Training seminar for Latvian structural engineers /
Apmācību seminārs Latvijas būvkonstrukciju projektētājiem:

Seismic-resistant structural design / Seismiski izturīgu konstrukciju projektēšana

Module 2:
Principles for Building Design in Seismic Zones

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CONTENT : Principles for Building Design in Seismic Zones

1. BASES OF DESIGN

- ✓ Seismic action
- ✓ Characteristics of earthquake resistant buildings

2. MODELLING AND STRUCTURAL ANALYSIS

- ✓ Modelling
- ✓ Methods of analysis

3. LATERAL FORCES METHOD OF ANALYSIS

- ✓ Concept
- ✓ Period estimation
- ✓ Distribution of forces between structural members
- ✓ Design of columns

4. DETAILING OF REINFORCED CONCRETE BUILDINGS

5. ANNEX - SPECIFIC RULES FOR STEEL BUILDINGS

GENERAL REFERENCE – EUROCODE 8 – Principles for Building Design in Seismic Zones

Part 1: General rules, seismic actions and rules for buildings Part 2: Bridges … Other

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1998-1

December 2004

ICS 91.120.25

Supersedes ENV 1998-1-1:1994, ENV 1998-1-2:1994, ENV 1998-1-3:1995

English version

Eurocode 8: Design of structures for earthquake resistance -
Part 1: General rules, seismic actions and rules for buildings

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 1: Règles générales, actions sismiques et règles pour les bâtiments

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben - Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten

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COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 36 B-1050 Brussels

2nd Generation:

Part 1-1: General rules and seismic action

Part 1-2: Buildings

FprEN 1998-1-2:2025

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1998-2

November 2005

ICS 91.120.25; 93.040

Supersedes ENV 1998-2:1994

English Version

Eurocode 8 - Design of structures for earthquake resistance -
Part 2: Bridges

Eurocode 8 - Calcul des structures pour leur résistance aux séismes - Partie 2: Ponts

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben - Teil 2: Brücken

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Part 2: Bridges

Part 3: Assessment and retrofitting of buildings

SEISMIC ACTION – Use **EN 1998-1** for the definition of the seismic action

CONSEQUENCE CLASS – Classify your BUILDING in one of the following CCs from **EN 1990**

Table 4.1 (NDP) — Qualification of consequence classes

Consequence class	Indicative qualification of consequences	
	Loss of human life or personal injury[a]	Economic, social or environmental consequences[a]
CC4 – Highest	Extreme	Huge
CC3 – High	High	Very great
CC2 – Normal	Medium	Considerable
CC1 – Low	Low	Small
CC0 – Lowest	Very low	Insignificant

[a] The consequence class is chosen based on the more severe of these two columns.

CC3 => "Special cases"

CC2 => "Normal buildings"

CONSEQUENCE CLASS – Classify your BUILDING in one of the following CCs from **EN 1990**

(3) Consequence class CC3 should be divided into CC3-a and CC3-b according to prEN 1998-1-1:2022, **4.2(3)**.

NOTE The definitions of buildings included in CC3-a and CC3-b are given in Table 4.1(NDP) unless a relevant Authority or the National Annex give different definitions for use in a country.

Table 4.1 (NDP) — Definitions of consequence classes CC3-a and CC3-b for buildings

CC3-a	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
CC3-b	Buildings of installations of vital importance for civil protection, e.g. hospitals, fire stations, etc. and their equipment, and buildings which should remain operational at all times, e.g. communications centres, data centres and their equipment.

NOTE Buildings that house very dangerous installations or materials are generally classified as CC4, which is not fully covered by the present standard.

LIMIT STATES AND ASSOCIATED SEISMIC ACTIONS – The seismic performance of a structure shall be measured by its state of damage under a given seismic action

a) LS of Near Collapse (NC) shall be defined as one in which the structure is heavily damaged, with large permanent drifts, but retains its vertical load bearing capacity; most ancillary components, where present, have collapsed.

b) LS of Significant Damage (SD) shall be defined as one in which the structure is significantly damaged, possibly with moderate permanent drifts, but retains its vertical-load bearing capacity; ancillary components, where present, are damaged (e.g., partitions and infills have not yet failed out-of-plane). The structure is expected to be repairable, but, in some cases, it may be uneconomic to repair.

ULS – Ultimate Limit States

c) LS of Damage Limitation (DL) shall be defined as one in which the structure is only slightly damaged and economic to repair, with negligible permanent drifts, undiminished ability to withstand future earthquakes and structural members retaining their full strength with a limited decrease in stiffness; ancillary components, where present, exhibit only minor damage that can be economically repaired (e.g. partitions and infills may show distributed cracking).

d) Fully Operational LS (OP) shall be defined as one in which the structure is only slightly damaged and economic to repair, allowing continuous operation of systems hosted by the structure remain in continuous operation.

SLS – Serviceability Limit States

PERFORMANCE FACTOR – Value to be multiplied by the seismic acceleration in function of the Limit State and CCs of the BUILDING (⇔"importance" factor)

Table 4.3 (NDP) — Return periods of seismic action in years

Limit state	Consequence class (CC)			
	CC1	CC2	CC3-a	CC3-b
NC	800	1600	2500	5000
SD	250	475	800	1600
DL	50	60	60	100

Table 4.4 (NDP) — Performance factors

Limit state (LS)	Consequence class (CC)			
	CC1	CC2	CC3-a	CC3-b
NC	1,2	1,5	1,8	2,2
SD	0,8	1	1,2	1,5
DL	0,4	0,5	0,5	0,6

"Normal Case" – Significant Damage ULS for CC2 building => Reference seismic action has been defined for such a case

CONCEPTUAL DESIGN – A building structure shall be able to resist horizontal seismic actions in any direction: ex. Combine the effects of the seismic actions $E_{\mathrm{E,x}}$ "+" $0{,}30\ E_{\mathrm{E,y}}$

PRIMARY AND SECONDARY SEISMIC MEMBERS

(1) A certain number of structural members (e.g. beams and/or columns) may be designated as "secondary" seismic members, not forming part of the seismic action resisting system of the building. The strength and stiffness of these members against seismic actions should be neglected in the models for seismic analysis.

(2) The secondary seismic members and their connections should be designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the most unfavourable seismic design condition. Due allowance of second order effects (P-Δ effects) in both the structure and these members should be made in the design of these members.

(3) Secondary columns which yield under the displacements caused by the seismic design condition in (2) should satisfy the requirements for local ductility of columns.

PRIMARY AND SECONDARY SEISMIC MEMBERS

(4) The designation of some structural members as secondary seismic members should not change the classification of the structure from non-regular to regular as described in the sequence.

(5) The contribution of lateral stiffness of secondary members to the total stiffness should not affect significantly the dynamic behaviour of the structure. To fulfil this requirement, should be satisfied:

The total contribution to lateral stiffness of all secondary seismic members does not exceed 15% of that of all primary seismic members.

(6) The primary structure should be assigned a ductility class according to EN 1998-1 – DC2 or DC3 (defined in the following). The ductility class should be unique for the building.

PRIMARY AND SECONDARY SEISMIC MEMBERS

Primary / secondary columns in the direction X

$$K_{x,1} = \sum_i E\, I_{xi}/h_i^3 = 4E\,\frac{0{,}3\cdot 1{,}5^3}{4^3} + 6E\,\frac{1{,}2\cdot 0{,}3^3}{4^3} = 0{,}0663E \text{ m}^4$$

$$K_{x,2} = \sum_i E\, I_{xi}/h_i^3 = 4E\,\frac{0{,}3\cdot 0{,}3^3}{4^3} + 6E\,\frac{0{,}5\cdot 0{,}5^3}{4^3} = 0{,}0064E \text{ m}^4$$

Ratio $K_{x,2} / K_{x,1} = 9{,}6\%$ ✓

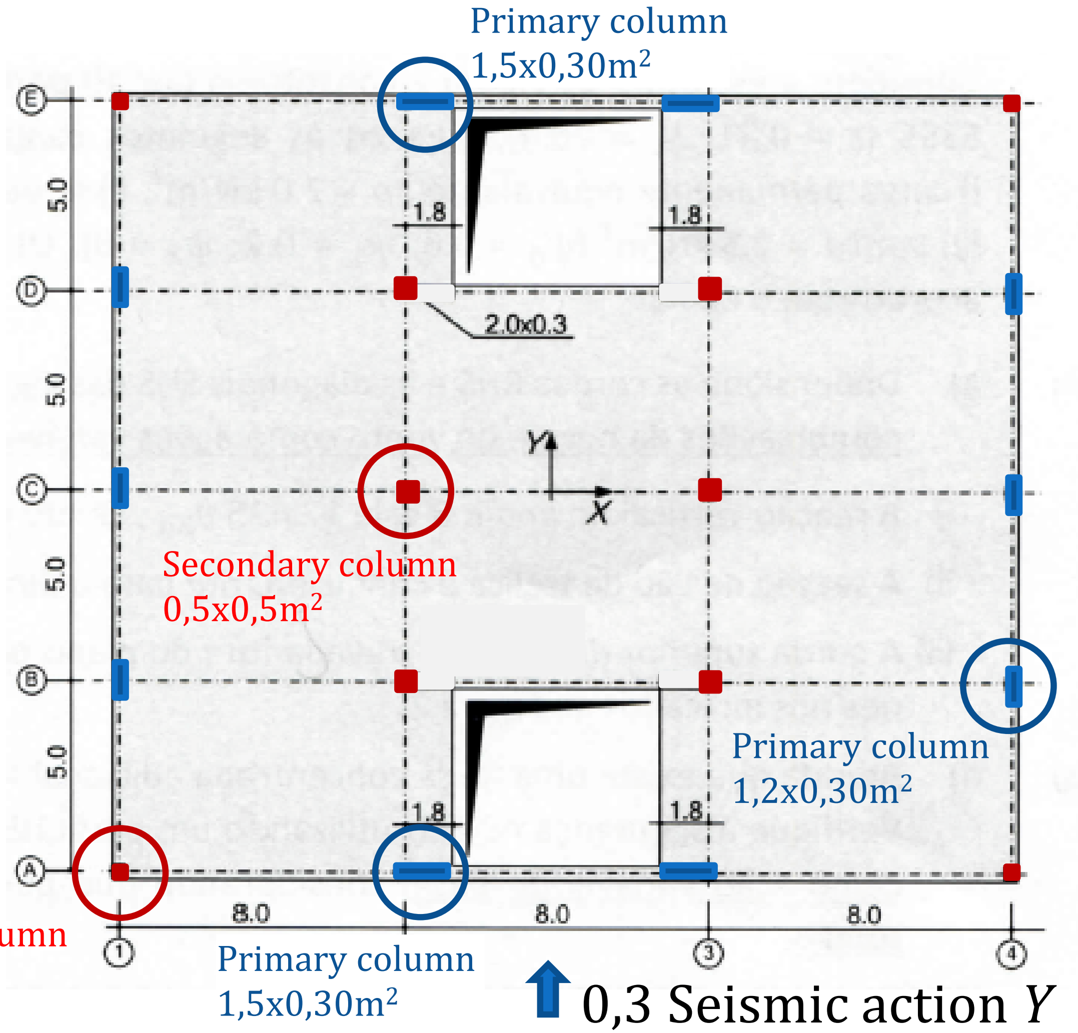
1,0 Seismic action X

Primary / secondary columns in the direction Y

$$K_{y,1} = \sum_i E\, I_{yi}/h_i^3 = 4E\,\frac{1{,}5\cdot 0{,}3^3}{4^3} + 6E\,\frac{0{,}3\cdot 1{,}2^3}{4^3} = 0{,}0511E \text{ m}^4$$

$$K_{y,2} = \sum_i E\, I_{yi}/h_i^3 = 4E\,\frac{0{,}3\cdot 0{,}3^3}{4^3} + 6E\,\frac{0{,}5\cdot 0{,}5^3}{4^3} = 0{,}0064E \text{ m}^4$$

Ratio $K_{y,2} / K_{y,1} = 12{,}5\%$ ✓



0,3 Seismic action Y

DUCTILITY CLASS – Regarding their deformation capacity and cumulative energy dissipation capacity, structures should be categorised in three ductility classes: DC1 (=DCL-Low Ductility), DC2 (=DCM-Medium Ductility) and DC3 (=DCH-High Ductility).

In DC1, the overstrength capacity is taken into account, while the deformation capacity and energy dissipation capacity are disregarded.
DCL "LOW DUCTILITY" > Ex. Unreinforced masonry structures; "low" behaviour factors ($q \leq 1{,}5$ to 2)

In DC2, the local overstrength capacity, the local deformation capacity and the local energy dissipation capacity are taken into account. Global plastic mechanisms are controlled.
DCM "MEDIUM DUCTILITY" > Reinforce concrete structures; "medium" behaviour factors ($2 \leq q < 3$)

In DC3, the ability of the structure to form a global plastic mechanism at SD limit state and its local overstrength capacity, local deformation capacity and local energy dissipation capacity are taken into account.
DCH "HIGH DUCTILITY" > Steel and Composite steel-concrete structures; "high" behaviour factors ($q \geq 3$)

DUCTILITY CLASS – Regarding their deformation capacity and cumulative energy dissipation capacity, structures should be categorised in three ductility classes: DC1 (=DCL-Low Ductility), DC2 (=DCM-Medium Ductility) and DC3 (=DCH-High Ductility).

Ex. Regular concrete building cast-in-situ with ductility class DC2

Table 10.1 — Default upper limit values of the behaviour factors

<table>
<tr><th colspan="2" rowspan="2">Structural type</th><th rowspan="2">q_R</th><th colspan="2">q_D</th><th colspan="2">$q = q_R\, q_S\, q_D$</th></tr>
<tr><th>DC2</th><th>DC3</th><th>DC2</th><th>DC3</th></tr>
<tr><td rowspan="3">Moment resisting frame or moment resisting frame-equivalent dual structures without infills or with non-interacting infills</td><td>multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual structures</td><td>1,3</td><td rowspan="3">1,3</td><td rowspan="3">2,0</td><td>2,5</td><td>3,9</td></tr>
<tr><td>multi-storey, one-bay moment resisting frames</td><td>1,2</td><td>2,3</td><td>3,6</td></tr>
<tr><td>one-storey moment resisting frames</td><td>1,1</td><td>2,1</td><td>3,3</td></tr>
<tr><td colspan="2">Moment resisting frame or moment resisting frame-equivalent dual structures with interacting masonry infills</td><td>1,1</td><td>1,2</td><td>1,7</td><td>2,0</td><td>3,0</td></tr>
<tr><td rowspan="4">Wall or wall-equivalent dual structures</td><td>wall-equivalent dual structures</td><td>1,2</td><td>1,3</td><td rowspan="3">2,0</td><td>2,3</td><td>3,6</td></tr>
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<tr><td>uncoupled walls structures</td><td>1,0</td><td>1,3</td><td>2,0</td><td>3,0</td></tr>
<tr><td>large walls structures</td><td>--</td><td colspan="2">--</td><td colspan="2">3,0 k_w</td></tr>
<tr><td colspan="2">Flat slab structures</td><td>1,1</td><td>1,2</td><td>--</td><td>2,0</td><td>--</td></tr>
</table>

IN-PLAN REGULARITY – Although not mandatory, achieving building regularity should be regarded as a good practice for the design of earthquake resistant buildings.

A building structure is classified as regular in-plan if:

- ✓ With respect to the lateral stiffness and mass distribution, the building structure is approximately symmetric in plan with respect to two orthogonal axes.
- ✓ The plan configuration is compact, i.e., each floor is delimited by a polygonal convex line. If in plan setbacks (re-entrant corners or edge recesses) exist, they do not affect the floor in-plan stiffness. For each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 15 % of the floor area.
- ✓ The in-plan stiffness of the floors is sufficiently large in comparison with the lateral stiffness of the vertical structural members, so that the deformation of the floor has a small effect on the distribution of the forces among the vertical structural members.
- ✓ The in-plan slenderness $\lambda = L_{max}/L_{min}$ of the building is not greater than 4, where L_{max} and L_{min} are respectively the greater and smaller in plan dimension of the building, measured in orthogonal directions.

IN-PLAN REGULARITY

✓ At each level i and for each direction of analysis x or y, the structural eccentricity e_o and the torsional radius r satisfy the two conditions given by the following formulas, expressed for the direction of analysis x (similar formulas apply for the direction y).

$$e_{ox} = |x_{CR} - x_{CM}| \leq 0{,}30 \cdot r_x \qquad , \qquad r_x = \sqrt{\frac{k_\theta}{k_y}} \geq \ell_s$$

where:

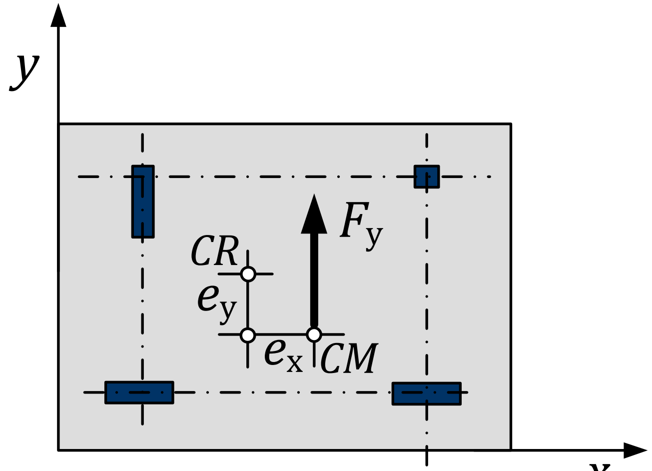
e_{ox} is the x component of the natural eccentricity, normal to the direction of analysis y considered

r_x is the x component of the torsional radius

ℓ_s is the radius of gyration of floor i ($l_{s,i}{}^2 = \frac{I_{x,i}+I_{y,i}}{m_i}$;)

$I_{x,i}$, $I_{y,i}$ are the mass moments of inertia of the floor around axis x and y respectively, going through the floor centre of mass CM_x

m_i is the mass of floor i



CR = centre of rigidity

CM = centre of mass

k_x , k_y – lateral stiffness of the vertical members of one floor

k_θ – torsional stiffness of the floor with respect CR_i

TORSIONALLY FLEXIBLE BUILDINGS

A building is classified as torsionally flexible in one direction if:

$$r_x = \sqrt{\frac{k_\theta}{k_y}} < \ell_s \quad \text{or} \quad r_y = \sqrt{\frac{k_\theta}{k_x}} < \ell_s$$

where:

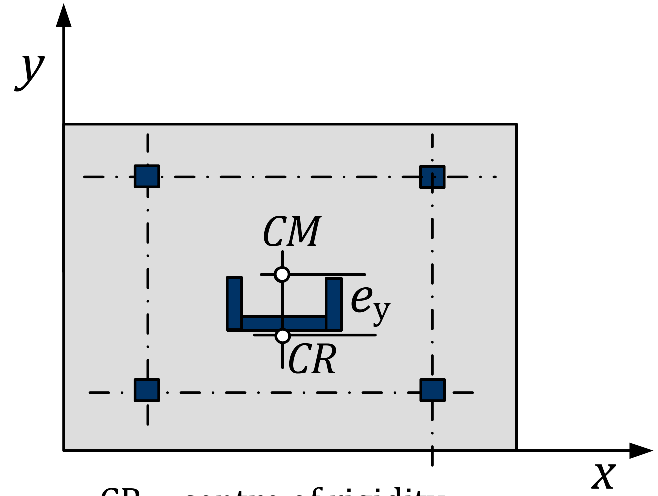
r_x is the x component of the torsional radius

ℓ_s is the radius of gyration of floor i ($l_{s,i}{}^2 = \frac{I_{x,i}+I_{y,i}}{m_i}$)

$I_{x,i}$, $I_{y,i}$ are the mass moments of inertia of the floor around axis x and y respectively, going through the floor centre of mass CM_x

m_i is the mass of floor i

In such a case the building can not be considered regular in-plan which **implies a 3D analysis and a lower behaviour factor** (q_R should be taken equal to 1,0 and the result value multiplied by 0,8)



CR = centre of rigidity

CM = centre of mass

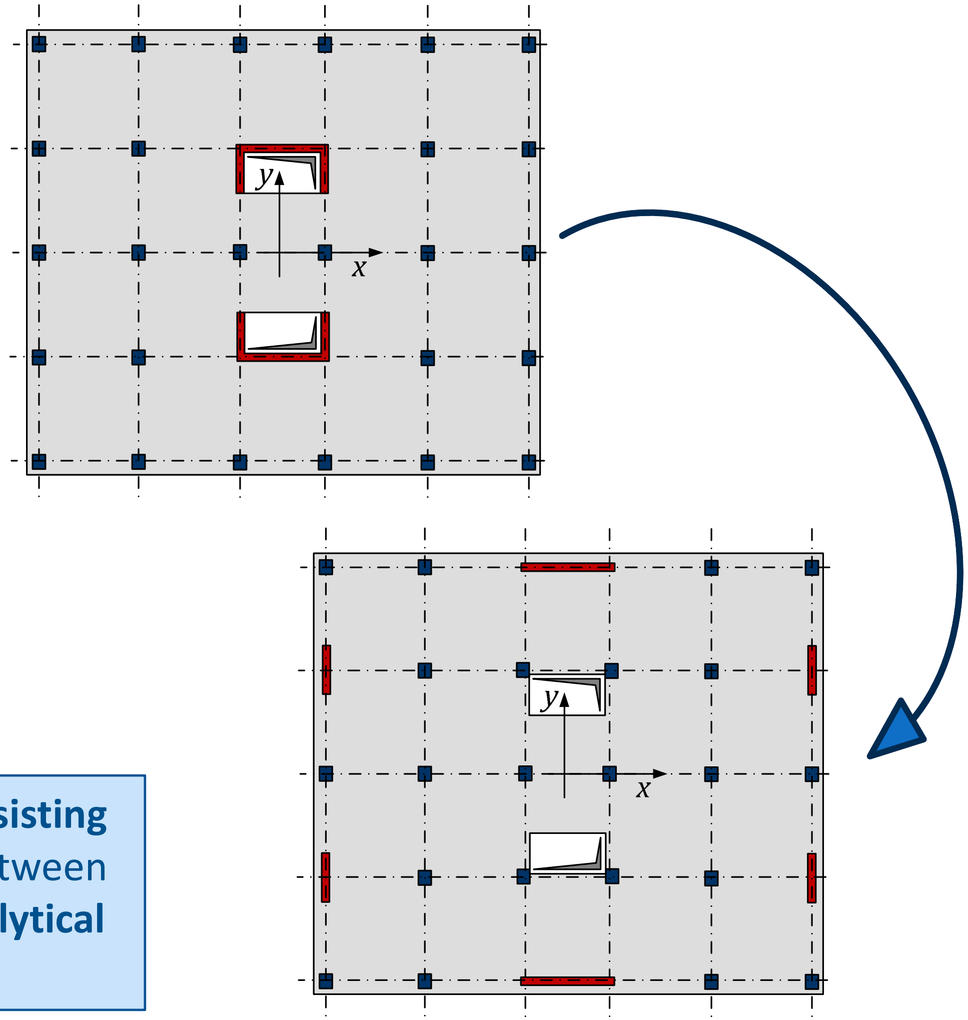
k_x , k_y – lateral stiffness of the vertical members of one floor

k_θ – torsional stiffness of the floor with respect CR_i

TORSIONALLY FLEXIBLE BUILDINGS –

How to avoid?

