

Training seminar for Latvian structural engineers /

Apmācību seminārs Latvijas būvkonstrukciju projektētājiem:

Seismic-resistant structural design / Seismiski izturīgu konstrukciju projektēšana

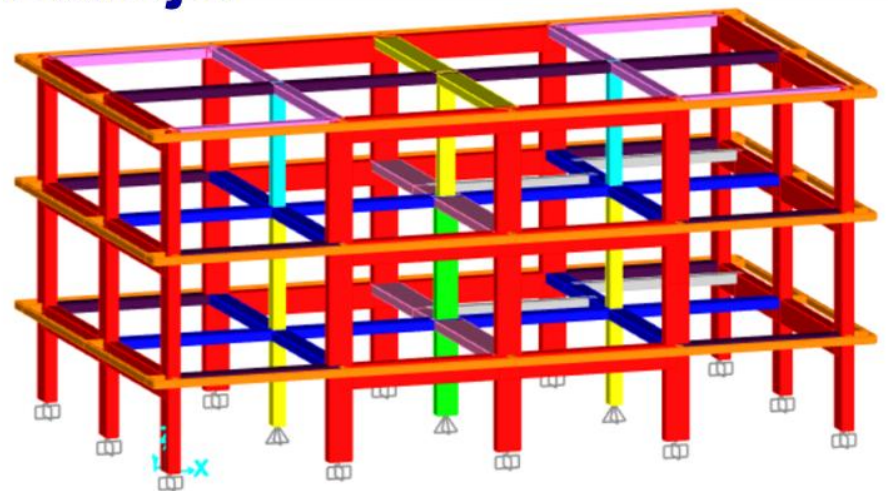
Module 3: Seismic design of structures

Online, 9th April 2025



LATVIJAS
BŪVKONSTRUKCIJU
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INTRODUCTION – GENERAL CONCEPTS

What is ductility?

Why is it Important?

Seismic Performance: Ductile structures dissipate seismic energy more effectively.

Structural Redundancy: Ability to redistribute stresses in case of localized damage.

Factors Influencing Ductility (Rc structures)

Quantity and Arrangement of Reinforcement

Concrete Class and Type of Steel

Concrete Confinement (e.g., closed stirrups)

Hierarchy of Strengths (capacity vs. demand)

Types of Ductility

Material Ductility: Behavior of the material (steel or concrete).

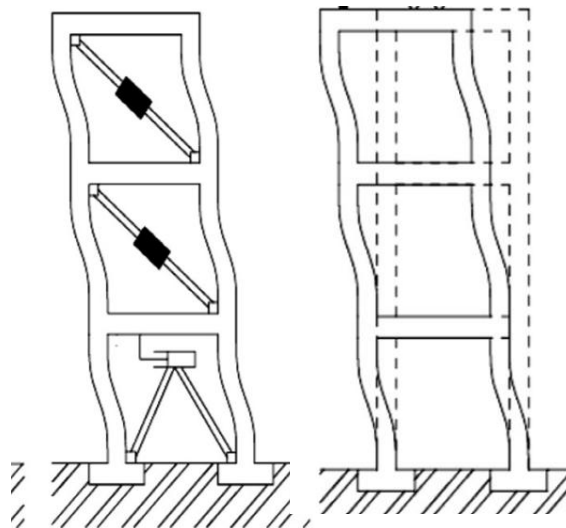
Local Ductility: Behavior of a single cross-section.

Member Ductility: Behavior of a single member (e.g., a member with **plastic hinges**).

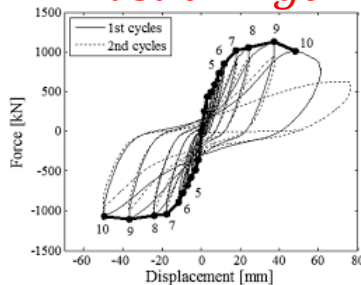
Global Ductility: Overall behavior of the structure (e.g., a **frame**).

INTRODUCTION – GENERAL CONCEPTS

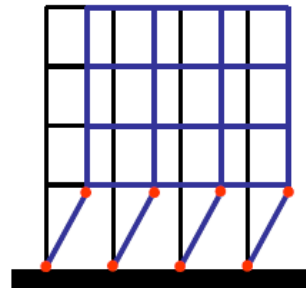
During an earthquake, the structure must dissipate the input energy to avoid collapse and protect lives. This is achieved by designing ductile mechanisms that allow controlled plastic deformation



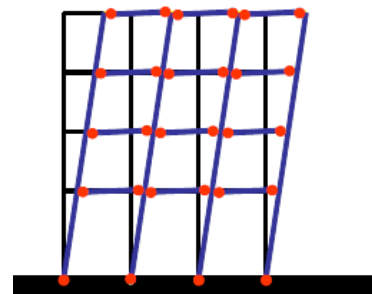
Plastic hinge



Frame ductility

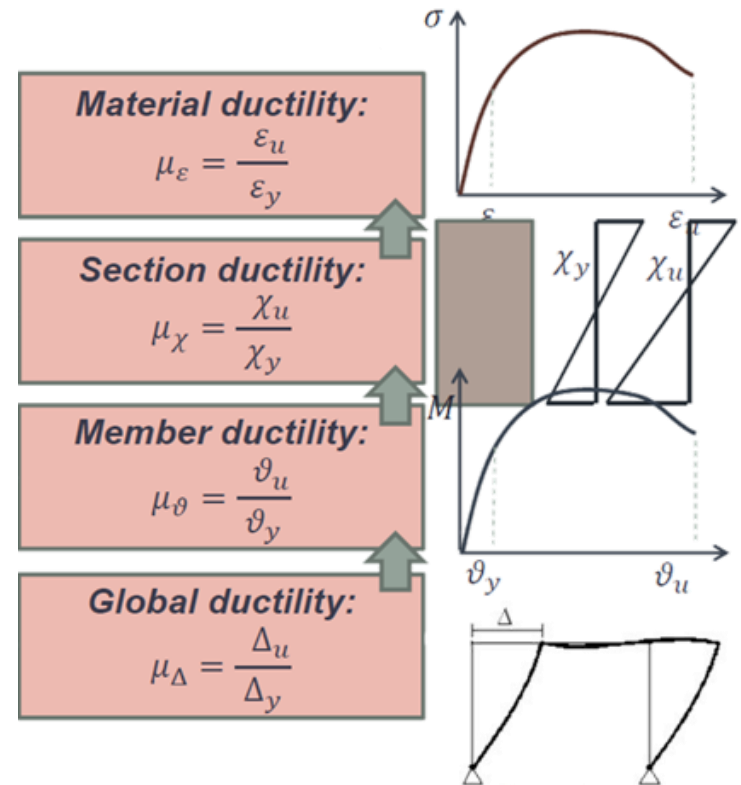


Story mechanism



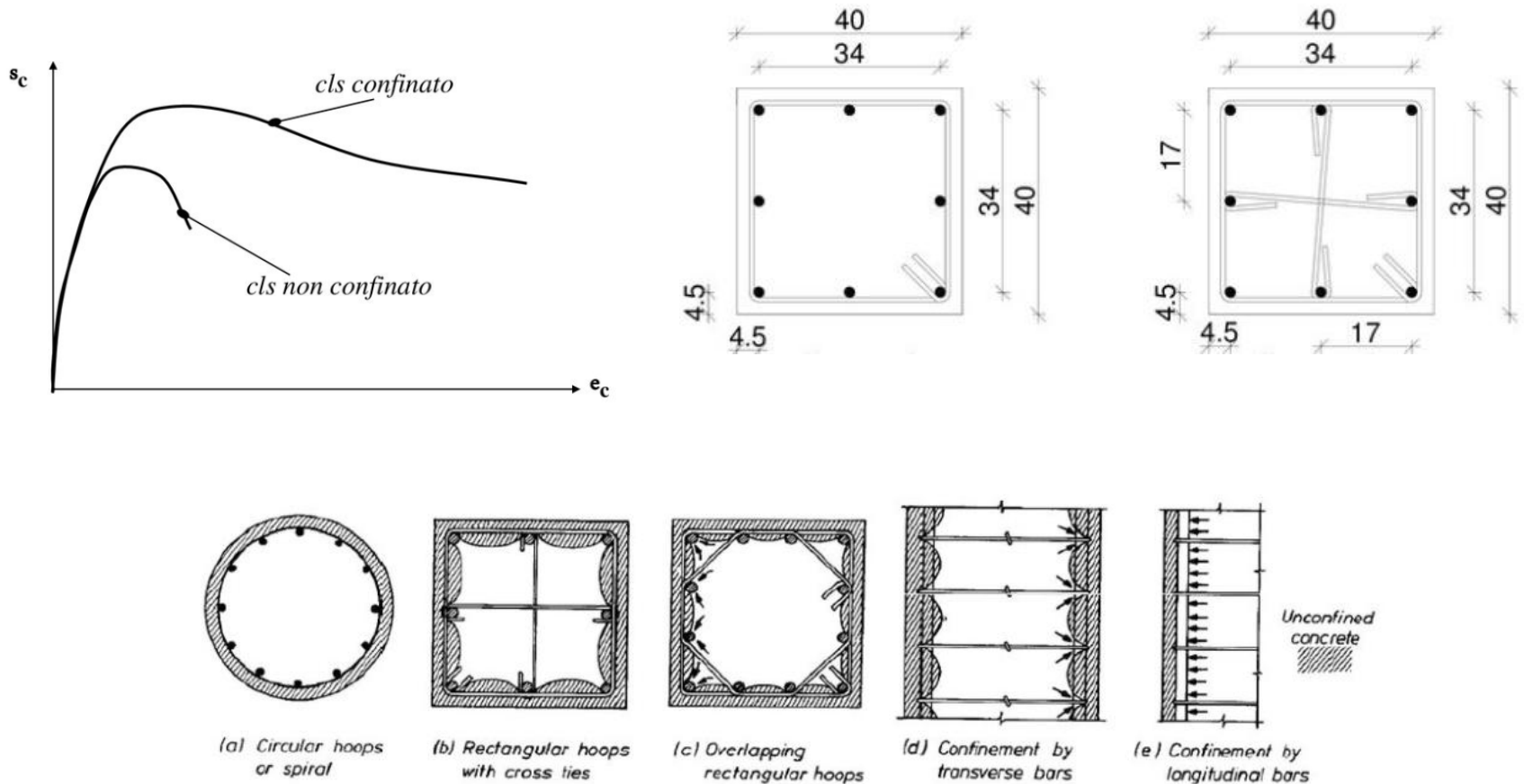
Sway mechanism

Global and local ductility



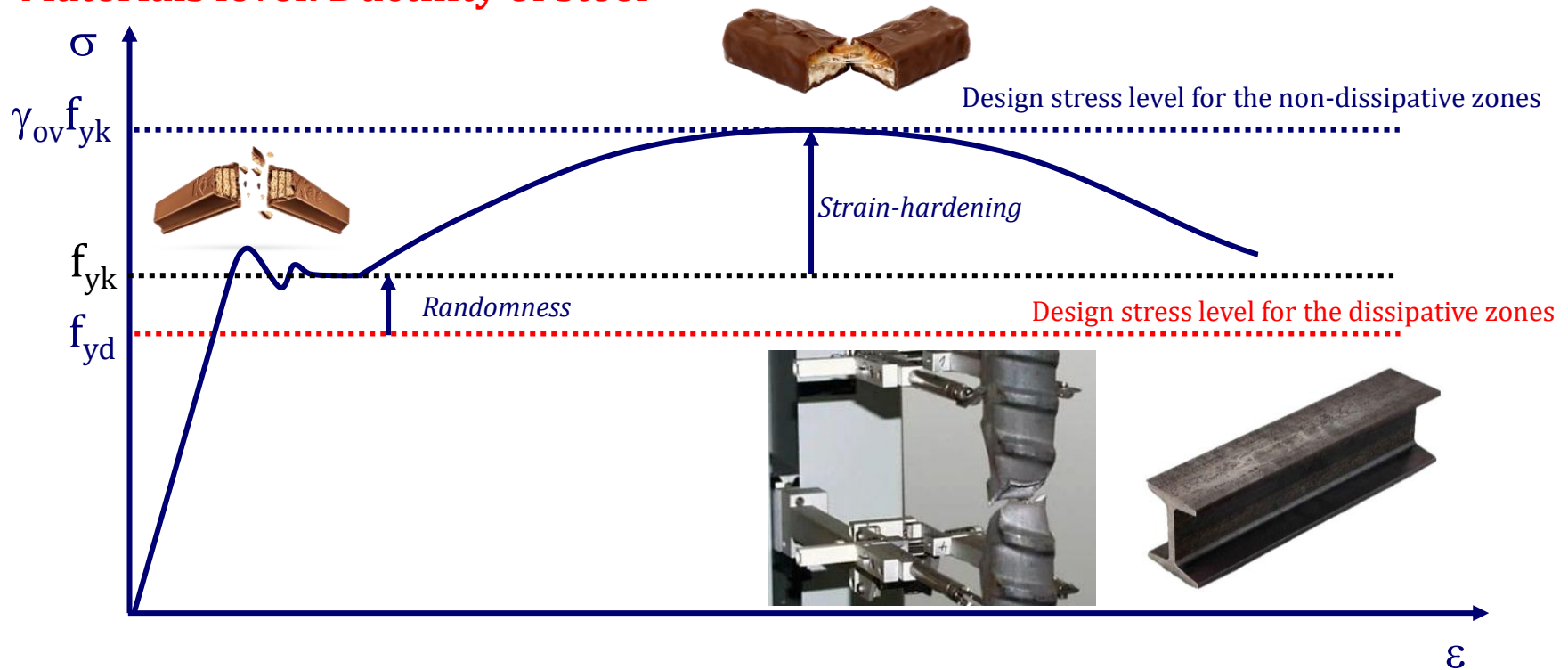
INTRODUCTION – GENERAL CONCEPTS

Materials level: Ductility of concrete



INTRODUCTION – GENERAL CONCEPTS

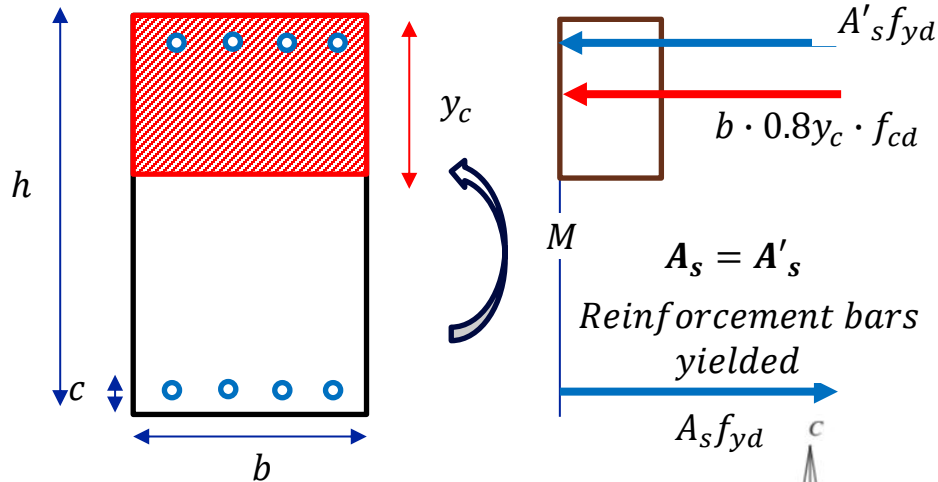
Materials level: Ductility of steel



Randomness and material strain-hardening both will affect the seismic design. According to the capacity design principles, the dissipative zone will be designed to yield under seismic actions, while all the other structural parts will be designed to resist to the maximum actions transferred by the dissipative zones accounting for both these effects.

INTRODUCTION – GENERAL CONCEPTS

Cross-section level: Ductility of rc sections



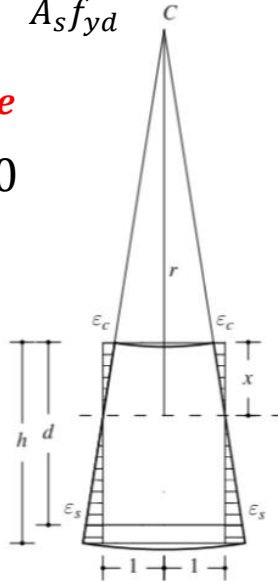
Translational equilibrium at failure

$$0.8 y_c b f_{cd} + A'_s f_{yd} - A_s f_{yd} = 0$$

$$y_c = h \frac{(\omega - \omega')}{0.8}$$

$$\omega = \frac{A_s f_{yd}}{b h f_{cd}}$$

$$\omega' = \frac{A'_s f_{yd}}{b h f_{cd}}$$



Ductility of a rc beam

$$\left(\frac{1}{r}\right)_y = \frac{\epsilon_{sy}}{d - y_{ce}} = \frac{\epsilon_{sy}}{d(1 - y_{ce}/d)}$$

$$\left(\frac{1}{r}\right)_u = \frac{\epsilon_{cu}}{y_c} = 0.8 \frac{\epsilon_{cu}}{h} \frac{1}{\omega - \omega'}$$

Typical Example

if $y_{ce} \cong 0.25 \div 0.3d = 0.25 \div 0.3(h - c)$

$$\left(\frac{1}{r}\right)_y = \frac{\epsilon_{sy}}{d(1 - y_{ce}/d)} \cong 1.4 \frac{\epsilon_{sy}}{d}$$

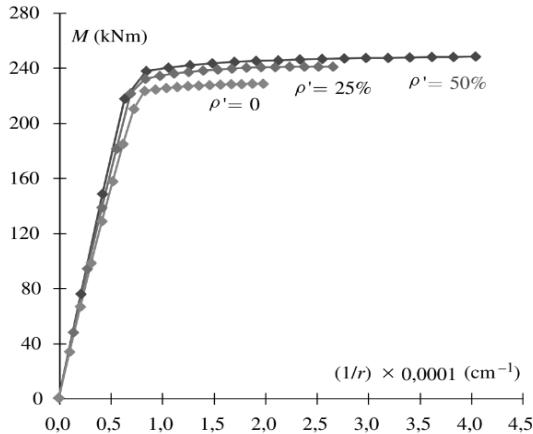
$$\mu_{1/r}^c = \frac{\left(\frac{1}{r}\right)_u}{\left(\frac{1}{r}\right)_y} = \frac{0.8 d \epsilon_{cu}}{1.4 h \epsilon_{sy}} \frac{1}{\omega - \omega'} \cong 0.51 \frac{\epsilon_{cu}}{\epsilon_{sy}} \frac{1}{\omega - \omega'}$$

with $c/h \cong 0.1$

$d/h \cong 0.9$

INTRODUCTION – GENERAL CONCEPTS

Cross-section level: Ductility of rc sections



$$\mu_{1/r}^c = \frac{\left(\frac{1}{r}\right)_u}{\left(\frac{1}{r}\right)_y} = \frac{0.8 d \varepsilon_{cu}}{1.4 h \varepsilon_{sy}} \frac{1}{\omega - \omega'} \cong 0.51 \frac{\varepsilon_{cu}}{\varepsilon_{sy}} \frac{1}{\omega - \omega'}$$

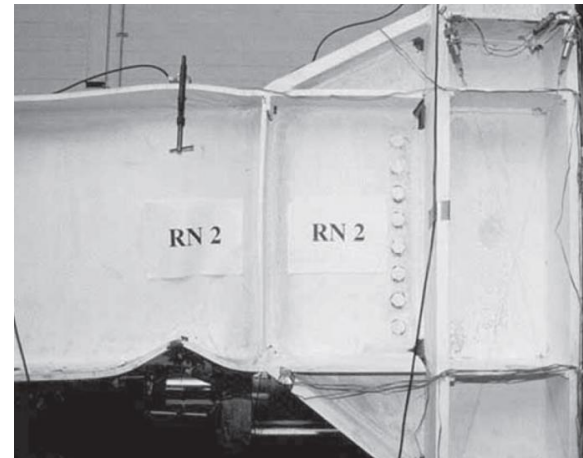
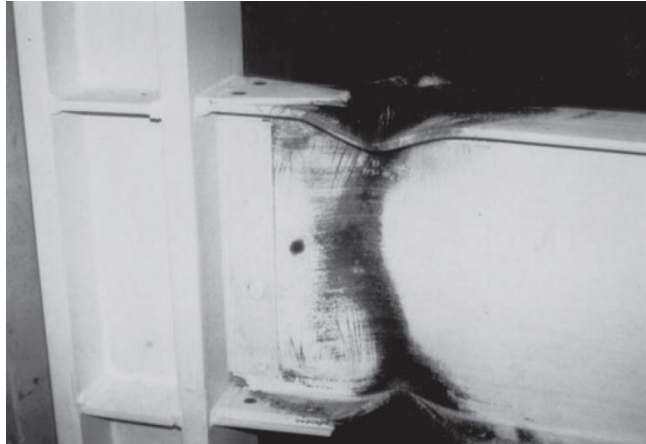
with $c/h \cong 0.1$ $d/h \cong 0.9$

To increase the value of the ultimate curvature in beams—thus making them more ductile—two approaches can be taken:

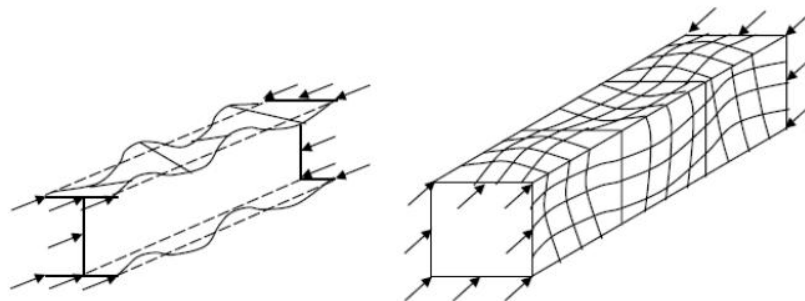
- By providing an adequate amount of **compressive reinforcement**. In seismic zones, the code requires that the amount of compressed reinforcement be at least **50%** of the tensile reinforcement in the **critical regions**, and at least **25%** in all other areas.
- By **increasing the ultimate strain of the concrete** through proper **confinement of the material**, as will be explained in more detail later. To this end, the code specifies minimum amounts of **transverse reinforcement (stirrups)** both in critical regions and elsewhere.

INTRODUCTION – GENERAL CONCEPTS

Cross-section level: Ductility of steel sections



The behaviour of the cross-section in plastic range depends on the slenderness of the plates composing the cross-section. Usually the slenderness of H sections is such to allow the achievement of the plastic resistance before the occurrence of local buckling, nevertheless, this is not a general rule and needs to be checked case by case.

