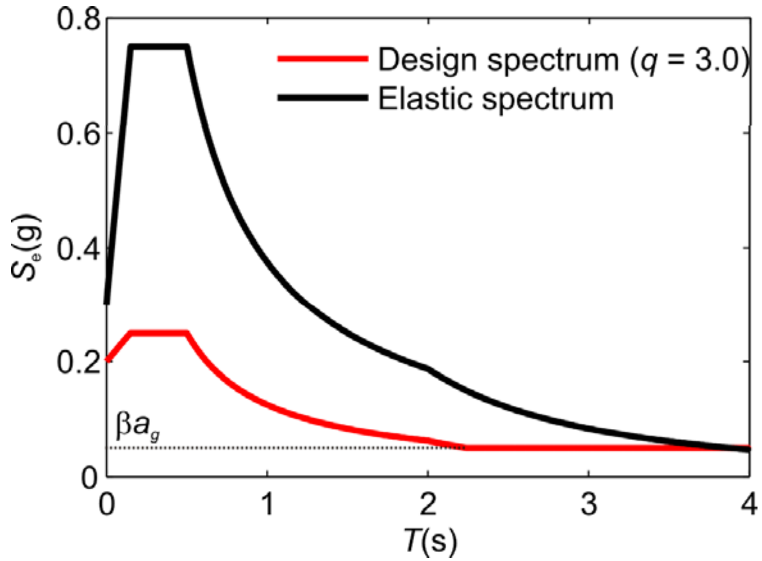
BASEMENT



Seismic actions

ELASTIC RESPONSE SPECTRUM

- $a_g = \gamma_I \cdot a_{gR} = 0.25g$
  - importance class II ($\gamma I = 1.0$)
- Soil B, Type 1
  - S = 1.2,
  - $T_B = 0.15$ s, $T_C = 0.5$ s, $T_D = 2.0$ s
- Damping 5%



Vertical actions

- Permanent loads “G”
  - self weight of the structure + 2 kN/m^2
- Variable – live loads “Q”
  - office building (category B) $\Rightarrow$ 2 kN/m^2
- Vertical loads (G, Q) were distributed to the elements

Seismic masses (1)

- Masses from permanent loads "G" $\Rightarrow$ **factor 1.0**
- Masses from live loads "Q" $\Rightarrow$ **factor Ψ_{Ei}**

$$\Psi_{Ei} = \varphi \cdot \Psi_{2i}$$

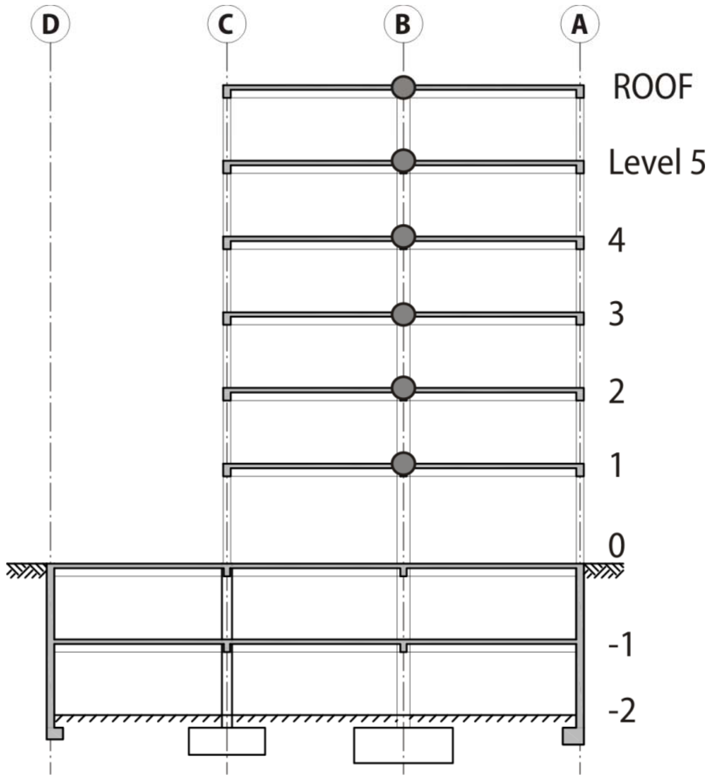
- factor φ = 1.0 (roof storey), φ = 0.5 (other)
- factor Ψ_{2i} = 0.3 (category B)
- 15% (30%) mass from Q is taken into account

Seismic masses (2)

Level	Storey mass *m* (ton)	Moment of inertia MMI (ton*m2)
ROOF	372	33951
5	396	36128
4	396	36128
3	396	36128
2	396	36128
1	408	37244
Σ =	**2362 ton**	

*** Only masses above level 0 are taken into account**

$$MMI = m \cdot l_s^2 = m \cdot \frac{l^2 + b^2}{12}$$



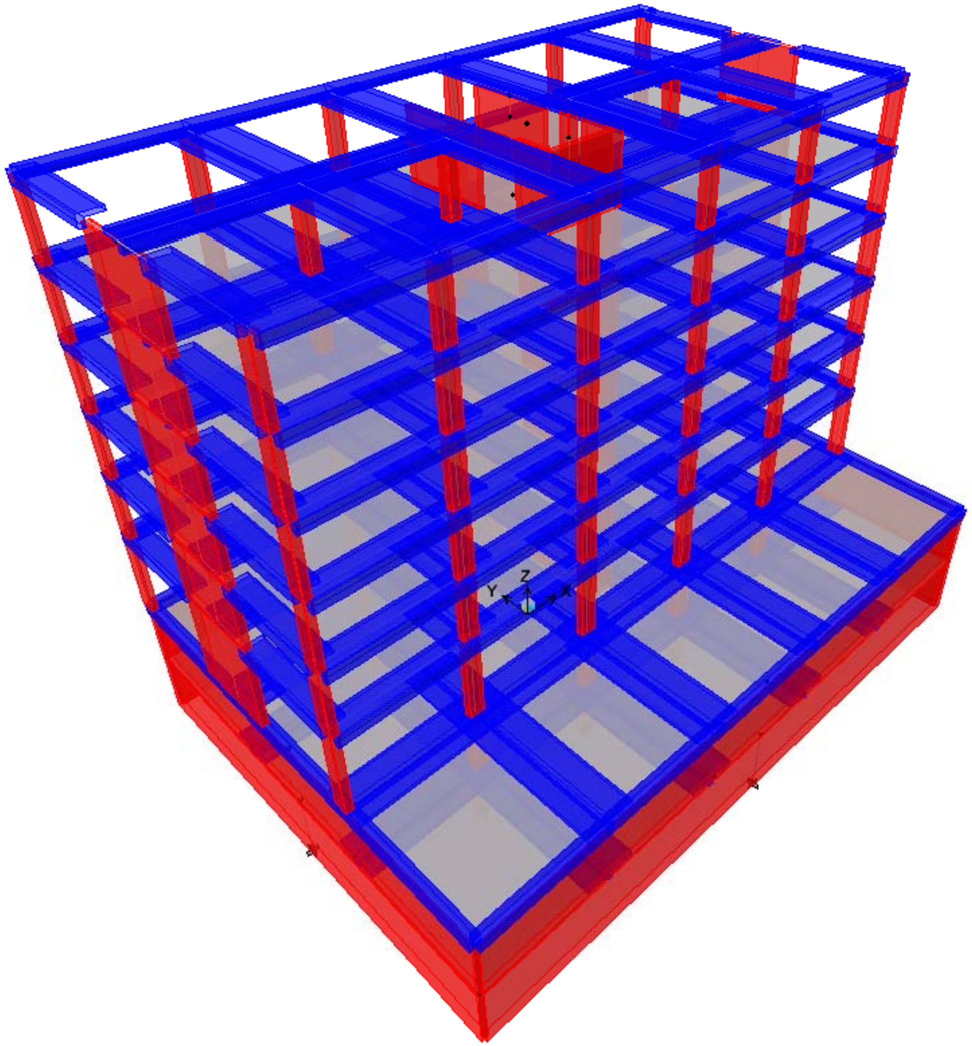
Structural model – general (1)

- 3D (spatial) model
- All element are modelled as line elements
  - peripheral walls are modelled with line elements and a rigid beam at the top of the each element
- Effective widths of beams (EC2)
- Rigid offsets are not taken into account
  - Infinitely stiff elements are used only in relation to walls W1 and W2
- Rigid diaphragms at each floor
  - slabs are not modelled

Structural model – general (2)

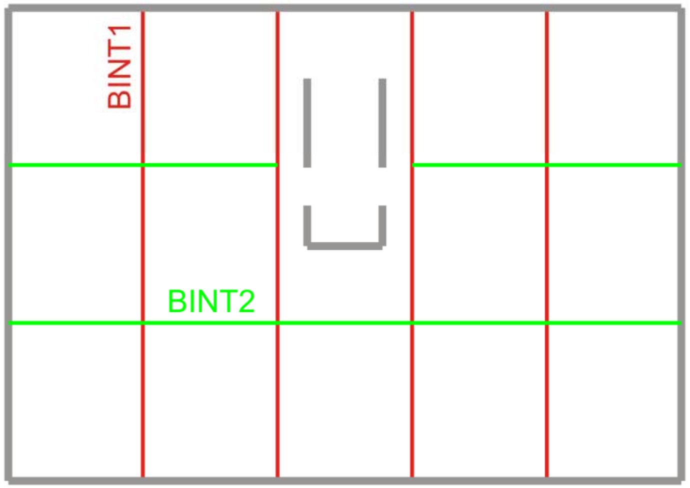
- Masses and mass moments of inertia are lumped at centres of masses
  - Only masses above the top of the peripheral walls are taken into account
- Cracked elements are considered
  - 0.5*As, 0.5*I, 0.1*It
- All elements are fully fixed in foundation
- Infills are not considered