- ✓ In addition to lateral rigidity, building structures should provide sufficient torsional rigidity to limit the development of torsional movements, which induce non-uniform stresses in structural members.
- ✓ Therefore, structural layouts where the primary seismic-resisting elements are positioned near the building's perimeter are much preferable.



NOTE – **Concrete buildings with stiff vertical earthquake-resisting structures near the perimeter**, symmetrically distributed between parallel sides, **may be deemed torsionally rigid without analytical verification**.

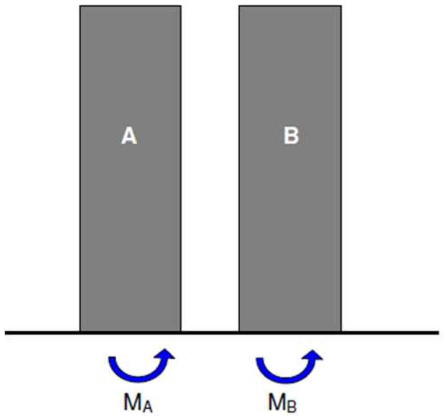
ELEVATION REGULARITY – A building structure is classified as regular in elevation if it complies with the following conditions:

- ✓ The primary structural elements (cores, structural walls, frames, and diaphragms) ensure a continuous resisting system from the foundation to the top of the building or the respective zone in the case of setbacks.

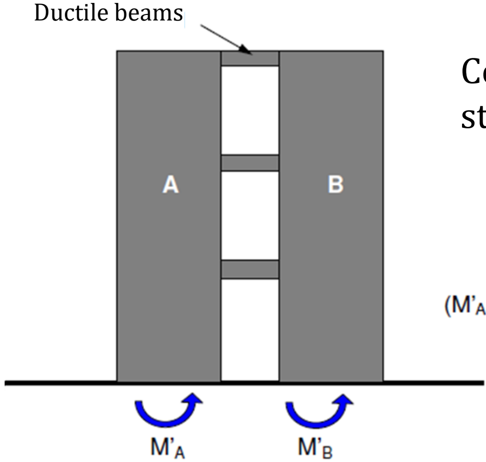
- ✓ Lateral rigidity and mass of the storeys remain constant or decrease gradually (≤20% relative to the storey below), so without abrupt changes.

If one of the conditions is not fulfil the building can not be considered regular in elevation which implies: i) **a 3D analysis using the response spectrum method** and ii) **adopting a behaviour factor reduced to 0,8 of the upper default value**.

BEHAVIOUR FACTORS – Concrete buildings should be classified into one of the structural types according to their behaviour under horizontal seismic actions:

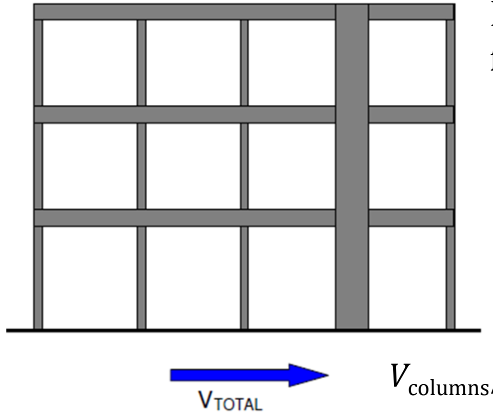


Uncoupled walls structures



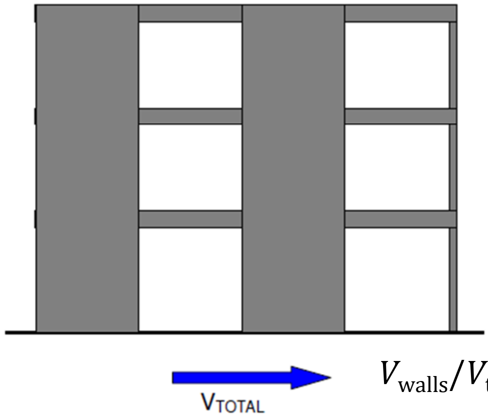
Coupled walls structures

$(M'_A + M'_B) < 75\% \, (M_A + M_B)$



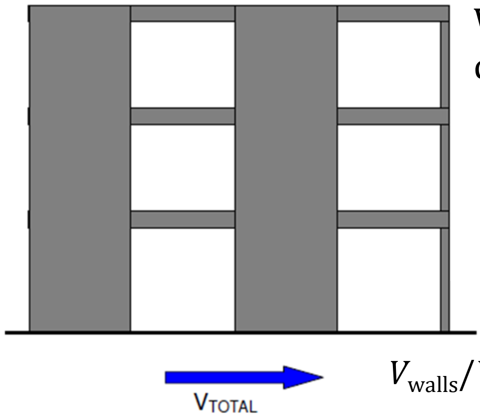
Moment resisting frame structure

$V_{columns}/V_{total} \geq 65\%$



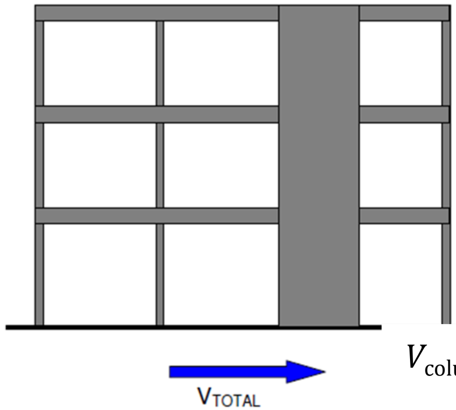
Wall structures

$V_{walls}/V_{total} \geq 65\%$



Wall-equivalent dual structures

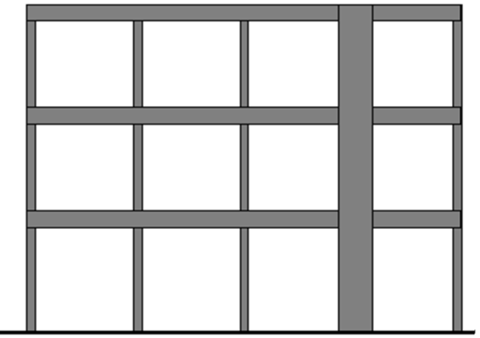
$V_{walls}/V_{total} \geq 50\%$

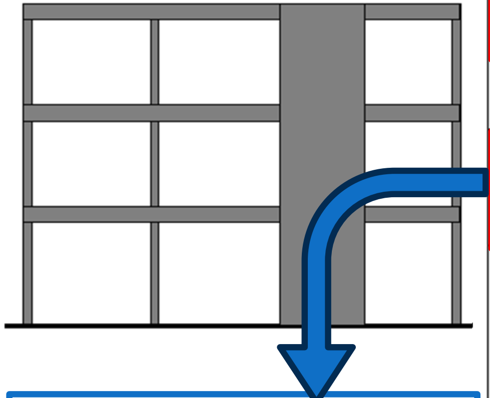


Moment resisting frame-equivalent dual structures

$V_{columns}/V_{total} \geq 50\%$

Table 10.1 — Default upper limit values of the behaviour factors





Structural type	q_S=1,5	q_R	q_D		$q = q_R\, q_S\, q_D$	
			DC2	**DC3**	**DC2**	**DC3**
Moment resisting frame or moment resisting frame-equivalent dual structures without infills or with non-interacting infills	multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual structures	1,3	1,3	2,0	2,5	3,9
	multi-storey, one-bay moment resisting frames	1,2			2,3	3,6
	one-storey moment resisting frames	1,1			2,1	3,3
Moment resisting frame or moment resisting frame-equivalent dual structures with interacting masonry infills		1,1	1,2	1,7	2,0	3,0
Wall or wall-equivalent dual structures	wall-equivalent dual structures	1,2	1,3	2,0	2,3	3,6
	coupled walls structures	1,2	1,4		2,5	3,6
	uncoupled walls structures	1,0	1,3		2,0	3,0
	large walls structures	--	--		3,0 k_w	
Flat slab structures		1,1	1,2	--	2,0	--

Note – For a DC3 building with MRF-eq. dual struct. torsionally flexible:
$q = 3 \cdot (1/1{,}1) \cdot 0{,}8 = 2{,}2$

METHODS OF ANALYSIS – A building structure due to seismic actions can be analysed using the following methods:

1. **Lateral forces method of analysis** – consists in applying a system of static forces in each principal direction, may be applied to structures whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.
2. **Response spectrum method** – consists of a modal linear-elastic analysis using the seismic acceleration spectrum. It may be applied in all cases and, particularly, to structures which do not satisfy the conditions for application of the lateral force method.
3. **Non-linear static analysis** – based on a pushover analysis and an equivalent single-degree-of-freedom (SDOF) model, may be applied: a) to verify the structural performance of newly designed structures; b) to assess the structural performance of existing or retrofitted structures.
4. **Response history analysis** – The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using different accelerograms.

METHODS OF ANALYSIS – WHEN THE LATERAL FORCE METHOD CAN BE ADOPTED?

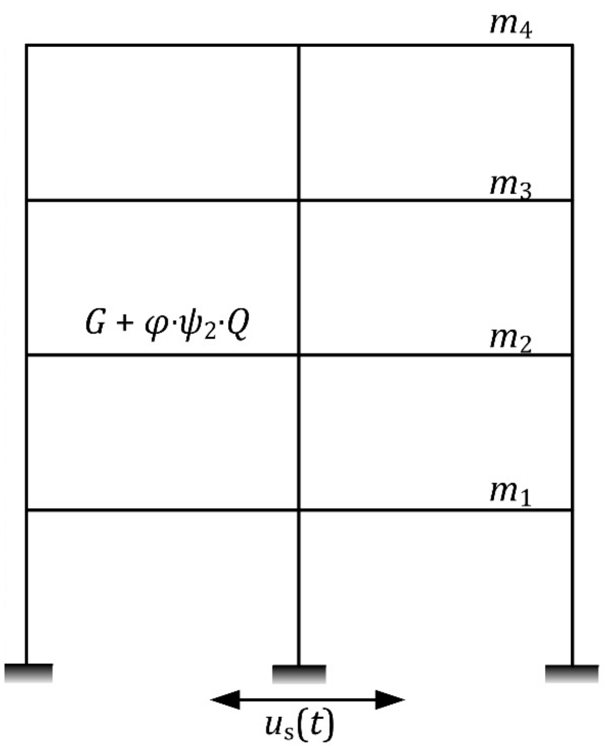
Table 4.1: Consequences of structural regularity on seismic analysis and design

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

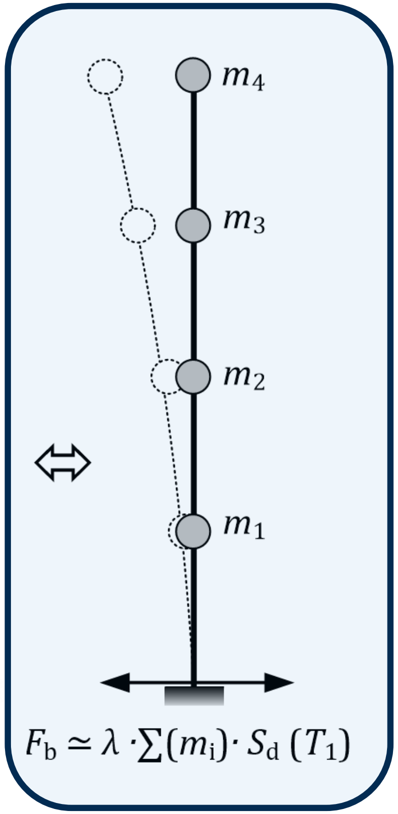
a) they have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2{,}0\,\mathrm{s} \end{cases}$$

Breakdown of masses for multi-storey buildings



Analysis model > $S_d(T_1)$



Masses of the storeys => Quasi permanent combination of actions:

$$\boxed{m_i = \{\Sigma G_{k,i} + \Sigma(\varphi \cdot \psi_{2,j} \cdot Q_{k,j})\}/g}$$

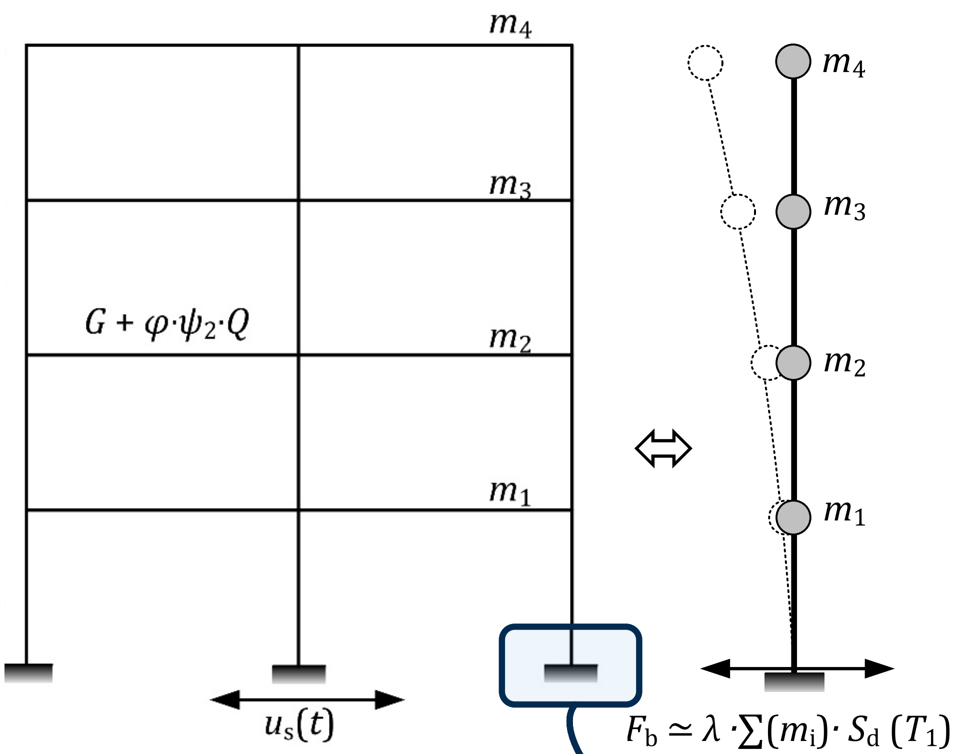
Minimum values of φ

Type of variable action	ψ_2	φ
Category A (domestic and residential activities)	0,3	0,5
Category C (public areas)	0,3	0,5
Other categories	Var.	1,0

NOTE: Several EC 8 — NA state φ = 1 always (on the “safe side”)

Simplified model for predesign the structure

Stiffness for multi-storey buildings



Analysis model > $S_d(T_1)$

- ✓ The stiffness of secondary seismic members against seismic actions should be neglected.
- ✓ In concrete, composite steel-concrete, and masonry buildings, the stiffness of the primary seismic members should be evaluated considering the effect of cracking.
- ✓ Unless a more accurate analysis of the cracked members is performed, the elastic flexural and shear stiffness properties of concrete may be taken to be equal to 50% of the corresponding stiffness of the uncracked concrete members.

Foundation usually considered fully retrained in the global model
(although flexible foundations may be used at later design stages)

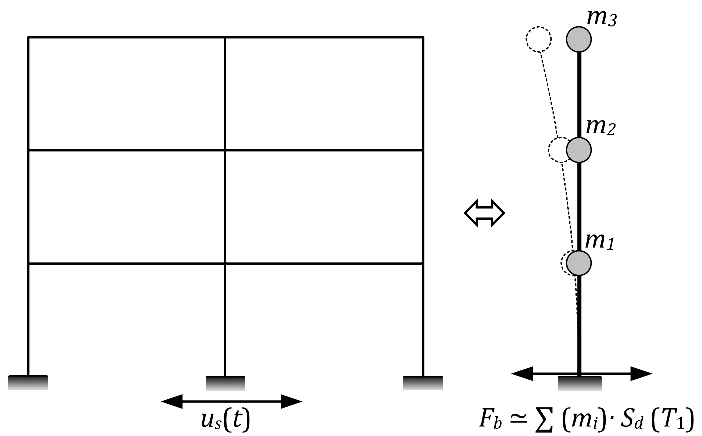
Lateral forces analysis method

Consider a building with n storeys and the following structural assumptions:

- ✓ To be used for buildings with heights up to 30 m and with $T_1 \leq \min(4T_C; 2s)$ [1st natural frequences ≥ 0,5Hz].
- ✓ Lateral rigidity and floor mass exhibit no abrupt variations between consecutive storeys, ensuring a *regular structure in elevation*.
- ✓ Floors act as rigid diaphragms, with mass and lateral rigidity distributions that are approximately symmetrical about two orthogonal axes (the principal directions), resulting in an *in-plan regular structure*.
- ✓ The seismic structural response can be assessed using two in-plan models, one for each principal direction.
- ✓ The response to seismic action is primarily governed by the fundamental vibration mode in each principal direction.

Oscillator with n DOF associated with building structure model

Factors to define the seismic action S_d



- Geographic location
- Seismic action type
- Type of foundation soil (A to E)

Adjacent area

- Importance factor (γ_I)
- Structure 1st period (T)
- Damping coef. ξ (generally 5% for concrete structures)
- Behaviour coefficient (q)

Structure charact.

$\boldsymbol{S_d}$

So, it is necessary to estimate the value of the period T

a) Based on the lateral deformability of the structure

δ_i – lateral displacements of the storeys due to the gravity loads G_i of each floor applied in the horizontal direction.

a1) Based on d – lateral displacement at the top of the building

$$\boxed{T_1 = \frac{2\pi}{\omega_1} \approx \frac{2\pi}{\sqrt{g}}\sqrt{d} = 2{,}0\,\sqrt{d} \quad (d \text{ in [m] and } T_1 \text{ in [s]})}$$

a2) Based on δ_i – lateral displacement of each storey due to G_i applied in the horizontal direction

$$\boxed{T_1 \approx 2\pi\sqrt{\frac{\sum(G_i\cdot\delta_i^2)}{g\cdot\sum(G_i\cdot\delta_i)}}}$$

b) Empirical formula (for buildings up to 40m tall)

$$\boxed{T_1 \approx C_t \cdot H^{3/4}}$$

being

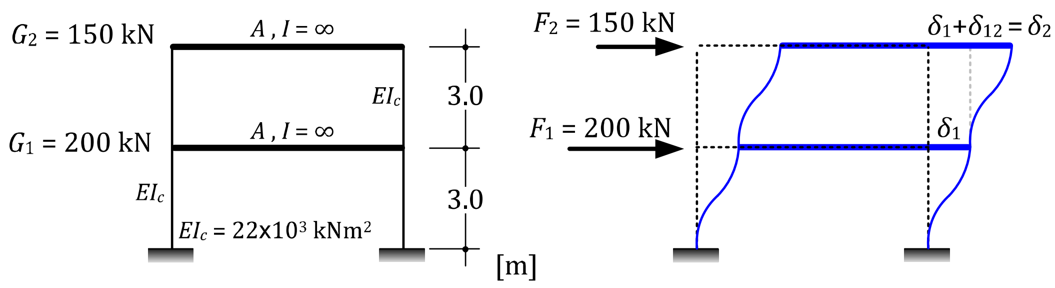
H – height of the building (relative to the ground level)

C_t = 0,085 for moment resistant space steel frames

0,075 for moment resistant space concrete frames

0,050 for all other structures of building

Example of estimating the period of the fundamental mode

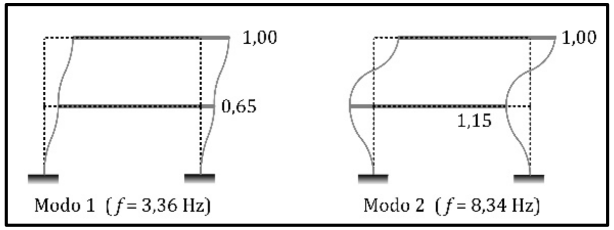


- Lateral rigidity of one storey $\quad k_{tot} = 2\times\frac{12(EI_c)}{h^3} = 19{,}56\times10^3\ \text{kN/m}$
- Displacement of storey 1 $\quad \delta_1 = \frac{F_1+F_2}{k_{tot}} = \frac{200+150}{19{,}56\times10^3} = 17{,}9\times10^{-3}\ \text{m}$
- Displacement of storey 2 $\quad d = \delta_2 = \delta_1 + \delta_{12} = \delta_1 + \frac{150}{19{,}56\times10^3} = 25{,}6\times10^{-3}\ \text{m}$
- Fundamental period of the struct. $\quad T_1 \approx 2{,}0\sqrt{d} = 2{,}0\sqrt{25{,}6\times10^{-3}} = 0{,}32\ \text{s}$