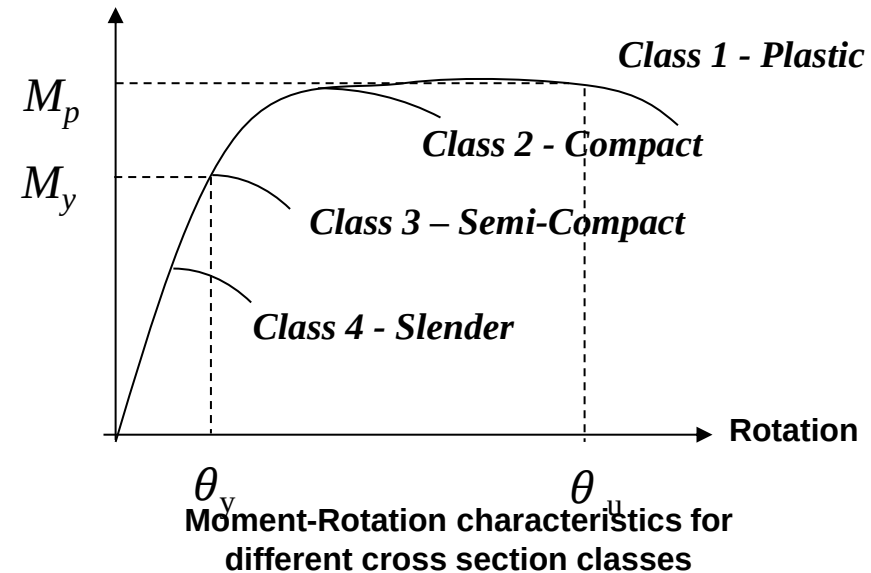


INTRODUCTION – GENERAL CONCEPTS

Cross-section level: Ductility of steel sections

Four section compactness classes to EC3:

- **Class 1 [Plastic]:**
Can develop M_p with significant rotation ductility (required for DCH)
- **Class 2 [Compact]:**
Can develop M_p with limited rotation capacity (required for DCM)
- **Class 3 [Semi-compact]:**
can develop M_y but not M_p
- **Class 4 [Slender]:**
unable to develop M_y before local buckling

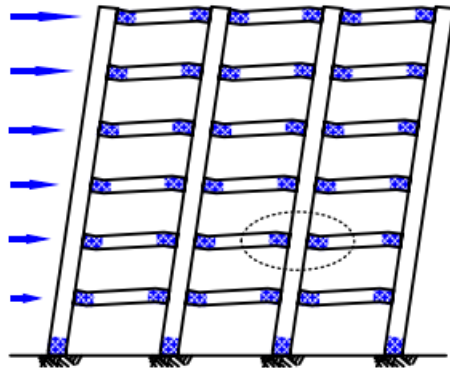


Requirements on cross-sectional class of dissipative elements depending on Ductility Class [DCH or DCM] and Behaviour Factor (q)

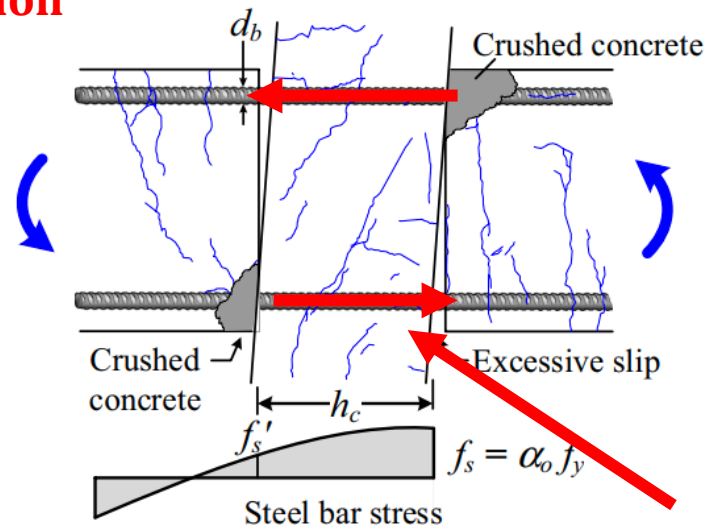
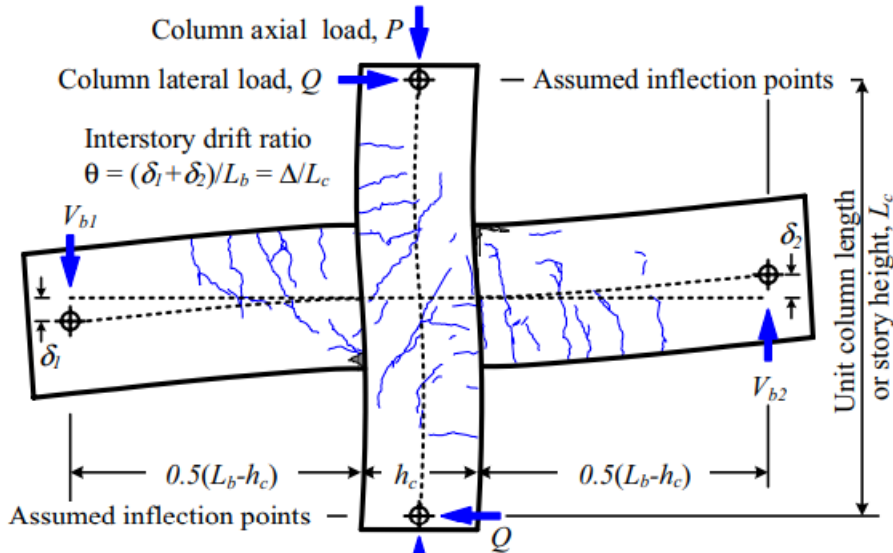
<i>Ductility Class</i>	<i>Reference q-factor</i>	<i>Cross-Section Class</i>
DCM	$1.5 < q \leq 2$	Class 1, 2 or 3
	$2.0 < q \leq 4$	Class 1 or 2
DCH	$q > 4$	Class 1

INTRODUCTION – GENERAL CONCEPTS

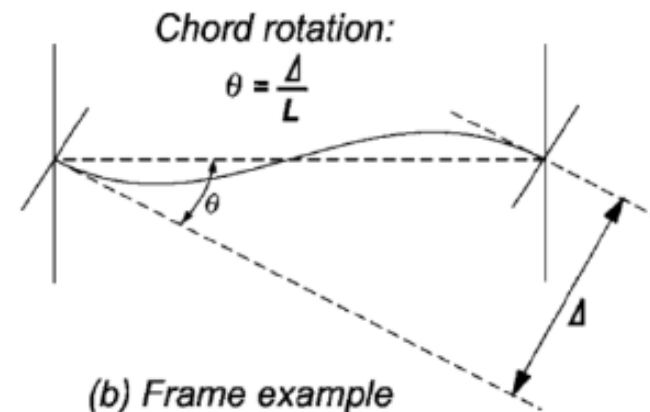
Member level: Ductility in terms of rotation



a Idealized beam-yielding mechanism



Shear force in the joint

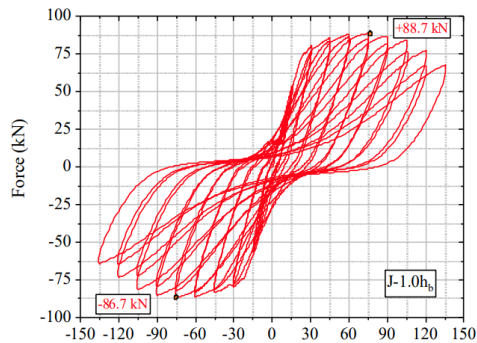


(b) Frame example

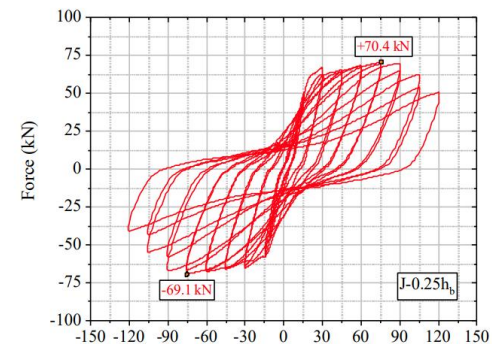
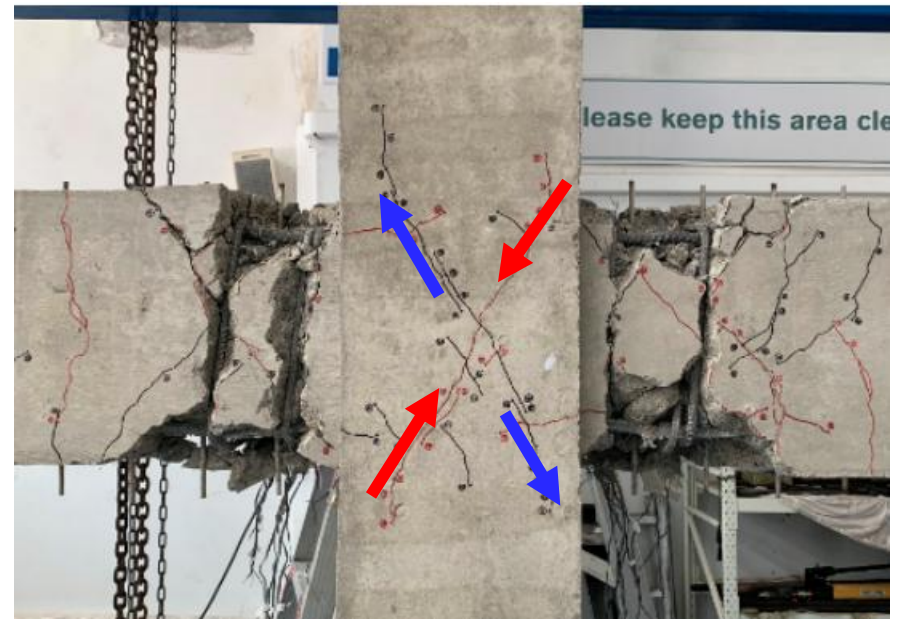
INTRODUCTION – GENERAL CONCEPTS

Member level: Ductility in terms of rotation

Joint failure in shear



Plastic hinge in the beam



INTRODUCTION – GENERAL CONCEPTS

Member level: Ductility in terms of rotation

(Weak beam-strong column-strong joint)



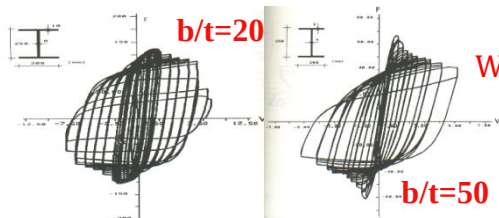
Dissipative zones are located in beams; joints and columns are properly overstrengthened

$$M_{Rj} > 1.1\gamma_{ov}M_{Rb}$$

Full-strength joints

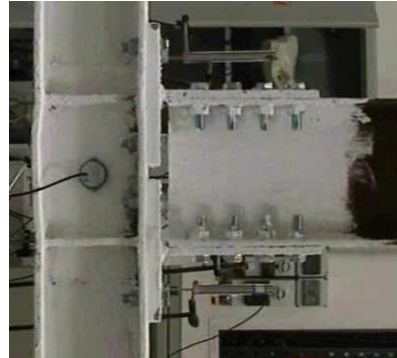
$$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$$

Beam-column hierarchy



Wide and stable hysteretic loops

(Strong beam-strong column-weak joint)



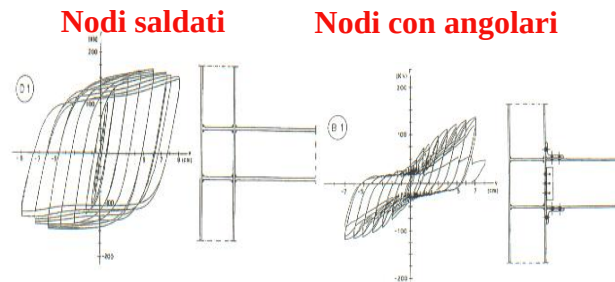
Dissipative zones are located in joints; beams and columns are properly overstrengthened

$$M_{Rj} < M_{Rb}$$

Partial-strength joints

$$\sum M_{Rc} \geq 1.3 \sum M_{Rj}$$

Beam-column hierarchy

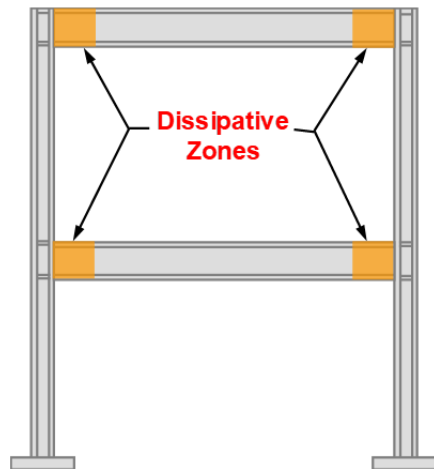


Dissipative capacity depends on the response of joints

INTRODUCTION – GENERAL CONCEPTS

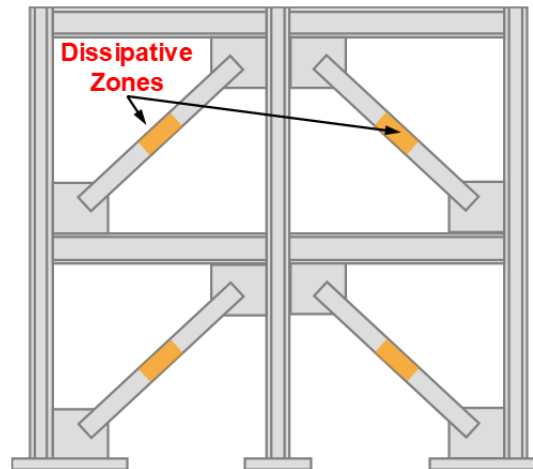
Structural level: Ductility in terms of sway displacement

Moment Resisting Frame (MRF)



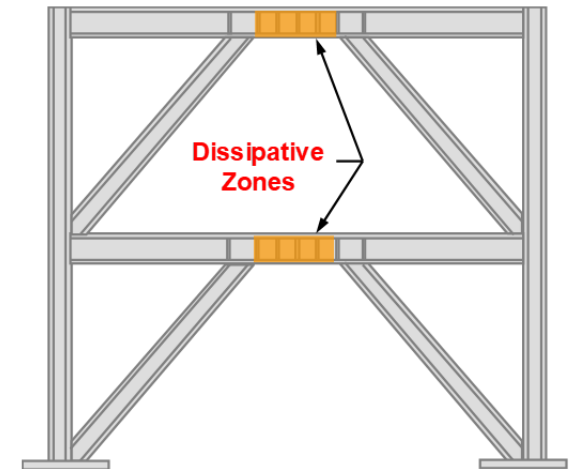
Seismic forces resisted
flexural

Centrally Braced Frame (CBF)



Seismic forces resisted
axial forces

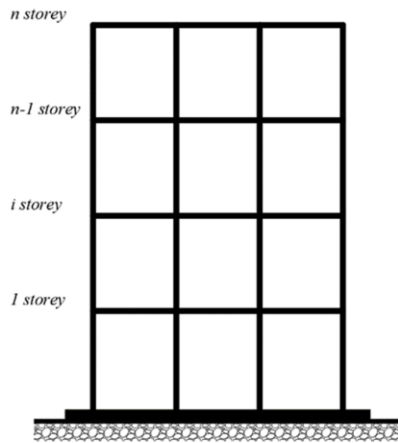
Eccentrically Braced Frame (EBF)



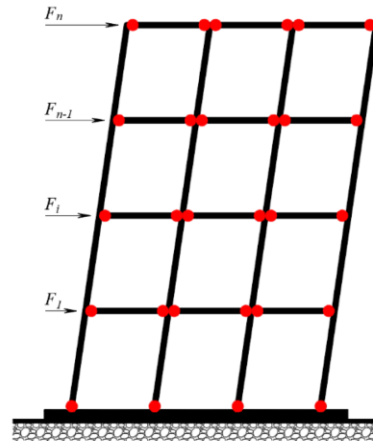
axially loaded members
dissipated in **Seismic Links**

INTRODUCTION – GENERAL CONCEPTS

Structural level: Ductility in terms of sway displacement



(a)

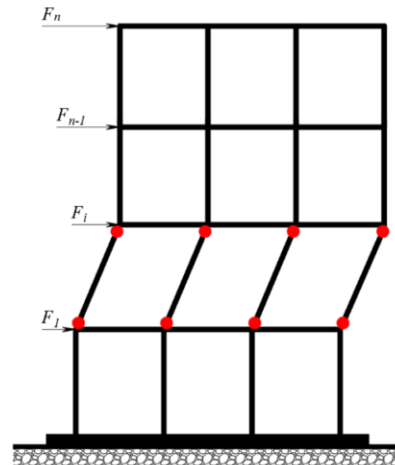
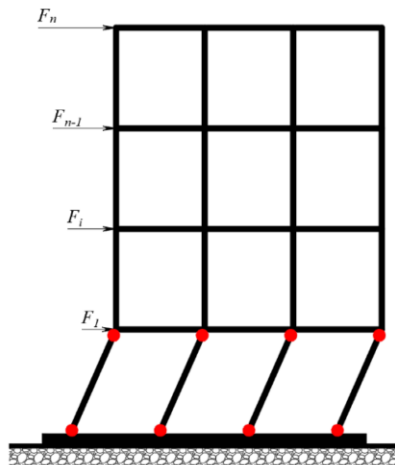


(b)

The energy dissipation and global ductility is related to the capacity of the structure to form plastic hinges in particular zones.

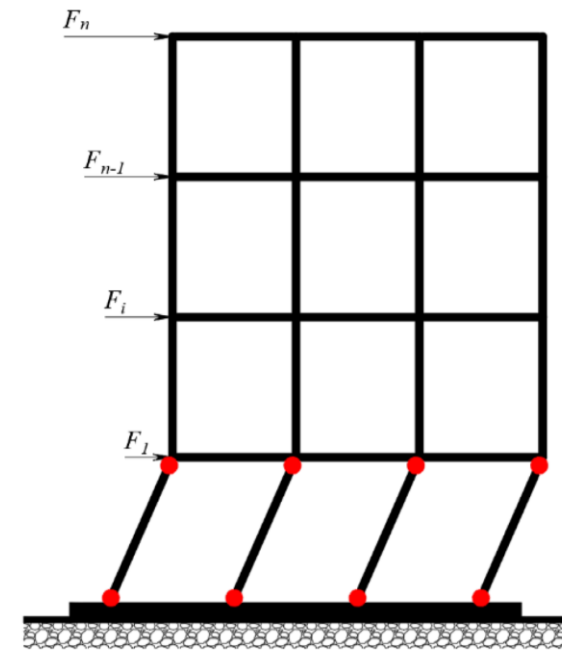
The hinges can be located in columns, beams (or even joints).

Nevertheless, it is clear that to improve global ductility the global mechanism is the most favourable, the storey mechanisms is the least favourable. To guarantee the global ductility, the local ductility has to be sufficient.



INTRODUCTION – GENERAL CONCEPTS

Structural level: Ductility in terms of sway displacement



How obtain a ductile behaviour?

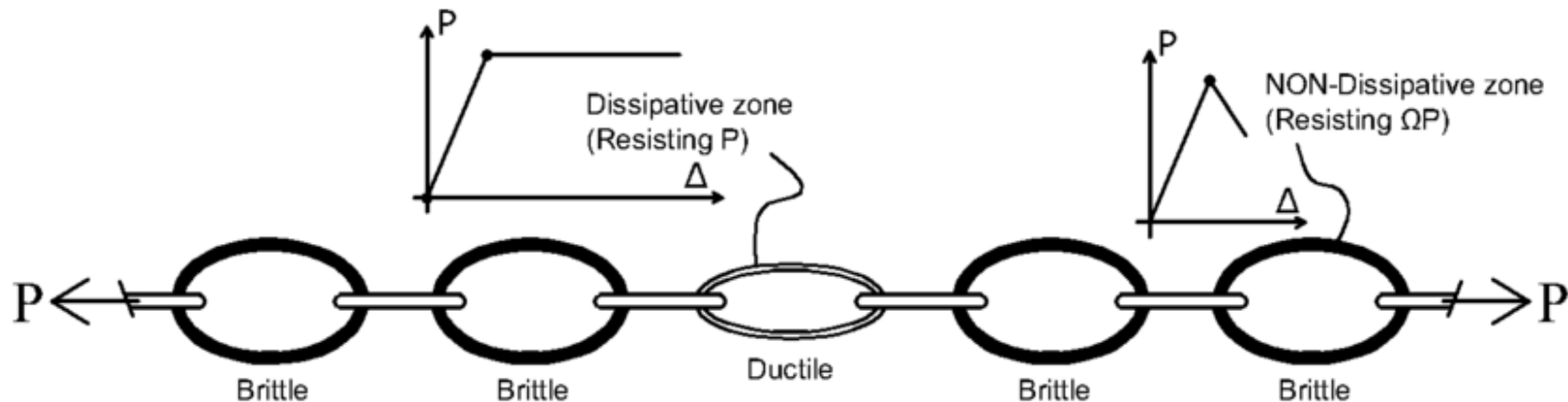
The principles of capacity design

1st principle

The dissipative zones must be designed and selected according to the design actions of the relevant seismic load combination. (In this way we assure that the dissipative zones will yield under the design seismic event $T_r=475$ years);

2nd principle

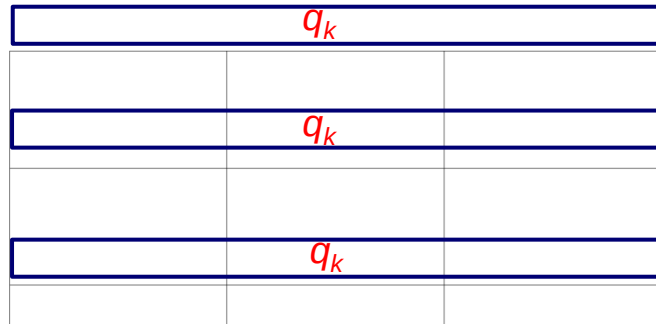
The non-dissipative zones must be designed for the maximum actions that the dissipative zones may transfer. (In this way we assure that the non-dissipative zones are in elastic range even when the full capacity of the dissipative zone is exploited);



What EC8 says?

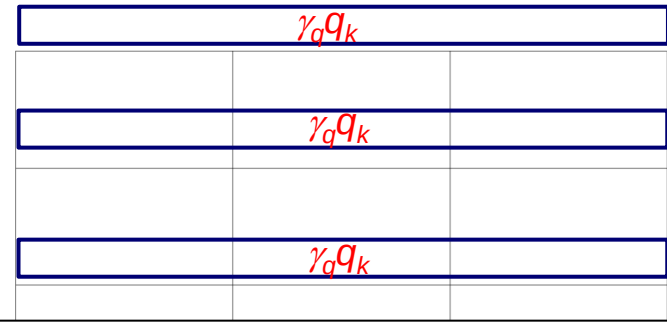
Typical design load combinations

SLS – Gravity loads



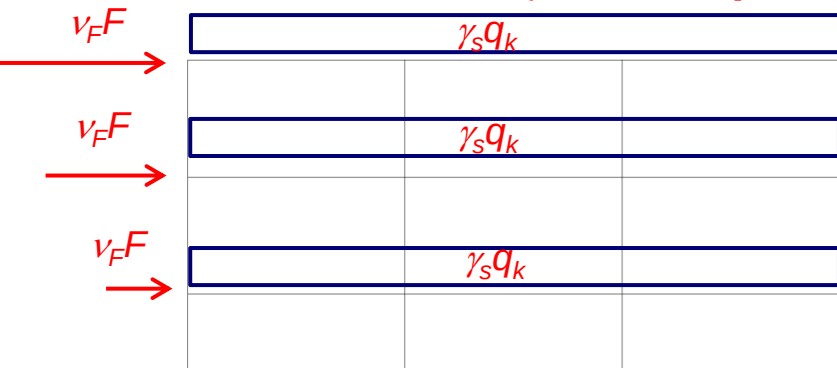
To check deflections of beam/control stresses

ULS – Gravity loads



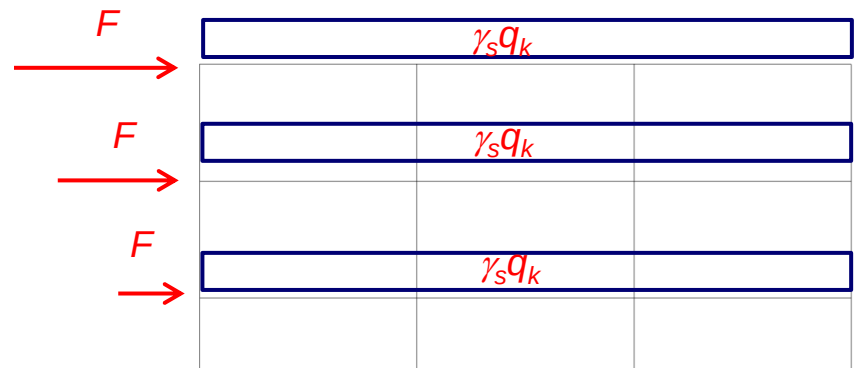
To check resistance of members (stability)

SLS – Seismic loads (low return period)



To check sway displacement (damage limitation)

ULS – Seismic loads (high return period)



To check resistance of members (stability), design ductility and hierarchy

Design steps for RC structures

Conceptual: seismic design approach

- 1) *General concept* of the structure, ensuring compliance with fundamental principles of good structural design in seismic area.
- 2) Definition of the *loads* acting on the structural elements.
- 3) Preliminary *sizing of structural sections*: 3.1. Secondary members; 3.2. Primary members.
- 4) *Structural analysis* and assessment of the overall behaviour.
- 5) Final *detailed design* of the structure and checks (stiffness, strength, *ductility*).