Structural model – general (3)

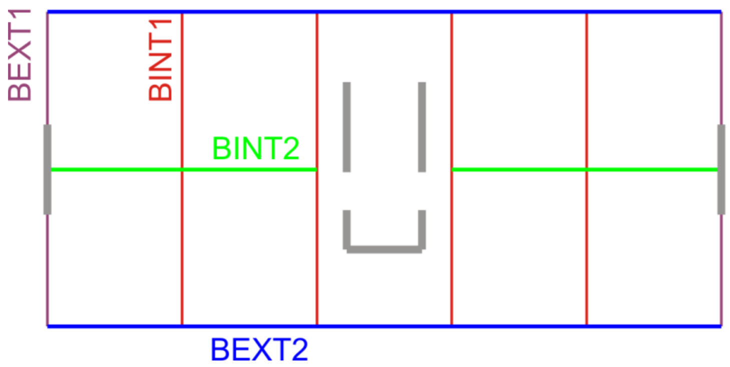


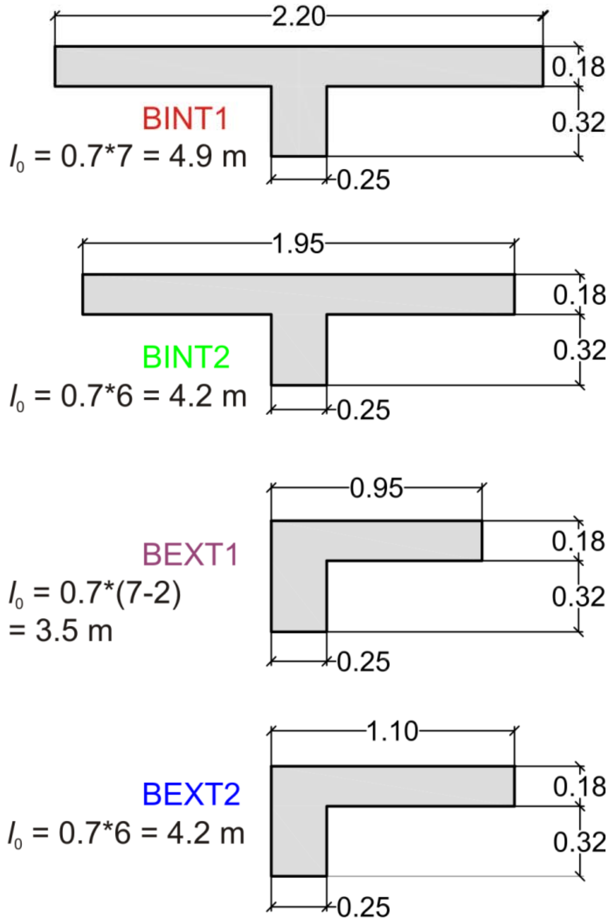
Structural model – effective width EC2

Level -1 and 0

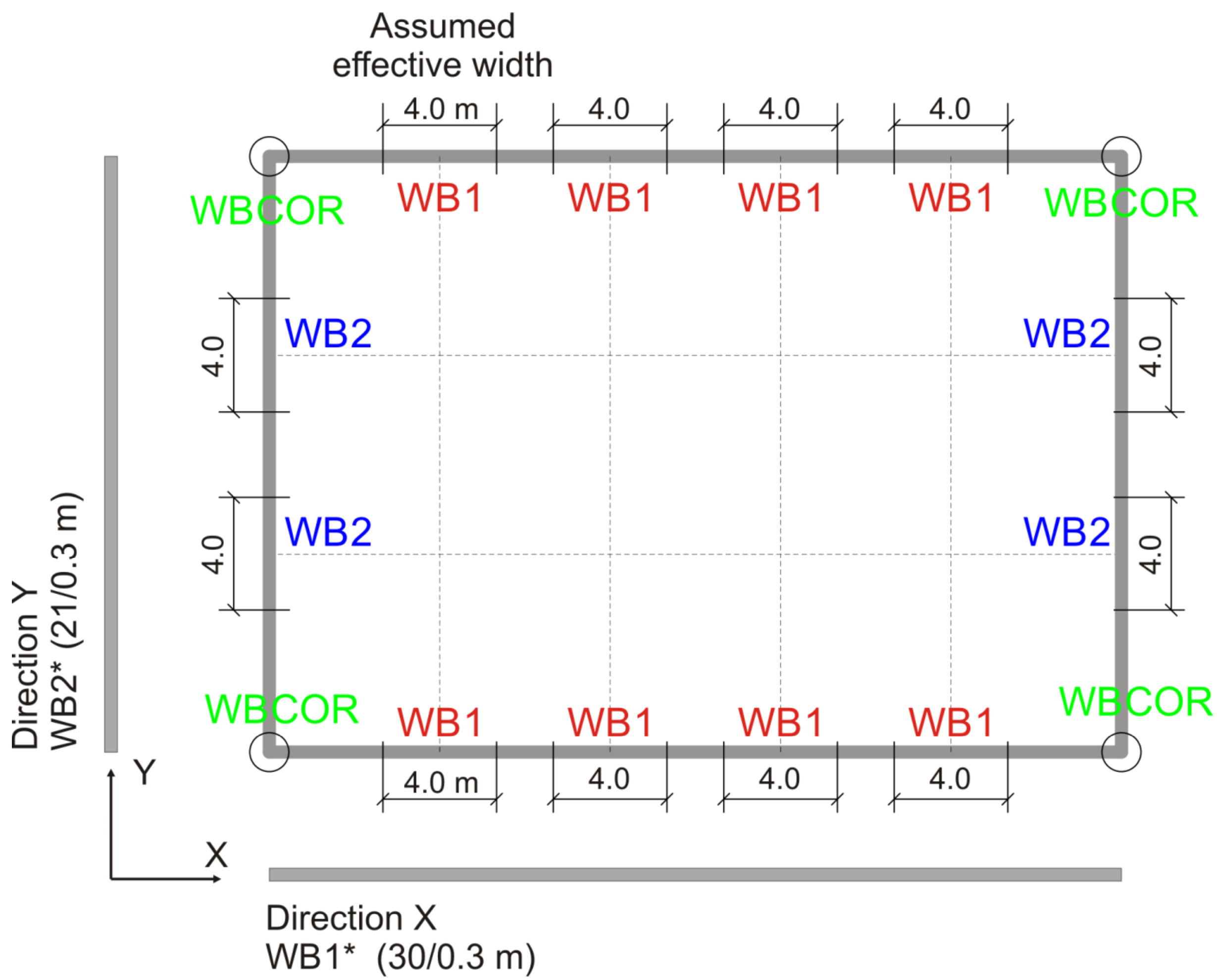


Level 1, 2, 3, 4, 5, Roof





Structural model – peripheral walls



Structural regularity

- Criteria for regularity in elevation

- Criteria for regularity in plan

1) slenderness < 4

$$\lambda = \frac{L_{max}}{L_{min}} < 4$$

2) eccentricity < 30%* torsional radius

Direction X: $e_{0X} \leq 0.30 \cdot r_X$

Direction Y: $e_{0Y} \leq 0.30 \cdot r_Y$

3) torsional radius < radius of gyration

Direction X: $r_X > l_s$

Direction Y: $r_Y > l_s$

Structural regularity in plan

- **Structural eccentricity e_0 and centre of stiffness**
  - 3 static load cases in each storey (F_{Xi} = 1, F_{Yi} = 1, M_i = 1)
  - Loads are applied in centres of mass (CM)
  - Determine rotation R_{Zi} due to F_{Xi}, F_{Yi} and M_i
  - Determine e_{0i} and centres of stiffness (XCR_i, YCR_i)

$$e_{0X,i} = \frac{R_{Z,i}\left(F_{X,i}=1\right)}{R_{Z,i}(M_i=1)} \quad \Rightarrow \quad XCR_i = e_{0X,i} + XCM_i$$

$$e_{0Y,i} = \frac{R_{Z,i}\left(F_{Y,i}=1\right)}{R_{Z,i}(M_i=1)} \quad \Rightarrow \quad YCR_i = e_{0Y,i} + YCM_i$$

Structural regularity in plan

- **Torsional radius (r_X, r_Y)**
  - 3 static load cases in each storey ($F_{TXi} = 1$, $F_{TYi} = 1$, $M_{Ti} = 1$)
  - Loads are applied in centres of stifness (CR)
  - Determine rotations R_{Zi} (M_{Ti}), displacement U_{Xi} (F_{Xi}) and U_{Yi} (F_{Yi})
  - Determine torsional ($K_{M,i}$) and lateral stiffnesses ($K_{FX,i}$, $K_{FY,i}$)
  - Determine r_{Xi} and r_{Yi}

$$K_{M,i} = \frac{1}{R_{Z,i}\left(M_{T,i} = 1\right)},\quad K_{FX,i} = \frac{1}{U_{X,i}\left(F_{TX,i} = 1\right)},\quad K_{FY,i} = \frac{1}{U_{Y,i}\left(F_{TY,i} = 1\right)}$$

$$r_{X,i} = \sqrt{\frac{K_{M,i}}{K_{FY,i}}} \quad and \quad r_{Y,i} = \sqrt{\frac{K_{M,i}}{K_{FX,i}}}$$

Structural regularity - criteria

- Criteria for regularity in elevation ✓
- Criteria for regularity in plan

1) $\lambda = \dfrac{L_{max}}{L_{min}} < 4$ ✓

2) Direction X: $e_{0X} \leq 0.30 \cdot r_X$
   Direction Y: $e_{0Y} \leq 0.30 \cdot r_Y$ ✓

3) Direction X: $r_X > l_s$
   Direction Y: $r_Y > l_s$ ✓

⇨ **Structure is regular in plan and in elevation**

Irregular in elevation if basement is also considered !?