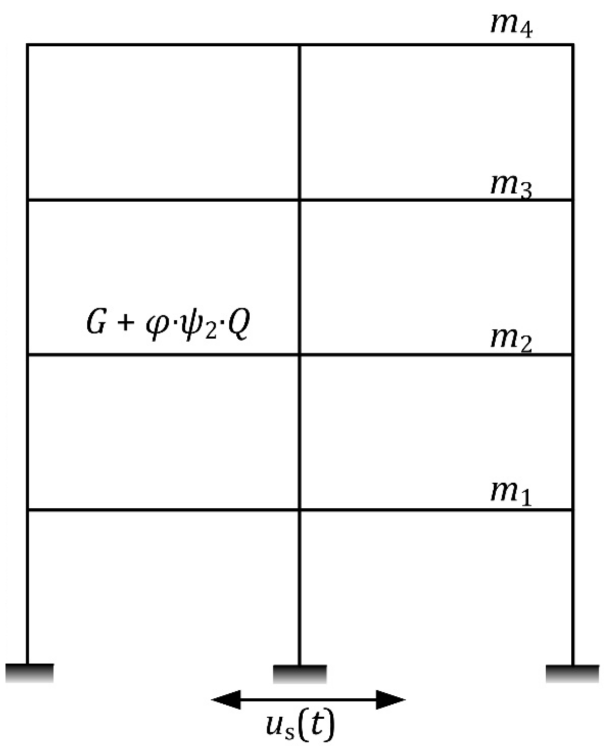
$$T_1 = 2\pi\sqrt{\frac{(200\times17{,}9^2 + 150\times25{,}6^2)\times10^{-6}}{9{,}8\times(200\times17{,}9+150\times25{,}6)\times10^{-3}}} = 0{,}30\ \text{s}$$

$$T_1 = 0{,}075\times6{,}0^{3/4} = 0{,}29\ \text{s}$$

More "rigorous" vibration modes
T_1 = 1/3,36=0,295 s; T_2 = 1/8,34=0,12 s



Base shear force for multi-storey buildings



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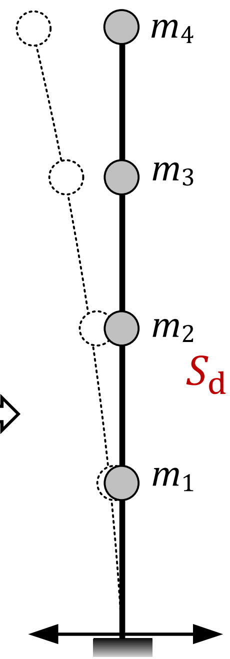
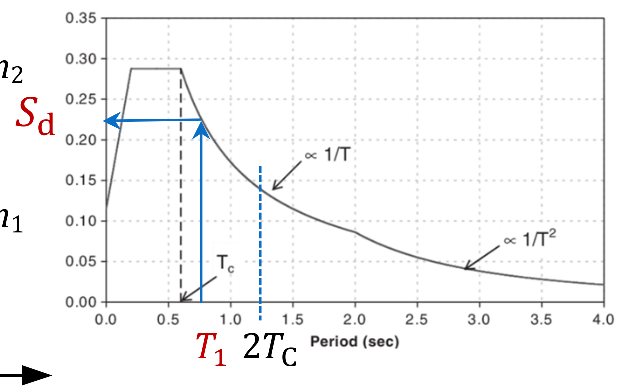


$F_b \simeq \lambda \cdot \sum(m_i) \cdot S_d\,(T_1)$



Value λ

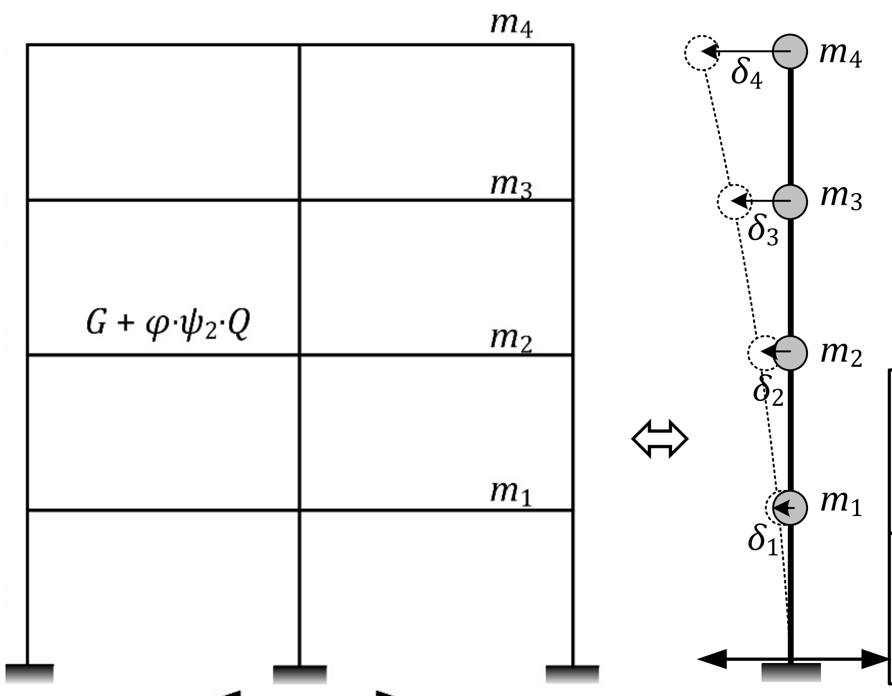
- $\lambda = 0{,}85$ if $T_1 <$ min $(2T_C\,;\,1{,}2\text{s})$ and the building has more than two storeys.
- $\lambda = 1{,}0$ otherwise.

NOTE: The correction factor λ accounts for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller, on average by 15% than the total building mass.

Analysis model > $S_d(T_1)$

$$\boxed{\text{Base shear force} \quad F_b = \lambda \cdot \Sigma\, m_i \cdot S_d(T_1)}$$

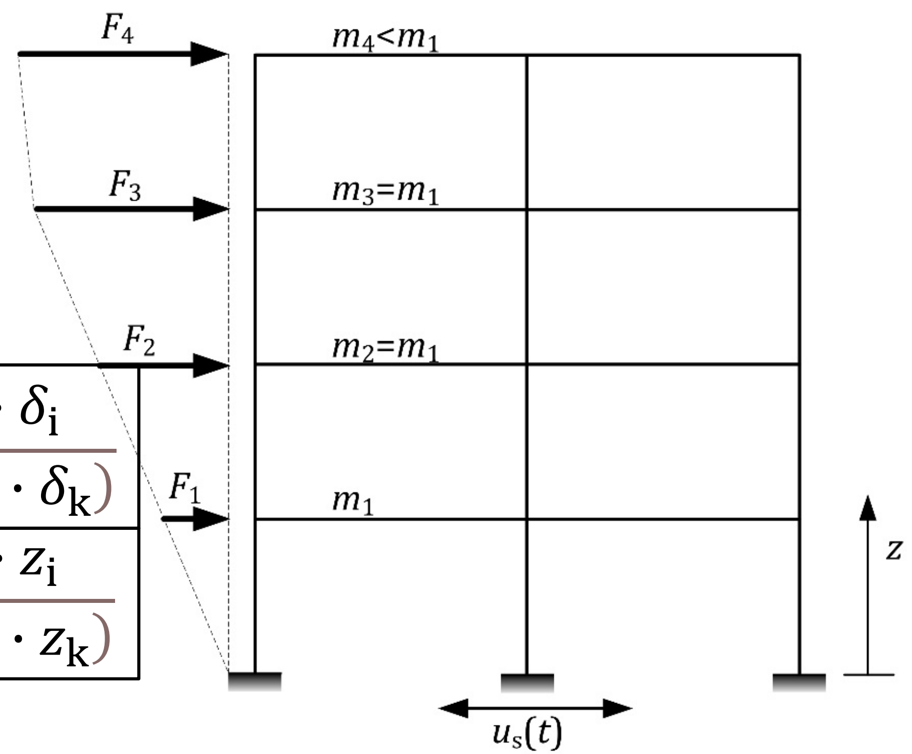
Breakdown of seismic force for multi-storey buildings



$F_b \simeq \lambda \cdot \sum(m_i) \cdot S_d\,(T_1)$

$$F_i = F_b \cdot \frac{m_i \cdot \delta_i}{\sum(m_k \cdot \delta_k)}$$

$$F_i \approx F_b \cdot \frac{m_i \cdot z_i}{\sum(m_k \cdot z_k)}$$

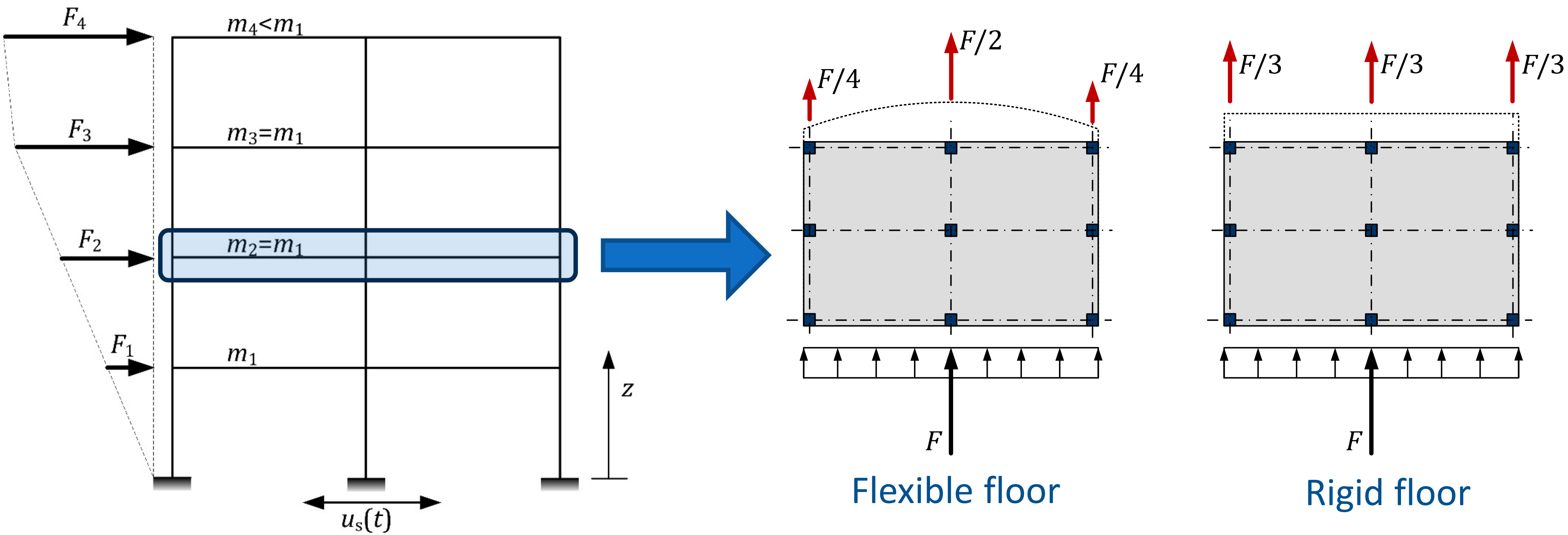


Analysis model > $S_d(T_1)$

Base shear force $F_b = \lambda \cdot \Sigma\, m_i \cdot S_d(T_1)$

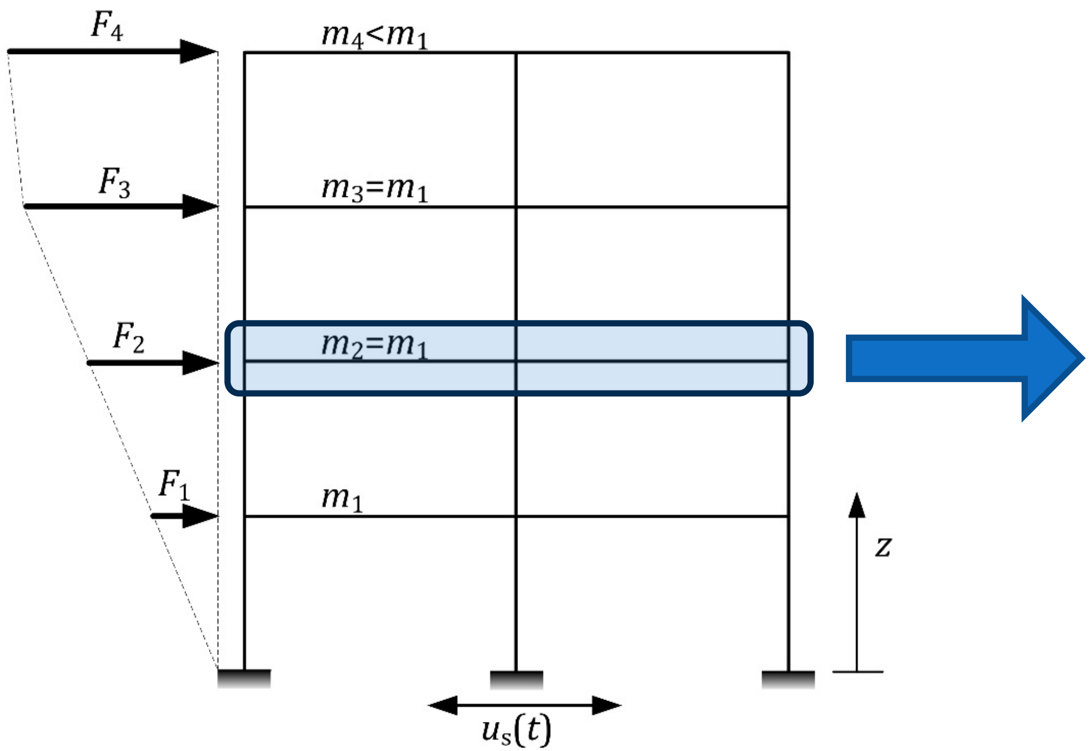
NOTE: The triangular distribution is a good approximation if the masses and stiffness of the storeys are constant in height

Breakdown of seismic force by each portal frame



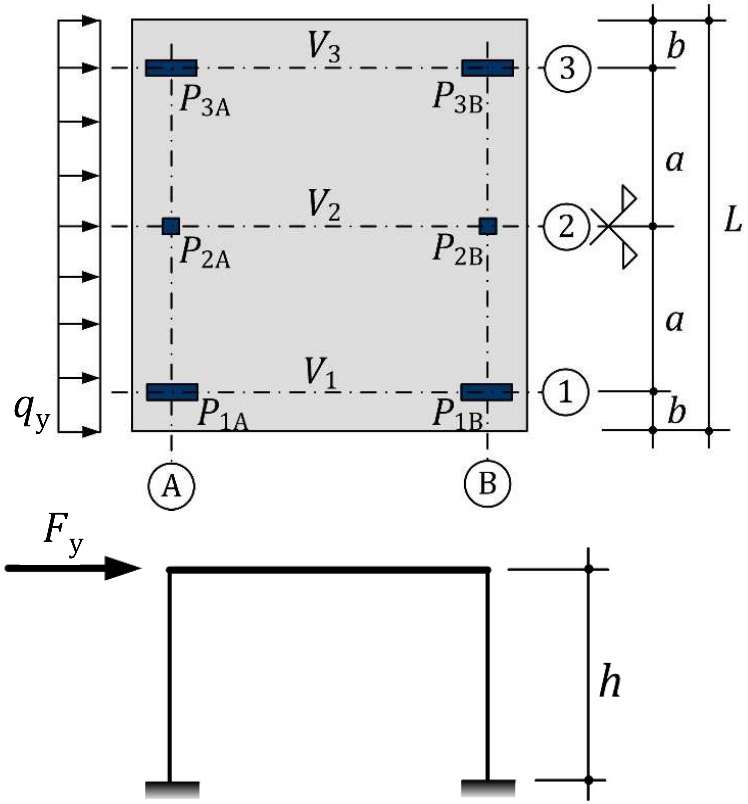
NOTE - Diaphragmatic behaviour at storey level ⇔ Floor and roof systems must have in-plane rigidity, resistance, and effective connections to vertical structural elements. Special attention is required for irregular layouts and large openings near primary vertical members, which may hamper these connections. **It is usually found that in-situ reinforced concrete solid slabs form a rigid diaphragm**.

Breakdown of seismic force by each portal frame



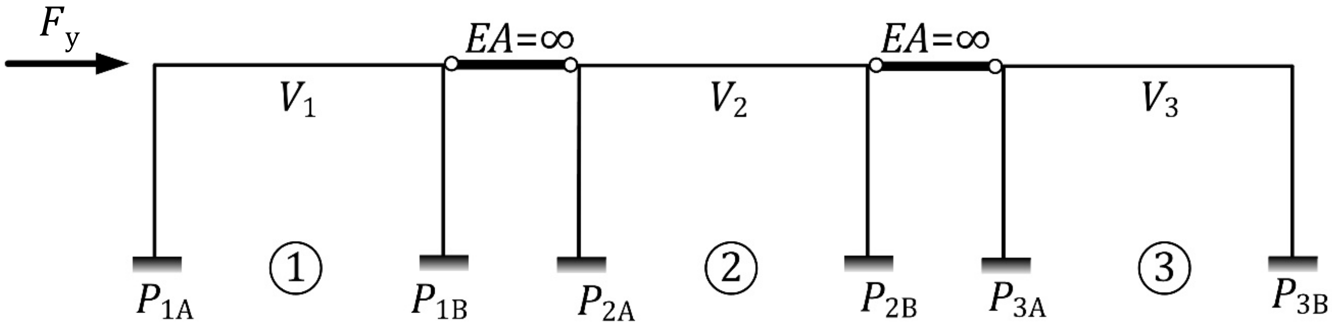
NOTE - Precast floor elements can be used on slabs (HCS - hollow-core slabs) although they are not the preferred solution for medium and high seismic regions. FprEN 1998-1-2:2025 requires to **cast a topping layer of 40 to 50 mm with a mesh reinforcement linked to beams and walls** (over the precast elements with a rough surface or connected to it through shear connectors), ensures a rigid diaphragm.

Breakdown of seismic force by the portal frames proportional to the relative stiffness of each frame in that direction:

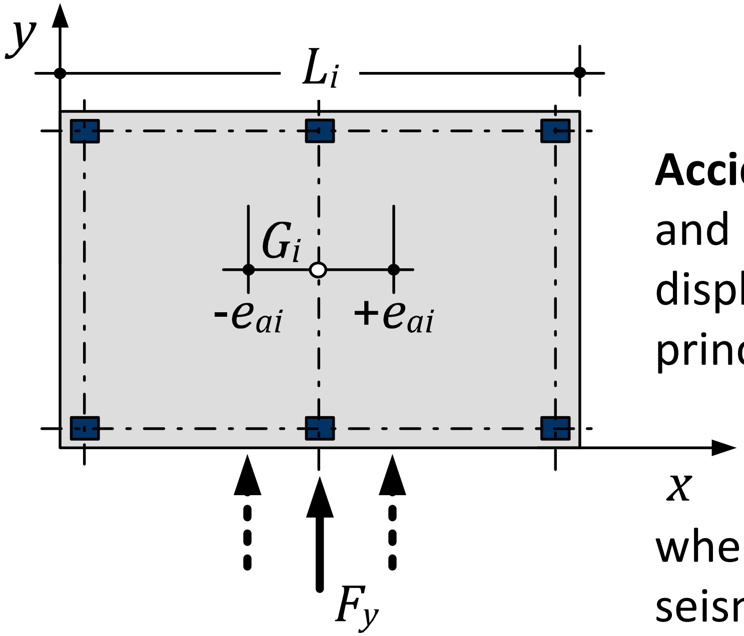


$$u_y^{(1)} = u_y^{(2)} = u_y^{(3)} = u_y = \frac{F_y}{\sum\left(k_y^{(i)}\right)}$$

$$\boxed{F_y^{(j)} = k_y^{(j)} \cdot u_y^{(j)} = \frac{k_y^{(j)}}{\sum\left(k_y^{(i)}\right)} \cdot F_y}$$



In symmetrical floors seismic design effects should consider a minimal torsion effect about the vertical axis to account for the eccentricity of application of the seismic force:



Accidental torsional effects – To account for mass location uncertainties and seismic motion variations, the centre of mass at each floor should be displaced from its nominal position by an accidental eccentricity in each principal direction,

$$e_{ai} = \pm 0{,}05 \cdot L_i$$

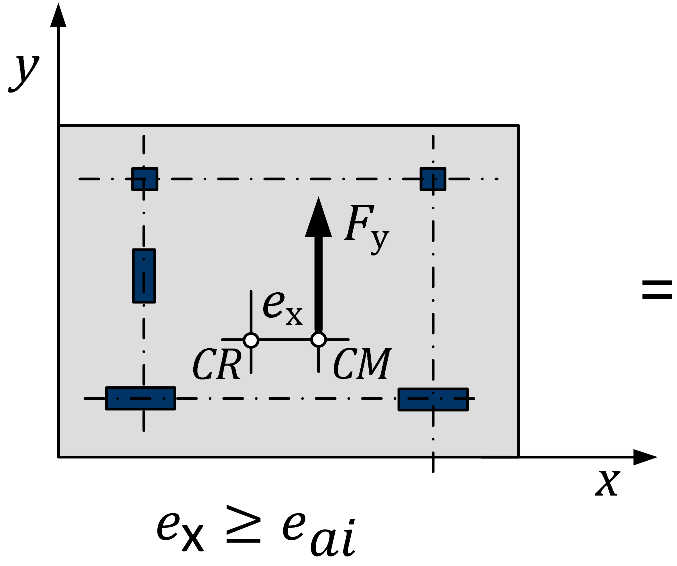
where L_i is the floor dimension perpendicular to the direction of the seismic action

For a symmetrical floor in terms of mass and stiffness => use an amplification coefficient of the internal forces due to translation

$$\xi_k = 1 + 0{,}6 \frac{x_k}{L_i}$$

ξ_k varies between 1,0 and 1,3

Effect of the floor's translation + rotation due to the eccentricity of application of the seismic force:



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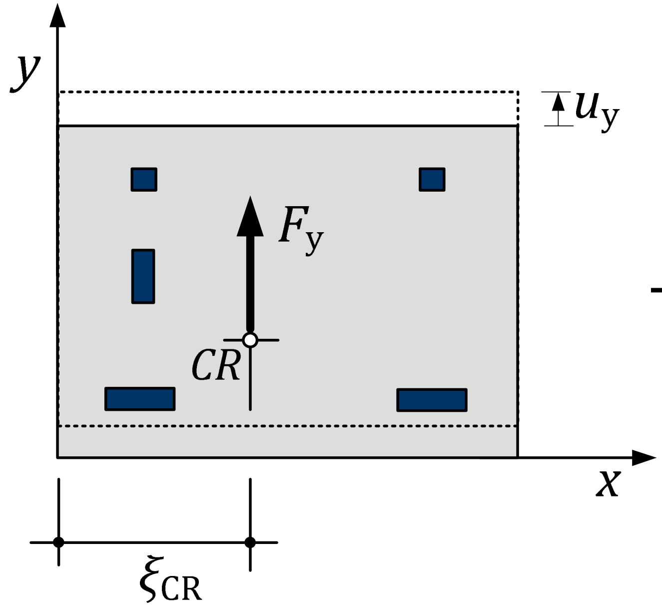