These early decisions are crucial, as they determine:

- The seismic performance of the structure: a *poorly conceived design* cannot be fully compensated by calculations—*flaws may be mitigated*, but not eliminated.
- The *overall cost*: a poor structural concept inevitably leads to higher construction costs.

Design steps for RC structures

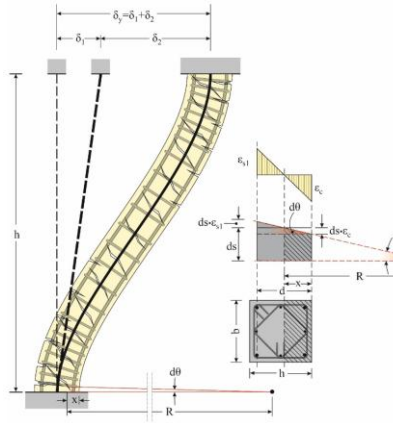
Typical design flow for RC MRFs

- 1) Define *conceptually the structural scheme* and subdivide the structural elements in primary (for which we will design the ductility more in detail) and secondary members (for which simple considerations can be made);
- 2) *Pre-design the size of the members* (beams and columns) according to the definition of primary (carry lateral forces) and secondary members (carry gravity loads) at step 1;
- 3) Perform the *structural analysis* considering all the relevant combinations;
- 4) Design the *reinforcement in the beams* taking care of proper detailing rule for the dissipative zones;
- 5) Design the *stirrups in the beams* respecting the so-called shear-bending hierarchy criterion;
- 6) Design the *column reinforcement* respecting the so-called beam-column hierarchy criterion;
- 7) Design the *stirrups in columns* as for the beams at step 5;
- 8) Design the *stirrups in the joints* to provide the necessary resistance.

Design steps for RC structures

Design rules for MRFs

Pre-design the size of the primary members promoting ductility (not mandatory rule)



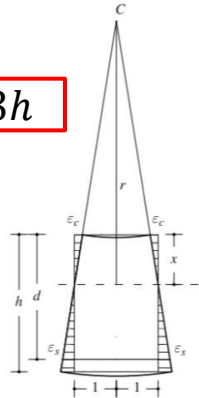
For beams

$$\left(\frac{1}{r}\right)_u = \frac{\varepsilon_{cu}}{y_c} = 0.8 \frac{\varepsilon_{cu}}{h} \frac{1}{\omega - \omega'} \quad \boxed{y_c \cong 0.15 \div 0.3h}$$

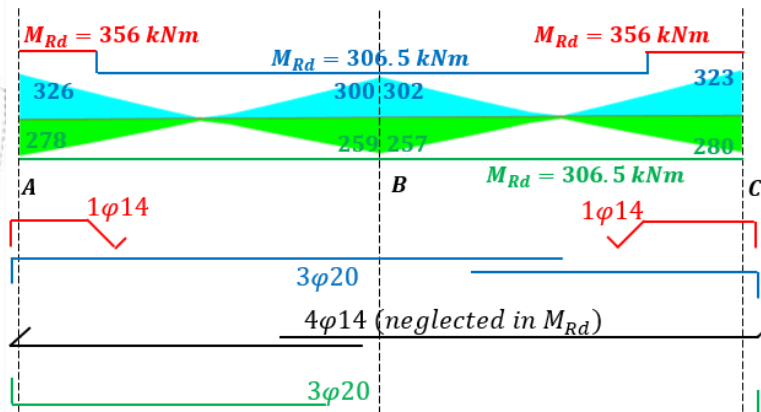
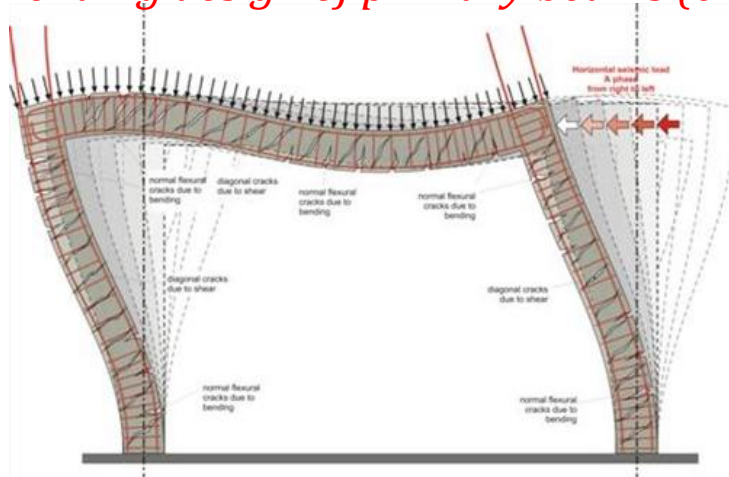
For columns

$$0.8 \frac{y_c}{h} = \frac{N}{bh f_{cd}} = \nu \quad \boxed{y_c \cong 0.2 \div 0.4h}$$

$$\nu \cong 0.15 \div 0.3$$



Bending design of primary beams (critical regions – plastic hinges)

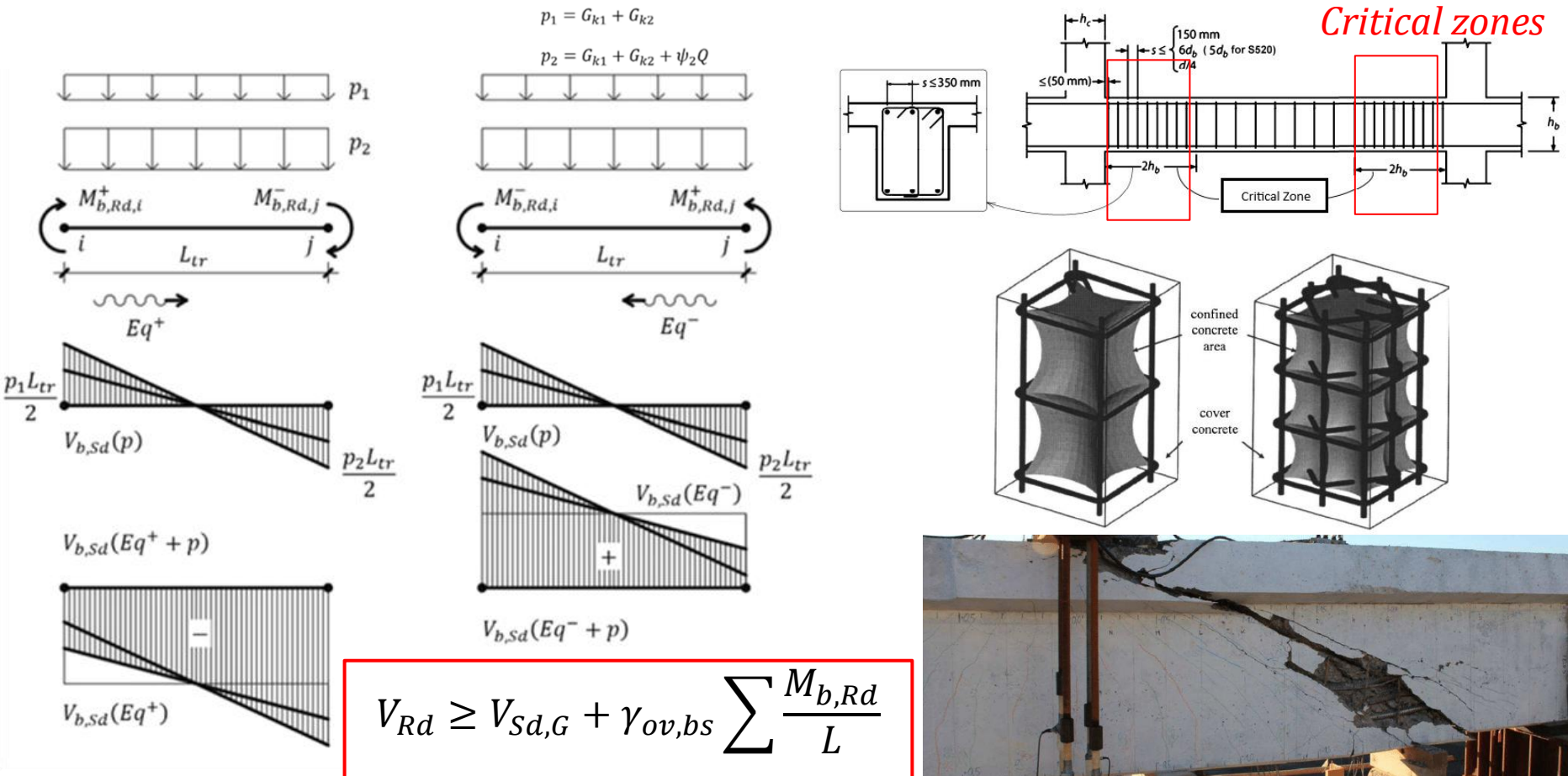


$$\boxed{M_{Rd} \geq M_{Sd}}$$

Design steps for RC structures

Design rules for MRFs

Shear design of primary beams + detailing rules (bending-shear hierarchy)



Design steps for RC structures

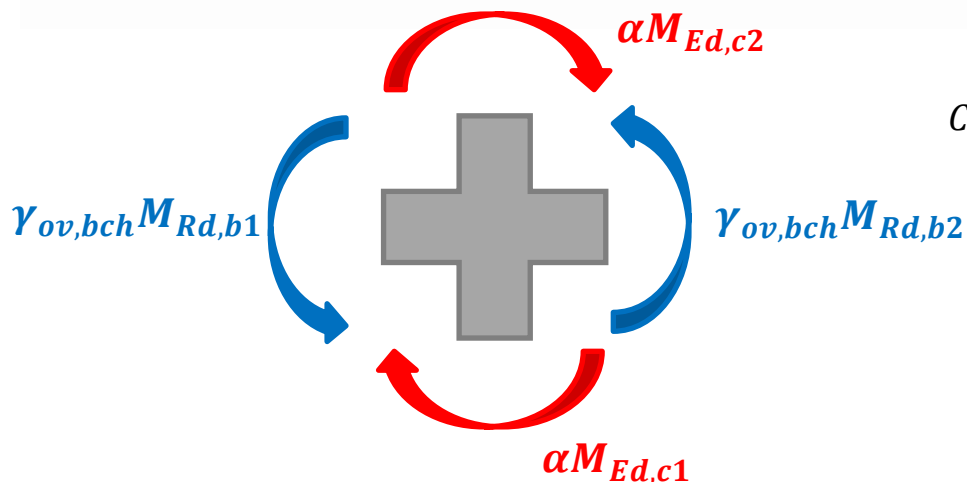
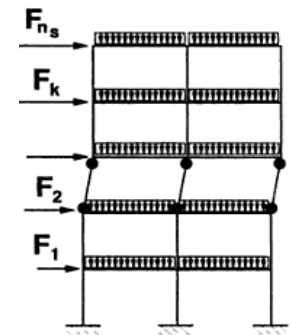
Design rules for MRFs

Overstrength primary columns (Beam-Column hierarchy)

Beam-Column members' hierarchy promotes the development of plastic hinges in beams. No soft storey

(4) Unless otherwise specified in Sections 5 to 8, to satisfy the requirement of (3)P, in frame buildings, including frame-equivalent ones as defined in 5.1.2(1), with two or more storeys, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns:

$$\sum M_{Rc} \geq 1,3 \sum M_{Rb} \quad (4.29)$$



Columns are properly oversized based starting from the evaluation of the beam resistance

$$\alpha = \frac{\gamma_{ov,bch} (M_{Rd,b1} + M_{Rd,b2})}{(M_{Ed,c1} + M_{Ed,c2})}$$

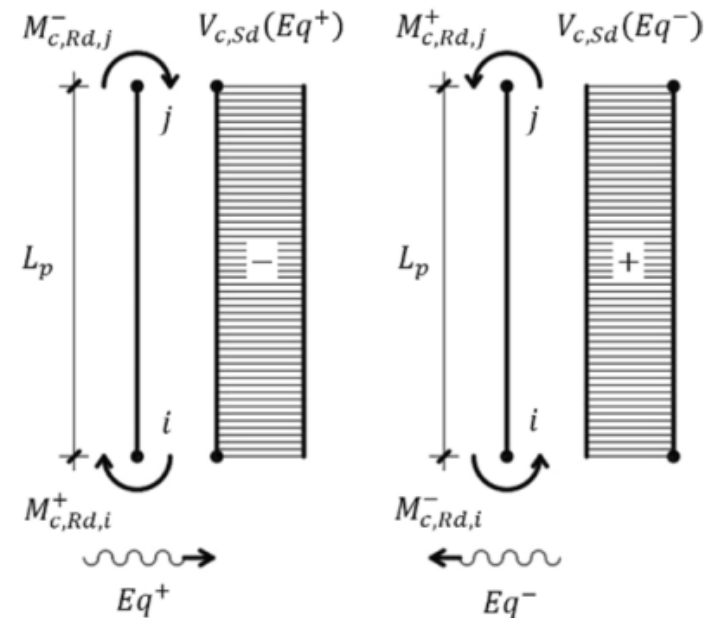
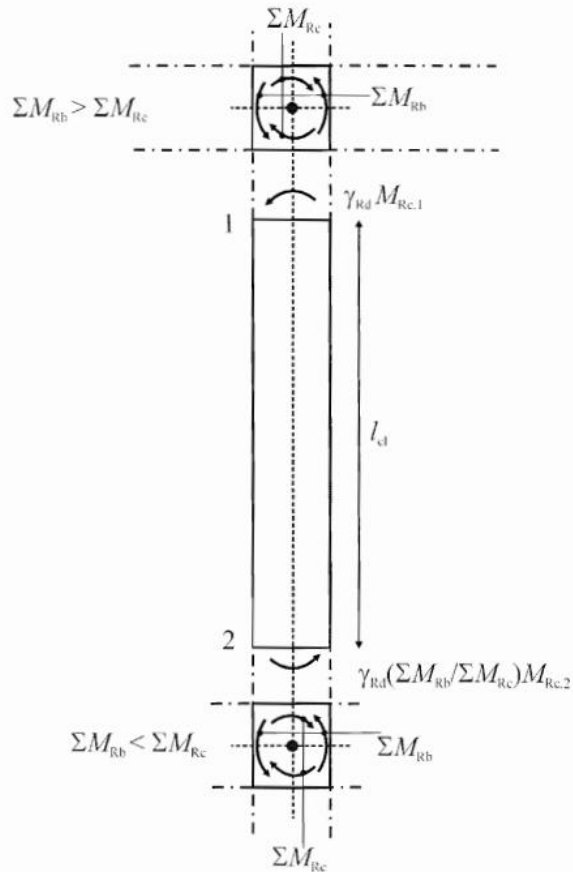
$$\gamma_{ov,bch} = 1.3$$

(6) The rules of (4) and (5) of this subclause are waived at the top level of multi-storey buildings.

Design steps for RC structures

Design rules for MRFs

Shear design of primary columns (bending-shear hierarchy)

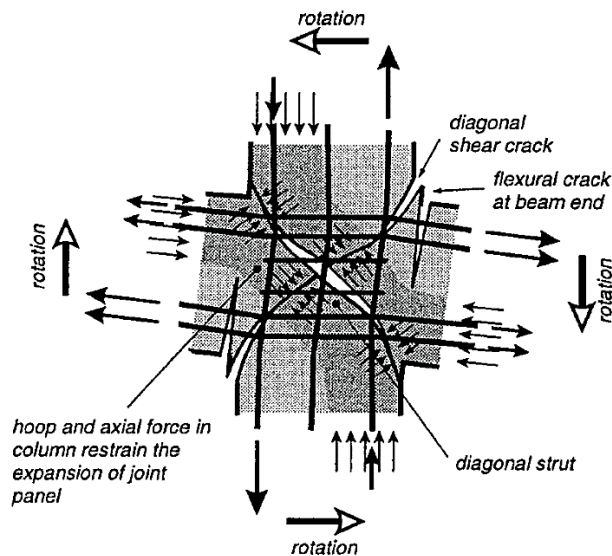


$$V_{Rd} \geq \gamma_{ov,cs} \sum \frac{M_{c,Rd}}{H}$$

Design steps for RC structures

Design rules for MRFs

Design of joints (Strut and Tie)



(3) Adequate confinement (both horizontal and vertical) of the joint should be provided, to limit the maximum diagonal tensile stress of concrete $\max \sigma_{ct}$ to f_{ctd} . In the absence of a more precise model, this requirement may be satisfied by providing horizontal hoops with a diameter of not less than 6 mm within the joint, such that:

$$\frac{A_{sh} \cdot f_{ywd}}{b_j \cdot h_{jw}} \geq \frac{\left(\frac{V_{jhd}}{b_j \cdot h_{jc}} \right)^2}{f_{ctd} + v_d \cdot f_{cd}} - f_{ctd} \quad (5.35)$$

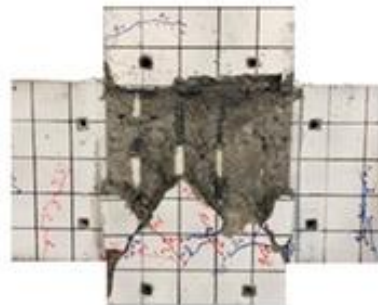
(4) As an alternative to the rule specified in (3) of this subclause, integrity of the joint after diagonal cracking may be ensured by horizontal hoop reinforcement. To this end the following total area of horizontal hoops should be provided in the joint.

a) In interior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} (1 - 0,8 v_d) \quad (5.36a)$$

b) In exterior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} A_{s2} f_{yd} (1 - 0,8 v_d) \quad (5.36b)$$



Design steps for RC structures

Design rules for MRFs

Ductility checks

Example related to base primary seismic columns

$$\alpha \omega_{wd} \geq 30 \mu_{\phi} v_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0,035 \quad (5.15)$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions

$$\left[\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} \right];$$

μ_{ϕ} is the required value of the curvature ductility factor;

v_d is the normalised design axial force ($v_d = N_{Ed}/A_c f_{cd}$);

$\varepsilon_{sy,d}$ is the design value of tension steel strain at yield;

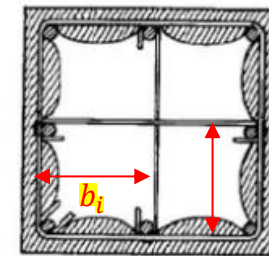
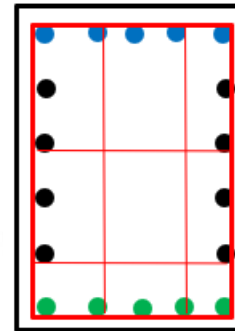
h_c is the gross cross-sectional depth (parallel to the horizontal direction in which the value of μ_{ϕ} used in (6)P of this subclause applies);

h_o is the depth of confined core (to the centreline of the hoops);

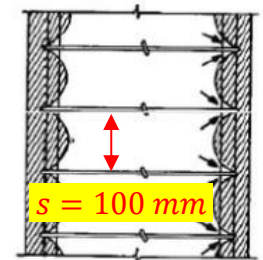
b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centreline of the hoops);

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:



(b) Rectangular hoops with cross ties



(d) Confinement by transverse bars

$$\alpha \omega_{wd} = \alpha_n \alpha_s \omega_{wd} \geq 30 \mu_{\phi} v_d \varepsilon_{sy,d} \frac{b}{b_{stirrup}} - 0.035$$

$$\omega_{wd} = \frac{\pi \frac{\phi_{stirrups}^2}{4} n_{stirrups} (b_{stirrup} + h_{stirrup}) f_{yd}}{b_{stirrup} \cdot h_{stirrup} \cdot s} \frac{f_{yd}}{f_{cd}}$$

Design steps for RC structures

Design rules for MRFs

Secondary members drift control

5.7 Design and detailing of secondary seismic elements

(1)P Clause 5.7 applies to elements designated as secondary seismic elements, which are subjected to significant deformations in the seismic design situation (e.g. slab ribs are not subject to the requirements of 5.7). Such elements shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation.

(2)P Maximum deformations due to the seismic design situation shall be calculated in accordance with 4.3.4 and shall account for P-Δ effects in accordance with 4.4.2.2(2) and (3). They shall be calculated from an analysis of the structure in the seismic design situation, in which the contribution of secondary seismic elements to lateral stiffness is neglected and primary seismic elements are modelled with their cracked flexural and shear stiffness.

(3) Secondary seismic elements are deemed to satisfy the requirements of (1)P of this subclause if bending moments and shear forces calculated for them on the basis of:
a) the deformations of (2)P of this subclause; and b) their cracked flexural and shear stiffness, do not exceed their design flexural and shear resistance M_{Rd} and V_{Rd} , respectively, as these are determined on the basis of EN 1992-1-1:2004.

Design steps for RC structures

Damage Limitation State – Stiffness

4.4.3 Damage limitation

4.4.3.1 General

(1) The “damage limitation requirement” is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the “no-collapse requirement” in accordance with 2.1(1)P and 3.2.1(3), the interstorey drifts are limited in accordance with 4.4.3.2.

(2) Additional damage limitation verifications might be required in the case of buildings important for civil protection or containing sensitive equipment.

4.4.3.2 Limitation of interstorey drift

(1) Unless otherwise specified in Sections 5 to 9, the following limits shall be observed:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_{Iv} \leq 0,005 h ; \quad (4.31)$$

b) for buildings having ductile non-structural elements:

$$d_{Iv} \leq 0,0075 h ; \quad (4.32)$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_{Iv} \leq 0,010 h \quad (4.33)$$

Depending on the national code, the assumed return period at DLS is 50-100 years.

To simplify EC8 requires to check directly that the displacements calculated at ULS (and therefore amplified of q) reduced by a coefficient $n=0.4-0.5$ are lower than specified limits.