Structural type of the building

- **UNCOUPLED WALL SYSTEM**
  - The structural system is defined as a wall system, when 65% (or more) of the shear resistance is contributed by walls
  - Application of shear resistance is difficult
  - EC8 allows that shear resistance may be substituted by shear forces
  - **Base (above basement) shear force taken by walls amounts to 72% (direction X) and 92% (direction Y) of the total shear force**

Dual wall equivalent system?

Behaviour factor *q*

- Structural type: **uncoupled wall system**
- Ductility class: **DCM**

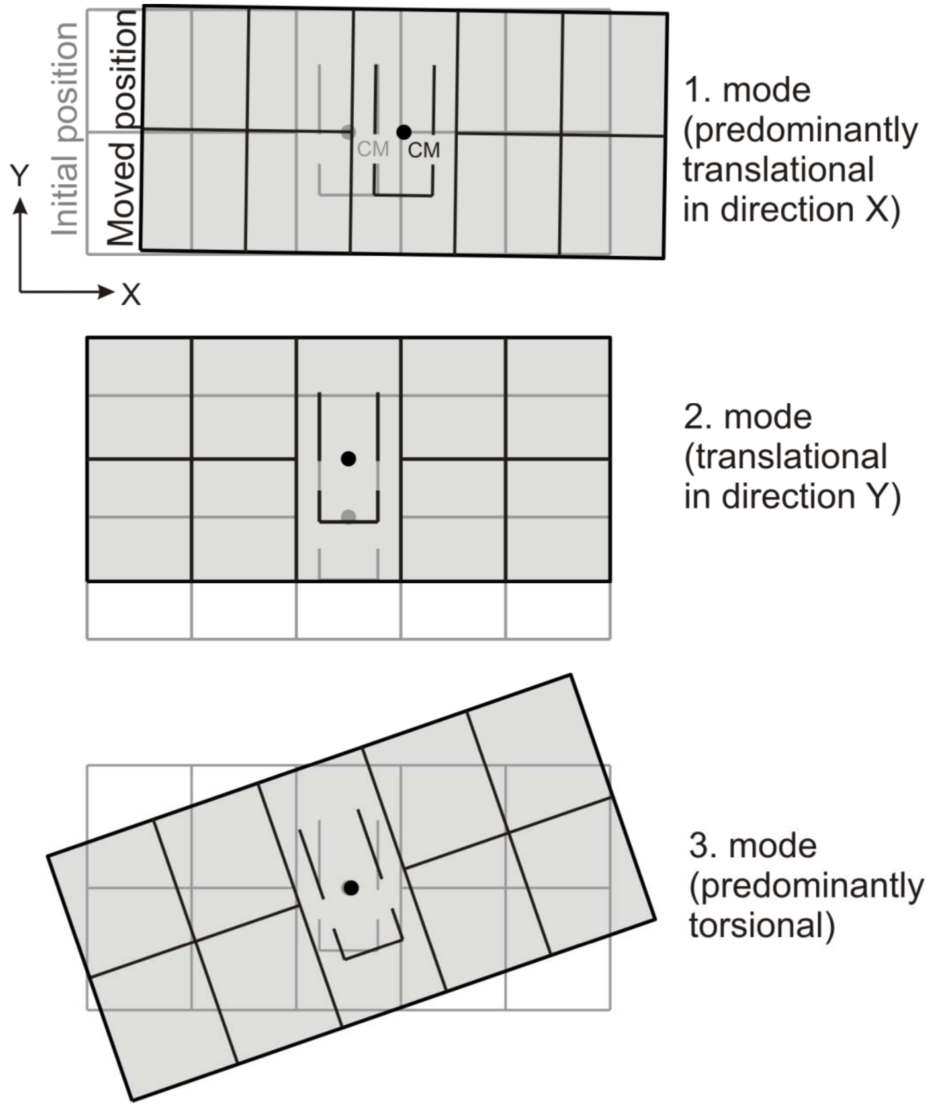
$$q_0 = 3.0$$

- Structural (ir)regularity:
  regular in elevation - no reduction q_0
- Factor associated with prevailing failure mode: $\boldsymbol{k_w = 1}$

$$q = k_w \cdot q_0 = 3.0$$

Periods, effective masses and modal shapes (1)

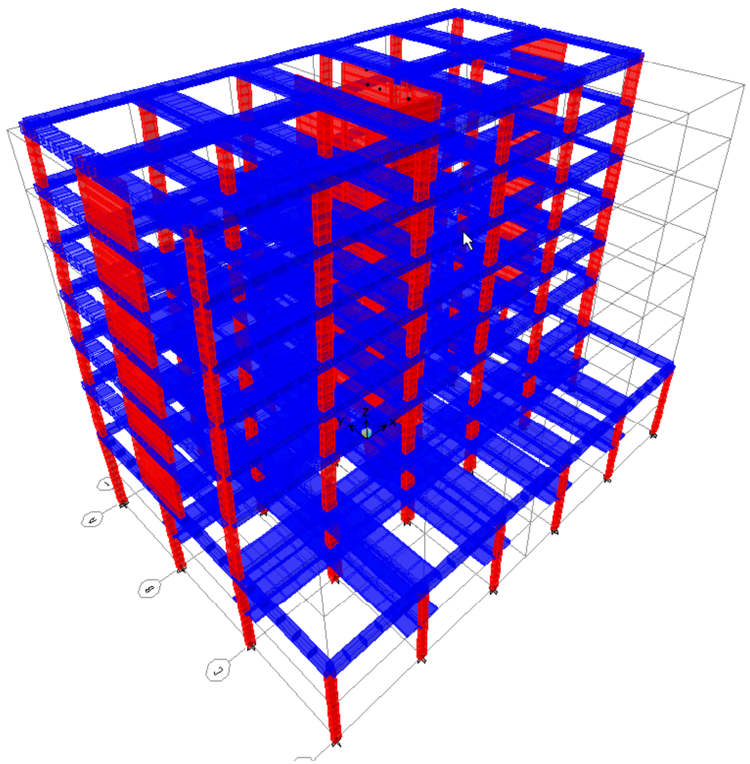
Mode	T (sec)	$M_{eff,UX}$ (%)	$M_{eff,UY}$ (%)	$M_{eff,MZ}$ (%)
1	0.92	80.2	0.0	0.2
2	0.68	0.0	76.3	0.0
3	0.51	0.2	0.0	75.2
4	0.22	15.0	0.0	0.2
5	0.15	0.0	18.5	0.0
6	0.12	0.2	0.0	17.6
	ΣM_{eff} =	95.7	94.7	93.1

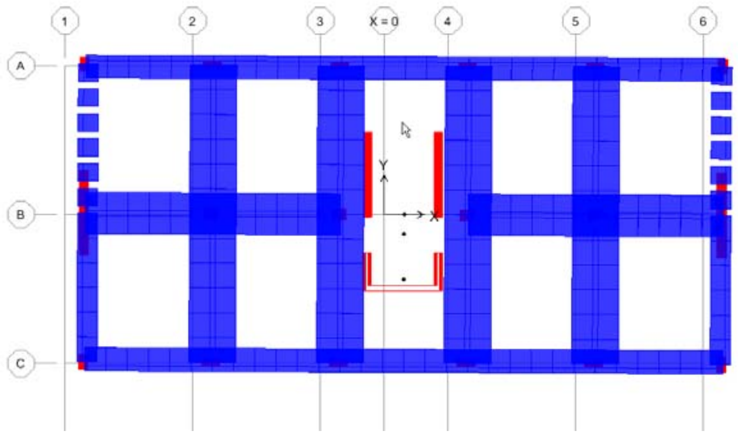


ETABS program

Periods, effective masses and modal shapes (2)

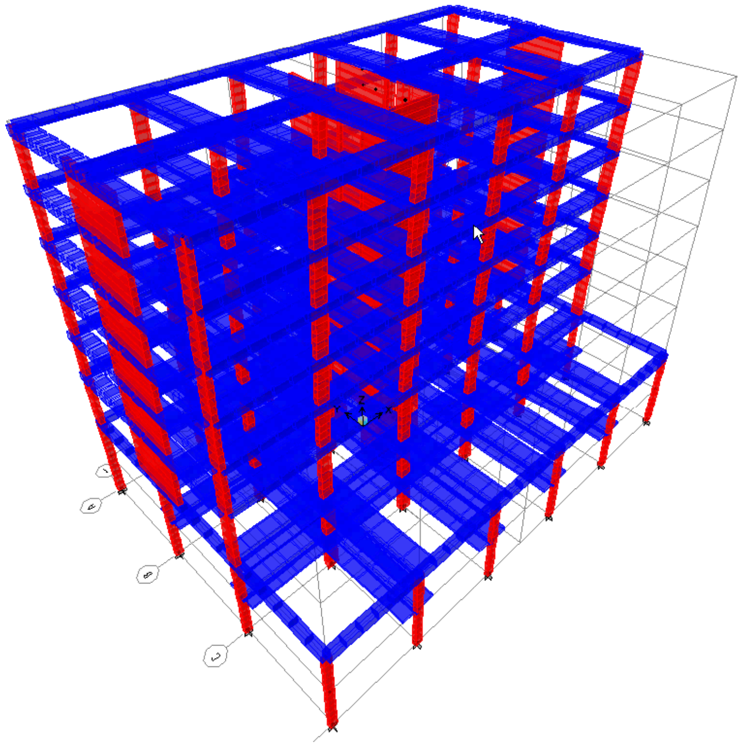
1. MODE – predominantly translational in X direction