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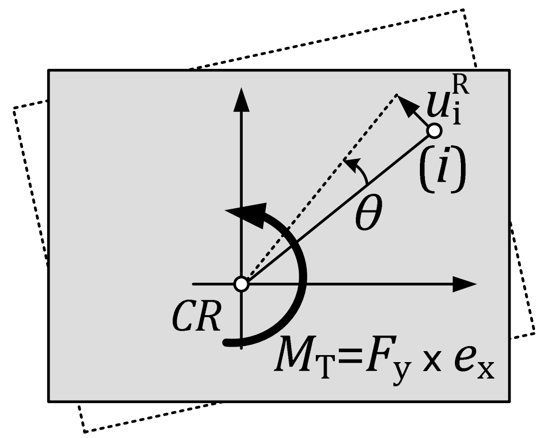
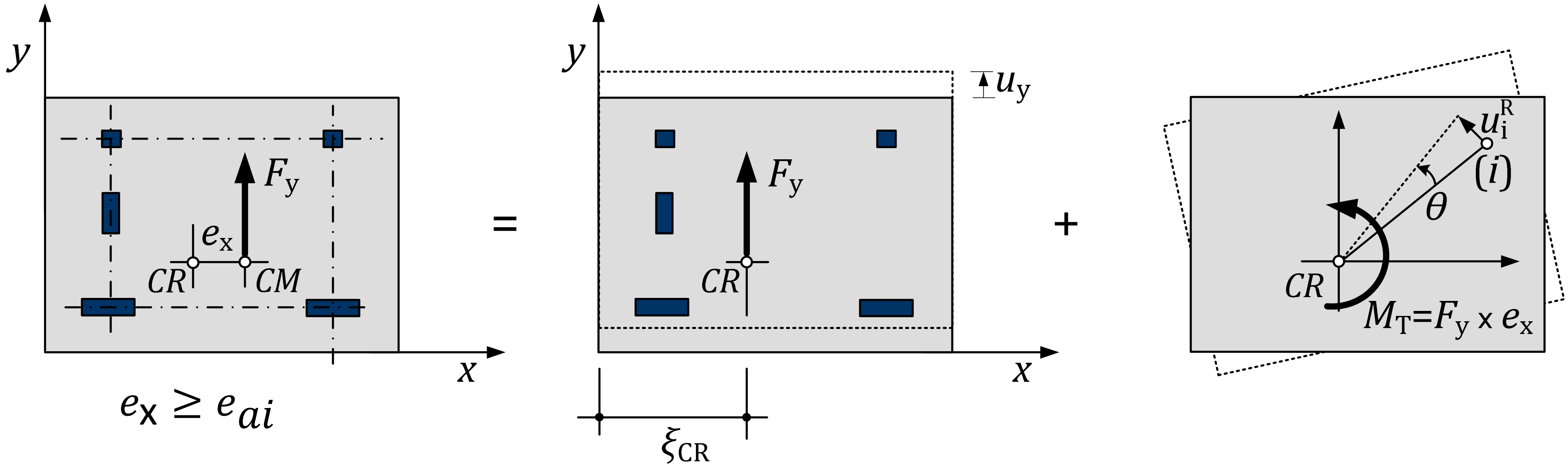


Floor with eccentric CR in relation to the CM

Floor translational effect

Floor rotational effect

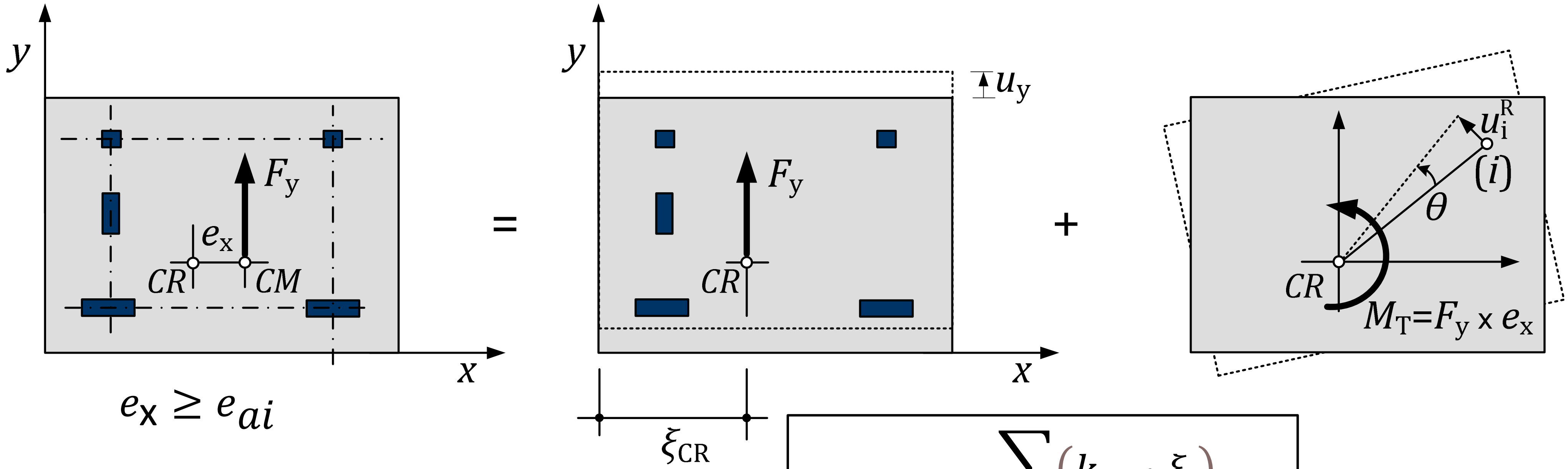
Effect of the floor's translation due to the eccentricity of application of the seismic force:



Redistribution of seismic forces by the frames due to the floor's translation

$$F_{y,i}^{T} = \frac{k_{y,i}}{\sum(k_{y,j})} \cdot F_y$$

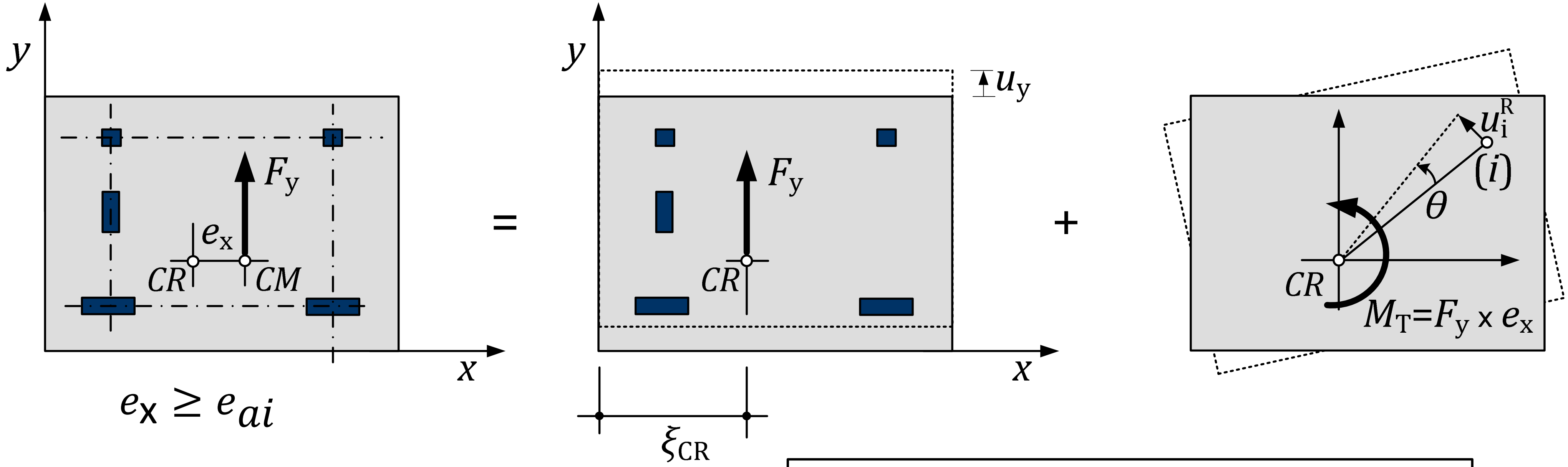
Effect of the floor's rotation due to the eccentricity of application of the seismic force:



Position of the centre of rigidity (CR) of the floor

$$\xi_{CR} = \frac{\sum (k_{y,i} \cdot \xi_i)}{\sum (k_{y,j})}$$

Effect of the floor's rotation due to the eccentricity of application of the seismic force:

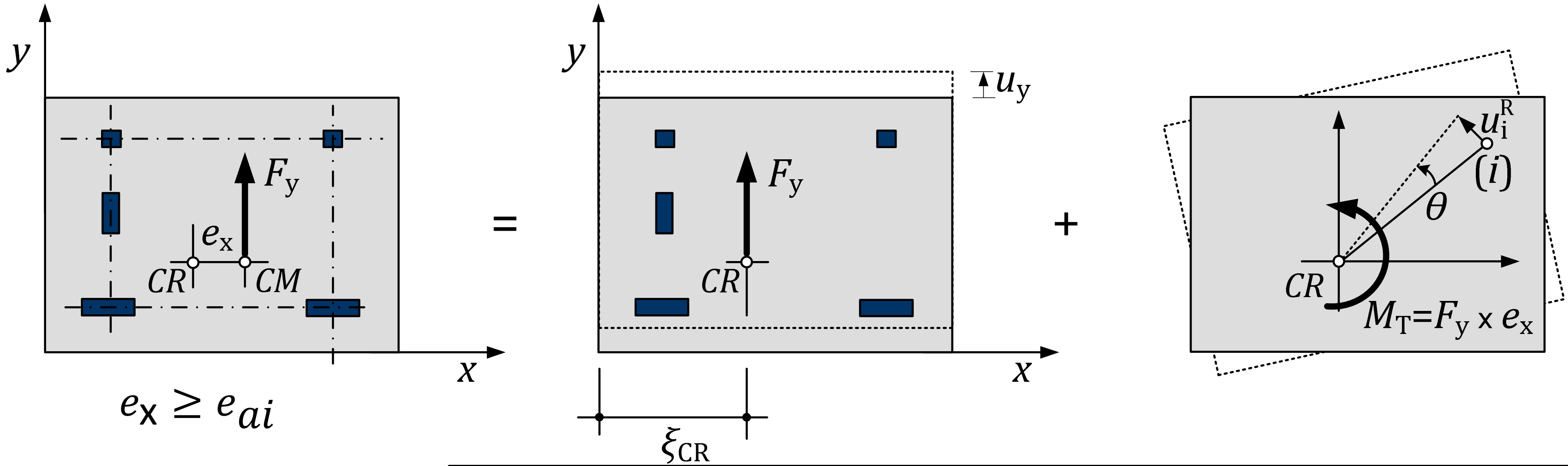


$e_x \geq e_{ai}$

Redistribution of seismic forces by the frames due to the floor's rotation

$$F_{y,i}^{R} = \frac{k_{y,i} \cdot x_i}{\sum\left(k_{y,j} \cdot x_j^2 + k_{x,j} \cdot y_j^2\right)} \cdot M_T$$

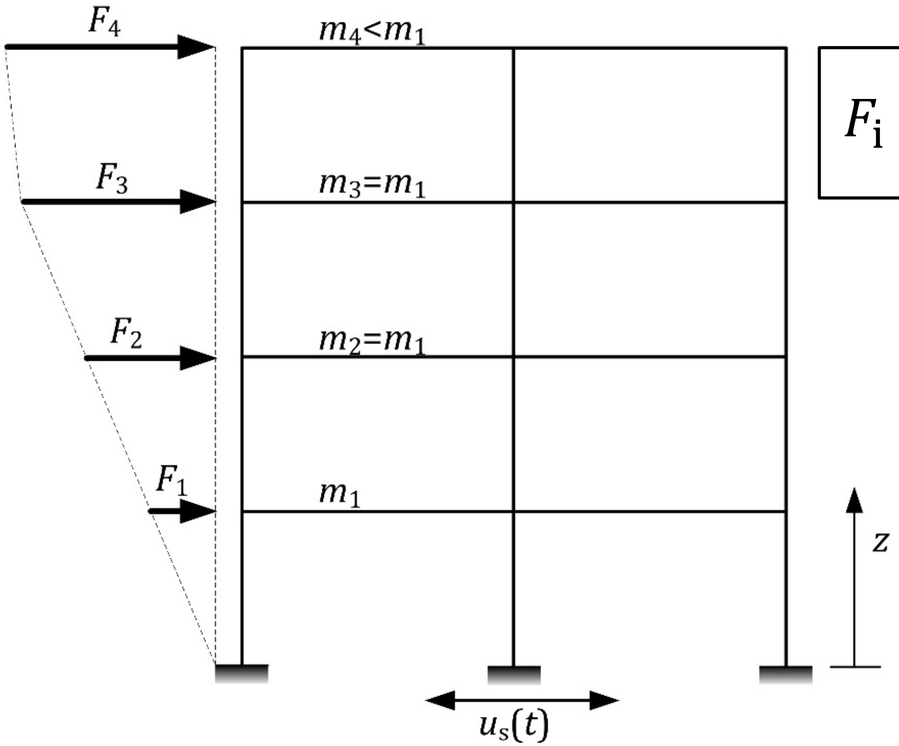
Effect of the floor's translation + rotation due to the eccentricity of application of the seismic force:



Forces on each frame due to floor translation + rotation

$$F_{y,i} = F_{y,i}^{T} + F_{y,i}^{R} = \frac{k_{y,i}}{\sum(k_{y,j})} \cdot F_y + \frac{k_{y,i} \cdot x_i}{\sum\left(k_{y,j} \cdot x_j^2 + k_{x,j} \cdot y_j^2\right)} \cdot M_T$$

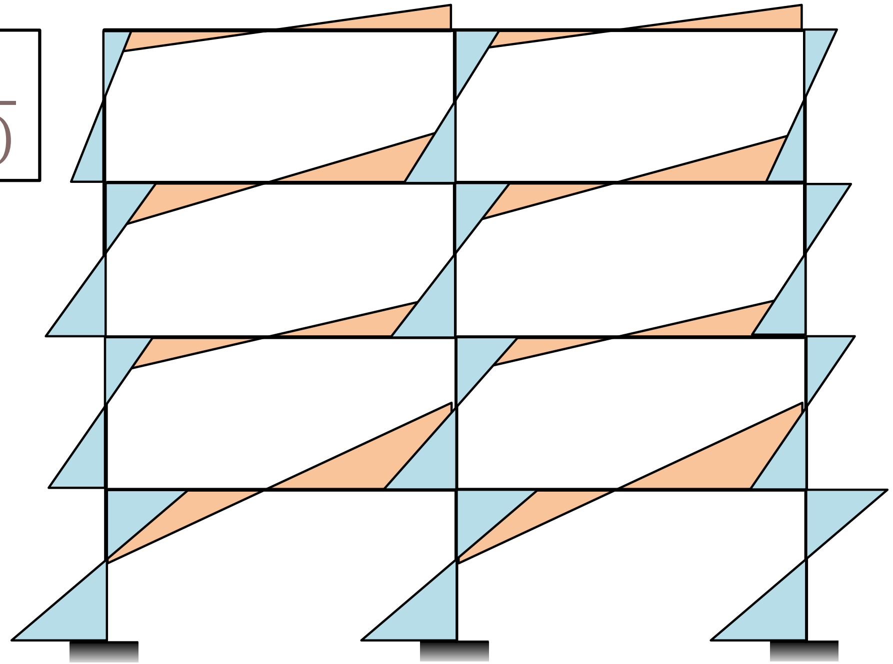
Internal forces resulting from seismic action



Analysis model

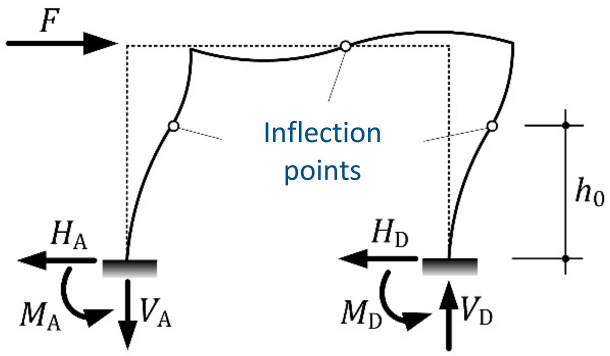
$$F_i = F_b \cdot \frac{m_i \cdot z_i}{\sum (m_k \cdot z_k)}$$

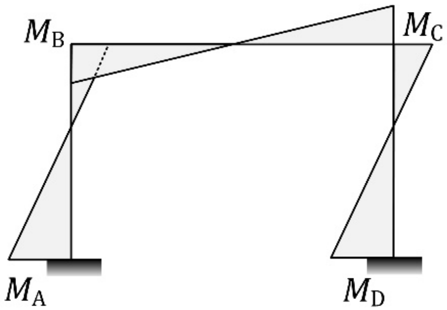
Equivalent static forces

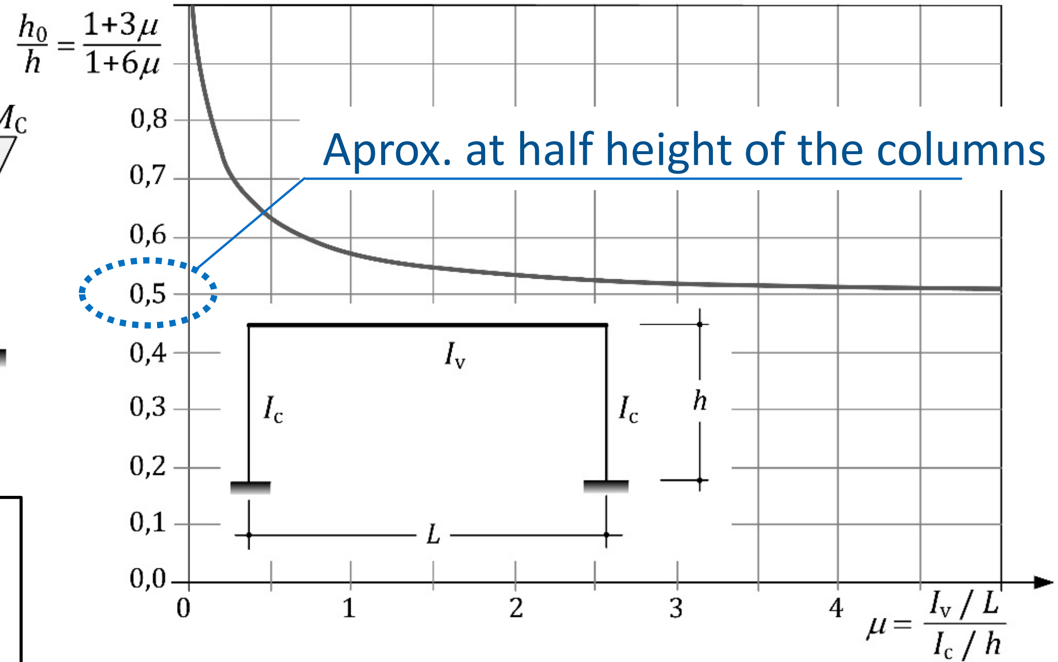


Bending moments

Internal forces resulting from seismic action – Position of null bending moment sections