CASE STUDY STRUCTURE



Laboratory STREngTH

1.200.000 m²

CASE STUDY STRUCTURE



Laboratory of Life

Research Institute

Base floor Open

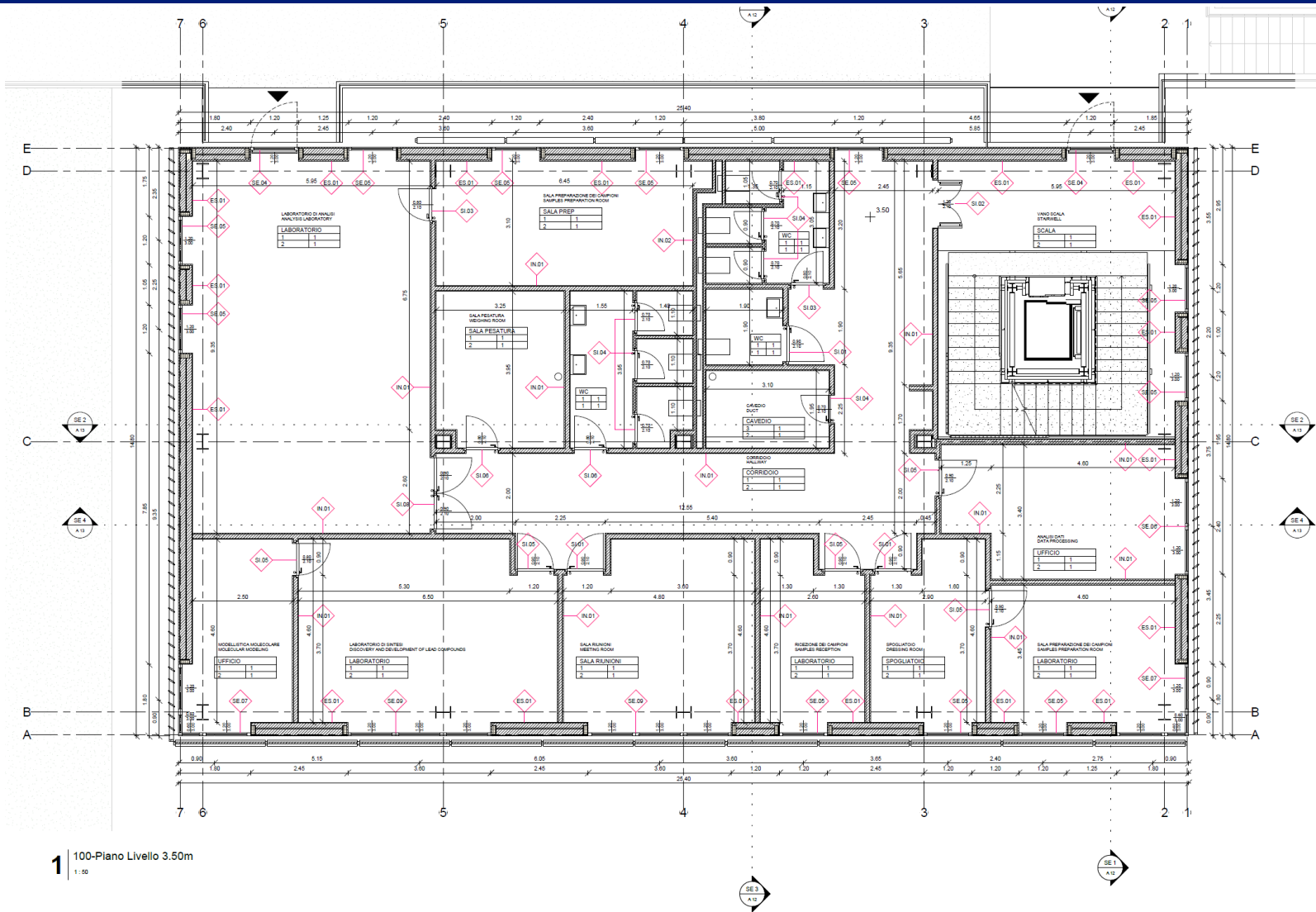
1 floor lab

1 floor office



Urbanistic data: covered area 375 sqm; volume about 3500 mc

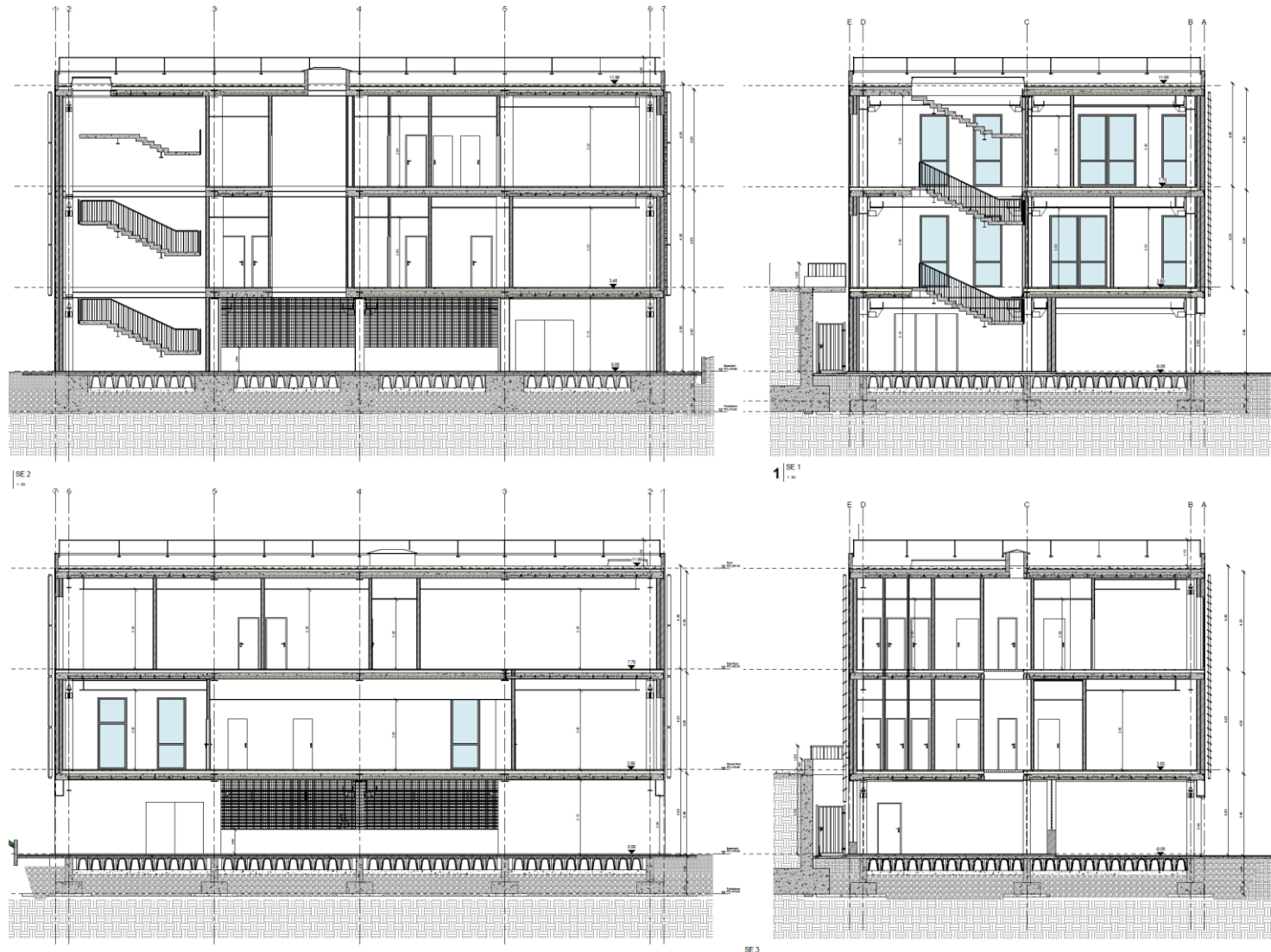
CASE STUDY STRUCTURE



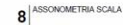
CASE STUDY STRUCTURE



CASE STUDY STRUCTURE



Design of a reinforced concrete structure

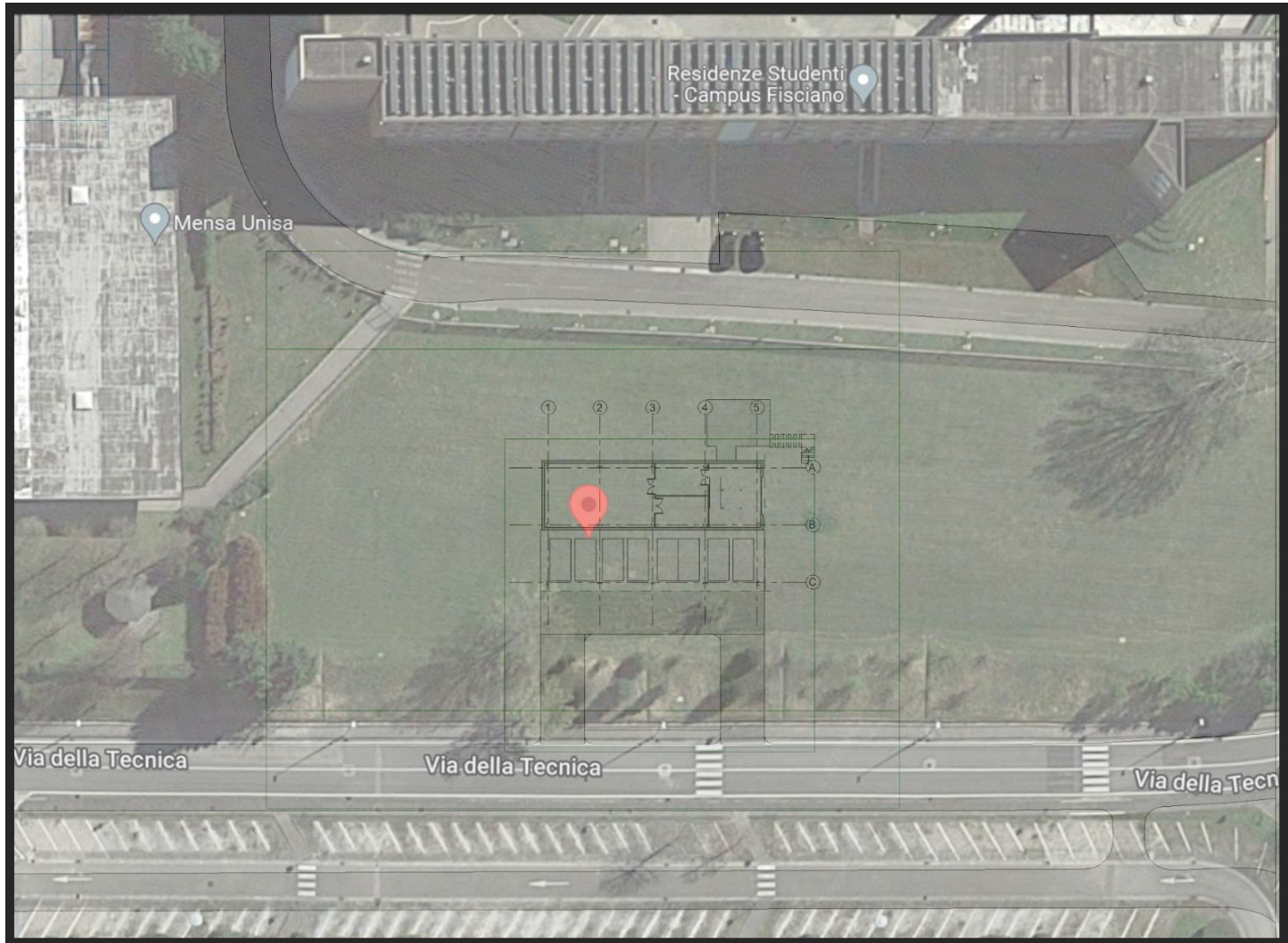


UNIVERSITÀ
DEGLI STUDI
DI SALERNO

Massimo Latour
University of Salerno

CASE STUDY STRUCTURE

Hole S1



CASE STUDY STRUCTURE

1-5 m

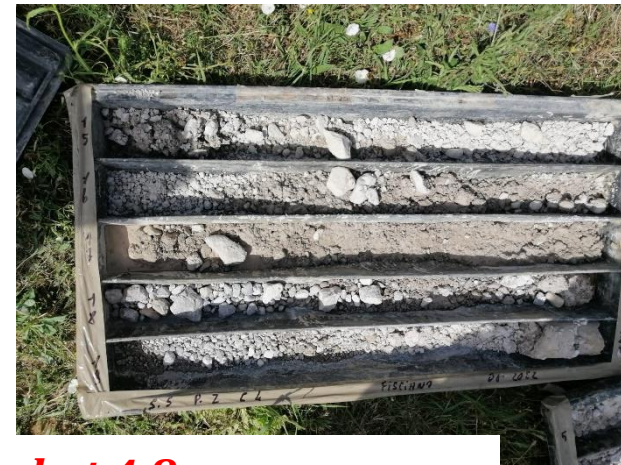
5-10 m

Fill dirt

Sand-gravel

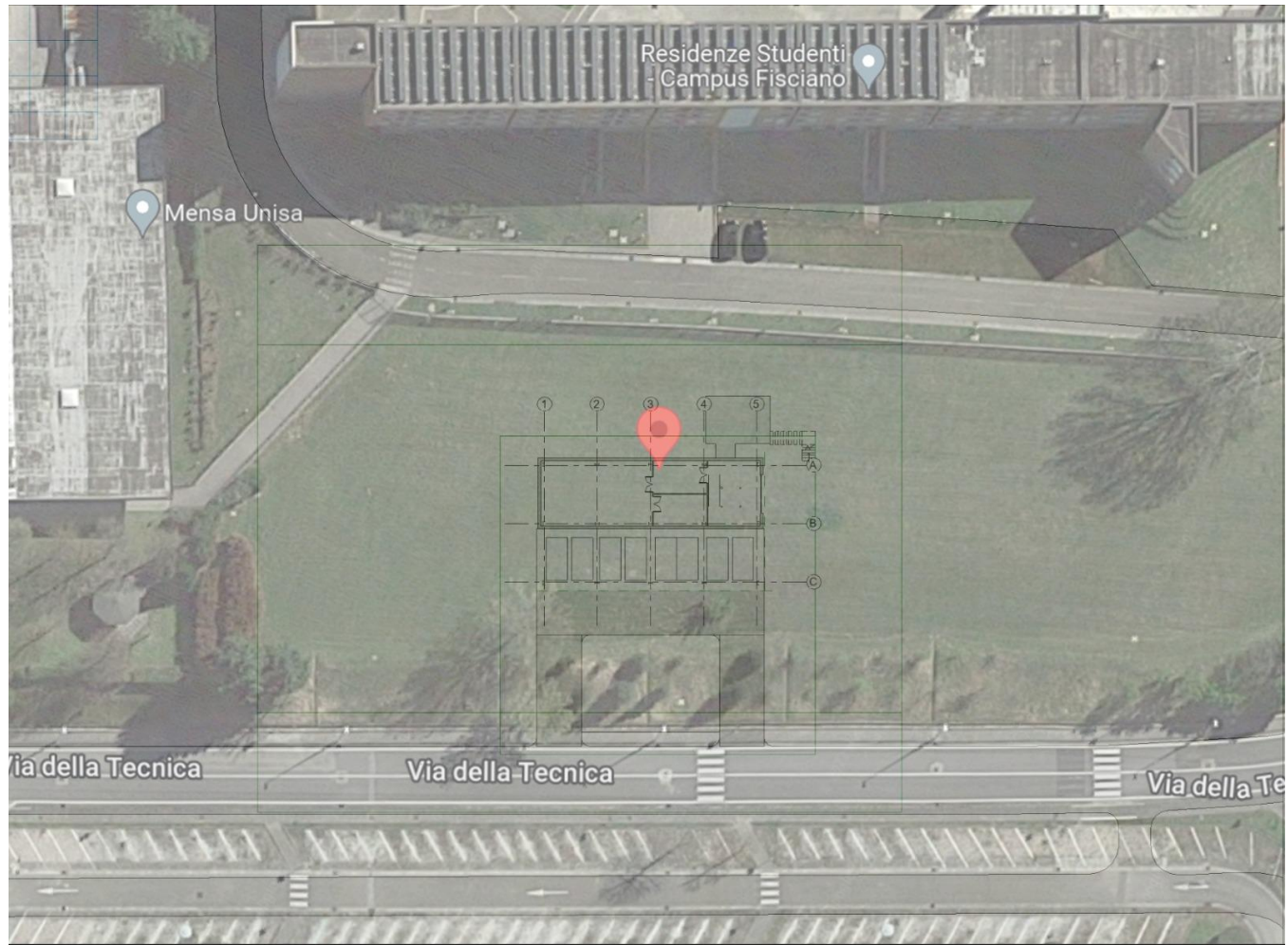
10-15 m

Sand & Gravel at 4.8 m



CASE STUDY STRUCTURE

Hole S2



CASE STUDY STRUCTURE

1-5 m

5-10 m

Sand & Gravel at 5.5 m

Fill dirt

Sand-gravel



CASE STUDY STRUCTURE

MASW

MASW stands for **M**ultichannel **A**nalysis of **S**urface **W**aves. It's a geophysical method used for soil and subsurface classification, particularly to evaluate shear wave velocity (V_s) profiles.

What it does

MASW helps determine the stiffness (shear wave velocity) of the ground layers, which is critical for earthquake engineering, site classification, foundation design, ground response analysis.

Why it is used in soil classification

The shear wave velocity (V_s) is a key parameter for classifying soil in seismic design codes like Eurocode 8 and others. Soils are classified into categories (e.g., A–E or I–III) based on V_{s30} (average shear wave velocity in the top 30 meters).

MASW is non-invasive, cost-effective, and can be done relatively quickly in the field.



CASE STUDY STRUCTURE

How it works

Source Generation:

a seismic source (e.g., hammer impact) generates surface waves (Rayleigh waves).

Wave Propagation:

surface waves travel through the soil; their velocity depends on material stiffness.

Data Acquisition:

a linear array of geophones records wave motion at regular spacing.

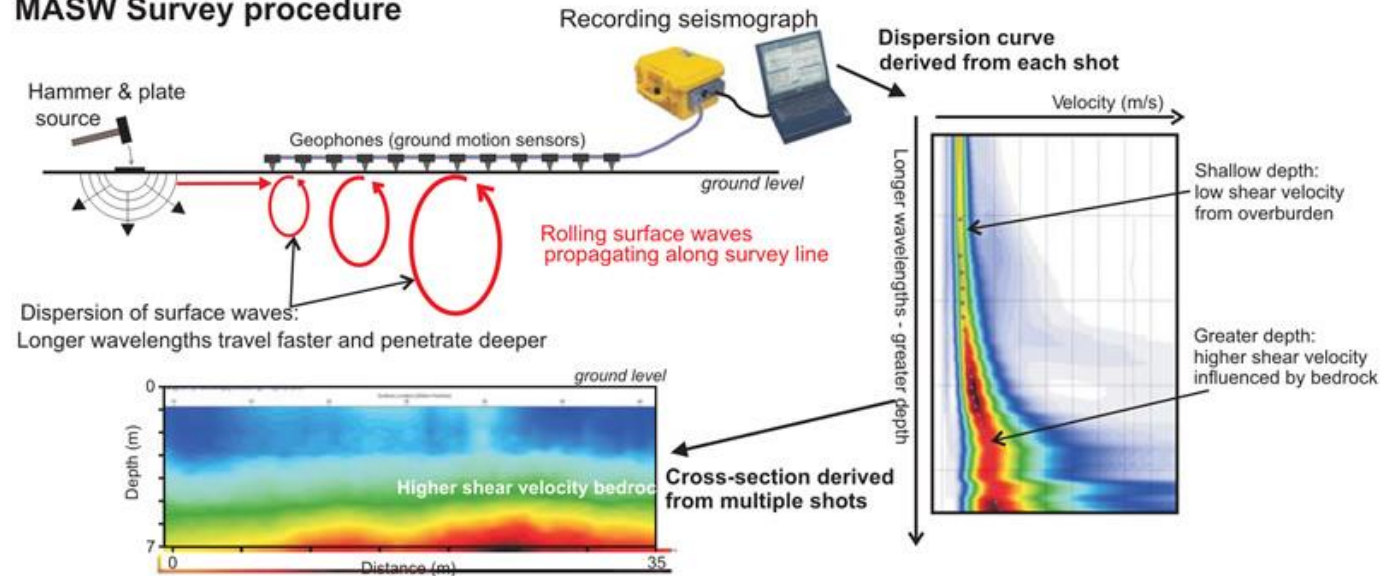
Signal Processing:

frequency–phase velocity data is extracted using Fourier analysis.

Inversion & Modeling:

surface wave dispersion curves are inverted to obtain the shear wave velocity (V_s) profile vs. depth.