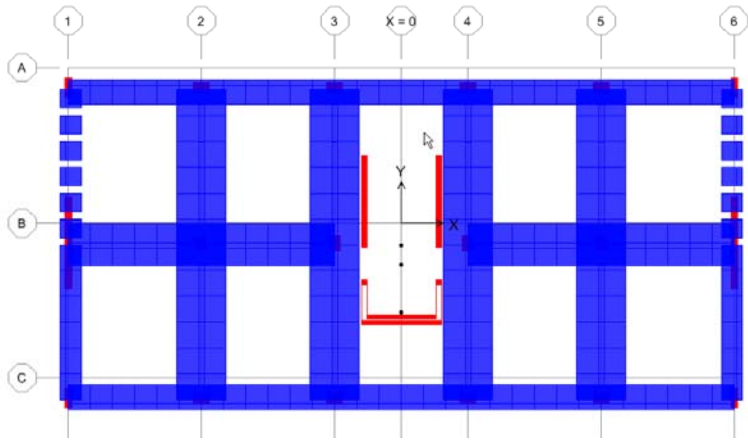




Periods, effective masses and modal shapes (3)

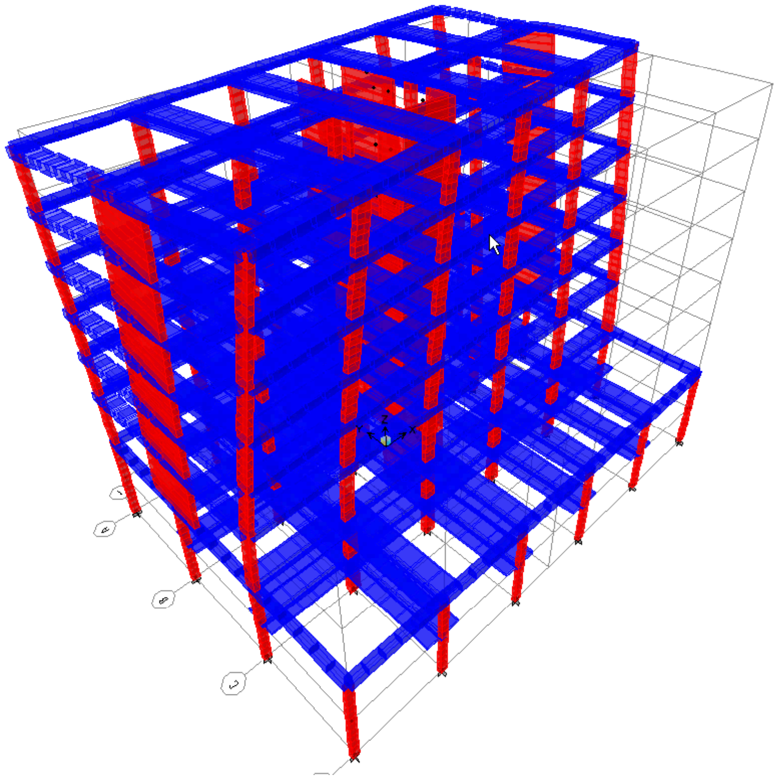
2. MODE – translational in Y direction

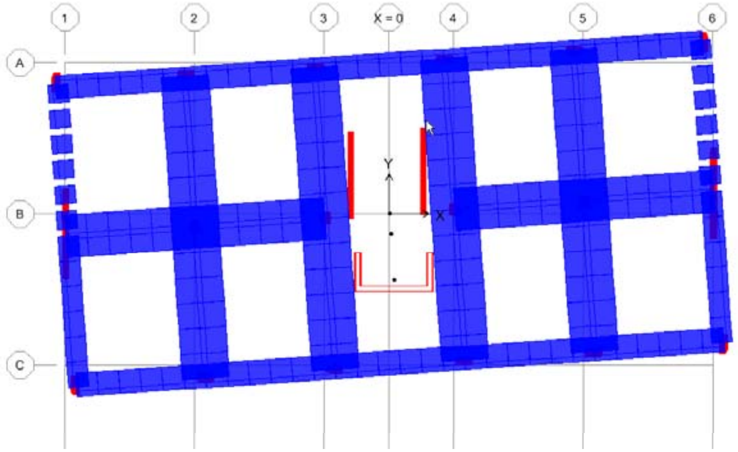




Periods, effective masses and modal shapes (4)

3. MODE – predominantly torsional





Modal response spectrum analysis RSA

- Modal response spectrum analysis was performed independently for the ground excitation in two horizontal direction
- Combination of diferent modes – CQC
- Combination of results in two directions – SRSS
- Design spectrum was used
- Accidental eccentricity was taken into account
- Seismic design situation

Accidental torsional effects

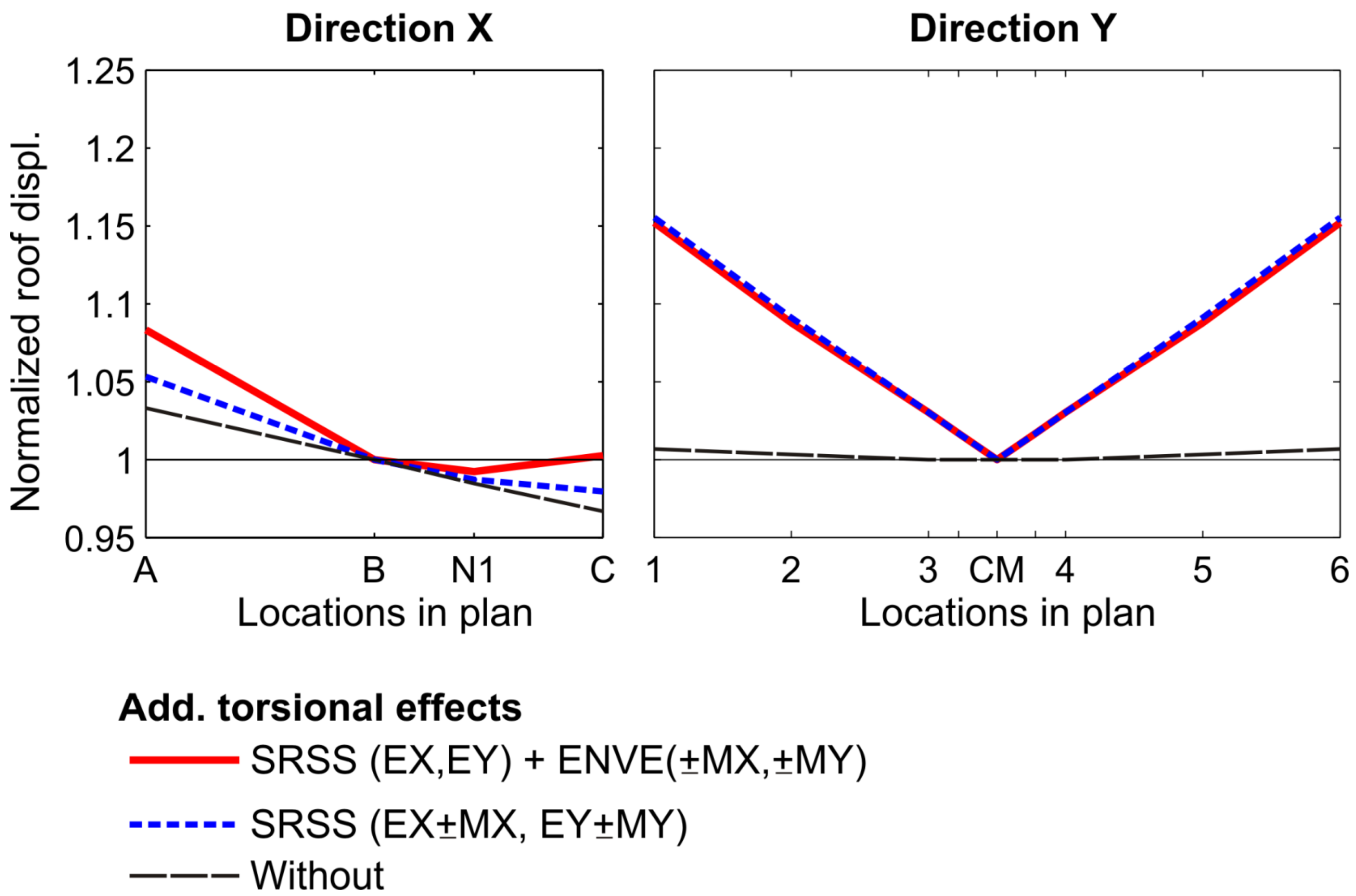
- Results of analysis without accidental torsion (SSRS of two horizontal directions) + envelope of accidental torsional effects