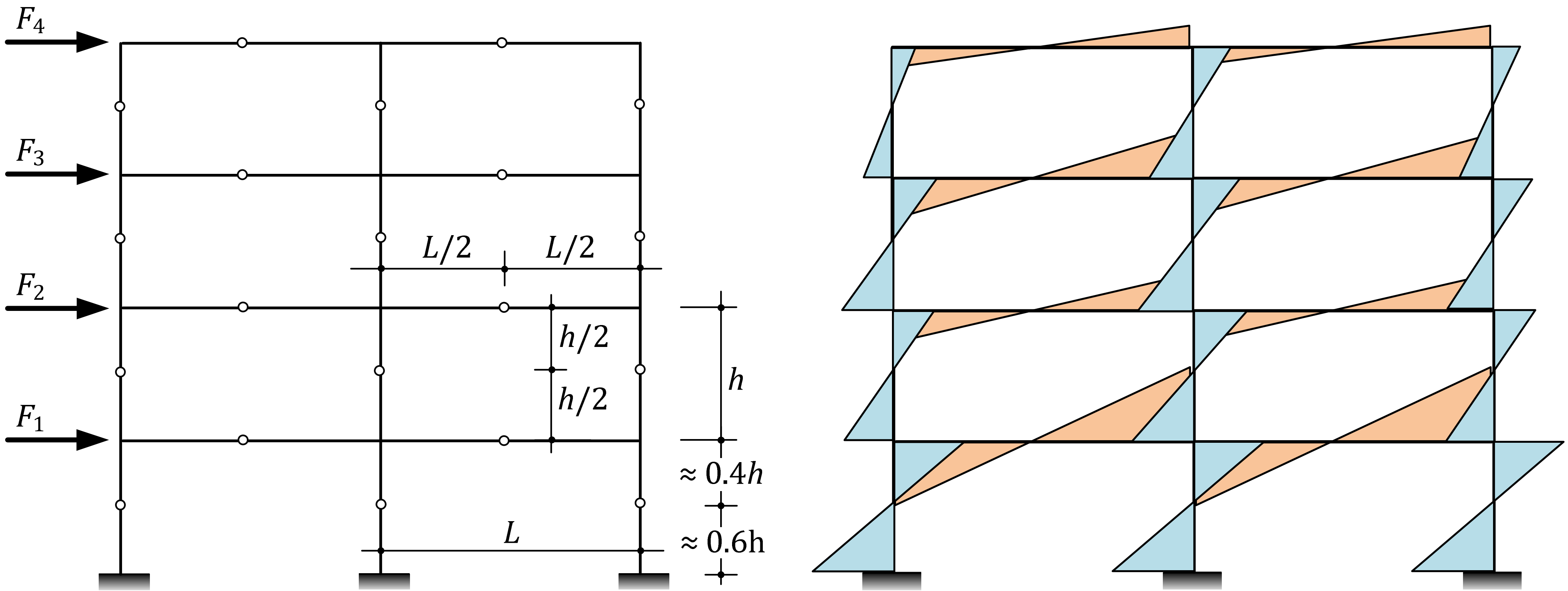




Ratio of rigidity between beams and columns

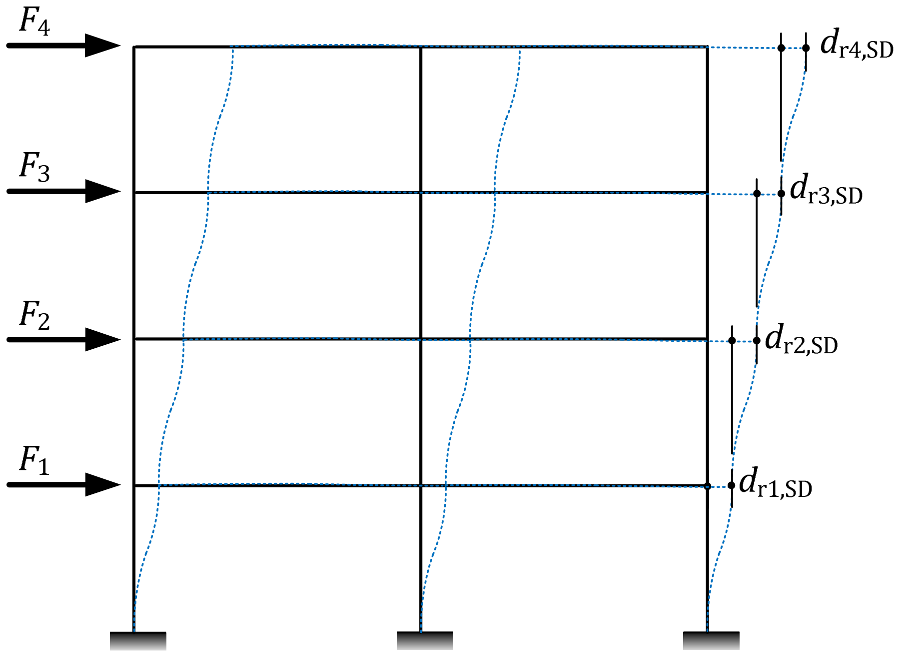
$$\mu = \frac{I_v/L}{I_c/h}$$

Internal forces resulting from seismic action – Position of null bending moment sections



Limitation of interstorey drift – maximum interstorey drift for the SD (significant damage) design situation should be limited to:

$$d_{r,SD} \leq \theta_s \, h_s$$



where:

$d_{r,SD}$ is the design interstorey drift, defined as the difference of lateral displacements at the top and bottom of the storey CM under consideration

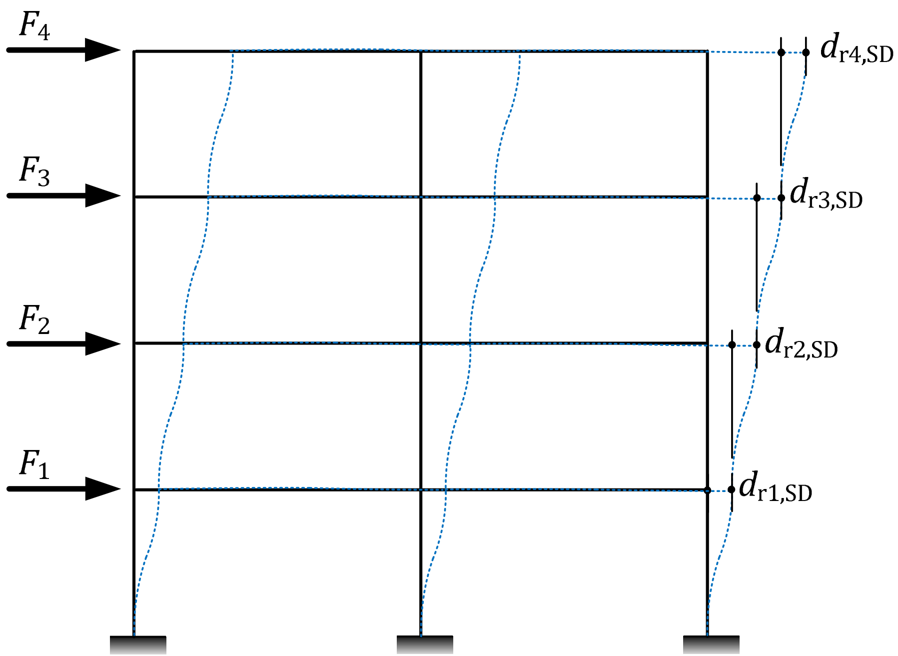
h_s is the interstorey height

θ_s is 0,02 for concrete / steel buildings designed to be dissipative, and without interacting masonry infills (between 0,006-0,011 with masonry infills)

NOTE: $d_{r,SD}$ should be obtained by the product of the lateral displacements obtained for each storey using the design seismic force by the behaviour factor

Second-order effects (P-Δ effects) may be neglected if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{\text{tot}}\, d_{\text{r,SD}}}{q_{\text{S}}\, q_{\text{R}}\, V_{\text{tot}}\, h_{\text{s}}} \leq 0{,}10$$



with,

θ the interstorey drift sensitivity coefficient

P_{tot} is the total gravity load at and above the storey, due to the masses considered in the seismic analysis of the structure

$d_{\text{r,SD}}$ is the design interstorey drift

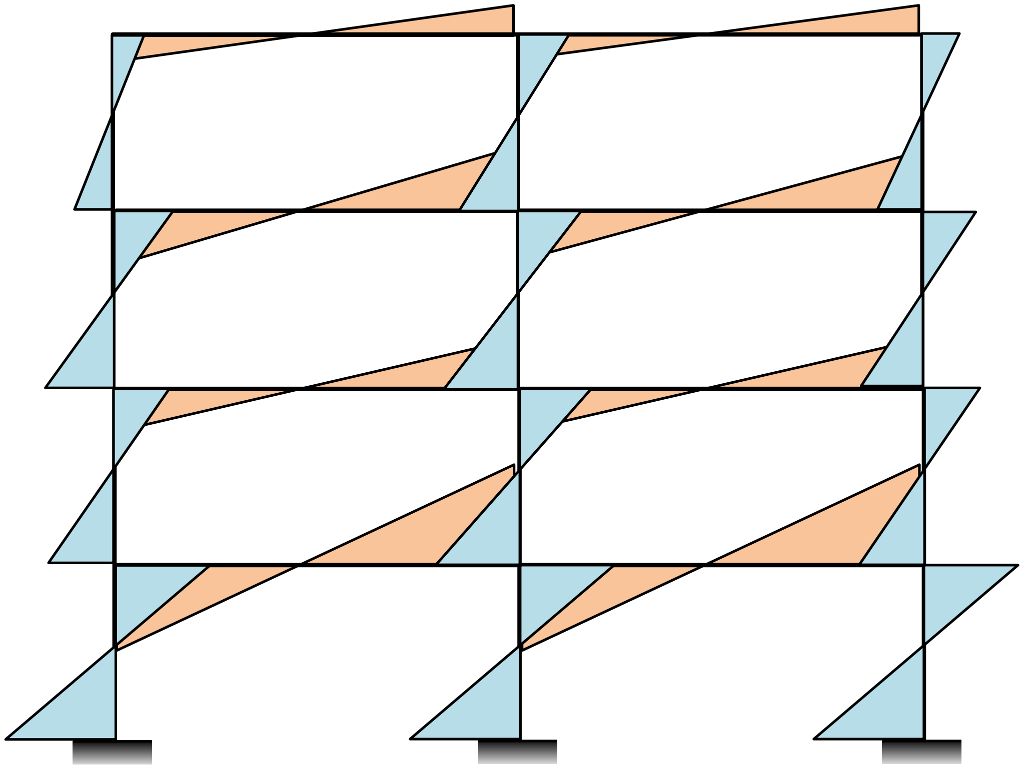
V_{tot} is the total storey shear in the seismic design situation

h_{s} is the interstorey height

q_{S} and q_{R} are the behaviour factor components

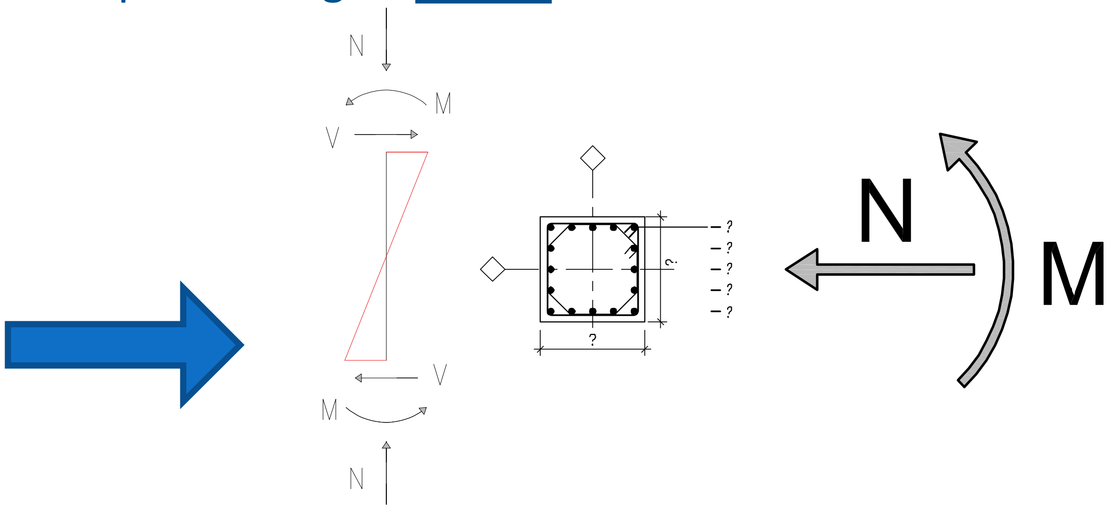
NOTE: If $0{,}10 < \theta \leq 0{,}20$ at any storey, the 2nd order effects may be considered by multiplying the seismic action effects by a factor $1 / (1 - \theta)$

Internal forces resulting from seismic action – Design of columns and concrete walls



Safety verifications are only possible if the cross-sections (geometry) and reinforcements of the columns are known

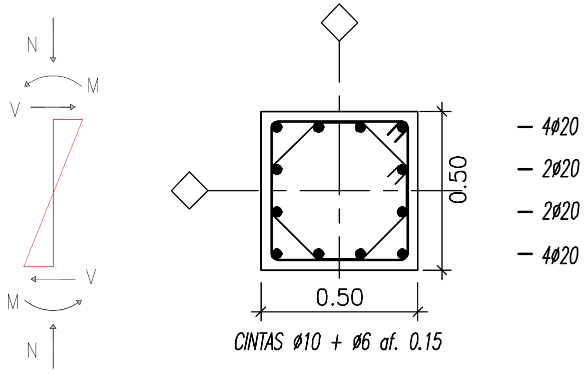
Analysis and Verification of the structure is only possible after performing its Design



Design of the column's cross-section and reinforcement – Example

1) Design internal forces

Columns with N_{Ed}, M_{Ed}, V_{Ed}



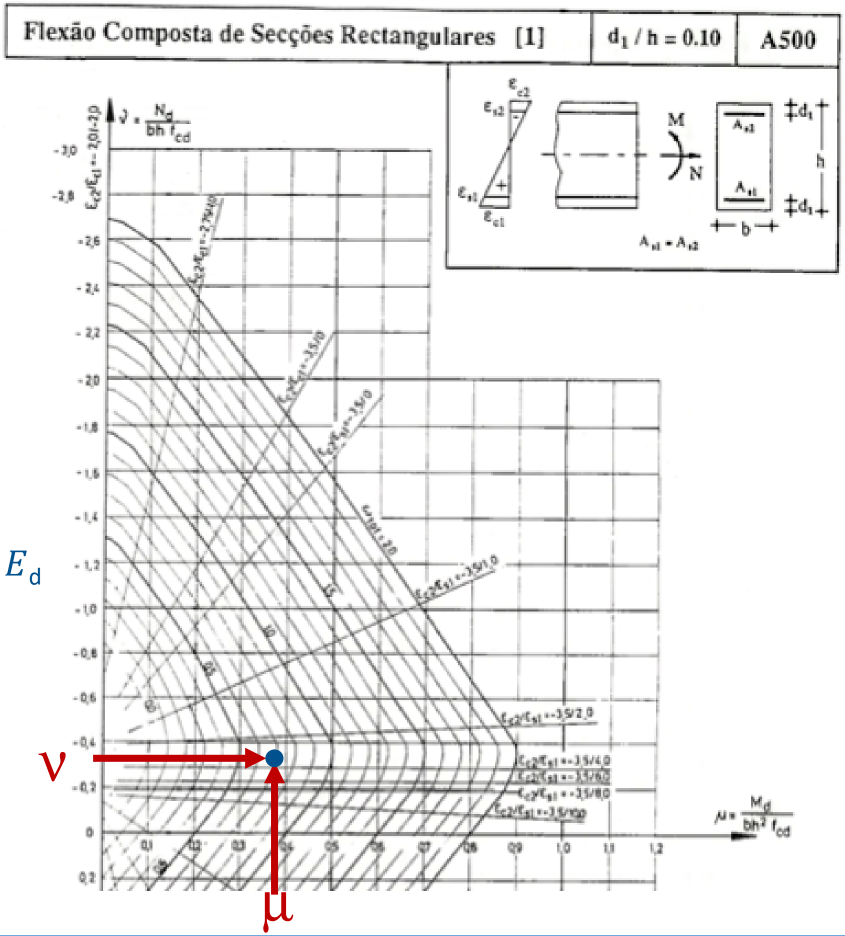
2) Design resistance

The column dimensions are assumed / changed

The design is an iterative process

Safety $\Rightarrow R_d \geq E_d$

Economy $\Rightarrow R_d$ not "much" sup. to E_d



Interactive process after materials are defined (C30; B500 C)

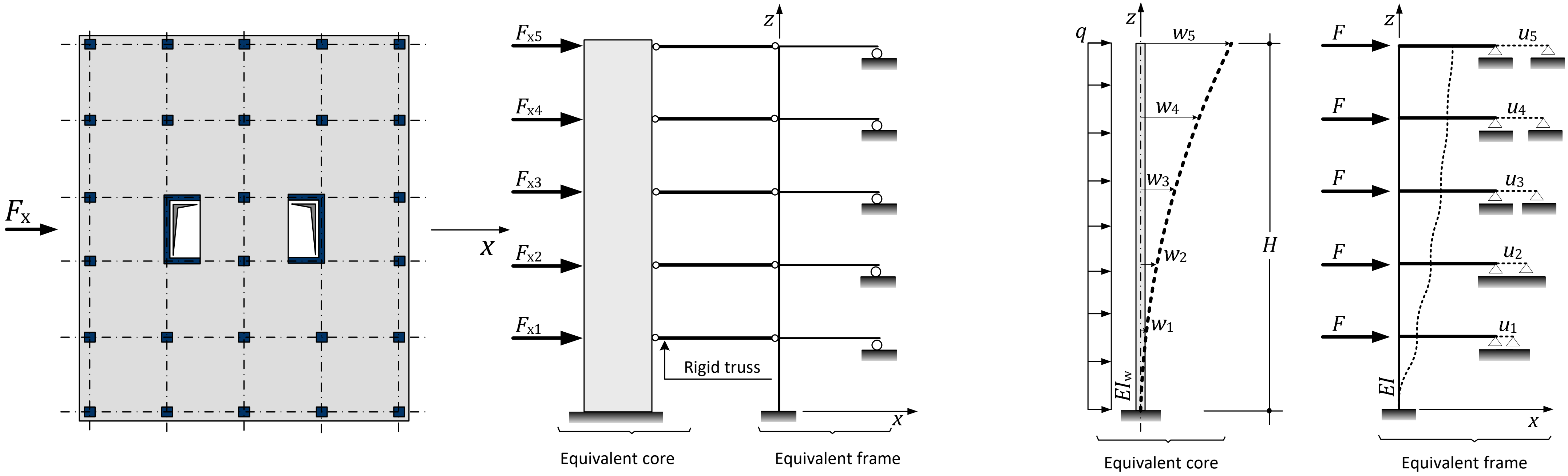
1 – Predesign cross-sections and obtain (N_{Ed}, M_{Ed}) for the seismic combination

2 – Evaluate (ν, μ) $\Rightarrow \omega \Rightarrow A_s$ (check safety) $R_d \geq E_d$

3 – Check the economy of the solution (R_d should not be "much" higher than E_d)

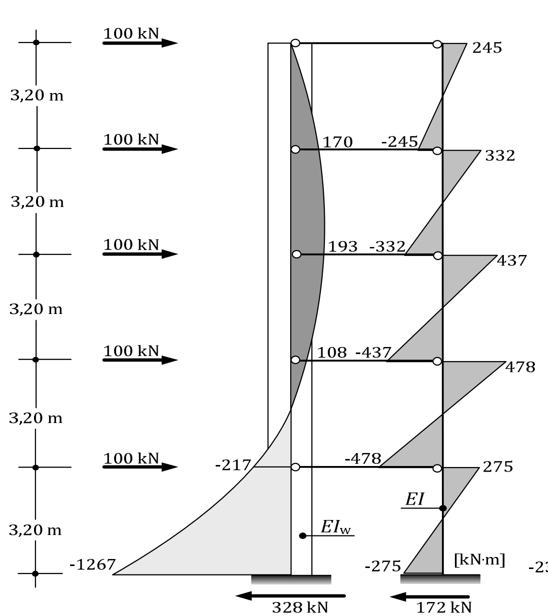
4 – If 2) or 3) are not satisfied, then resize cross-sections and new (N_{Ed}, M_{Ed}) in 1)

LATERAL FORCES METHOD OF ANALYSIS – Effects of the concrete cores

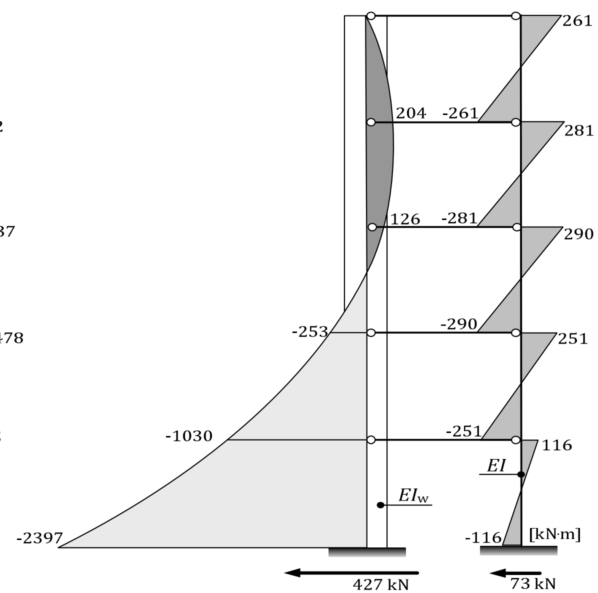


Effects of the concrete cores :

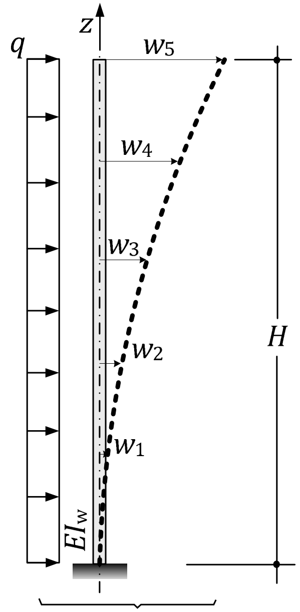
- ✓ Allow to reduce column cross-sections and reinforcement ratios
- ✓ Concentrates bending moments at the foundation of the walls
- ✓ Decreases and equalises inter-storey displacements.
- ✓ Significantly increases global resistance to horizontal forces



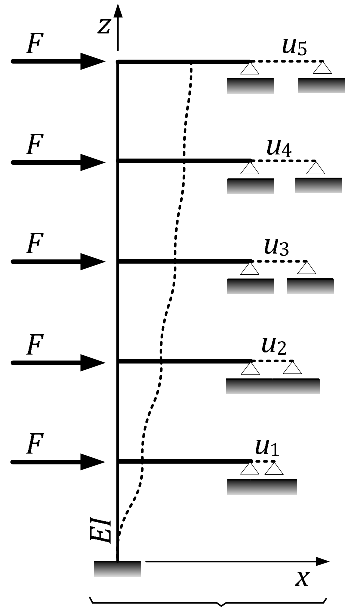
$EI_w = 10EI$



$EI_w = 50EI$



Equivalent core



Equivalent frame

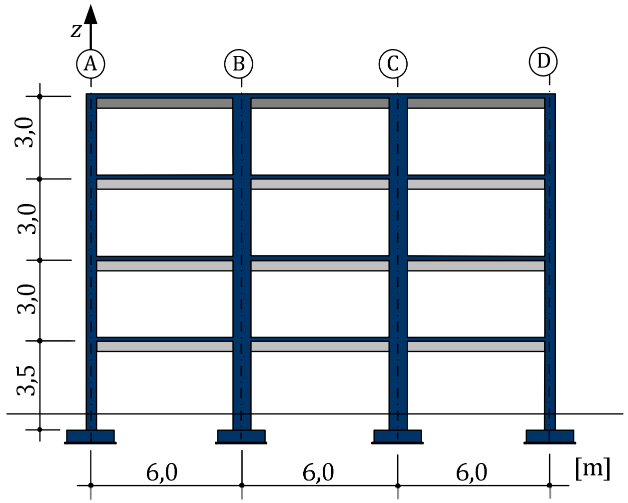
Effects of the concrete cores :

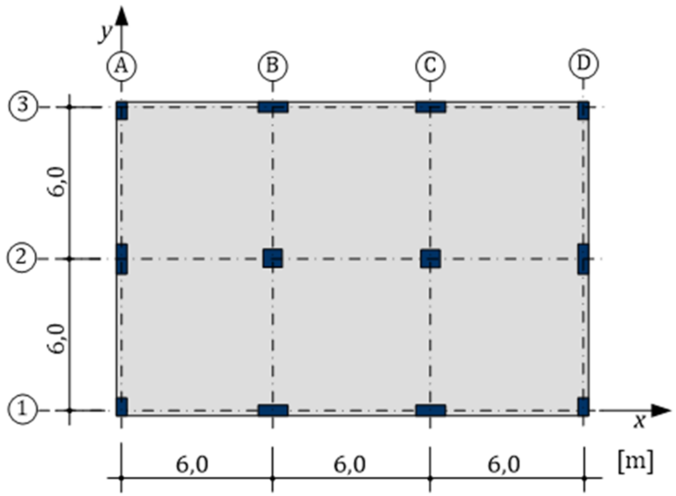
- ✓ Allow to reduce column cross-sections and reinforcement ratios
- ✓ Concentrates bending moments at the foundation of the walls
- ✓ Decreases and equalises inter-storey displacements.
- ✓ Significantly increases global resistance to horizontal forces

Objective – Preliminary design of beams and slabs and initial verification of the seismic design requirements.