MASW Survey procedure



Aschematic illustration of the MASW data acquisition and processing procedure leading to a final section.□

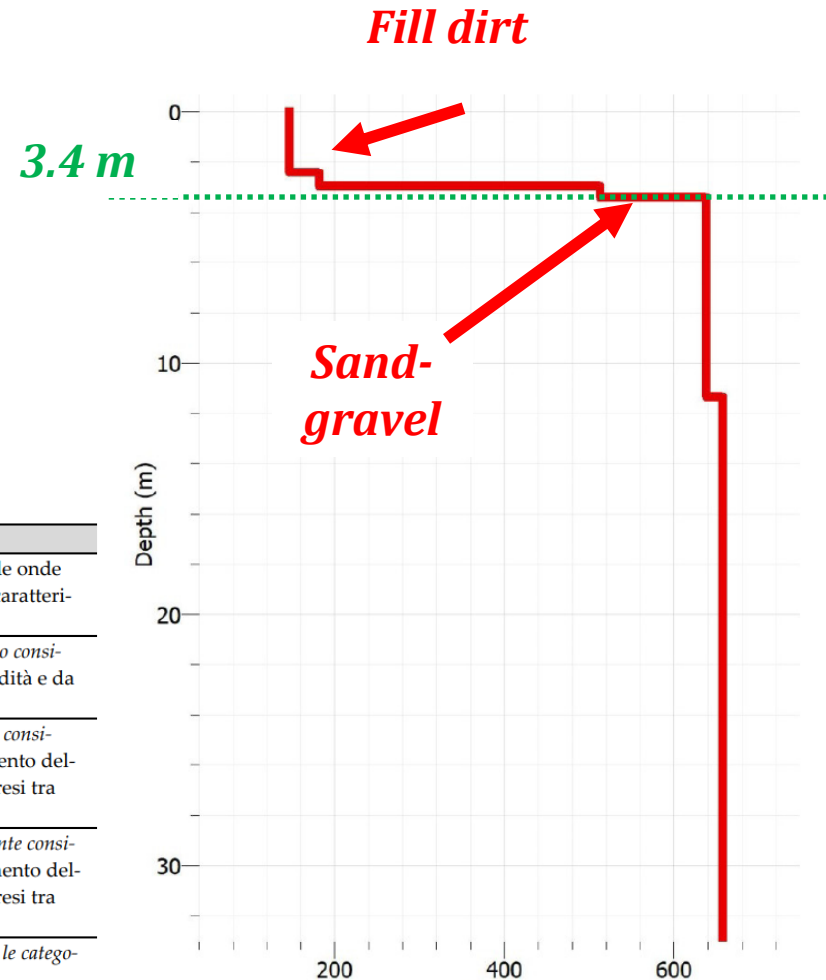


CASE STUDY STRUCTURE

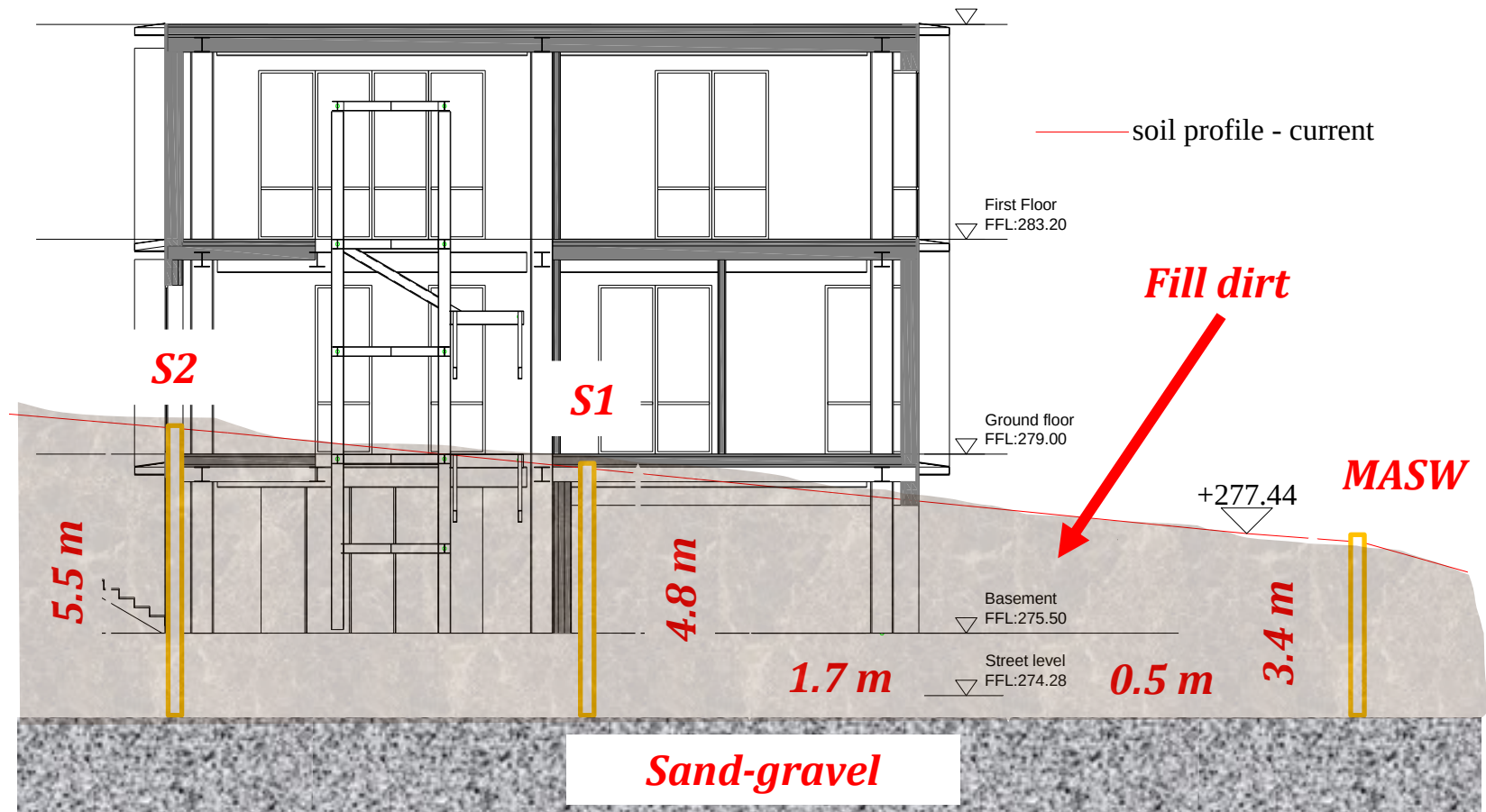
Strato	h(m)	z (m)	V _{Sfin}
1	2,4	-2,4	143
2	0,6	-3,0	181
3	0,4	-3,4	514
4	7,9	-11,3	629
5	18,7	-30,0	654

Tab. 3.2.II – Categorie di sottosuolo che permettono l'utilizzo dell'approccio semplificato.

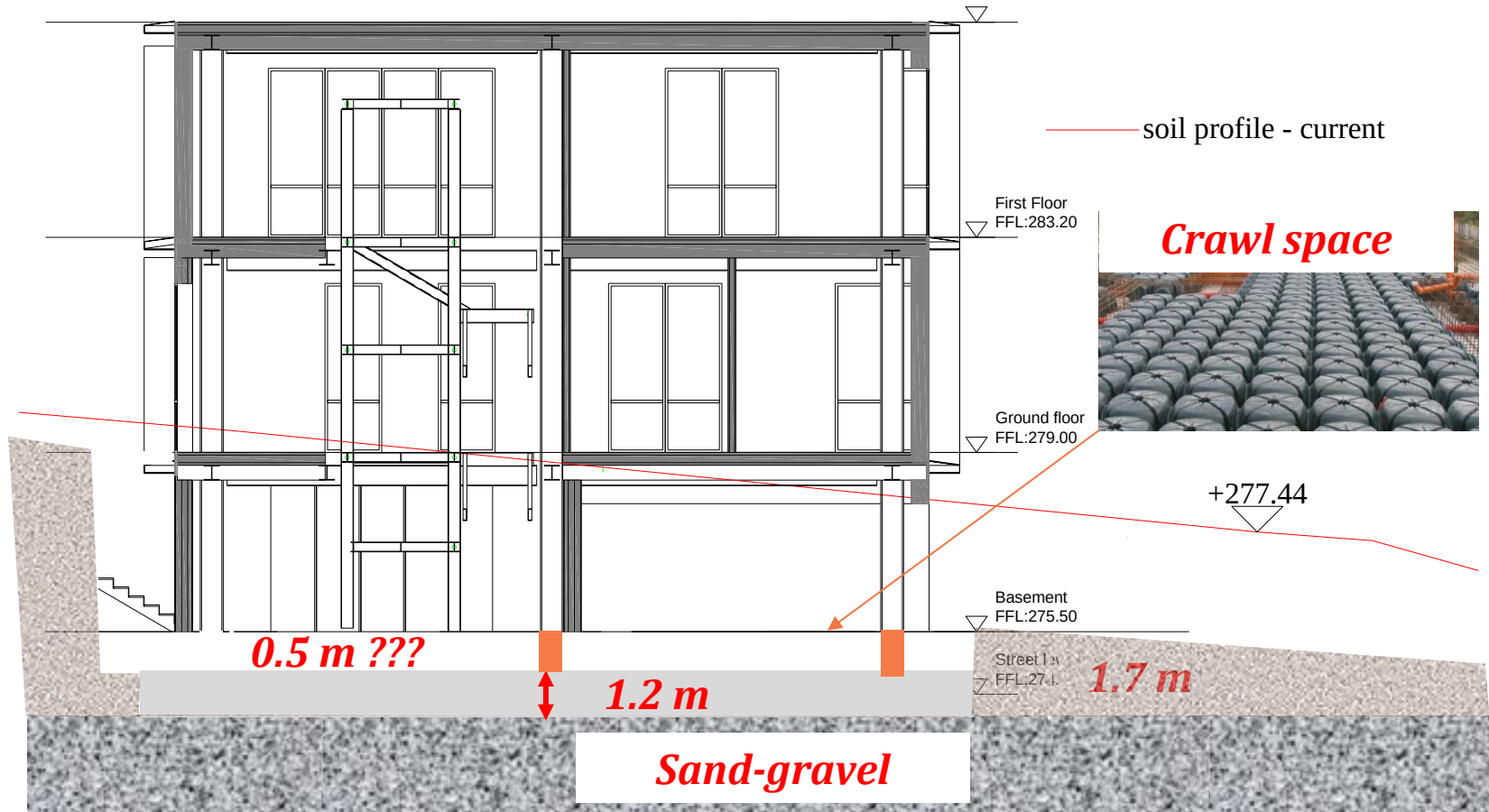
Categoria	Caratteristiche della superficie topografica
A	Ammassi rocciosi affioranti o terreni molto rigidi caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 m/s e 800 m/s.
C	Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 180 m/s e 360 m/s.
D	Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti, con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D, con profondità del substrato non superiore a 30 m.



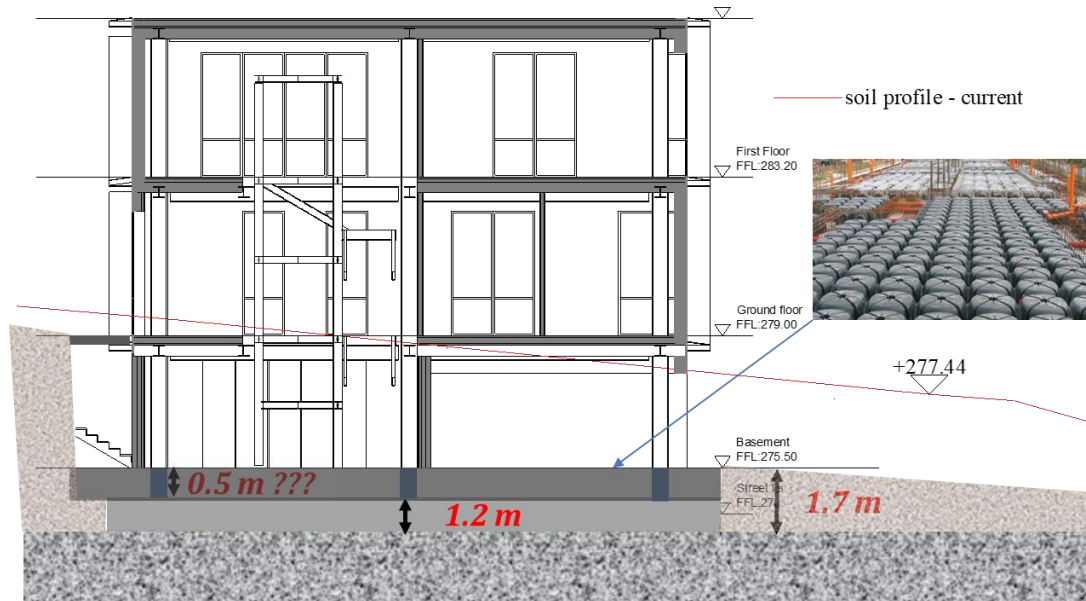
CASE STUDY STRUCTURE



CASE STUDY STRUCTURE



CASE STUDY STRUCTURE



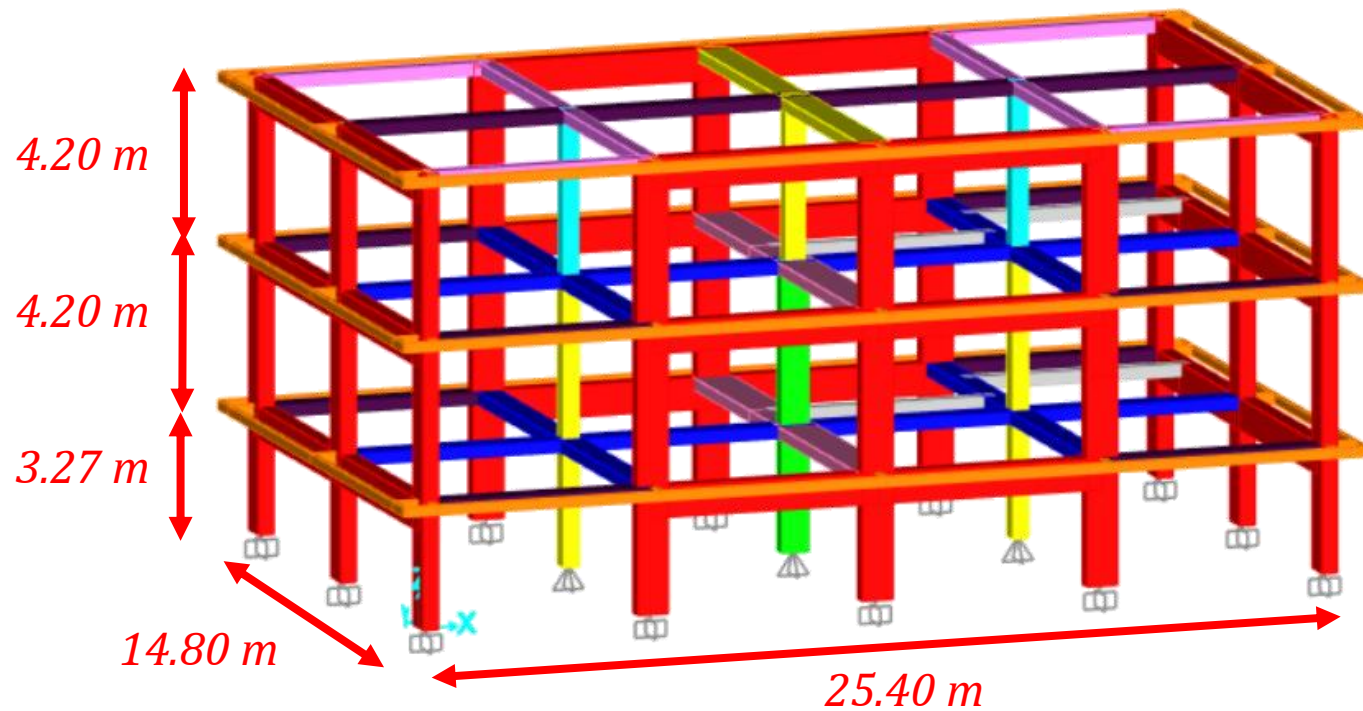
Weight of the soil removed

$$1000 \text{ kg/mc} \times 380 \text{ smq} \times 5 \text{ m} = 1900 \text{ tons}$$

***Weight of the building SLS
(Characteristic combo)***

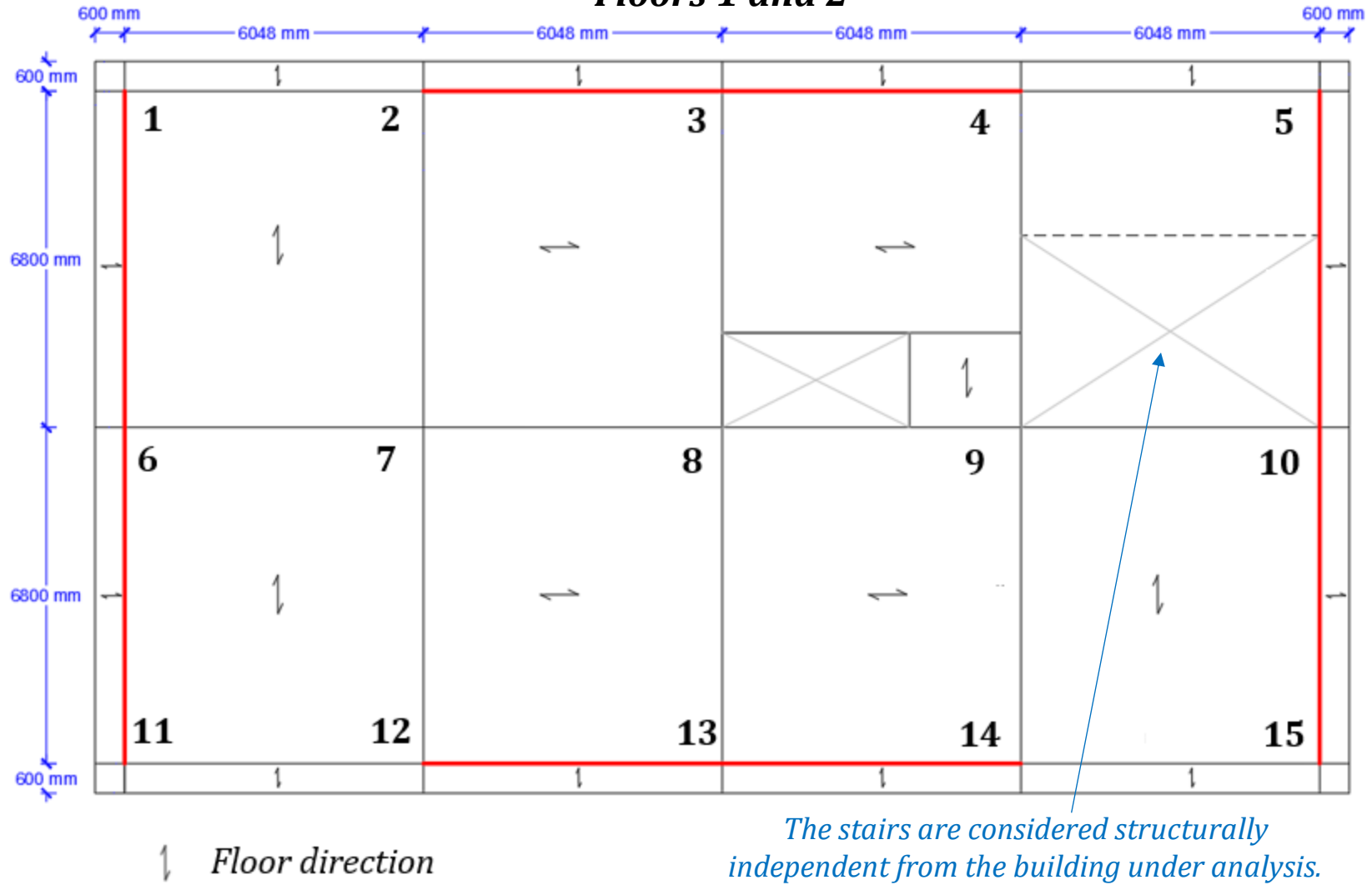
1700 tons

GEOMETRY



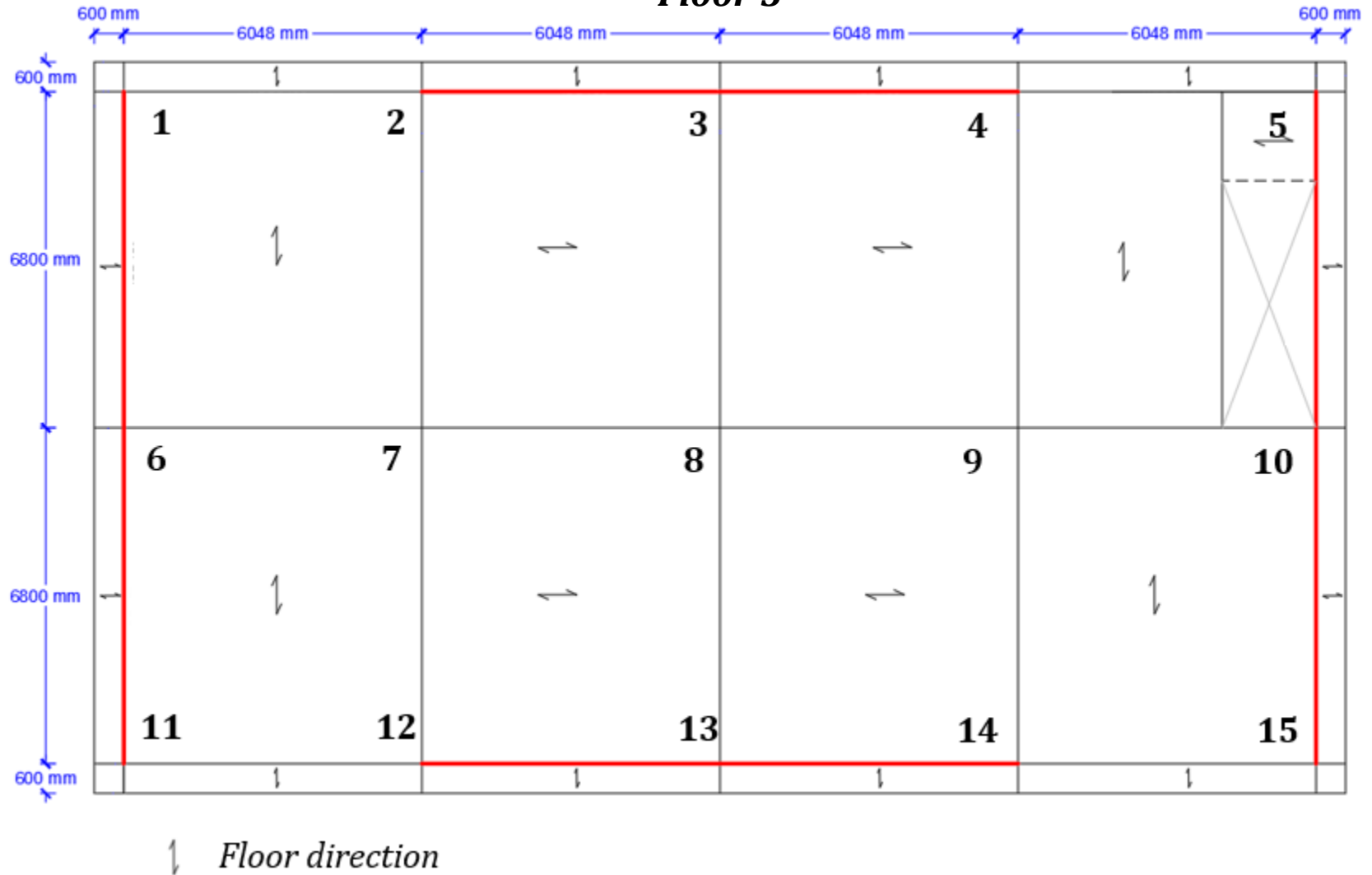
PLAN VIEW

Floors 1 and 2



PLAN VIEW

Floor 3



MATERIALS

Concrete

Class: C35/45

$R_{ck} = 45 \text{ MPa}$

$f_{ck} = 35 \text{ MPa}$

$f_{cd} = 19.83 \text{ MPa}$

$E_{cm} = 34000 \text{ MPa}$

$E_{cm} = 17000 \text{ MPa}$ (in cracked condition)

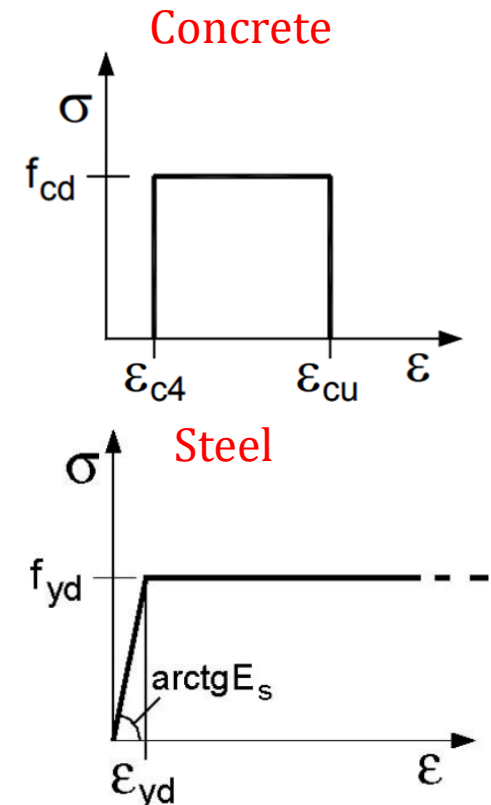
Steel for the reinforced concrete

Type: B500

$f_{yk} = 500 \text{ MPa}$

$f_{yd} = 435 \text{ MPa}$

$E_y = 210000 \text{ MPa}$



The stress-strain law of concrete is modelled according to a stress-block.

The stress-strain laws of steel are assumed elastic perfectly plastic.