  $$SRSS\ (E_X, E_Y) + ENVE(\pm M_X, \pm M_Y)$$

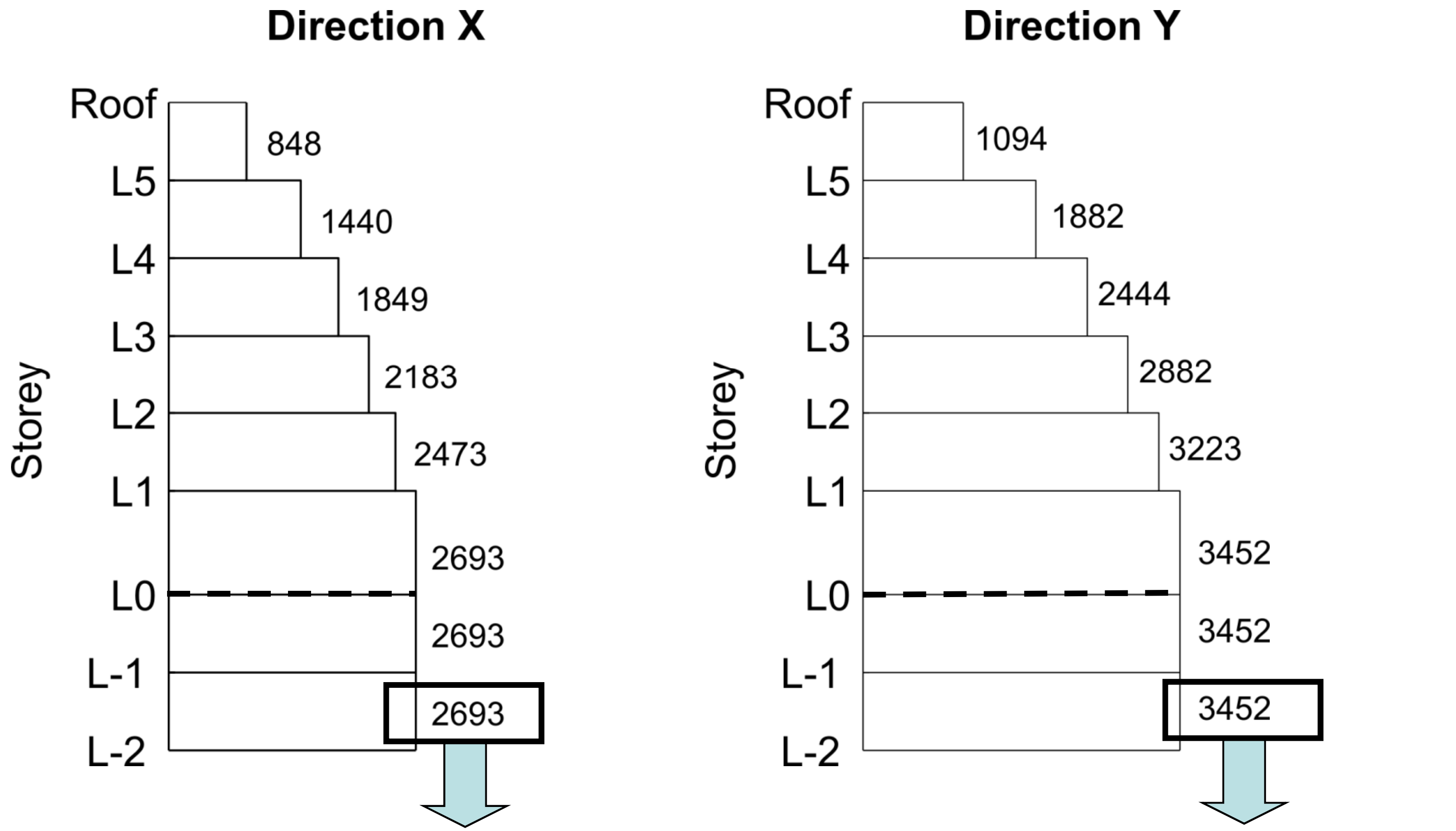
- Results of analysis without accidental torsion + accidental torsional effects, for each horizontal direction. SRSS combination of two horizontal directions