Consider a moment-resisting frame residential building located in southern Portugal. The building consists of four upper levels (three standard floors and an accessible rooftop terrace) and a ground floor (Level 0).

The structure is composed of reinforced concrete (C30/37 concrete; B500B steel) in an orthogonal grid. All floors feature solid slab construction and share the same floor plan configuration.





Foundation Soil: Thick deposit of very dense sand – Type B ground according to EC8-1.

Shallow Foundations: Columns are fully embedded at their base.

Columns:

(i) Constant cross-section along the height.

(ii) Central columns – 0.50 × 0.50 m².

(iii) Corner columns – 0.35 × 0.50 m².

(iv) Intermediate side columns – 0.35 × 0.80 m².

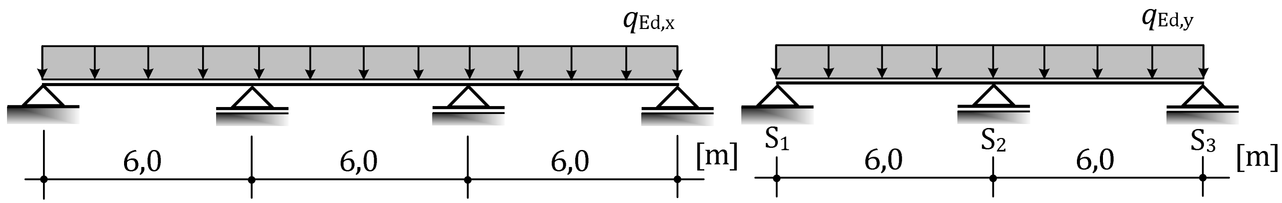
Residential Building ⇒ Importance Class II, hence performance factor is $\gamma_{\mathrm{I}} = 1{,}0$ ($\therefore a_{\mathrm{g}} = a_{\mathrm{gR}}$) for both types of seismic action.

Gravitational Actions:

- Self-weight of structural elements.
- "Remaining permanent load" = 4,0 kN/m².
- Weight of external walls = uniformly distributed linear load of 7,0 kN/m, applied to the edge beams of the typical floors.
- Live load: $q_{\mathrm{k}} = 2{,}0$ kN/m²; $\psi_2 = 0{,}3$.

Preliminary Slab Thickness Design:

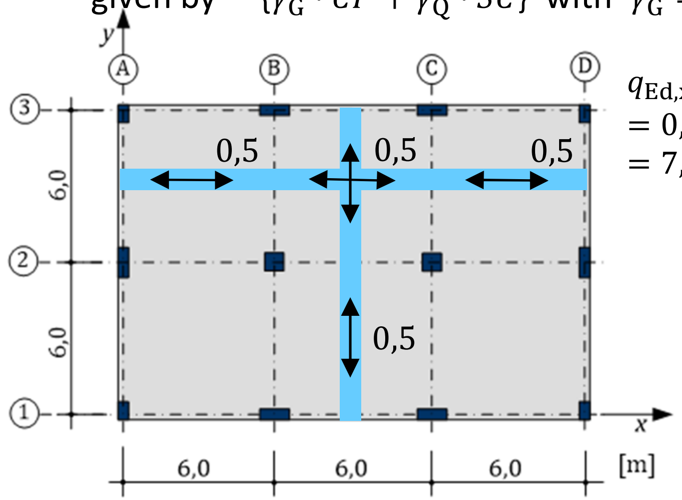
- Square slab panels, supported on all four edges, with a span of ℓ = 6,0 m.
- Slab thickness h = 0,17 m ≈ ℓ/35.



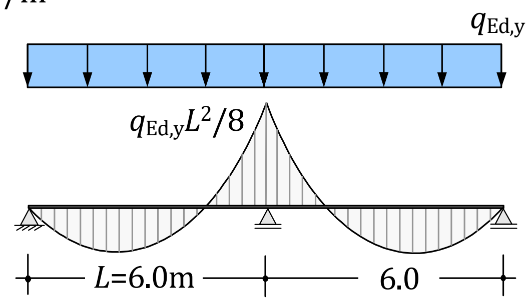
a) direction x b) direction y

Analysis Model for Estimating Slab Forces:

- The strip method is used, assuming that in all panels, the applied load is equally distributed in both principal directions (x and y).
- The design value of the uniformly distributed load for the ULS combination is given by $\{\gamma_{\mathrm{G}} \cdot CP + \gamma_{\mathrm{Q}} \cdot SC\}$ with $\gamma_{\mathrm{G}} = 1{,}35$ & $\gamma_{\mathrm{Q}} = 1{,}50$:



$$\begin{aligned} q_{\mathrm{Ed,x}} &= q_{\mathrm{Ed,y}} \\ &= 0{,}5 \times \{1{,}35 \times (25 \times 0{,}17 + 4{,}0) + 1{,}50 \times 2{,}0\} \\ &= 7{,}1\ \mathrm{kN/m^2} \end{aligned}$$



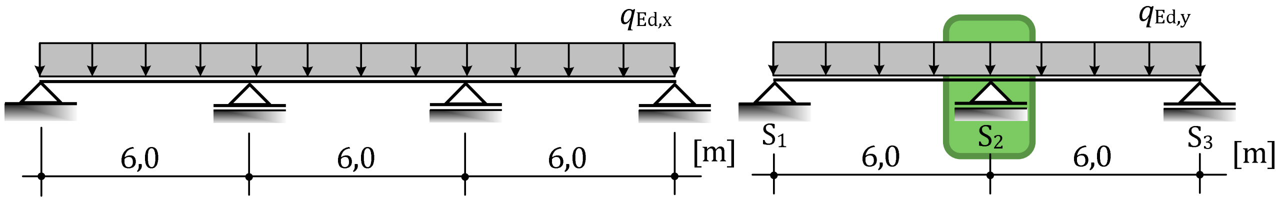
b) direction y

Gravitational Actions:

- Self-weight of structural elements.
- "Remaining permanent load" = 4,0 kN/m².
- Weight of external walls = uniformly distributed linear load of 7,0 kN/m, applied to the edge beams of the typical floors.
- Live load: $q_{\mathrm{k}} = 2{,}0$ kN/m²; $\psi_2 = 0{,}3$.

Preliminary Slab Thickness Design:

- Square slab panels, supported on all four edges, with a span of ℓ = 6,0 m.
- Slab thickness h = 0,17 m ≈ ℓ/35.



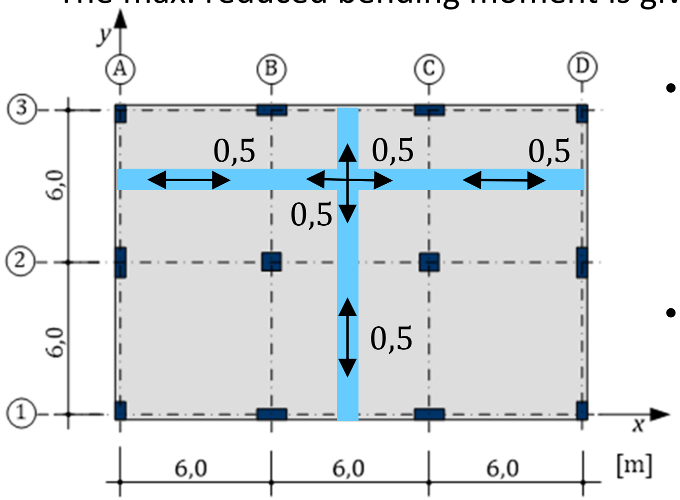
a) direction x b) direction y

Assessment of Maximum Flexural Reinforcement of the Slab:

- The maximum bending moment occurs at section S2 and is given by:

$$|M_{\mathrm{Ed}}|_{\max} = |M_{\mathrm{y,S2}}| = 7{,}1 \times \frac{6{,}0^2}{8} = 31{,}8\ \mathrm{kN} \cdot \frac{\mathrm{m}}{\mathrm{m}}$$

- The max. reduced bending moment is given by: $\mu_{\max} = \frac{31{,}8}{0{,}130^2 \times 20{,}0 \times 10^3} = 0{,}10$ ✓



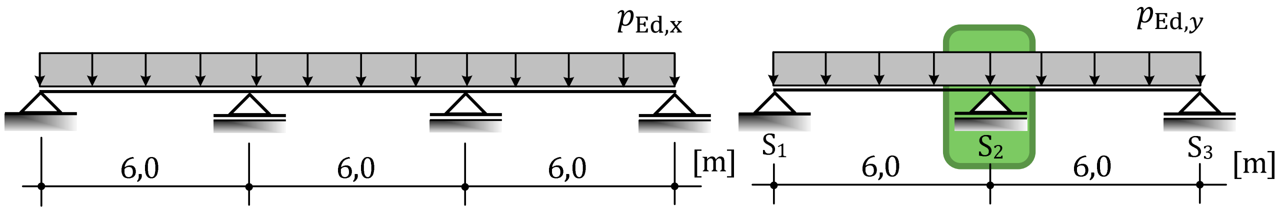
- The section with the highest flexural reinforcement in the slab must have: $A_{\mathrm{s}} \geq 6{,}16$ cm²/m (using B500 steel);
- This value can be achieved, for example, with reinforcement bars $\phi = 12$ mm spaced at 150 mm ($A_{\mathrm{s}} = 7{,}54$ cm²/m, $\rho_\ell = 0{,}58\%$).

Gravitational Actions:

- Self-weight of structural elements.
- "Remaining permanent load" = 4,0 kN/m².
- Weight of external walls = uniformly distributed linear load of 7,0 kN/m, applied to the edge beams of the typical floors.
- Live load: $q_{\mathrm{k}} = 2{,}0$ kN/m²; $\psi_2 = 0{,}3$.

Preliminary Beam Geometry Design:

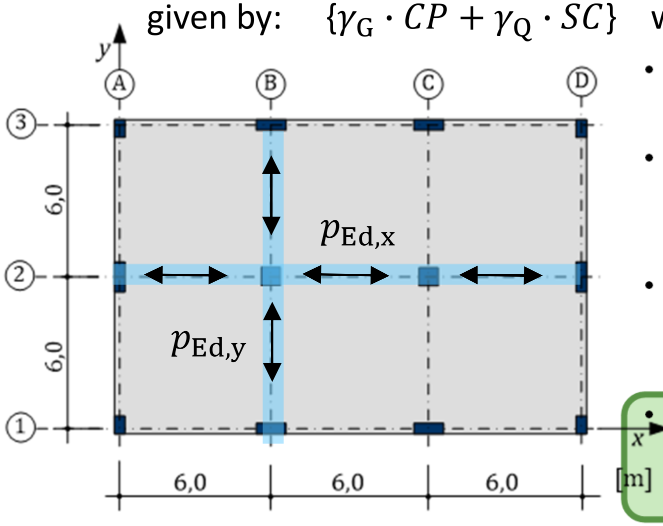
- Continuous span beam of ℓ = 6,0 m.
- Beam height h = 0,50 m ≈ $\ell/12$.
- Beam width b = 0,30 m ≈ h x 0.6.



a) direction x b) direction y

Analysis Model for Assessing Beam Internal Forces:

- To maintain consistency with the strip method, which assumes that the applied load in all slab panels is equally distributed in both principal directions.
- The design value of the uniformly distributed load for the ULS combination is given by: $\{\gamma_{\mathrm{G}} \cdot CP + \gamma_{\mathrm{Q}} \cdot SC\}$ with $\gamma_{\mathrm{G}} = 1{,}35$ & $\gamma_{\mathrm{Q}} = 1{,}50$



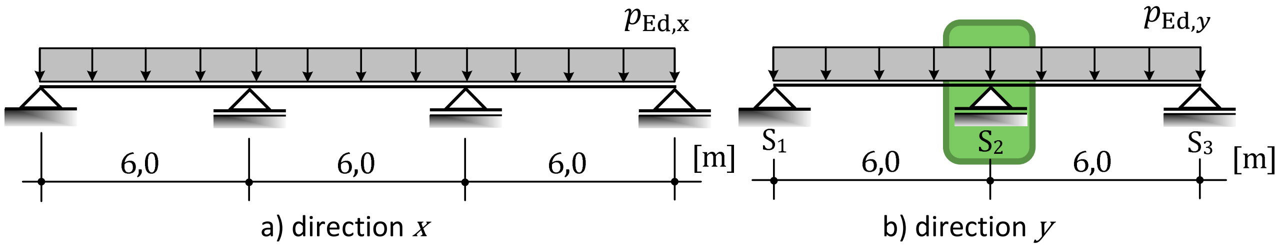
- Self-weight of the beam: $p_{\mathrm{beam}} = 25 \times (0.30 \times 0.50) = 3.75$ kN/m
- Edge beams (floors 1 to 3): $p_{\mathrm{Ed}} = 1{,}35 \times (3{,}75 + 7{,}0) + 3/8 \times (7{,}1 \times 6{,}0) = 30{,}4$ kN/m
- Interior beams in the x-direction: $p_{\mathrm{Ed},x} = 1{,}35 \times 3{,}75 + 2 \times [5/8 \times (7{,}1 \times 6{,}0)] = 58{,}1$ kN/m
- Interior beams in the y-direction: $p_{\mathrm{Ed,y}} = 1{,}35 \times 3{,}75 + [(5/8 + 1/2) \times (7{,}1 \times 6{,}0)] = 52{,}8$ kN/m

Gravitational Actions:

- Self-weight of structural elements.
- "Remaining permanent load" = 4,0 kN/m².
- Weight of external walls = uniformly distributed linear load of 7,0 kN/m, applied to the edge beams of the typical floors.
- Live load: $q_k = 2{,}0$ kN/m²; $\psi_2 = 0{,}3$.

Preliminary Beam Geometry Design:

- Continuous span beam of ℓ = 6,0 m.
- Beam height h = 0,50 m ≈ ℓ/12.
- Beam width b = 0,30 m ≈ h x 0.6.

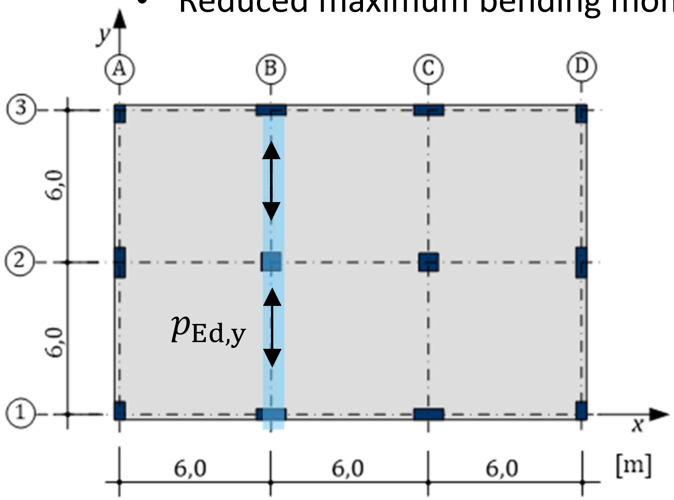


Estimation of Maximum Bending Reinforcement for the Beam

- Maximum bending moment at section S_2 of beam alignment B:

$$M_{Ed,S2} = -52{,}8 \times \frac{6{,}0^2}{8} = -237{,}5 \text{ kN} \cdot \text{m}$$

- Reduced maximum bending moment: $\mu_{max} = \frac{237{,}5}{0{,}30 \times 0{,}45^2 \times 20{,}0 \times 10^3} = 0{,}20$ ✓



- The section with the highest bending reinforcement in the beam must have: $A_{s,beam} \geq 14{,}1$ cm² (for B500 steel);
- This requirement can be achieved using, for example: 3 ϕ 25 mm ($A_s = 14{,}7$ cm²).

Seismic Action:

(i) Regular frame-type building;

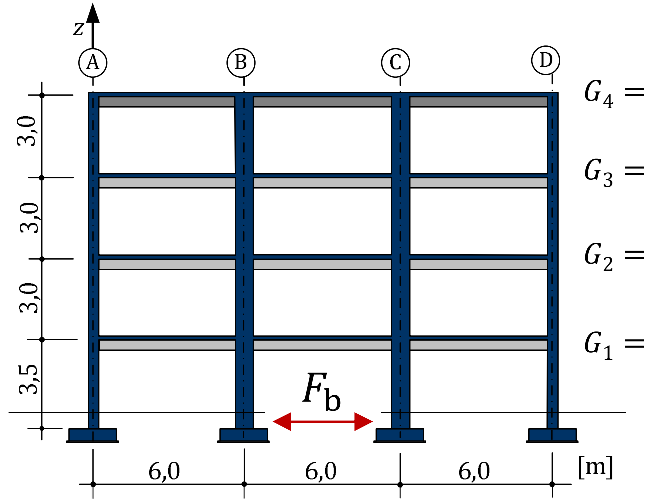
(ii) Maximum ground acceleration: $S_{\mathrm{b,max}} = 1{,}30$

(iii) Behaviour factor: assume $q = 3{,}9$ – the maximum value from EC8-1 for a regular building in plan and elevation with high ductility – DCH;

(iv) Fundamental natural frequency and period
$T_{1\mathrm{x}} = T_{1\mathrm{y}} = 0{,}075 \cdot H^{3/4} = 0{,}50\ \mathrm{s}$
$f_{1\mathrm{x}} = f_{1\mathrm{y}} = 2\ \mathrm{Hz}$ for H=12,5 m

Design Response Spectrum:

Seismic action	$a_\mathrm{g} = \gamma_\mathrm{I} \cdot a_\mathrm{gR}$ [m/s²]	S	T_C [s]	$S_\mathrm{d}(T_1 = 0{,}50\ \mathrm{s})$ [m/s²]
zone 1.2	2,0	1,23	0,60	1,58
zone 2.2	1,7	1,27	0,25	0,69



$G_4 = 2402$ kN

$G_3 = 2885$ kN

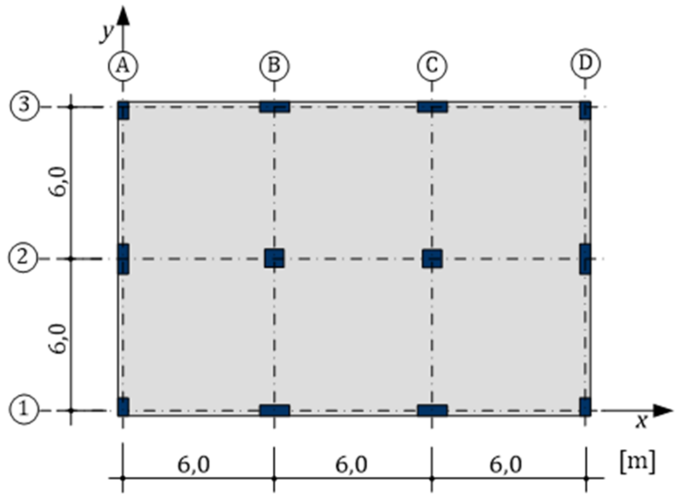
$G_2 = 2885$ kN

$G_1 = 2885$ kN

$mass =$
$= (3 \times 2885 + 2402)/9{,}8$
$= 1128{,}2$ t

Base seismic shear force =

$F_\mathrm{b} = \boxed{0{,}85} \times 1128{,}2 \times 1{,}58$
$= 1516$ kN

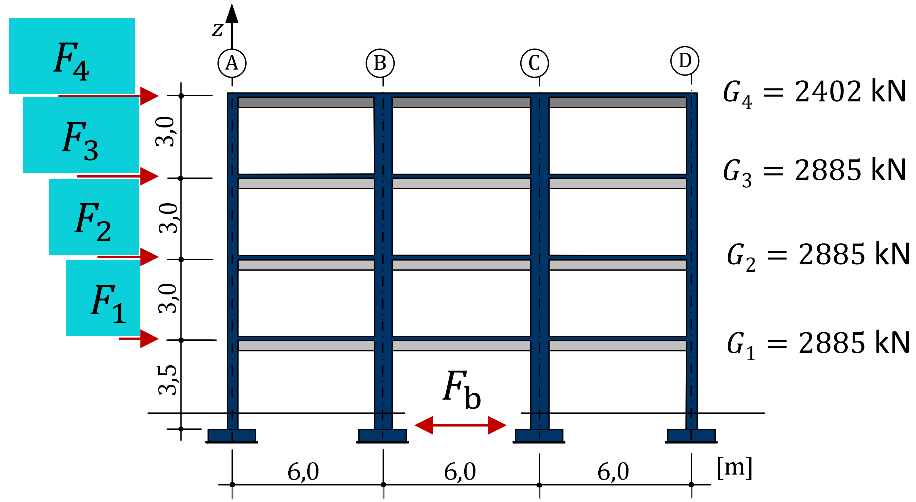


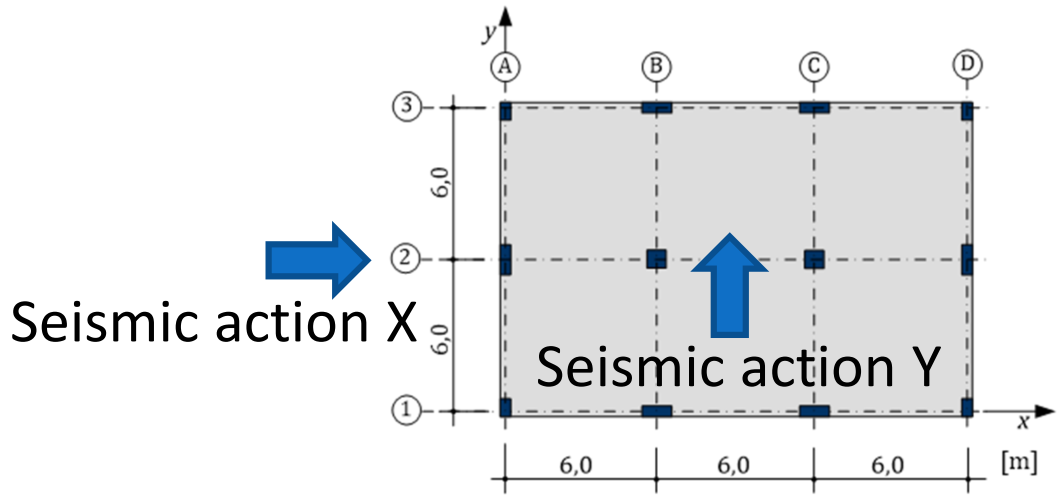
NOTE: According to EC8-1, the base seismic shear force is given by: $F_\mathrm{b} = \lambda \cdot m \cdot S_\mathrm{d}(T_1)$, where $\lambda = 0{,}85$ if the building has three or more storeys and $T_1 \le 2T_\mathrm{C}$ and $\lambda = 1{,}0$ for all other cases. The purpose of this factor λ is to account for the fact that, in such cases, the participation mass factor vibrating in the first mode does not correspond to the total mass of the building.