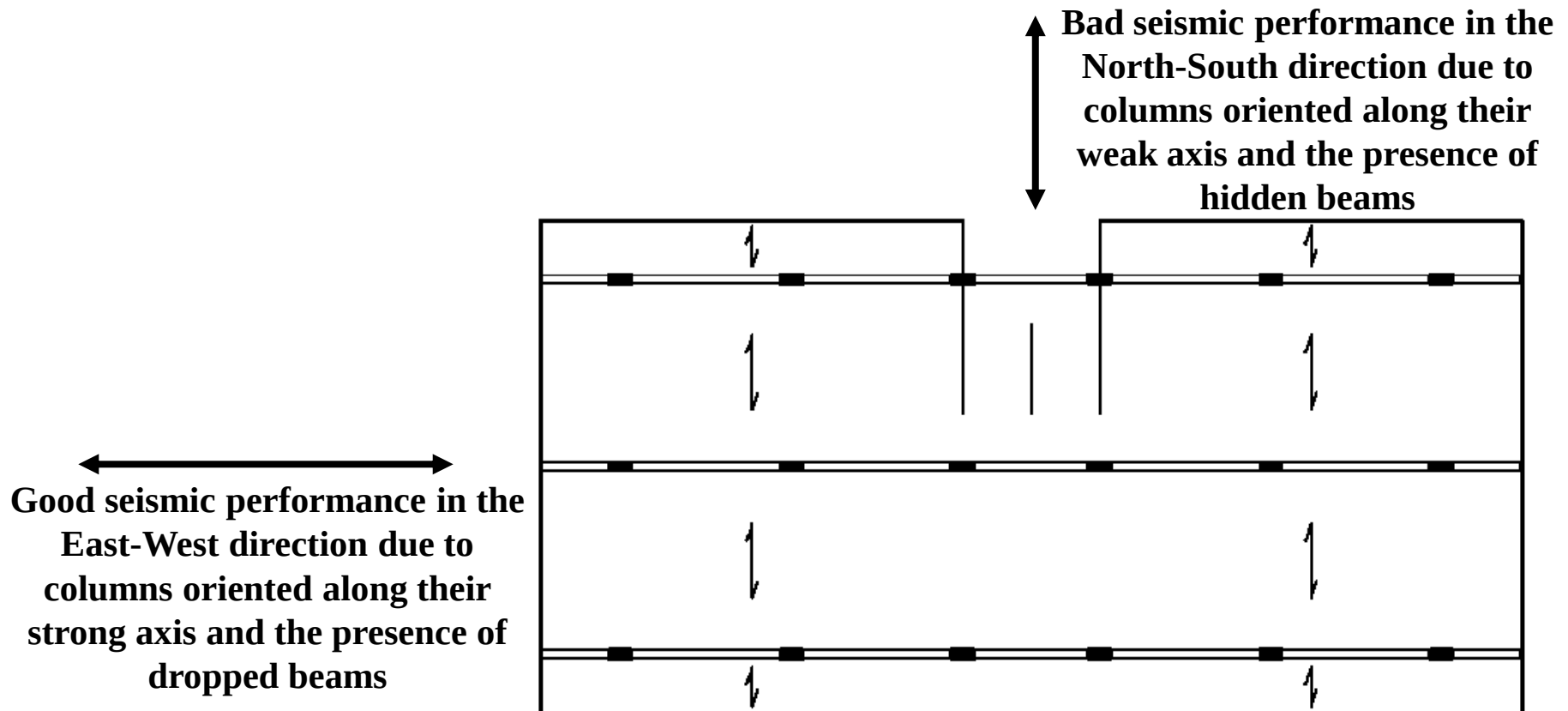
DEFINITION OF THE FRAMES LAYOUT

Bi-directional resistance and stiffness

Seismic motion always has two horizontal components.

The building structure must be capable of resisting horizontal forces acting in any direction.

Significant differences in stiffness can make typically negligible aspects become highly relevant.



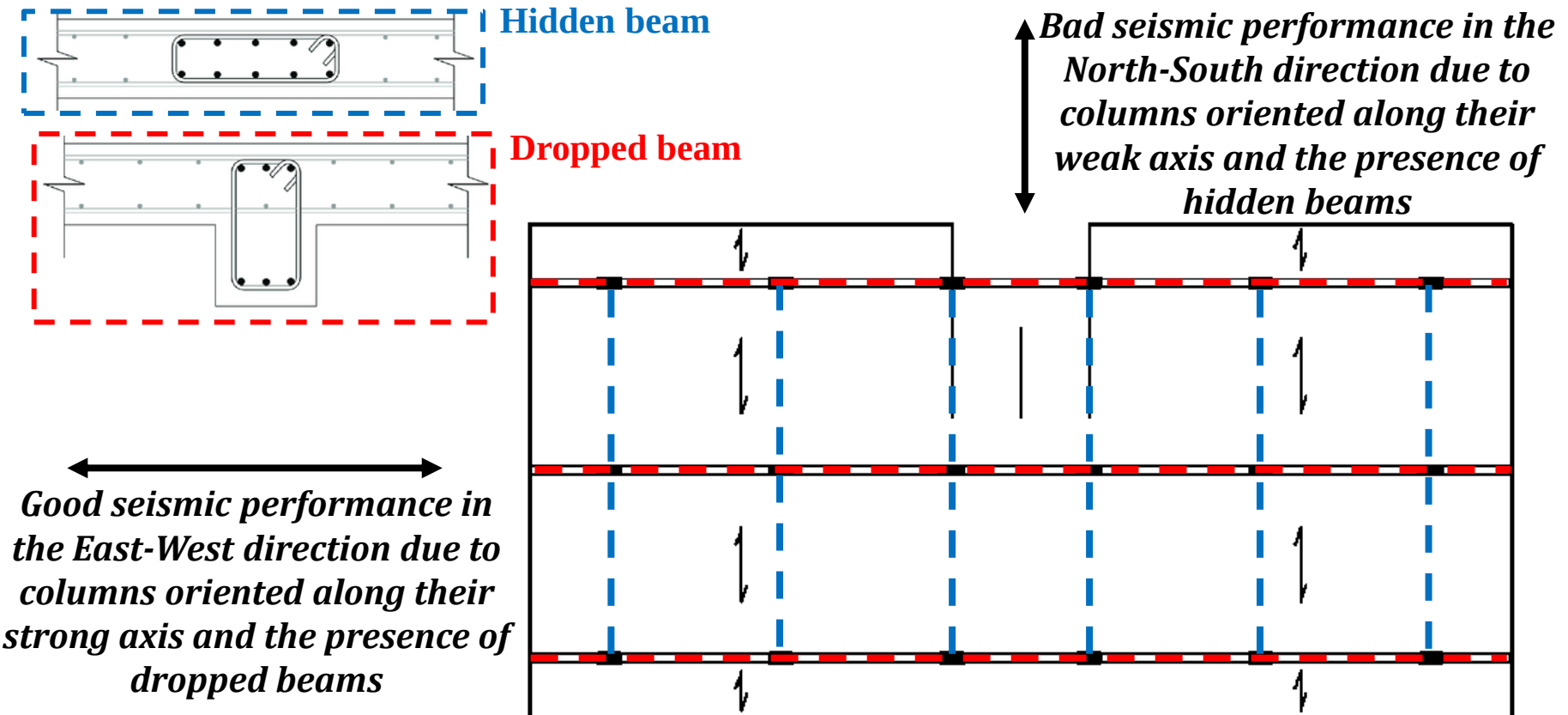
DEFINITION OF THE FRAMES LAYOUT

Bi-directional resistance and stiffness

Seismic motion always has two horizontal components.

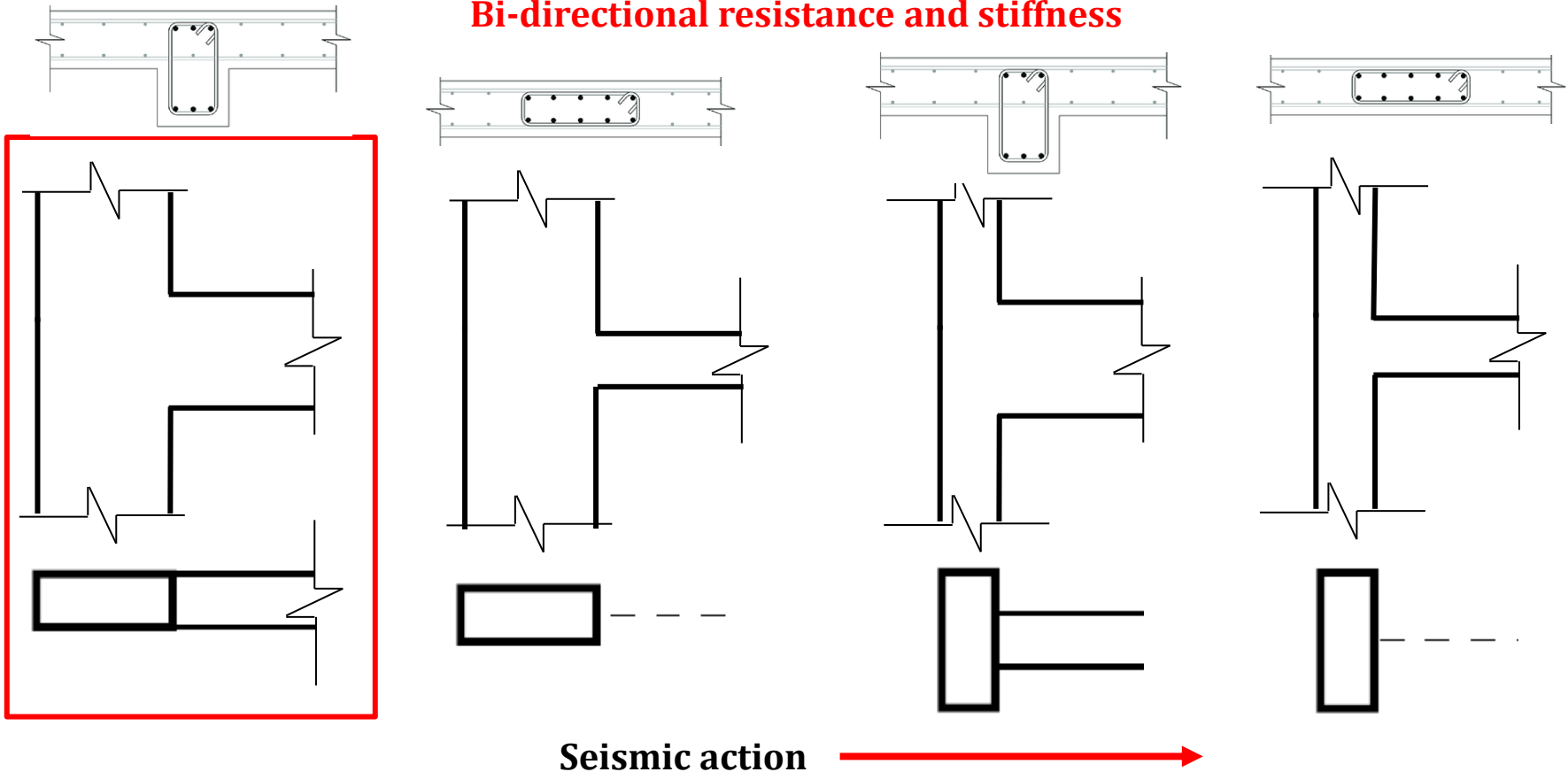
The building structure must be capable of resisting horizontal forces acting in any direction.

Significant differences in stiffness can make typically negligible aspects become highly relevant.



DEFINITION OF THE FRAMES LAYOUT

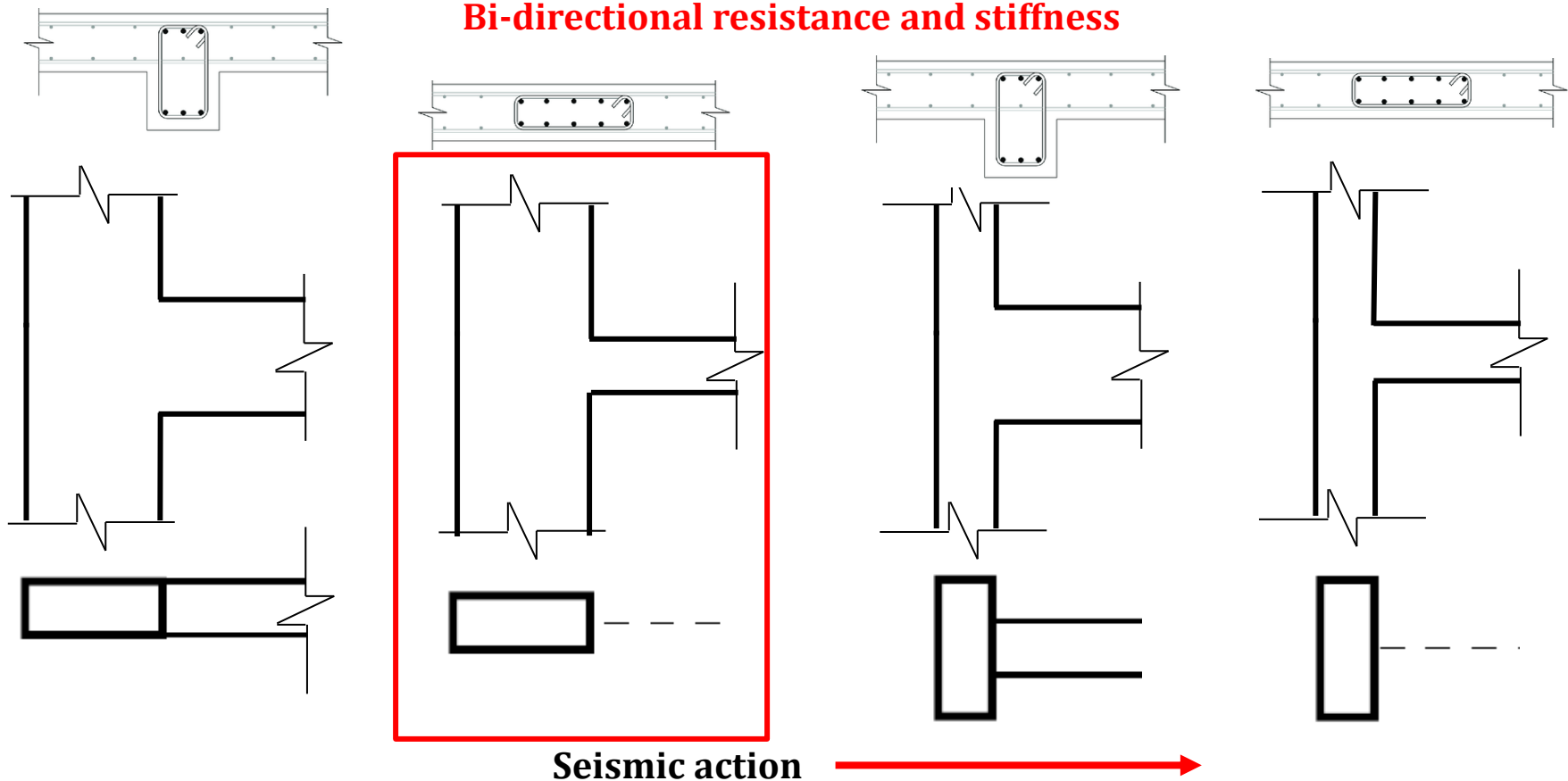
Bi-directional resistance and stiffness



The column is oriented to provide high stiffness and resistance in the direction of seismic action, complemented by the presence of a dropped beam. This combination effectively ensures strong stiffness and resistance in the seismic direction at all the storeys.

DEFINITION OF THE FRAMES LAYOUT

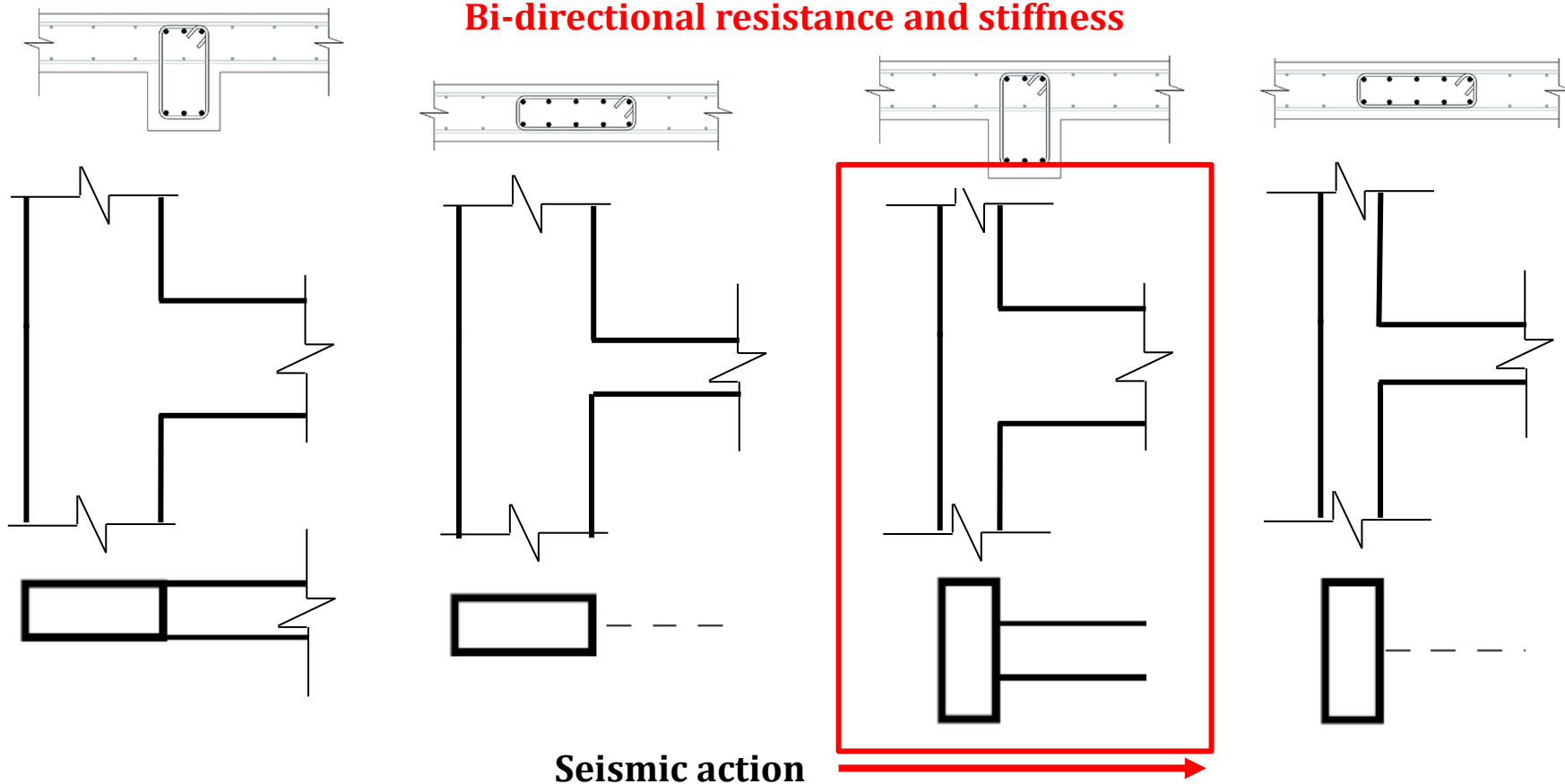
Bi-directional resistance and stiffness



The column is oriented to provide high stiffness and resistance in the direction of seismic action, complemented by the presence of a hidden beam. This combination is able to ensure strong stiffness and resistance only at the first storey.

DEFINITION OF THE FRAMES LAYOUT

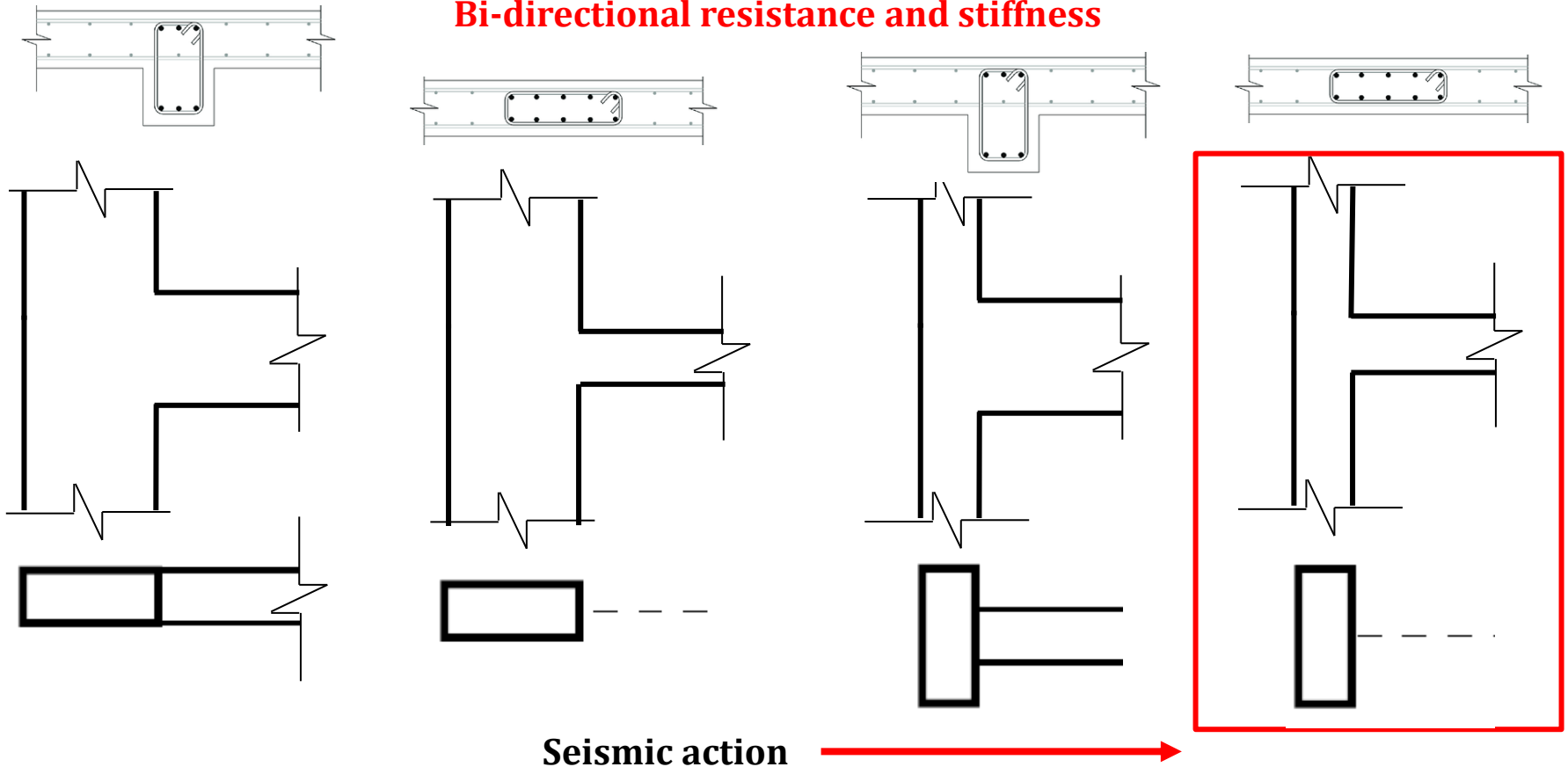
Bi-directional resistance and stiffness



The column is oriented to provide low stiffness and resistance in the direction of seismic action, complemented by the presence of a dropped beam. This combination provides limited stiffness and resistance across all storeys.