  $$SRSS\ (E_X \pm M_X, E_Y \pm M_Y)$$

RSA – Accidental torsional effects



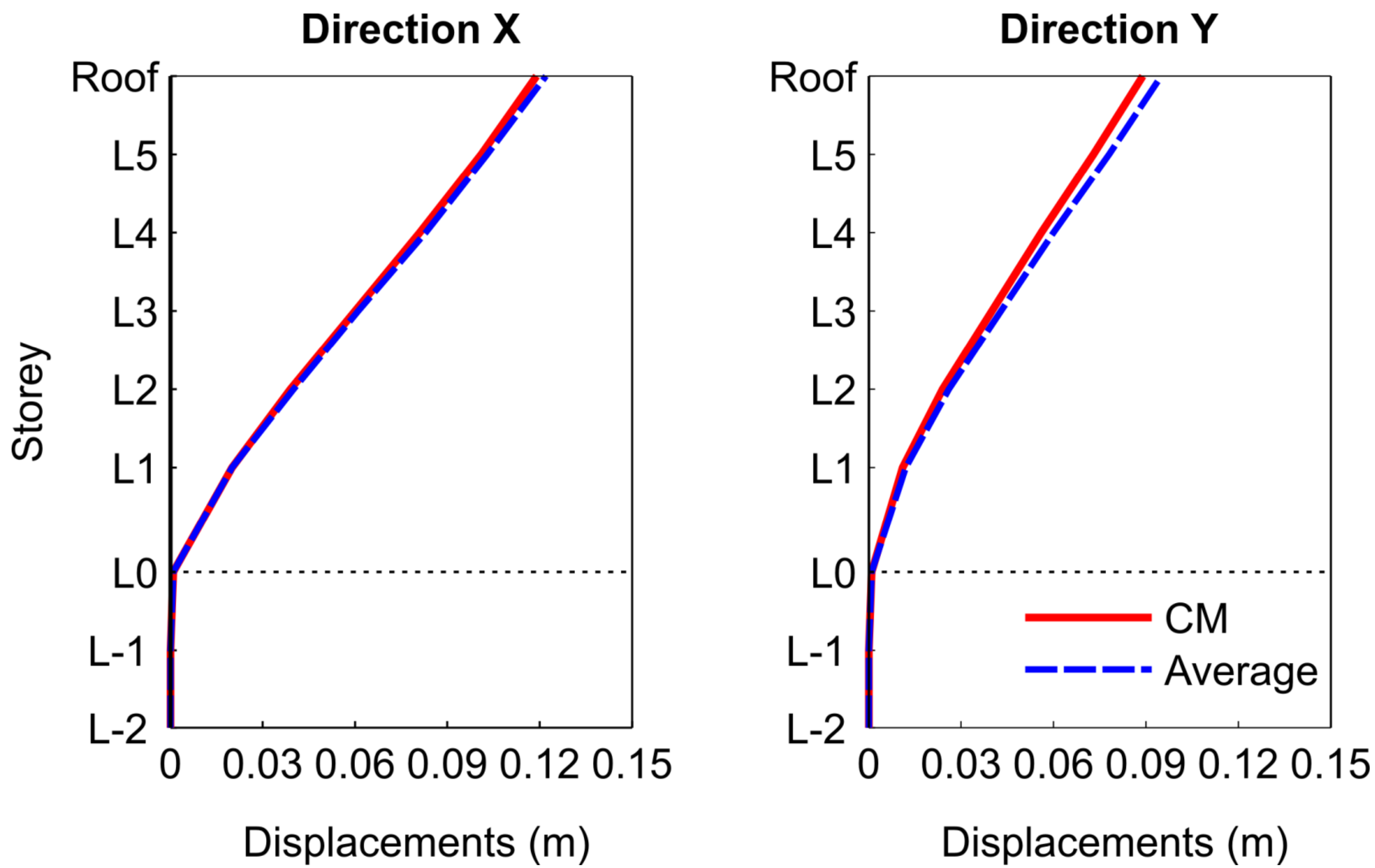
RSA – shear forces



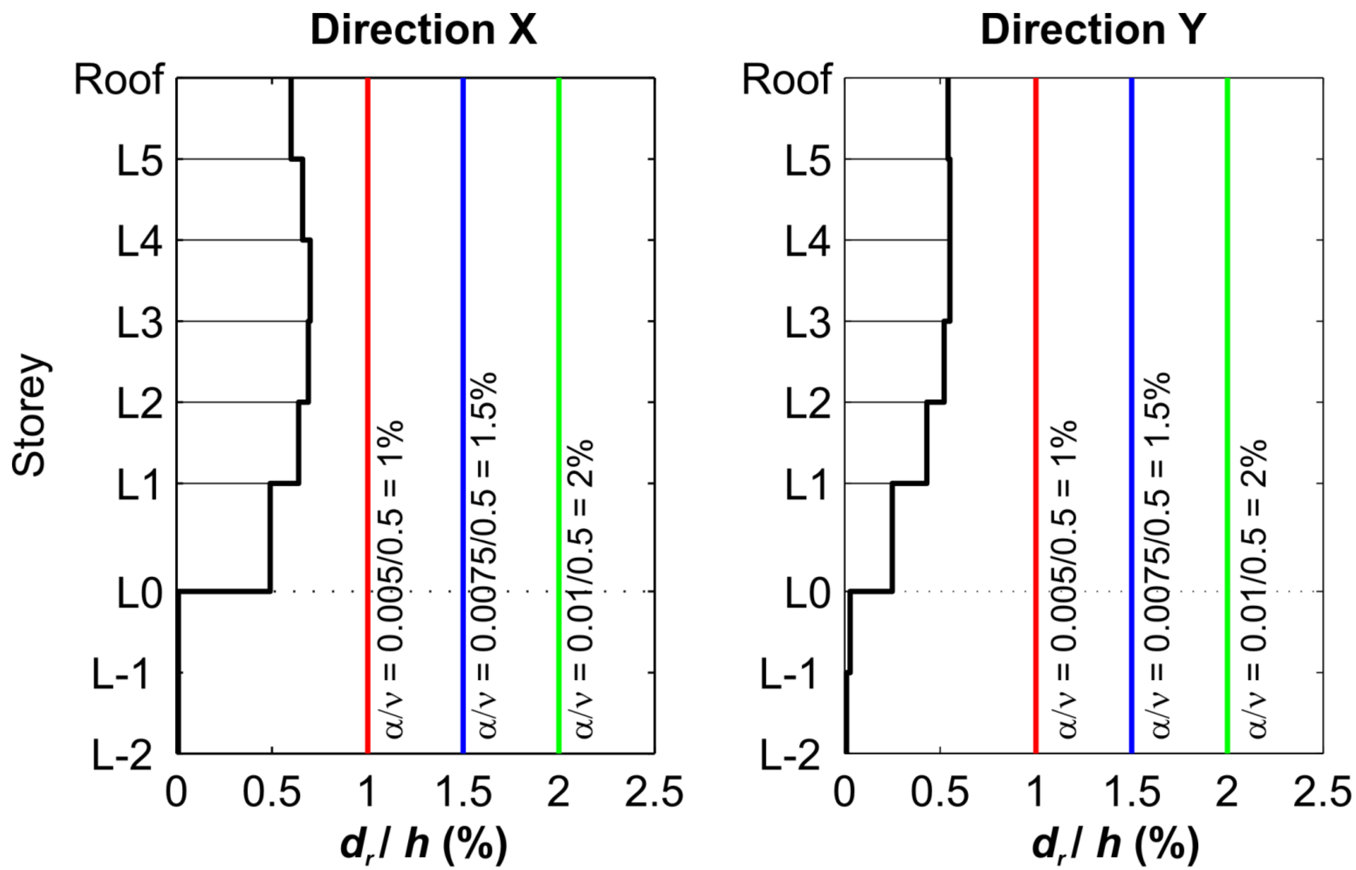
12% of the total weight

15% of the total weight

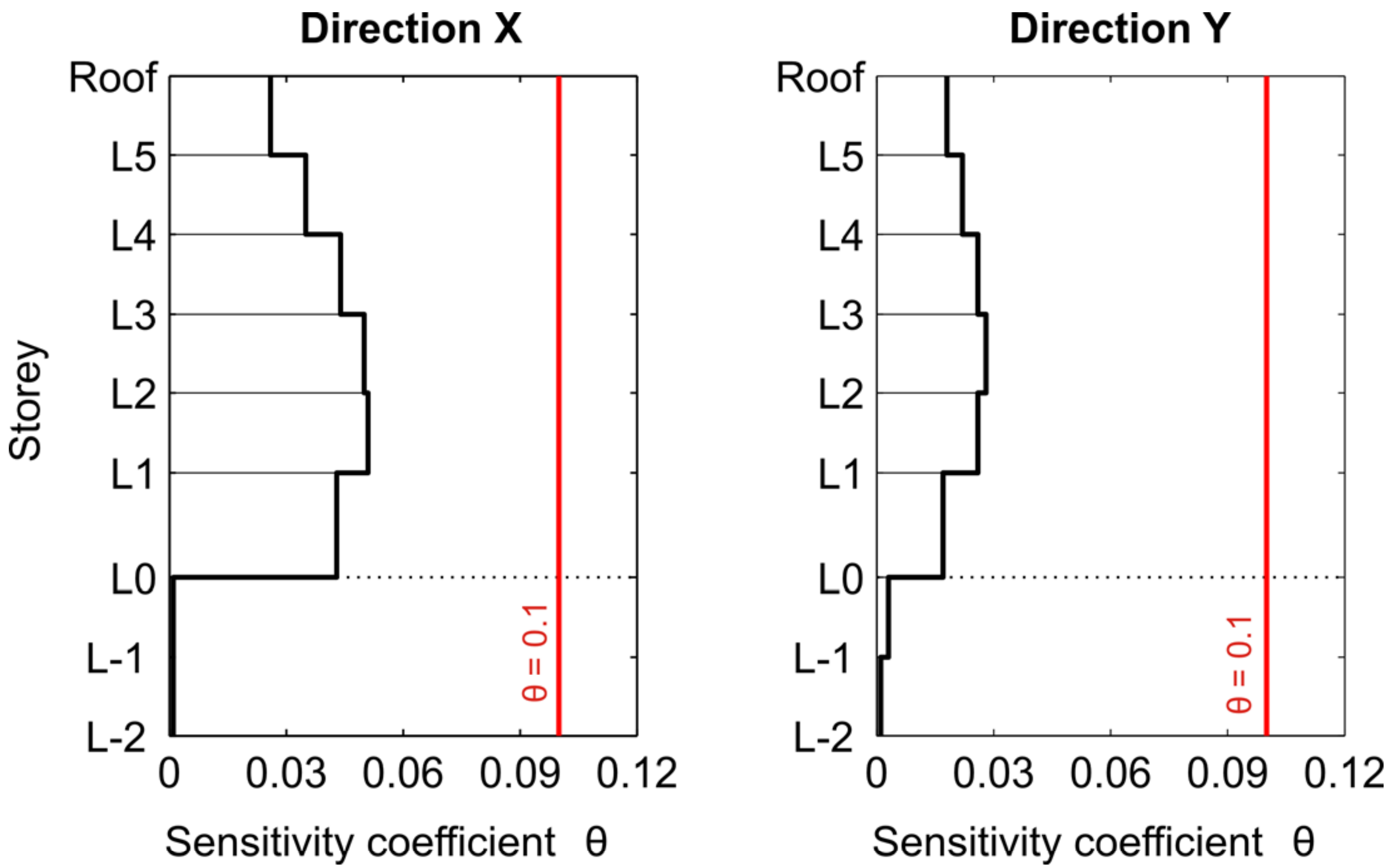
RSA - displacements



RSA – Damage limitations

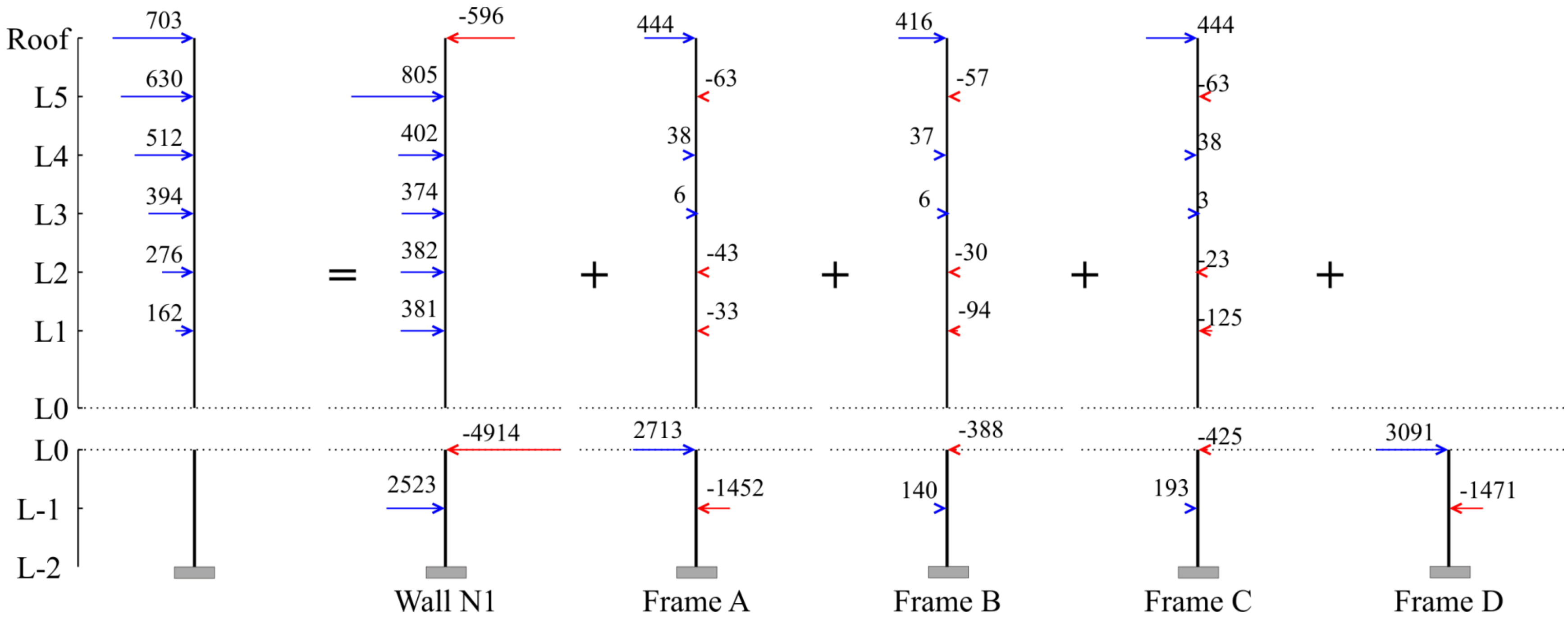


RSA – second order effects



Force distribution

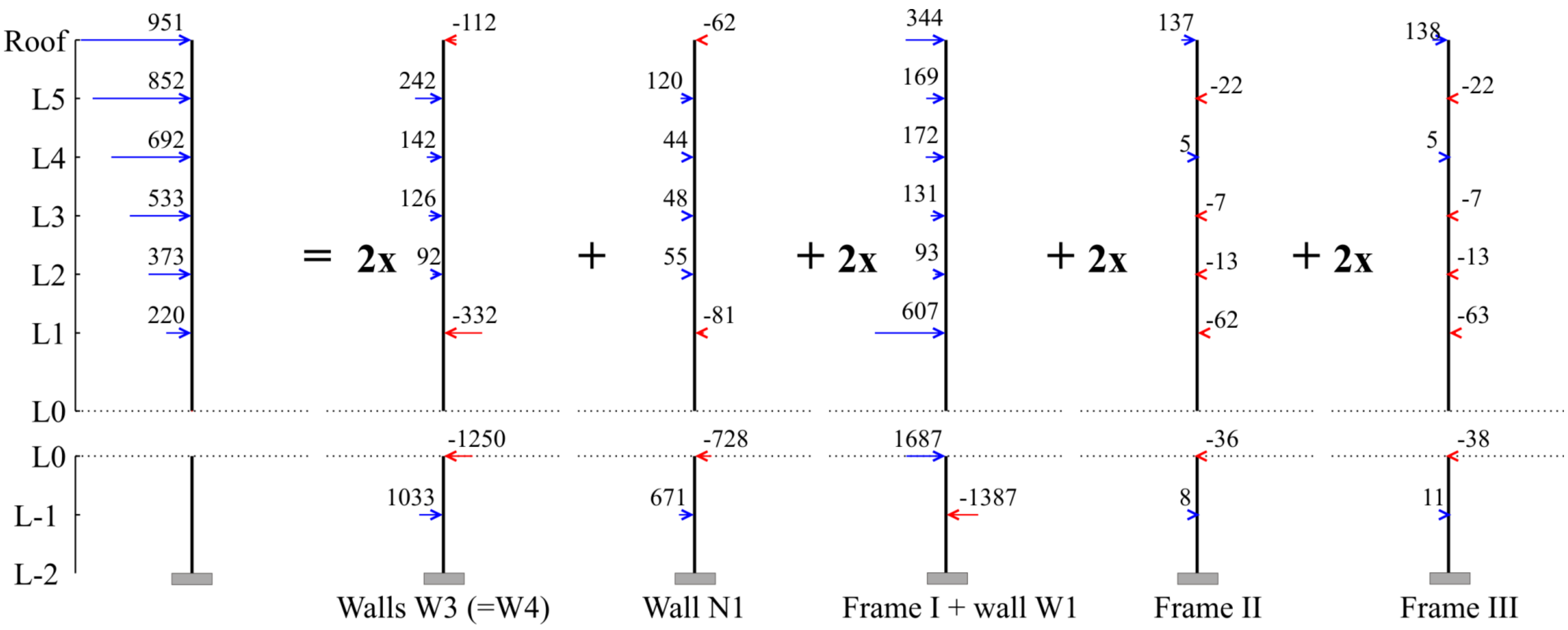
Direction X



Lateral force method