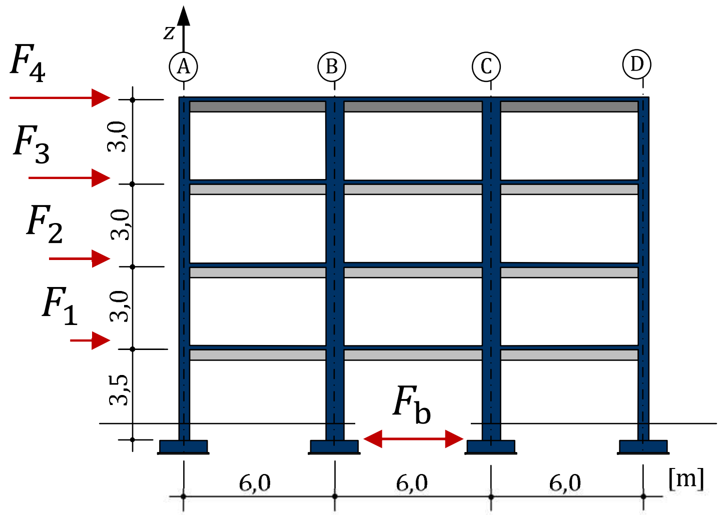
Lateral equivalent forces to the seismic action at each floor given by :

$$F_i \approx F_b \cdot (m_i \cdot z_i) / \sum (m_j \cdot z_j)$$

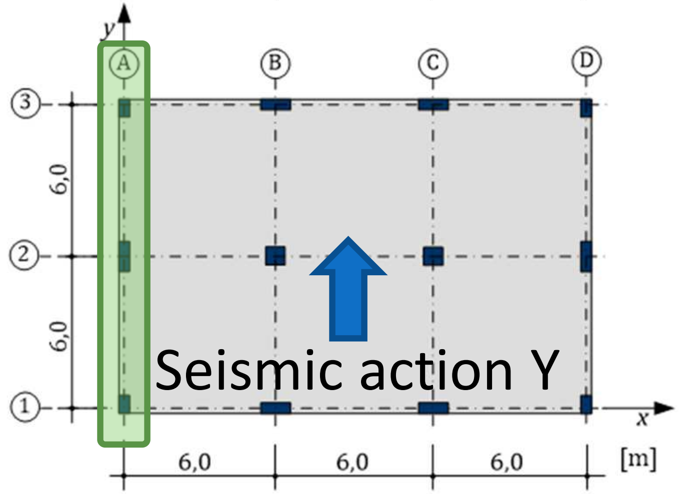
Floor (i)	z_i [m]	m_i [t]	$m_i \cdot z_i$ [t·m]	F_i [kN]
4 (roof)	12,5	245,1	3064	**528**
3	9,5	294,3	2796	**482**
2	6,5	294,3	1913	**329**
1	3,5	294,3	1030	**177**
	SUM	1128,2	8804	**1516**



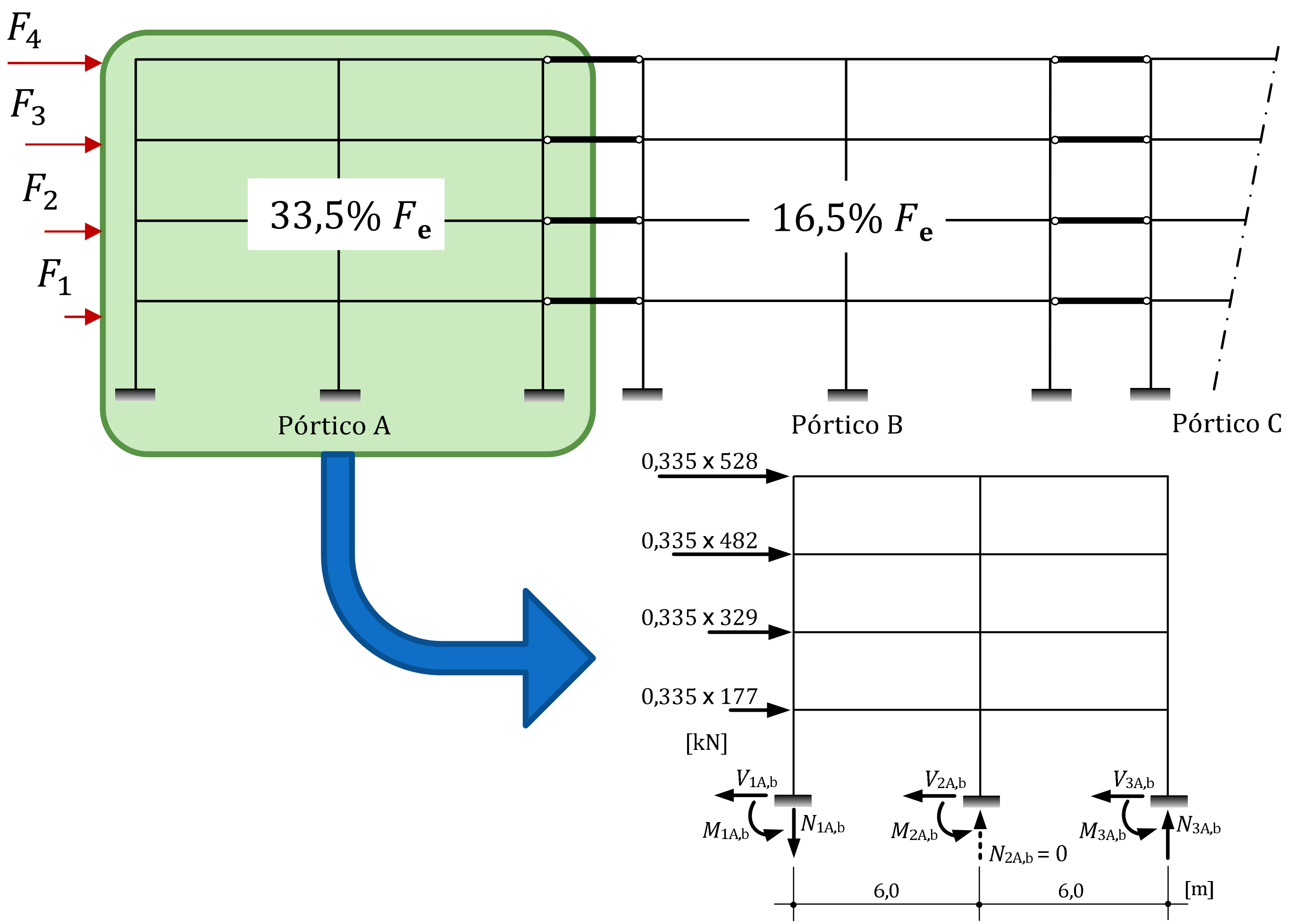




F_4
F_3
F_2
F_1
F_b
z
A B C D
3,0
3,0
3,0
3,5
6,0 6,0 6,0 [m]

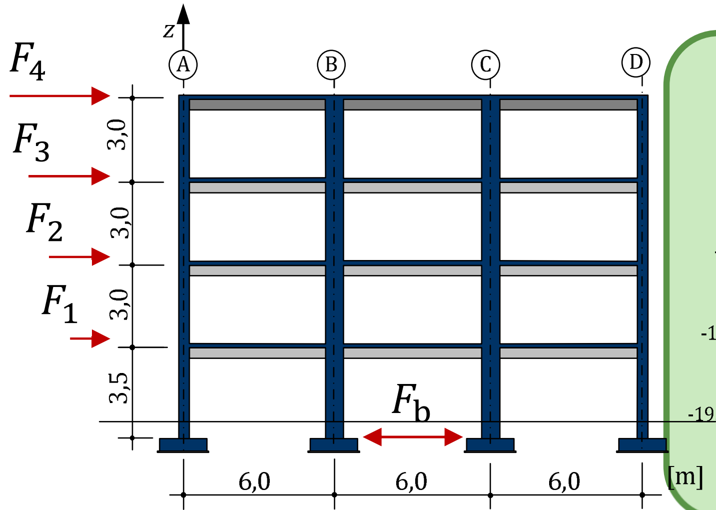


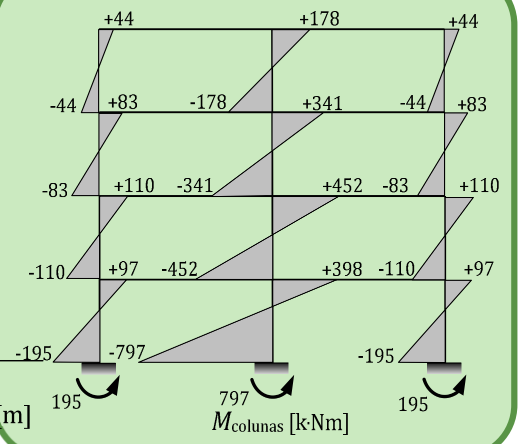
y
x
A B C D
1 2 3
6,0
6,0
6,0 6,0 6,0 [m]
Seismic action Y

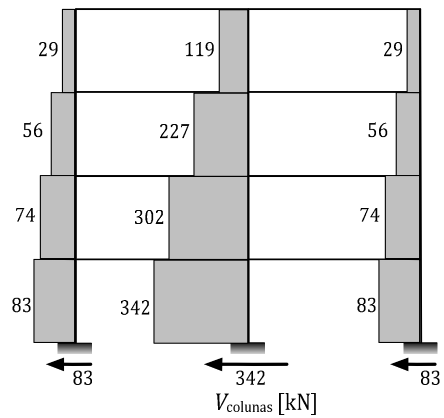


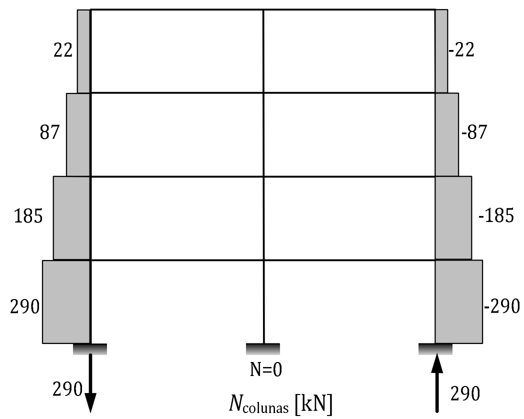
F_4
F_3
F_2
F_1
33,5% F_e
16,5% F_e
Pórtico A
Pórtico B
Pórtico C
0,335 x 528
0,335 x 482
0,335 x 329
0,335 x 177
[kN]
$V_{1A,b}$
$M_{1A,b}$
$N_{1A,b}$
$V_{2A,b}$
$M_{2A,b}$
$N_{2A,b} = 0$
$V_{3A,b}$
$M_{3A,b}$
$N_{3A,b}$
6,0 6,0 [m]

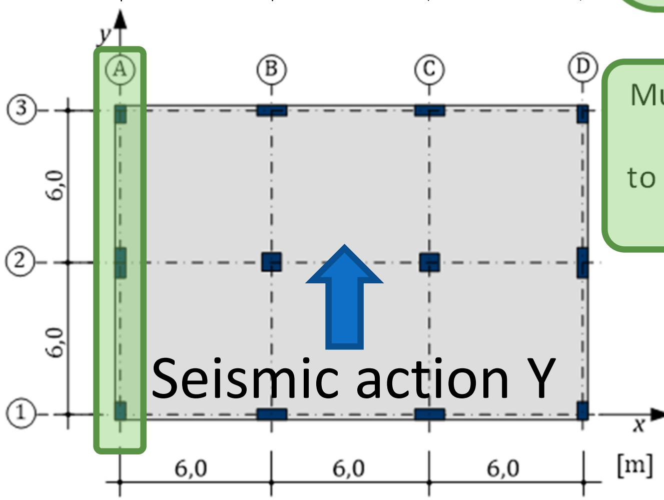
LATERAL FORCES METHOD OF ANALYSIS – Worked example





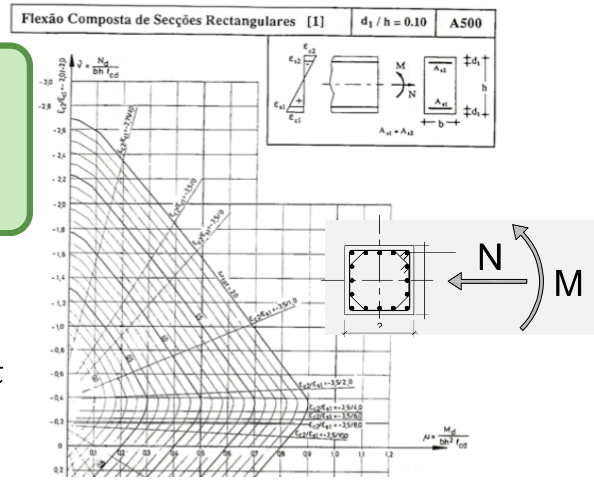






Multiply these internal forces by
$\xi = 1 + 0{,}6 \cdot {}^{9}/_{18} = 1{,}3$
to account for the floor torsional effect

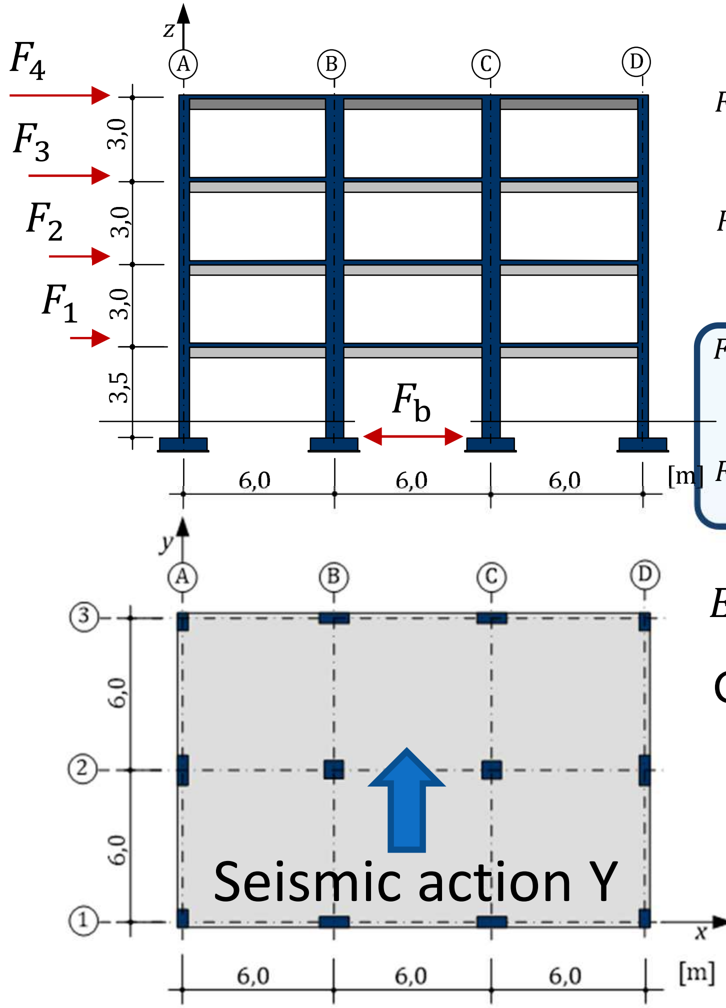
Bending reinforcement on the columns

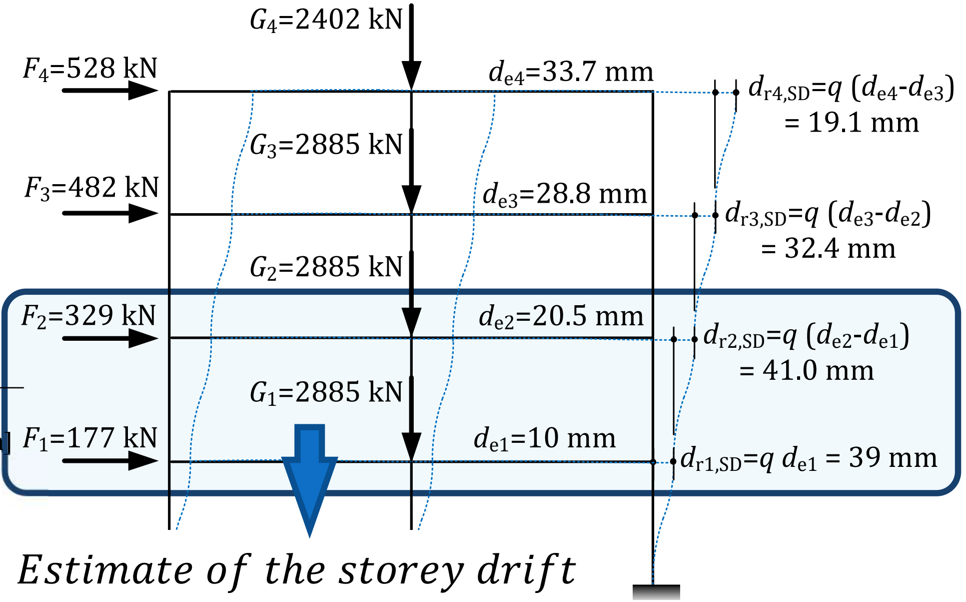


NOTE: According to EC8-1, the design forces in structural elements due to seismic action must be determined for the following two combinations.

$E_{E,x}$ " + " $0{,}30\ E_{E,y}$

$0{,}30\ E_{E,x}$ " + " $E_{E,y}$





Estimate of the storey drift

Governing storey 1-2:

$d_{r2,SD} = 41\text{ mm} \leq 0{,}02\ h_2 = 0{,}02 \cdot 3000 = 60\text{ mm}$ ✓
(assuming no masonry infills which is may not the case)

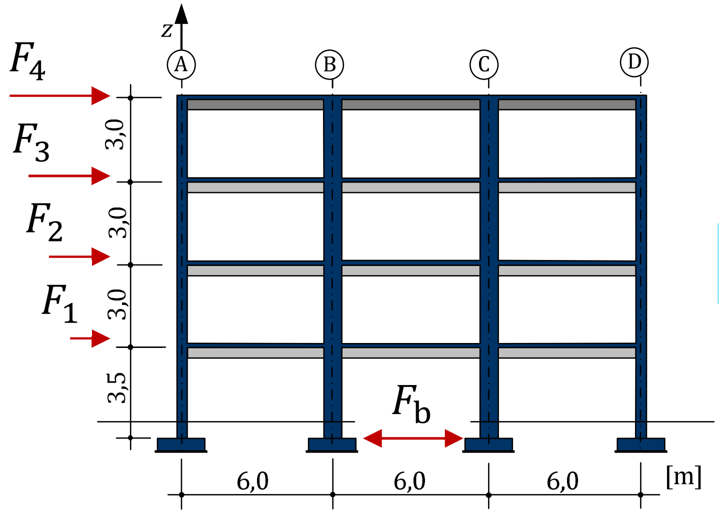
$$\theta = \frac{(2402 + 2 \cdot 2885) \cdot 41}{1{,}5 \cdot 1{,}3 \cdot (528 + 482 + 329) \cdot 3000} = 0{,}04 \leq 0{,}10 \checkmark$$

(2^{nd} order P–Δ effects are not required to be considered)

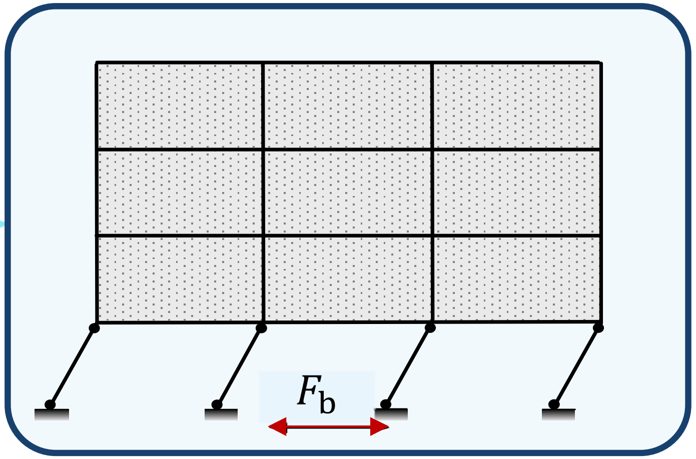
NOTE - For evaluating the seismic lateral displacement, the model lateral displacements of each floor should be multiplied by the behaviour factor.

Additionally, the modulus of elasticity of concrete in the columns is taken as half the reference value (for C30/37, E_c= 33/2 = 16,5GPa) to consider the cracking effects on concrete bending rigidity.