DEFINITION OF THE FRAMES LAYOUT

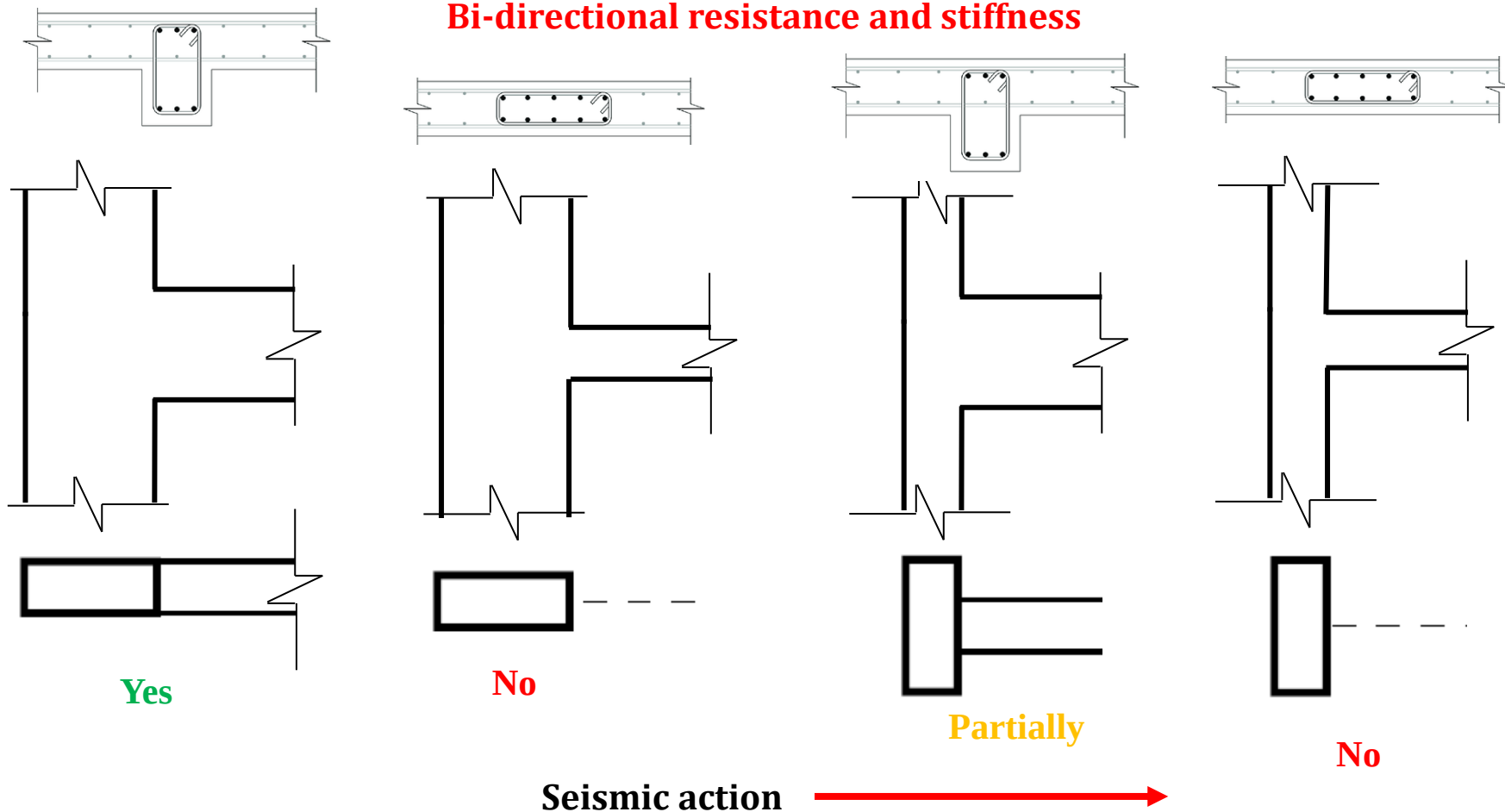
Bi-directional resistance and stiffness



The column is oriented to provide low stiffness and resistance in the direction of seismic action, complemented by the presence of a hidden beam. This combination does not guarantee stiffness and resistance across all storeys.

DEFINITION OF THE FRAMES LAYOUT

Bi-directional resistance and stiffness



During the design phase, can the frames incorporating these elements be considered as seismic-resistant frames?

DEFINITION OF THE FRAMES LAYOUT

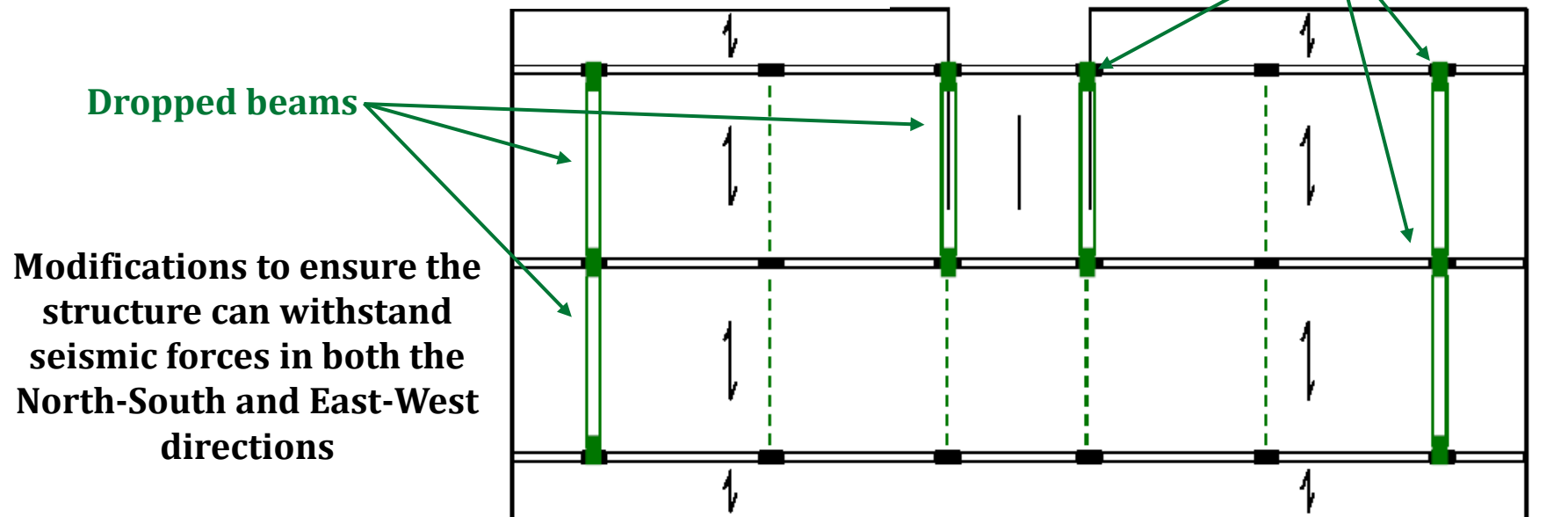
Bi-directional resistance and stiffness

4.2.1.3 Bi-directional resistance and stiffness

(1)P Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction.

(2) To satisfy (1)P, the structural elements should be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.

(3) The choice of the stiffness characteristics of the structure, while attempting to minimise the effects of the seismic action (taking into account its specific features at the site) should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.



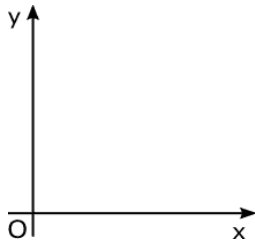
1) General concept of the structure

The depth of the deck is assumed equal to **30 cm**.

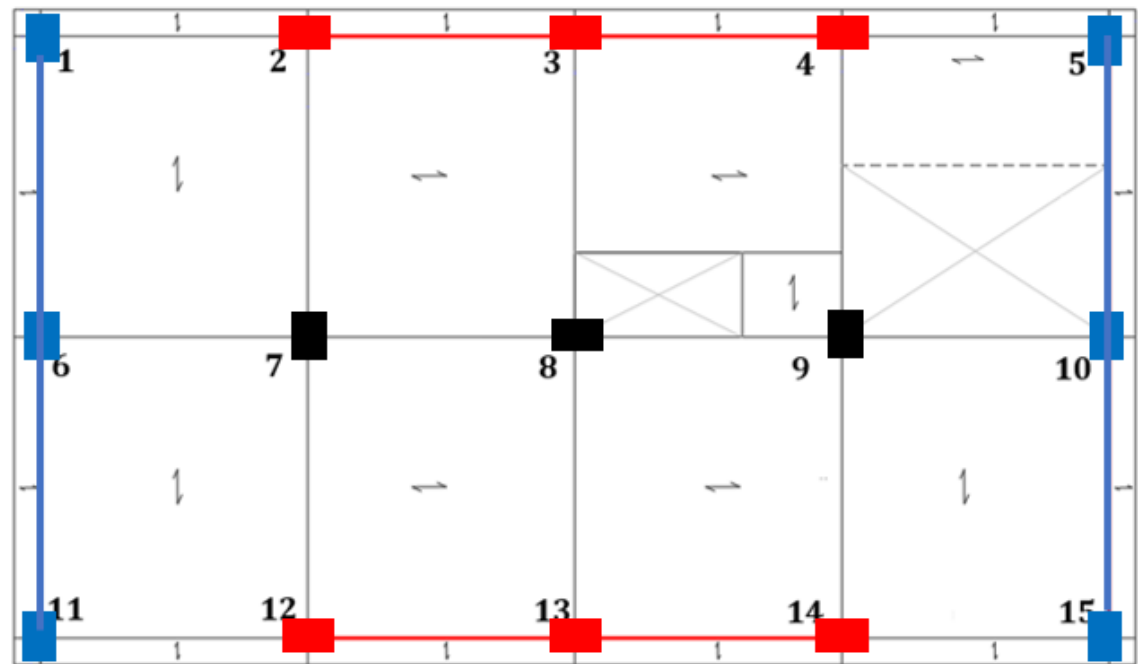
The existence of a slab of adequate thickness ensures the transmission of inertial forces from the masses to the load-bearing elements and allows for limiting the number of modes to be considered when evaluating the dynamic response of the structure.

Seismic frame in x-direction (dropped beams)

Frames with hidden beams are generally not contributing significantly to seismic resistance. According to the Eurocode, these frames can eventually be modelled as secondary members, *provided that they do not contribute to the structure's lateral stiffness by more than 15% (secondary members)*.



Seismic frame in y-direction (dropped beams)



 Columns in seismic frame in x-direction

 Columns in seismic frame in y-direction

 Columns in “pendular” frame

Pendular frame (hidden beams)

2) Definition of the loads

The depth of the deck is assumed equal to 30 cm.

Floors 1 and 2

Permanent structural load: $G_1 = 3.25 \text{ kN/m}^2$

Permanent non-structural load: $G_2 = 1.85 \text{ kN/m}^2$

Variable load: $Q_1 = 3.00 \text{ kN/m}^2$

Claddings on the external beams: $g_2 = 4.50 \text{ kN/m}$

Floor 3

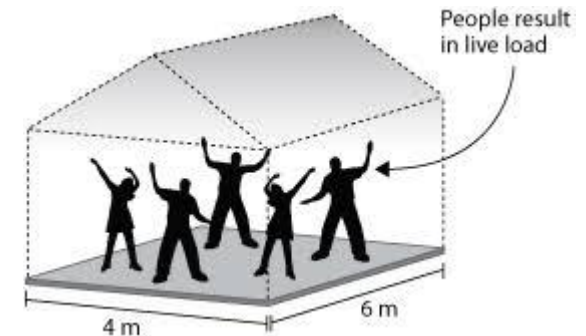
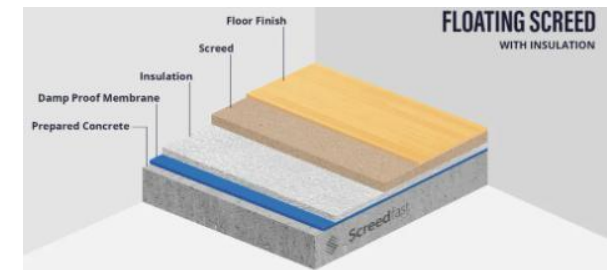
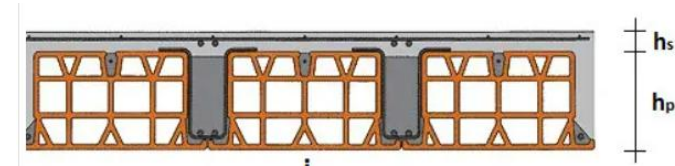
Permanent structural load: $G_1 = 3.25 \text{ kN/m}^2$

Permanent non-structural load: $G_2 = 2.20 \text{ kN/m}^2$

Variable load: $Q_1 = 0.50 \text{ kN/m}^2$

Claddings on the external beams: $g_2 = 1.12 \text{ kN/m}$

Snow: $Q_{\text{snow}} = 0.59 \text{ kN/m}^2$



2) Definition of the loads

The seismic action has been defined according to the Italian code and referring to the construction site located in Fisciano, characterised by *type-B soil and topography class T1, Importance class III*.

FASE 1. INDIVIDUAZIONE DELLA PERICOLOSITÀ DEL SITO

Ricerca per coordinate: LONGITUDINE 14.8003, LATITUDINE 40.7738

Ricerca per comune: REGIONE Campania, PROVINCIA Salerno, COMUNE Fisciano

Elaborazioni grafiche: Grafici spettri di risposta, Variabilità dei parametri

Elaborazioni numeriche: Tabella parametri

Nodi del reticolo intorno al sito: km 7.5, 33430, 33431, 33652, 33653

Reticolo di riferimento: Controllo sul reticolo (Sito esterno al reticolo, Interpolazione su 3 nodi, Interpolazione corretta), Interpolazione superficie rigata

La "Ricerca per comune" utilizza le coordinate ISTAT del comune per identificare il sito. Si sottolinea che all'interno del territorio comunale le azioni sismiche possono essere significativamente diverse da quelle così individuate e si consiglia, quindi, la "Ricerca per coordinate".

INTRO **FASE 1** FASE 2 FASE 3

FASE 2. SCELTA DELLA STRATEGIA DI PROGETTAZIONE

Vita nominale della costruzione (in anni) - V_N : 50 info

Coefficiente d'uso della costruzione - c_U : 1.5 info

Valori di progetto: Periodo di riferimento per la costruzione (in anni) - V_R : 75 info

Periodi di ritorno per la definizione dell'azione sismica (in anni) - T_R : info

Stati limite di esercizio - SLE: SLO - $P_{VR} = 81\%$: 45, SLD - $P_{VR} = 63\%$: 75

Stati limite ultimi - SLU: SLV - $P_{VR} = 10\%$: 712, SLC - $P_{VR} = 5\%$: 1462

Elaborazioni: Grafici parametri azione, Grafici spettri di risposta, Tabella parametri azione

Strategia di progettazione: T_R [anni] vs S_{pa} [g]

LEGENDA GRAFICO: --- Strategia per costruzioni ordinarie, --- Strategia scelta

INTRO **FASE 1** **FASE 2** FASE 3

<https://www.stacec.com/WebApp/ismoGis-Calcolo-parametri-sismici-online>

<https://www.dlubal.com/en/load-zones-for-snow-wind-earthquake/seismic-lvs-en-1998-1.html?srsId=AfmBOopyN19C0iRdhtjn0jwogu0R53nSmsKDXQ92witQLa23SpbUTura#¢er=56.88200056778468,26.773681640625004&zoom=7&marker=56.947,24.104>

FASE 3. DETERMINAZIONE DELL'AZIONE DI PROGETTO

Stato Limite: Stato Limite considerato SLV info

Risposta sismica locale: Categoria di sottosuolo B info, $S_B = 1.200$, $C_C = 1.305$ info, Categoria topografica T1 info, $h/H = 0.000$, $S_T = 1.000$ info

Compon. orizzontale: Smorzamento ξ_s (%) 5, $\eta_1 = 1.000$ info

Spettro di progetto elastico (SLE): Fattore q_s 1, Regol. in altezza si info

Compon. verticale: Spettro di progetto: Fattore q_s 1, $\eta_1 = 1.000$ info

Elaborazioni: Grafici spettri di risposta, Parametri e punti spettri di risposta

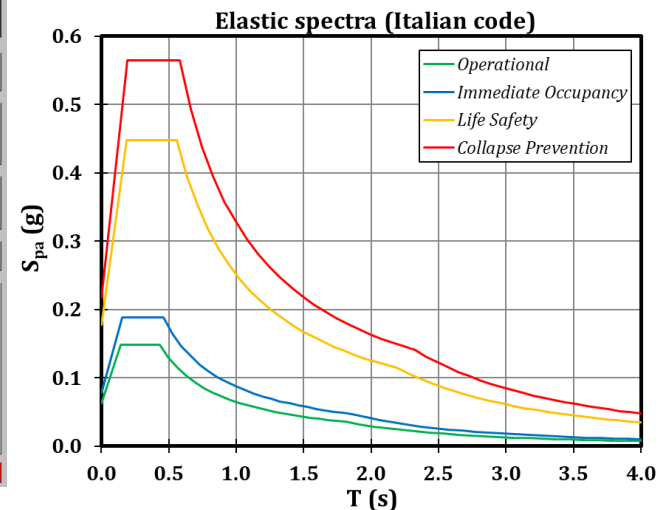
Spettri di risposta: S_{ax} [g], S_{ay} [g], S_z [g]

--- Spettro di progetto - componente orizzontale

--- Spettro di progetto - componente verticale

--- Spettro elastico di riferimento (Cat. A-T1, $\xi = 5\%$)

INTRO **FASE 1** **FASE 2** **FASE 3**



2) Definition of the loads

The seismic action has been defined according to the Italian code and referring to the construction site located in Fisciano (Salerno), characterised by type-B soil and topography class T1. **The spectrum at the Life Safety Limit State corresponds to the spectrum specified in Eurocode 8.**

3.2.2.2 Horizontal elastic response spectrum

(1)P For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2,5 - 1) \right] \quad (3.2)$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \quad (3.3)$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \left[\frac{T_C}{T} \right] \quad (3.4)$$

$$T_D \leq T \leq 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \left[\frac{T_C T_D}{T^2} \right] \quad (3.5)$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma \cdot a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see (3) of this subclause.

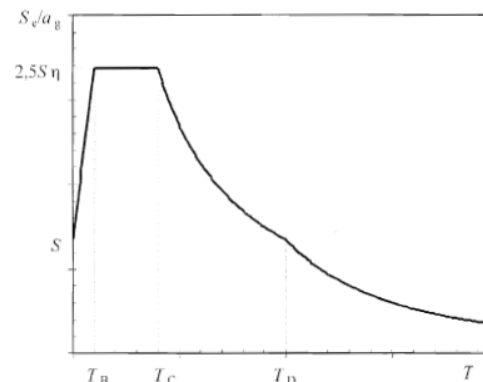


Figure 3.1: Shape of the elastic response spectrum

(2)P The values of the periods T_B , T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

NOTE 1 The values to be ascribed to T_B , T_C , T_D and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see 3.1.2(1)), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S , T_B , T_C and T_D are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalised by a_g for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

2) Definition of the loads

The analysed structure belongs to importance class III and the corresponding importance factor equal to 1.2, proposed by the Eurocode 8, is assumed for the calculations.

Importance class	Buildings	Importance factor γ_I (recommended value)
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0,8
II	Ordinary buildings, not belonging in the other categories.	1,0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,4

3. The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}) chosen by the National Authorities (see 2.1(1)P). An importance factor γ_I equal to 1,0 is assigned to this reference return period. For return periods other than the reference (see importance classes in 2.1(3)P and (4)), the design ground acceleration on type A ground a_g is equal $t_0 a_{gR}$ times the importance factor γ_I (**$a_g = \gamma_I \times a_{gR}$**). (See Note to 2.1(4) of EC8).

The importance factor 1.0 is related to a recurrence period of 475 years (SLV), while 0.8 is related to 243 years, 1.2 to 821 years, and 1.4 to 1303 years.

2) Definition of the loads

Cases

- G_1 = permanent structural load;
- G_2 = permanent non-structural load;
- Q_1 = accidental load;
- E_X = seismic action along longitudinal direction;
- E_Y = seismic action along transverse direction.

ULS Combinations

$$g_{G1} G_1 + g_{G2} G_2 + g_Q Q_1$$

$$G_1 + G_2 + y_{21} Q_1 + E_X + 0.3 E_Y$$

$$G_1 + G_2 + y_{21} Q_1 + 0.3 E_X + E_Y$$

Load	Condition	Coefficient	Value
Structural	Favourable	γ_{G1}	1.00
	Unfavourable		1.30
Non-structural	Favourable	γ_{G2}	0.80
	Unfavourable		1.50
Accidental	Favourable	γ_Q	0.00
	Unfavourable		1.50

SLS Combinations

$$G_1 + G_2 + Q_1$$

$$G_1 + G_2 + y_{11} Q_1$$

$$G_1 + G_2 + y_{21} Q_1$$

Load	ψ_1	ψ_2
Offices	0.7	0.3 (Italian Code)
Roof	0.5	0.2

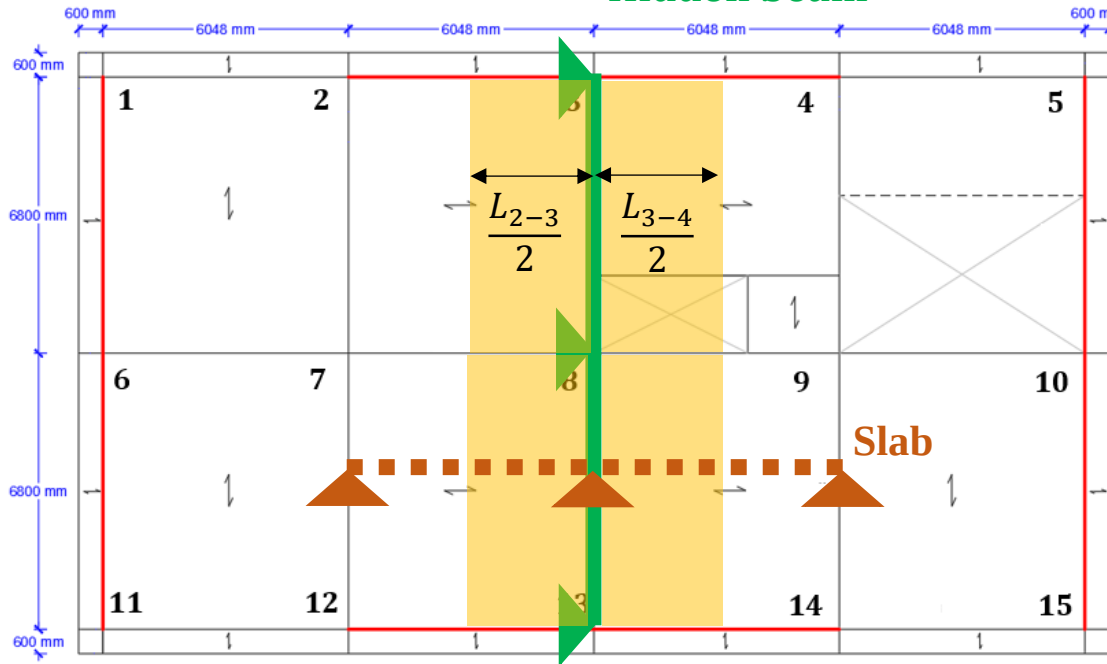
3) Sizing of sections and preliminary structural verification