Force distribution

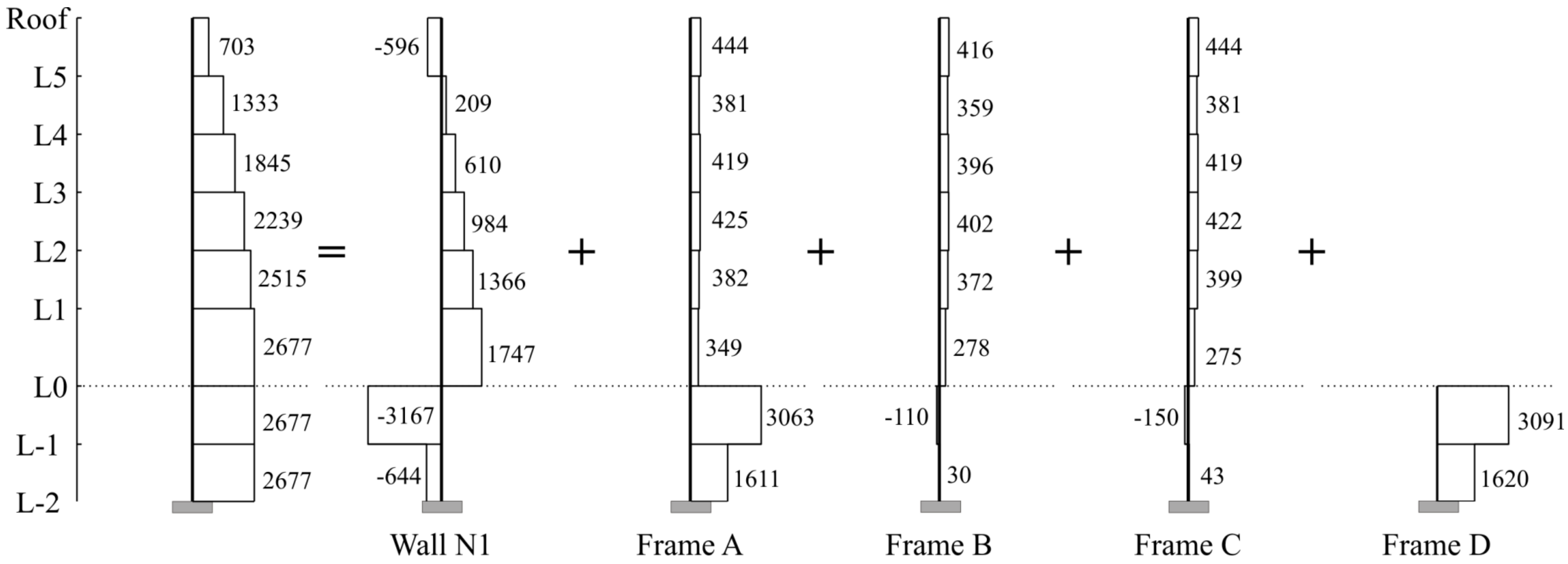
Direction Y



Lateral force method

Shear forces

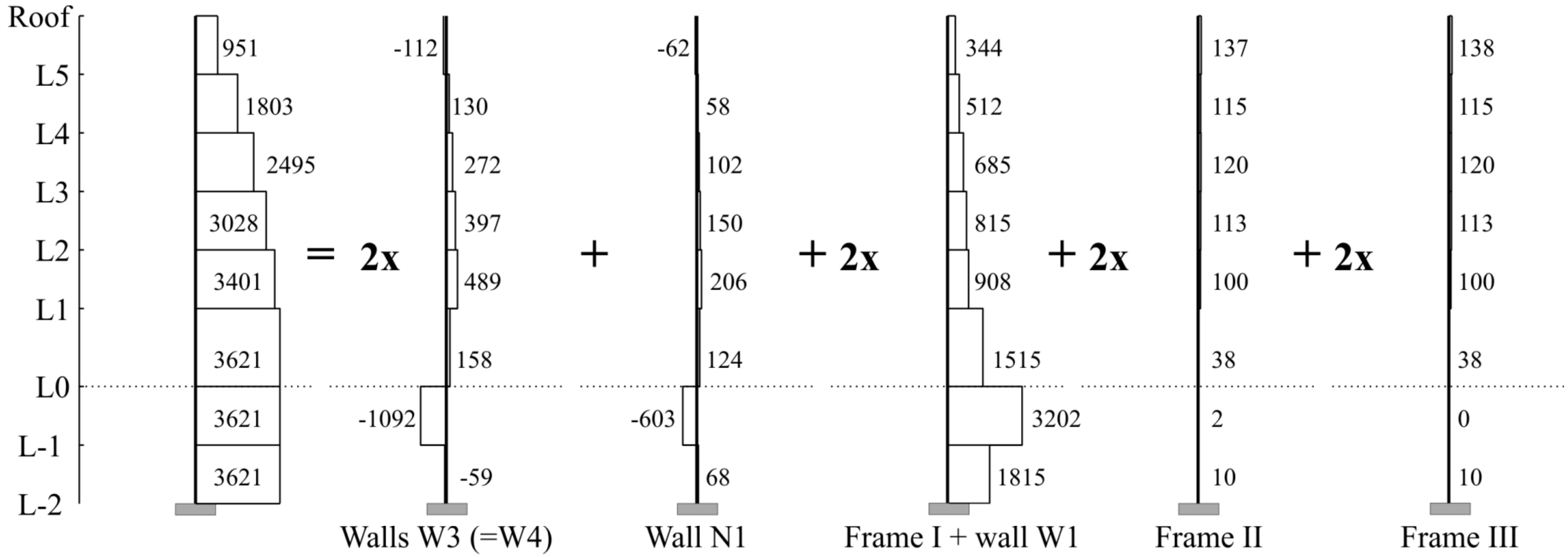
Direction X



Lateral force method

Shear forces

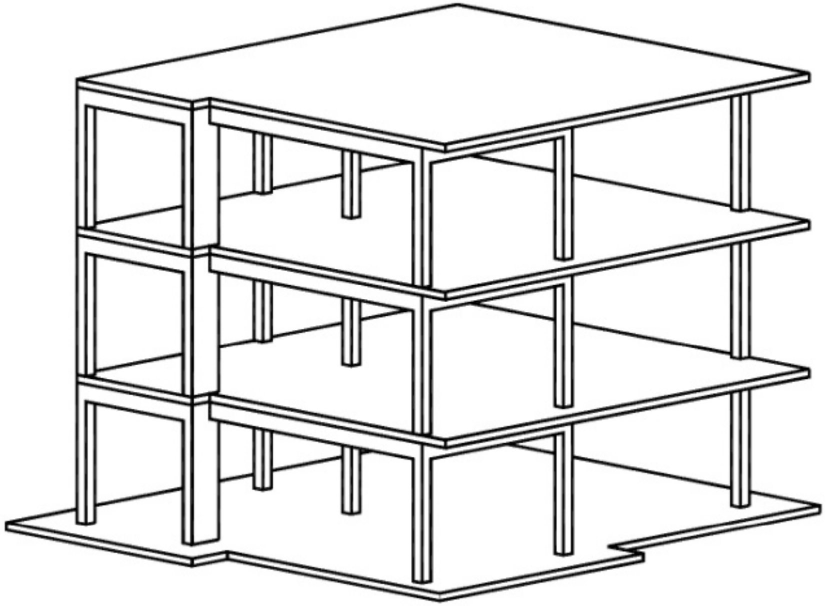
Direction Y



Lateral force method

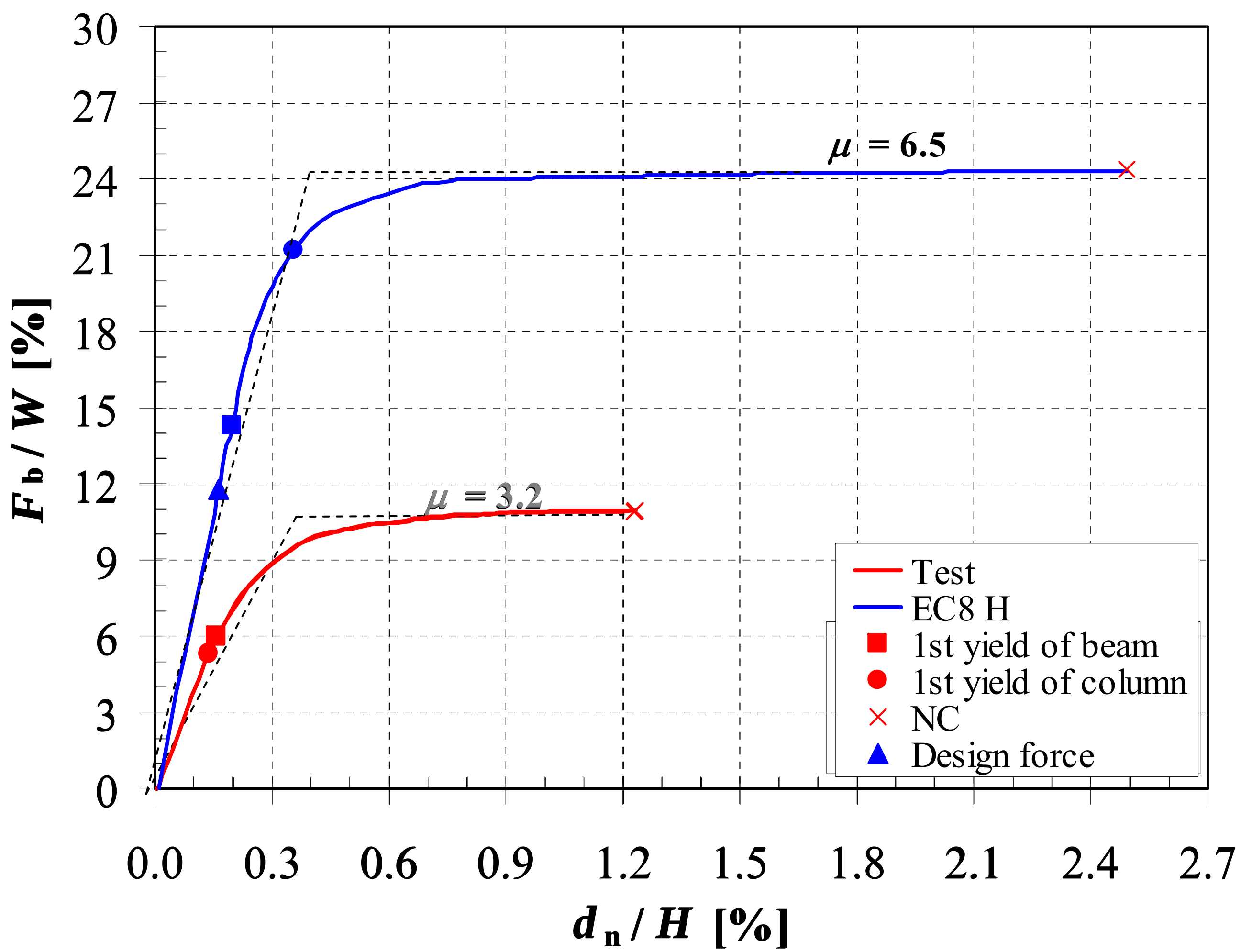
Code designed versus old buildings

SPEAR BUILDING



Pushover curves

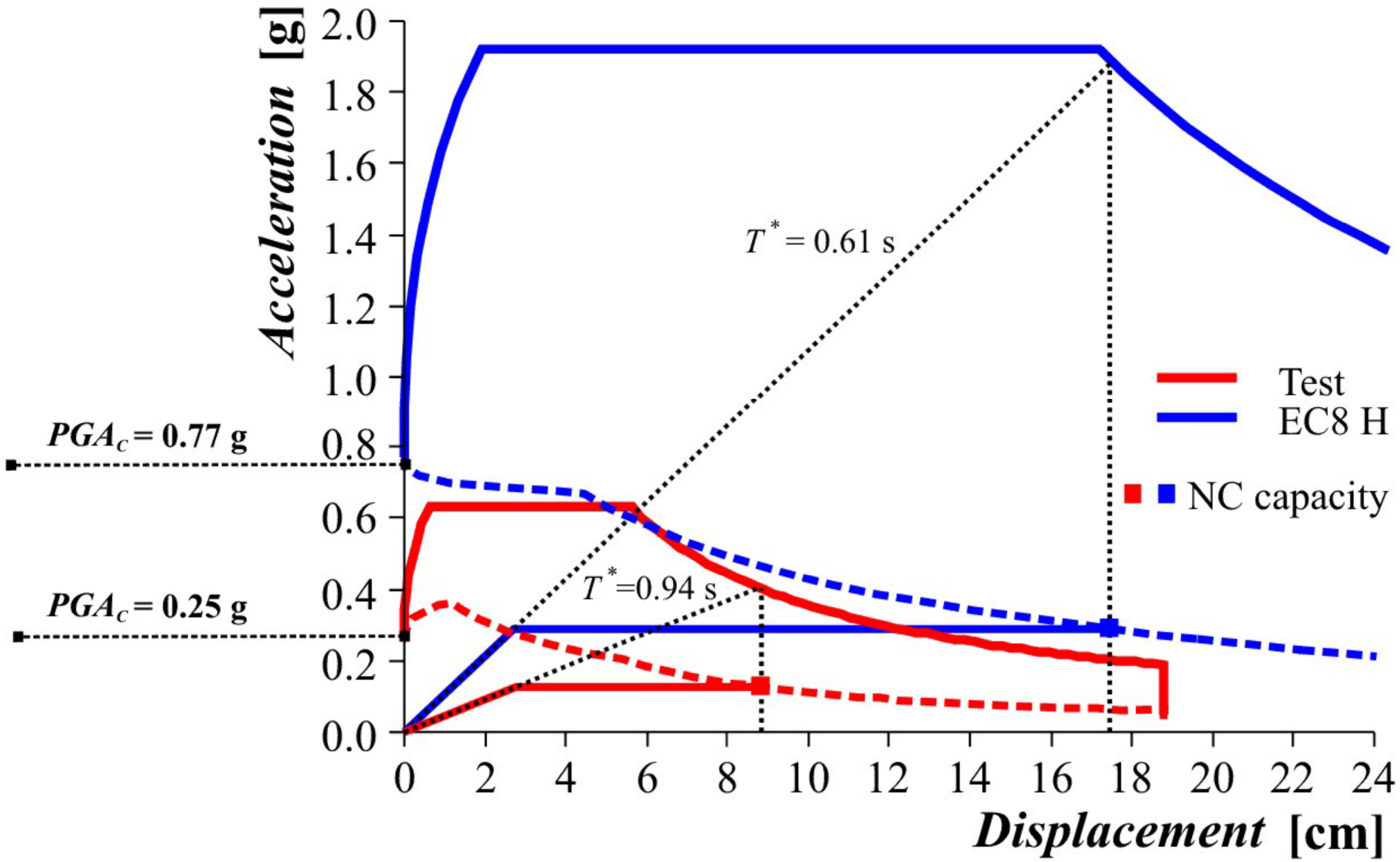
- Test
- EC8 H



X direction

Determination of seismic capacity (NC)

- Test
- EC8 H



Probability of “failure”

PGA_{475} = 0.25 g x 1.15 = 0.29 g (seismic hazard map, soil type C)

PGA_C = 0.25 g (test building), PGA_C = 0.77 g (EC8 building)

$P_{NC} = 0.78 \times 10^{-2}$ or 32% in 50 years (test building)

$P_{NC} = 2.67 \times 10^{-4}$ or 1.3% in 50 years (EC8 building)

Discussion of results

PGA_C = 0.77 g

“The code is too conservative!?”

$P_{NC,50}$ = 1.3 %

“The probability is too high!?”

How high is the tolerable probability?

How safe is safe enough?