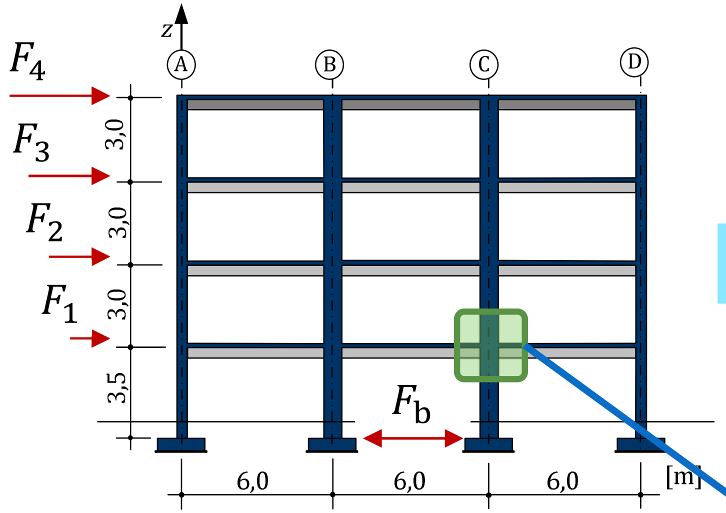
Capacity design rule for moment resisting frames



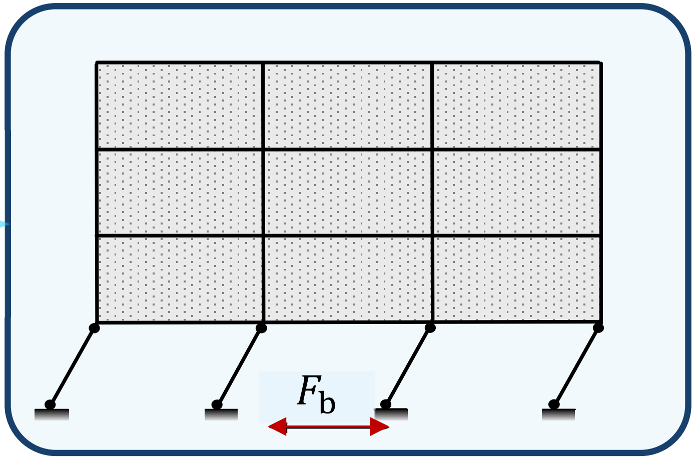
To prevent the formation of a flexible storey mechanism (soft storey) – activation of plastic hinges at the base and top sections of multiple columns between two consecutive floors.



Capacity design rule for moment resisting frames

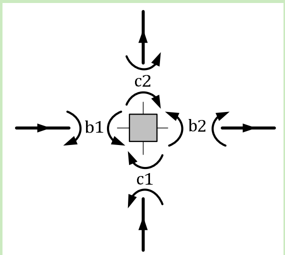


To prevent the formation of a flexible storey mechanism (soft storey) – activation of plastic hinges at the base and top sections of multiple columns between two consecutive floors.



Bending resistance of primary columns

$$\Sigma M_{Rd,c} \geq 1{,}30 \cdot (\Sigma M_{Rd,b})$$

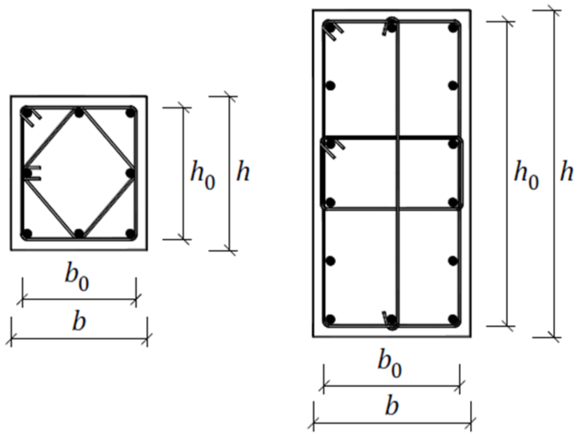


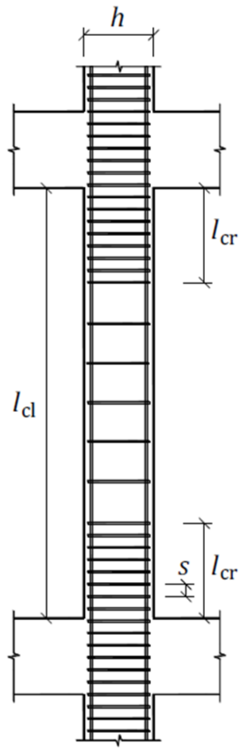
$$\begin{cases} \Sigma M_{Rd,c} = M^{+}_{Rd,c1} + M^{-}_{Rd,c2} \\ \Sigma M_{Rd,b} = M^{-}_{Rd,b1} + M^{+}_{Rd,b2} \end{cases}$$

Bending resistance of the beams

Detailing for local ductility (Rules from FprEN 1998-1-2:2025)

Transverse reinforcement in primary columns





distance $l_{cr} = \max\ \{b_{max}; l_{cl}/6; 0{,}45\ m\}$

with $b_{max} = \max\ \{b; h\}$

For DC2 – Spacing

$$s = \min\{b_o/2; 9d_{bL,min}; 0{,}200\ m\}$$

For DC3 – Spacing

$$s = \min\{b_o/2; 8d_{bL,min}; 0{,}175\ m\}$$

Detailing for local ductility (Rules from FprEN 1998-1-2:2025)

Confined reinforcement in wall ends

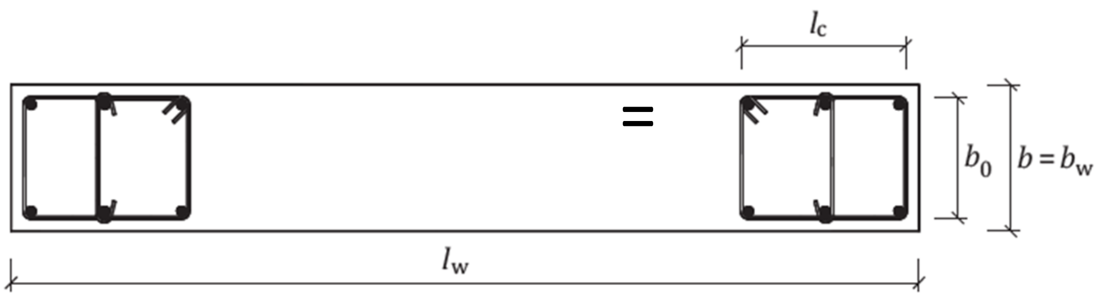


Figure 10.8 — Confined boundary element of free-edge wall end

Confined reinforcement should be put at least within the height of $h_{cr} = \max\ \{l_w ; h_w / 6\}$

$$h_{cr} \leq \min\ \{2l_w ; h_s \text{ , if } n \leq 6 \text{ storeys}\}$$
$$\leq \min\ \{2l_w ; 2h_s \text{ , if } n \leq 7 \text{ storeys}\}$$

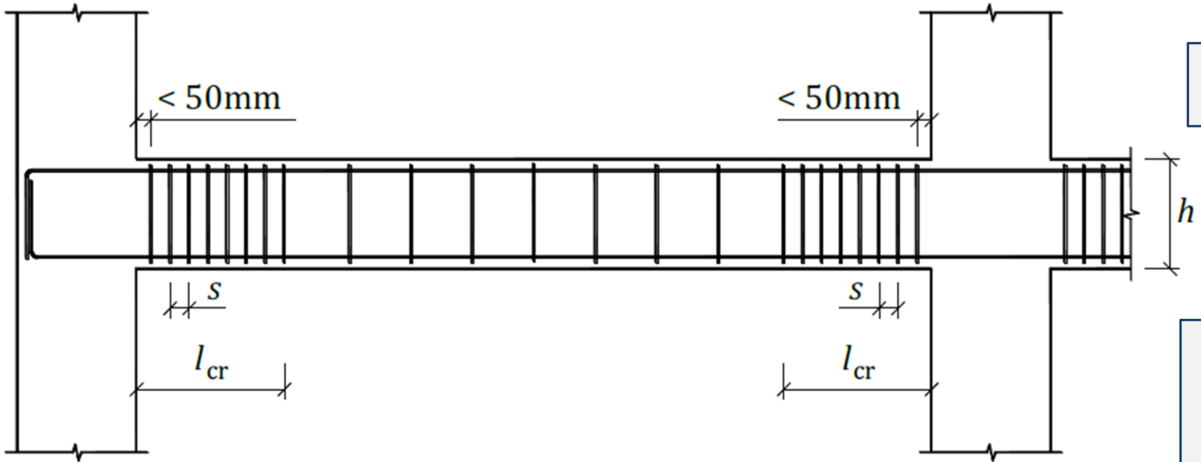
$2h_s$ and n are the storey height and the number of storeys of the wall

Spacing

$$l_c = \max\ \{0{,}15\ l_w ; 1{,}5\ d_w\}$$

Detailing for local ductility (Rules from FprEN 1998-1-2:2025)

Transverse reinforcement in critical regions of beams



Critical distance $l_{cr} = h$ (= beam depth)

For DC2 – Spacing

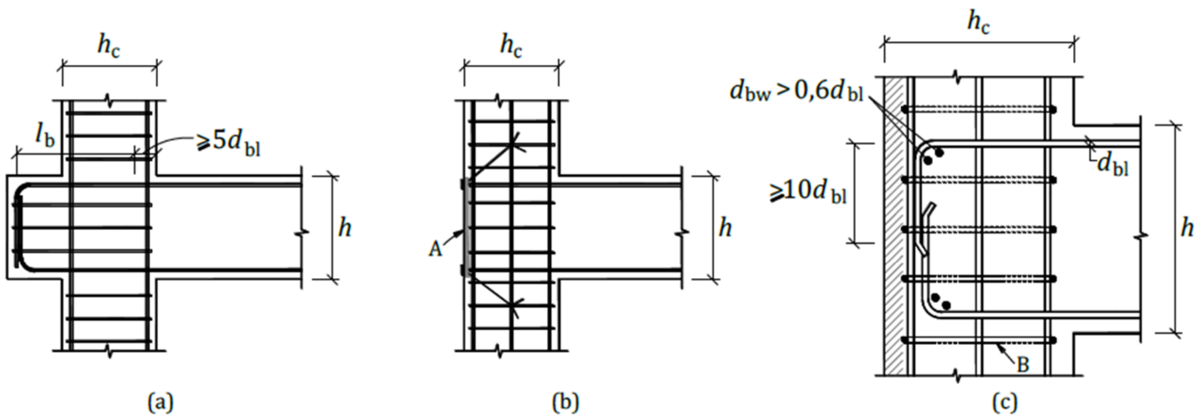
$$s = \min\{h/4;\, 30d_{bw};\, 12d_{bL,min}\}$$

For DC3 – Spacing

$$s = \min\{h/4;\, 24d_{bw};\, 8d_{bL,min}\}$$

Detailing for local ductility (Rules from FprEN 1998-1-2:2025)

Exterior beam – columns joint detailing:
Anchorage of reinforcement in beams



Key

A anchor plate
B hoops around column bars

Table 10.7 — Maximum diameter of beam longitudinal bars passing through joints

	Concrete grade	Reinforcing steel strength class	
		B400	B500
Exterior beam-column joints	C20/25 C25/30	DC2 or DC3: $d_{bL} \leq 0{,}045\ h_c$	DC2 or DC3: $d_{bL} \leq 0{,}035\ h_c$
	C30/37 C45/55	DC2 or DC3: $d_{bL} \leq 0{,}065\ h_c$	DC2 or DC3: $d_{bL} \leq 0{,}055\ h_c$
	C50/60 C90/105	DC2 or DC3: $d_{bL} \leq 0{,}095\ h_c$	DC2 or DC3: $d_{bL} \leq 0{,}075\ h_c$

Example – column with h_c = 0,45 m in C45/55 for an external beam-column joint
d_{bL} = 0,055 x 450 mm ≤ 25 mm

ANNEX – Additional information related to STEEL BUILDINGS - For steel structures EC8-1 defines steel buildings shall be designed in accordance with one of the following design concepts:

- **Concept a)** **Low-dissipative structural behaviour** – only accepted with DC1 – Low Ductility Class
- **Concept b)** **Dissipative structural behaviour** – should be adopted with DC2 – Medium Ductility and DC3 – High Ductility Classes

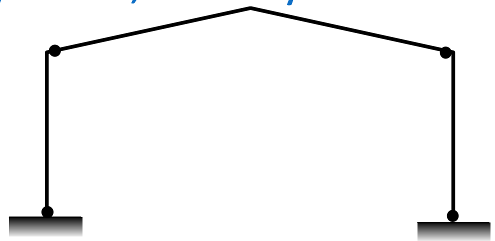
For each ductility class, the standard defined upper values for the behaviour factor should be:

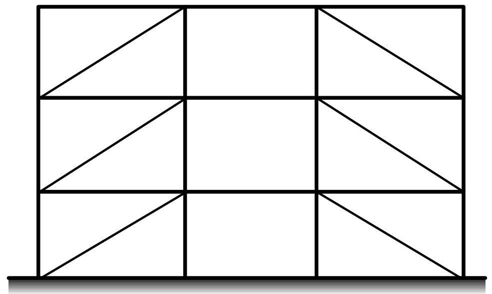
Table 11.1 — Structural ductility classes and corresponding upper limit reference values of the behaviour factors

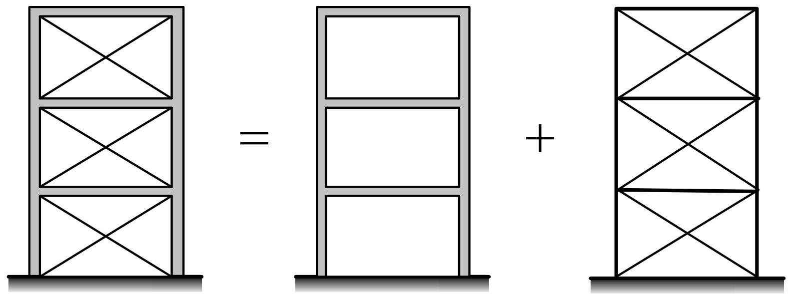
Structural ductility class	Upper limit reference values of the behaviour factor q
DC1	1,5
DC2	3,5
DC3	limited by the values of Table 11.3

ANNEX – Additional information related to STEEL BUILDINGS - For steel structures EC8-1 defines calculation rules and construction provisions with various types of structural systems, namely:

- **Moment resisting frames** – in which horizontal forces are mainly resisted by members acting in an essentially flexural manner;
- **Frames with braces** – in which horizontal forces are mainly resisted by members subjected to axial forces (under tension); bracings may be single storey bracings or double storey bracings;
- **Inverted pendulum structures** – in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building member or at the bases of columns;
- **Structures with concrete cores or walls** – in which horizontal forces are mainly resisted by these cores or walls (shear resistance at the concrete cores or walls exceeds 75% of the total shear resistance);
- **Moment resisting frames combined with concentric bracings** – in which horizontal forces are resisted by both the concrete frames and steel bracings.

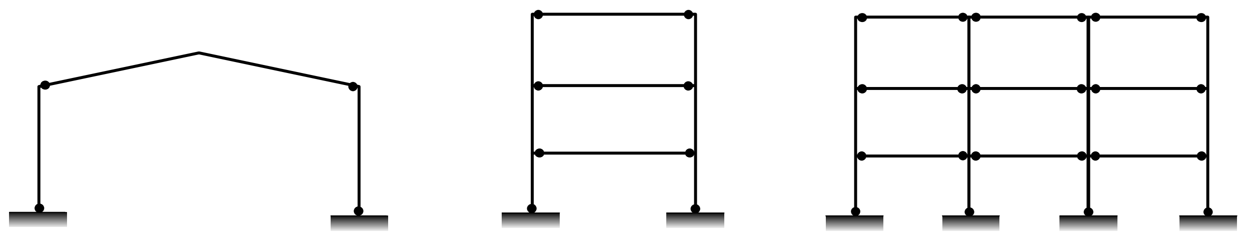




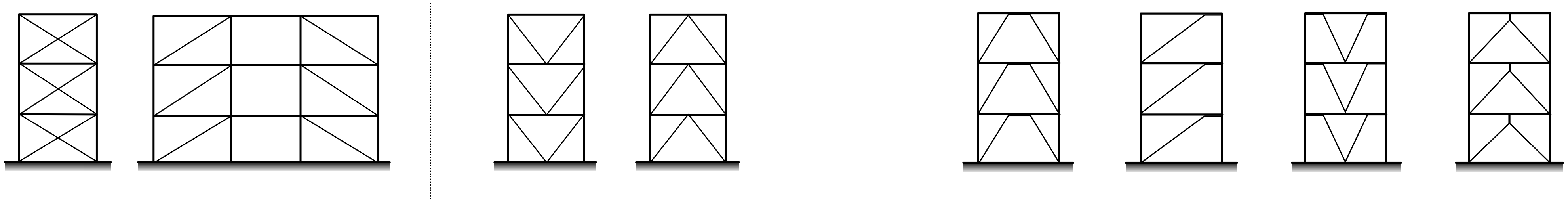


ANNEX – Additional information related to STEEL BUILDINGS - For steel structures EC8-1 defines calculation rules and construction provisions with various types of structural systems, namely:

- **Moment resisting frames** – in which horizontal forces are mainly resisted by members acting in an essentially flexural manner; MRF can have 1, 2 or more storeys



- **Frames with braces** – can have concentric braces or eccentric braces, in in which horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear



SPECIFIC RULES FOR STEEL BUILDINGS: Behaviour factors

ANNEX – Additional information related to STEEL BUILDINGS - Behaviour factors for steel structures are relatively high, taking advantage of the high ductility of these types of structures. For regular buildings with β=1.0, and considering a high ductility class (DC3), the values below are obtained.

$$q = \max\{\beta \cdot q;\ 1,5\}$$

Default upper limit values of behaviour factors for steel systems regular in elevation

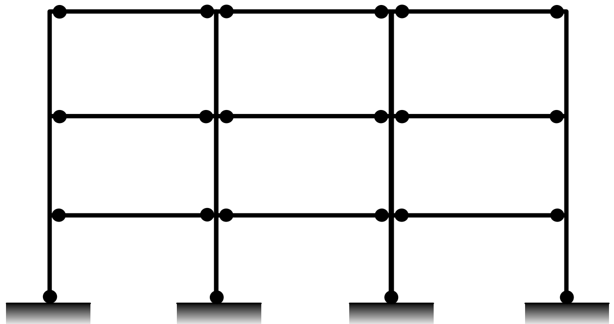
Structural Type	Ductility Class DC2	Ductility Class DC3
Moment resisting frames with:		
• single-storey with class 1 and 2 cross sections	3,0	5,5
• multi-storey	3,5	6,5
Frames with concentric braces: Diagonal bracings and V bracings	2,5	4,0
Frames with eccentric braces	3,0	6,0
Inverted pendulum	2,0	2,3
Structures with concrete cores or concrete walls	See Table 10.1: walls or wall equivalent structures	

with:

- ✓ $\beta = 1{,}0$ or $\beta = 0{,}8$ depending on whether the structure is regular or irregular in elevation,
- ✓ the default coefficient q is obtained from the table, based on the type of structural system

SPECIFIC RULES FOR STEEL BUILDINGS: Additional requirements

ANNEX – Additional information related to STEEL BUILDINGS - EC8-1 provides some additional rules for the design of the main steel structure and bracing:



- ✓ In simple frames with two or more stories, the condition of *'columns being more resistant than beams'* must be satisfied at all joints, except at the base of the frame and at the joints on the top floor.
- ✓ To ensure sufficient local ductility, the cross-section of the dissipative elements in compression or bending must comply with the class based on the structure's ductility class and the behaviour coefficient.

DC2	$q \leq 2$	Cross-section of class 1, 2, or 3 (or even 4)
	$2 < q \leq 3{,}5$	Cross-section of class 1 or 2
DC3	$q \leq 3{,}5$	Cross-section of class 1 or 2
	$q > 3{,}5$	Cross-section of class 1

- ✓ Also, to ensure plastic deformation capacity, the dissipative elements under tension (such as diagonals) with bolted connections must satisfy the condition $N_{u,Rd} > N_{pl,Rd}$ (the design resistance of the connection must be greater than the design plastic resistance of the element).

SPECIFIC RULES FOR STEEL BUILDINGS: Additional requirements

ANNEX – Additional information related to STEEL BUILDINGS - EC8-1 provides some additional rules for the design of the main steel structure and bracing:

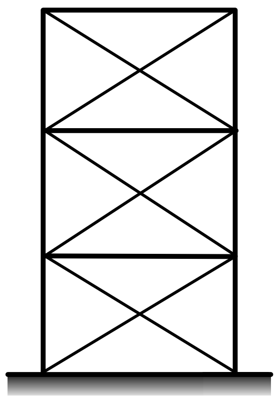
- ✓ The non-dissipative connections of dissipative elements must have sufficient over-resistance to prevent restricting the cyclic yielding of the connected element. This criterion is automatically considered satisfied in connections made by full penetration butt welds; bolted connections or connections made by fillet welds must satisfy the following condition,

$$R_{\mathrm{d,lig}} \geq 1{,}1 \cdot \gamma_{\mathrm{ov}} \cdot R_{\mathrm{yd,elem}}$$

  where $R_{\mathrm{d,lig}}$ is the design value of the connection resistance, $R_{\mathrm{yd,elem}}$ is the design value of the plastic resistance of the connected dissipative element, and γ_{ov}=1.25 is a typical overstrength factor. In practice, these connections must be designed for a design force 38% higher than $R_{\mathrm{yd,elem}}$.

Additionally, in frames with centered bracing:

- ✓ It must be ensured that the yielding of the diagonals under tension occurs before the failure of any connection and before the yielding or buckling of beams or columns.
- ✓ In frames with more than two stories, the normalized slenderness of the diagonals $\bar{\lambda}$ should not exceed 2,0.



UZ PRIEKSU INŽENIERIJĀ

Paldies par uzmanību un sadarbību!

Sirsnīgi pateicamies visiem Latvijas inženieriem par izrādīto interesi un sniegto uzticēšanos.

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Tower of Belém – Lisbon - completed in 1521

RC buildings – Lisbon – under construction