*Gravity part
(secondary beams and columns)*

3) Sizing of sections and preliminary structural verification

Floors 1 and 2

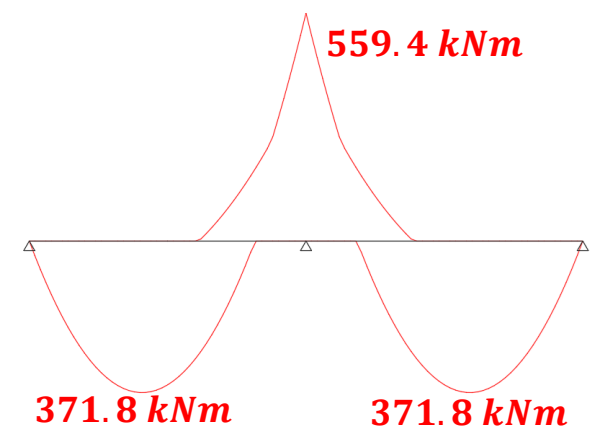
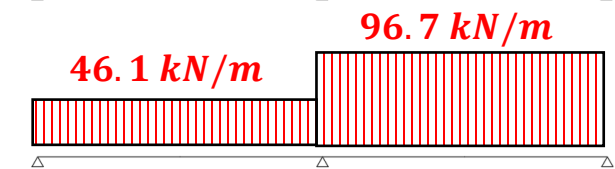
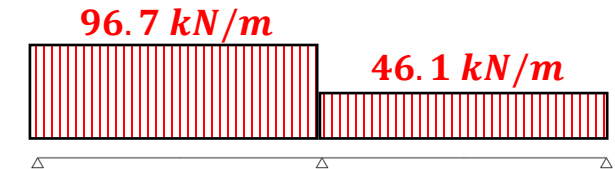
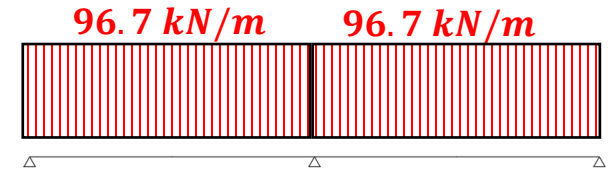
Hidden beam



$$G_1 = 3.25 \text{ kN/m}^2 \quad G_2 = 1.85 \text{ kN/m}^2$$

$$Q_1 = 3.00 \text{ kN/m}^2 \quad g_{1,100 \times 30} = \gamma_{cls} b h = 7.50 \text{ kN/m}$$

$$\gamma_{G1} = 1.30 \quad \gamma_{G2} = 1.50 \quad \gamma_Q = 1.50 \quad CC = 1.25$$



$$q_{3-8,ULS,max} = (\gamma_{G1}G_1 + \gamma_{G2}G_2 + \gamma_Q Q_1) \frac{L_{2-3} + L_{3-4}}{2} CC + \gamma_{G1} g_{1,100 \times 30} = 96.7 \text{ kN/m}$$

$$q_{8-13,ULS,min} = (G_1 + G_2) \frac{L_{2-3} + L_{3-4}}{2} CC + g_{1,100 \times 30} = 46.1 \text{ kN/m}$$

$$q_{8-13,ULS,max} = (\gamma_{G1}G_1 + \gamma_{G2}G_2 + \gamma_Q Q_1) \frac{L_{2-3} + L_{3-4}}{2} CC + \gamma_{G1} g_{1,100 \times 30} = 96.7 \text{ kN/m}$$

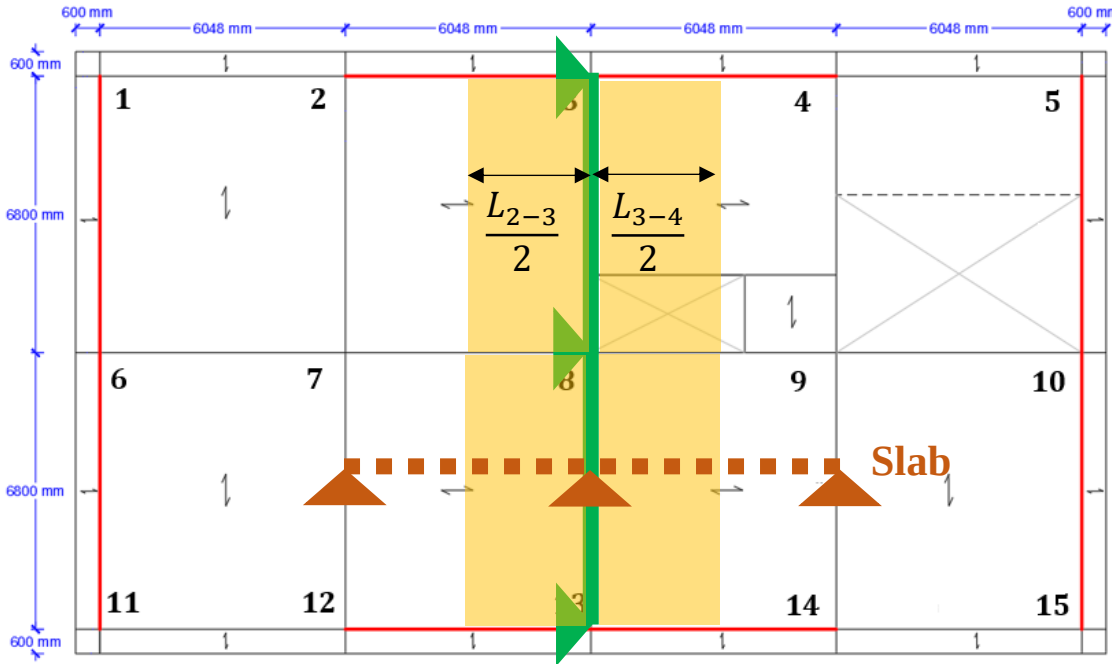
$$q_{3-8,ULS,min} = (G_1 + G_2) \frac{L_{2-3} + L_{3-4}}{2} CC + g_{1,100 \times 30} = 46.1 \text{ kN/m}$$

3) Sizing of sections and preliminary structural verification

Floors 1 and 2

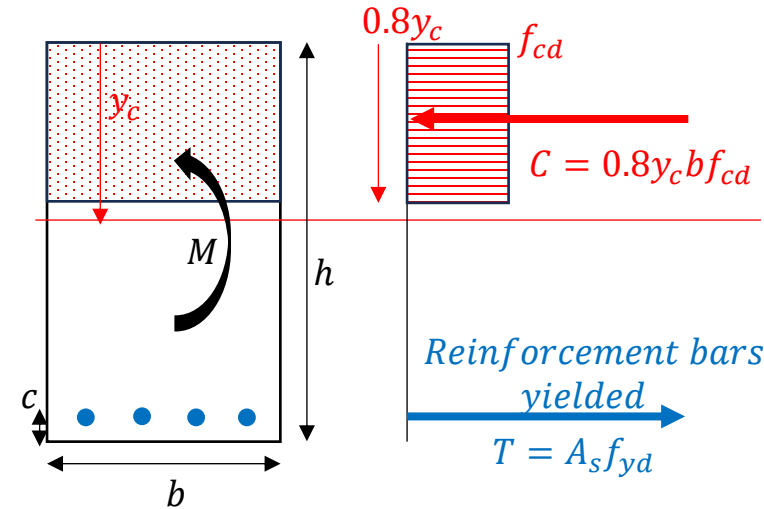
Hidden beam

Design assumptions



$$\xi = \frac{y_c}{h} = 0.6 \quad c = 30 \text{ mm} \Rightarrow \delta = \frac{c}{h} = 0.1$$

$h = 300 \text{ mm}$ (hidden beam)



Rotational equilibrium around the bars:

$$b \cdot 0.8y_c \cdot f_{cd} \cdot (h - c - 0.4y_c) = M$$

$$\frac{b \cdot 0.8y_c \cdot f_{cd} \cdot (h - c - 0.4y_c)}{bh^2 f_{cd}} = \frac{M}{bh^2 f_{cd}}$$

$$0.8\xi \cdot (1 - \delta - 0.4\xi) = \frac{M}{bh^2 f_{cd}} \quad b = \frac{1}{0.8\xi \cdot (1 - \delta - 0.4\xi)} \cdot \frac{M}{h^2 f_{cd}}$$

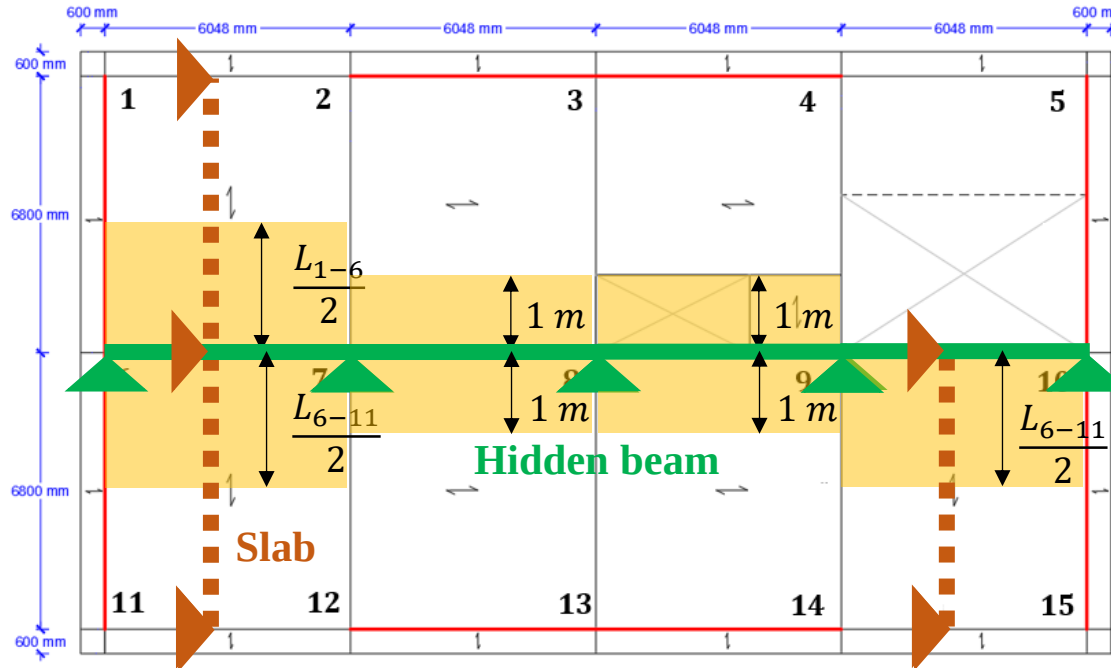
Design equation for the beam width:

$$b = \frac{1}{0.8\xi \cdot (1 - \delta - 0.4\xi)} \cdot \frac{M_{\max, 3-8-13, \text{ULS, gravity}}}{h^2 f_{cd}} = 989 \text{ mm}$$

$b = 1000 \text{ mm}$ (100 cm)

3) Sizing of sections and preliminary structural verification

Floors 1 and 2



$$G_1 = 3.25 \text{ kN/m}^2$$

$$G_2 = 1.85 \text{ kN/m}^2$$

$$Q_1 = 3.00 \text{ kN/m}^2$$

$$g_{1,60 \times 30} = \gamma_{cls} b h = 25 \text{ kN/m}^3 \cdot 0.6 \text{ m} \cdot 0.3 \text{ m} = 4.50 \text{ kN/m}$$

$$\gamma_{G1} = 1.30 \quad \gamma_{G2} = 1.50$$

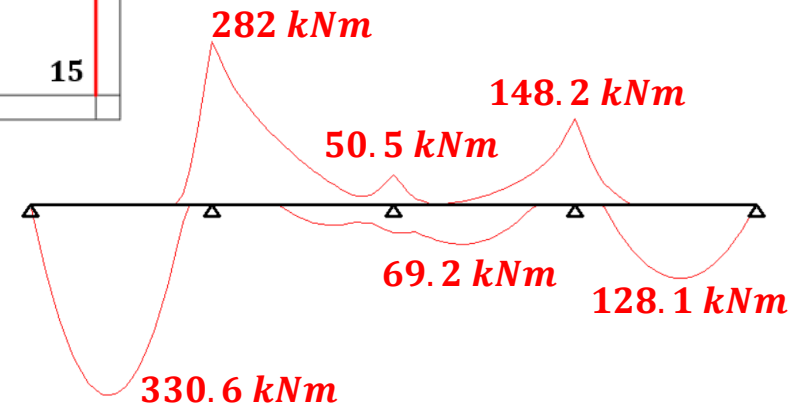
$$\gamma_Q = 1.50 \quad \psi_{21} = 0.60$$

$$CC = 1.25$$

$$q_{6-7,ULS,max} = 103.6 \text{ kN/m}$$

$$q_{7-8,ULS,max} = 28.9 \text{ kN/m}$$

$$q_{9-10,ULS,max} = 45.0 \text{ kN/m}$$

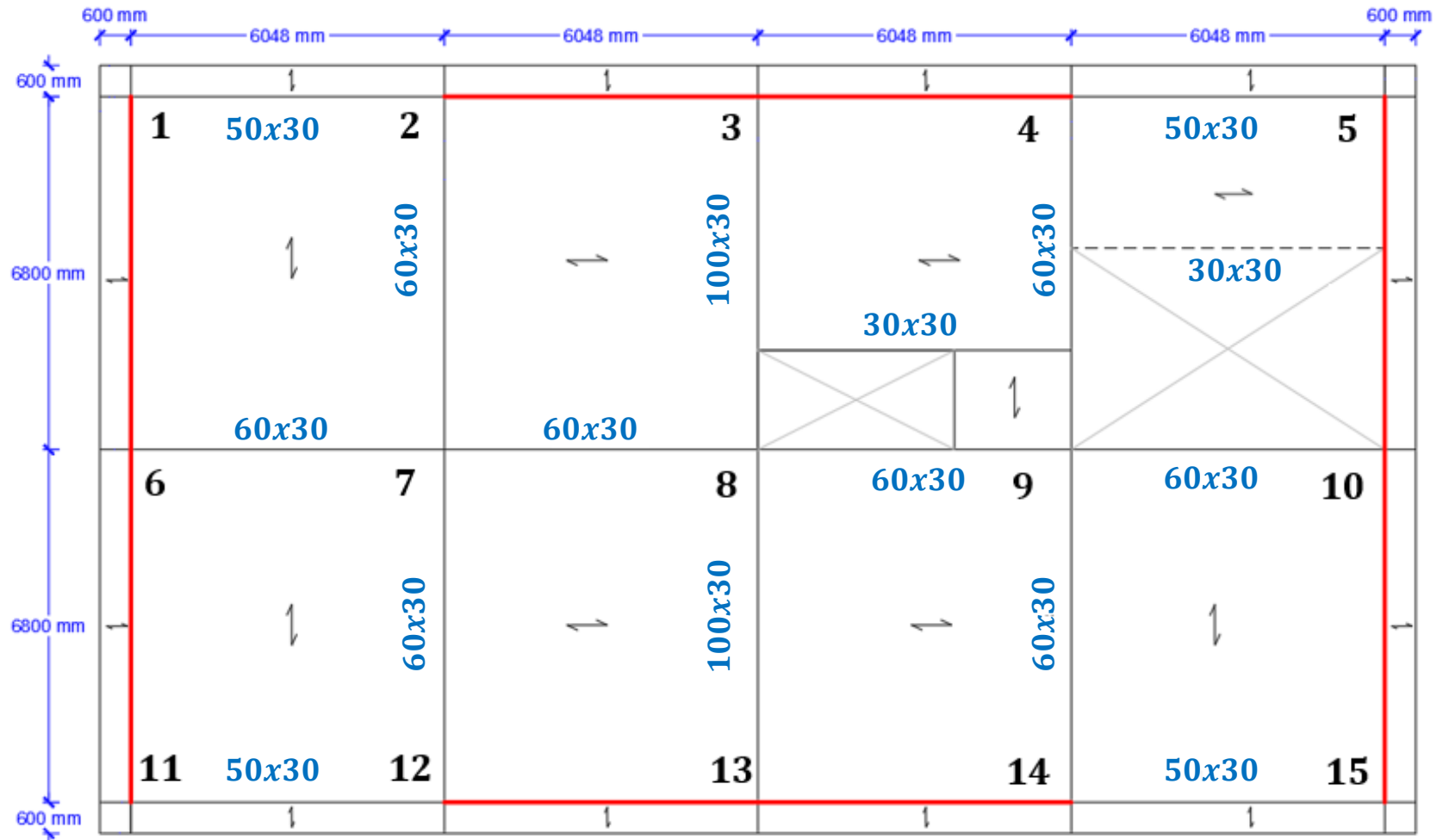


$$M_{max,6-7-8-9-10,ULS,gravity} = 330.6 \text{ kNm} \quad b = \frac{1}{0.8\xi \cdot (1 - \delta - 0.4\xi)} \cdot \frac{M_{max,6-7-8-9-10,ULS,gravity}}{h^2 f_{cd}} = 585 \text{ mm}$$

$b = 60 \text{ cm}$

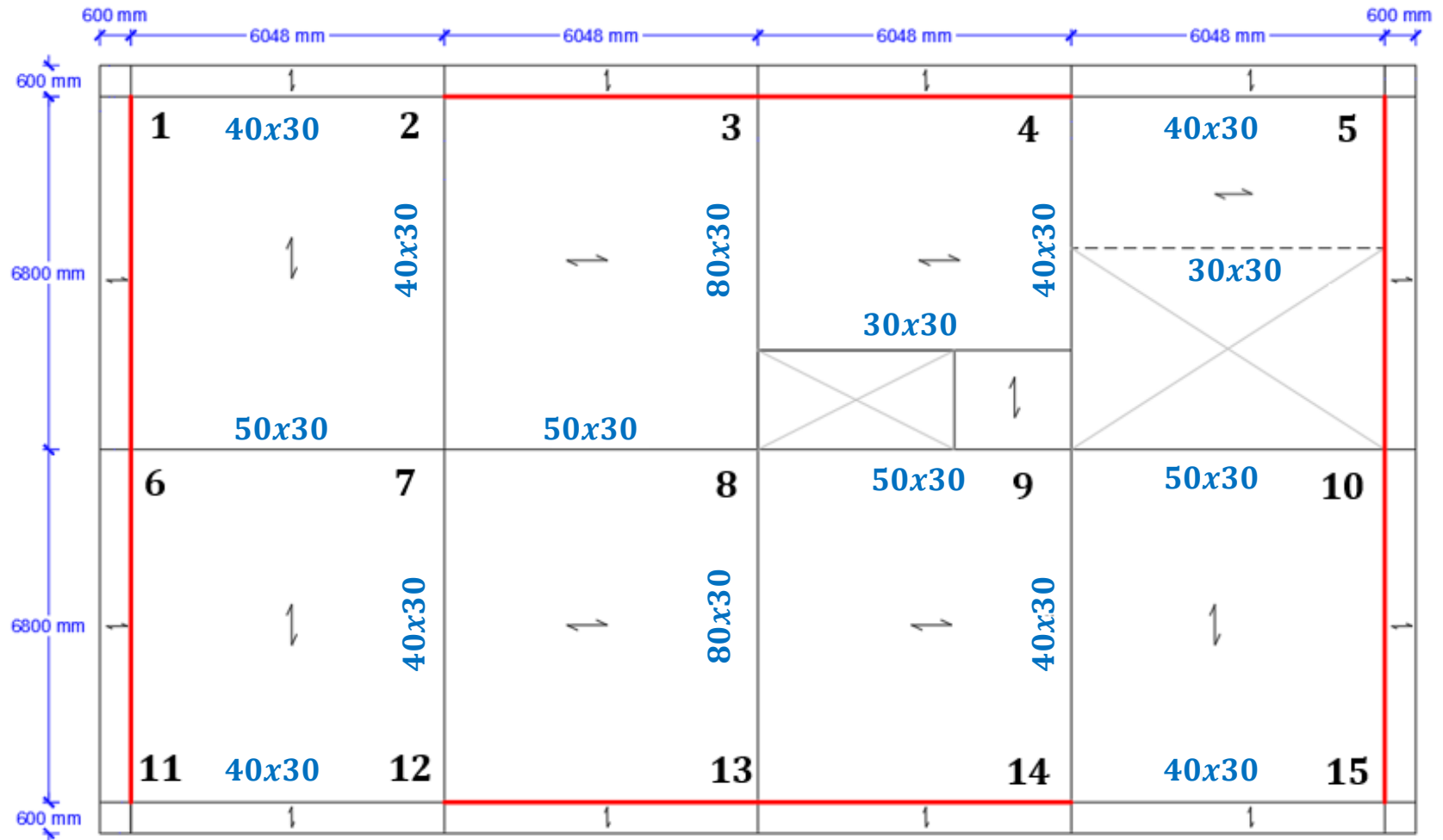
3) Sizing of sections and preliminary structural verification

Floors 1 and 2 (secondary hidden beams)



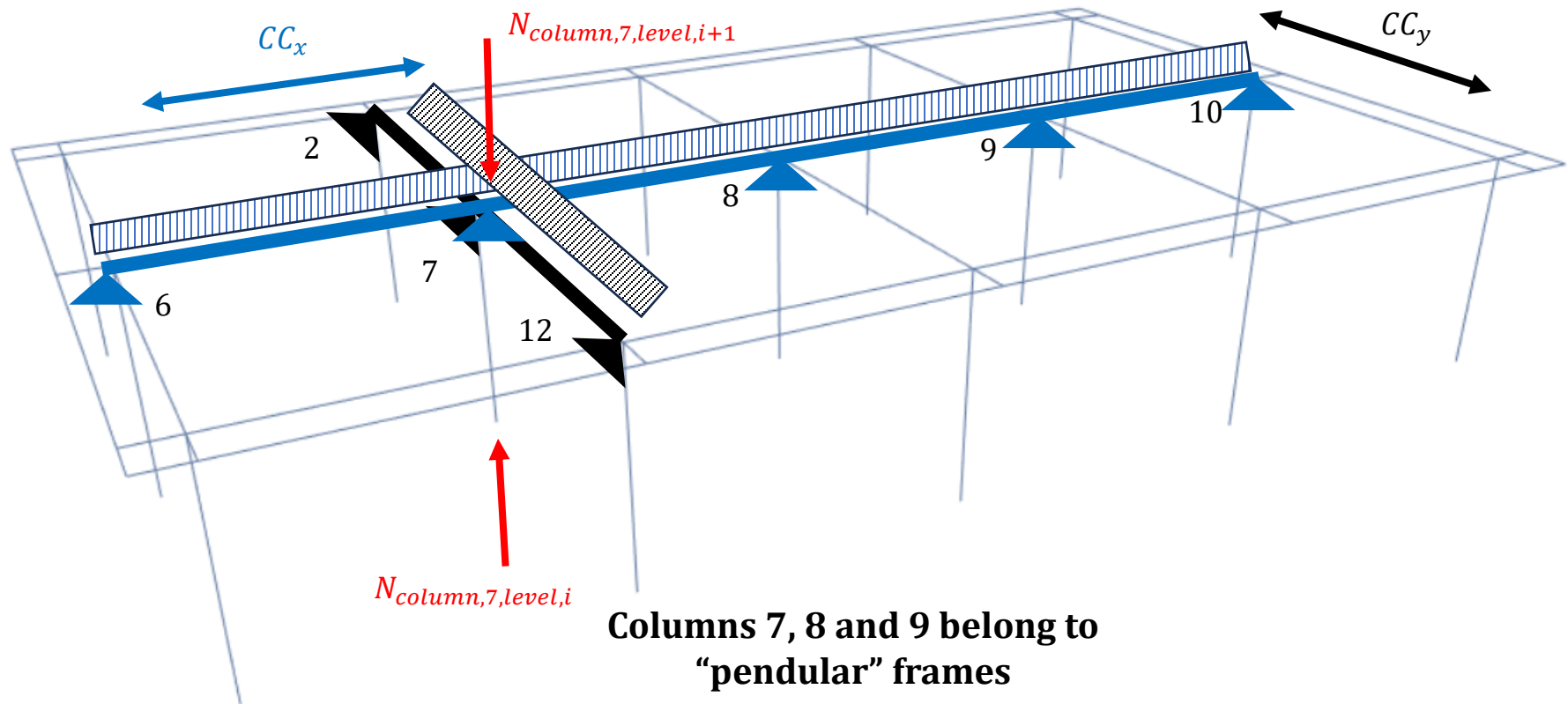
3) Sizing of sections and preliminary structural verification

Floor 3 (secondary hidden beams)

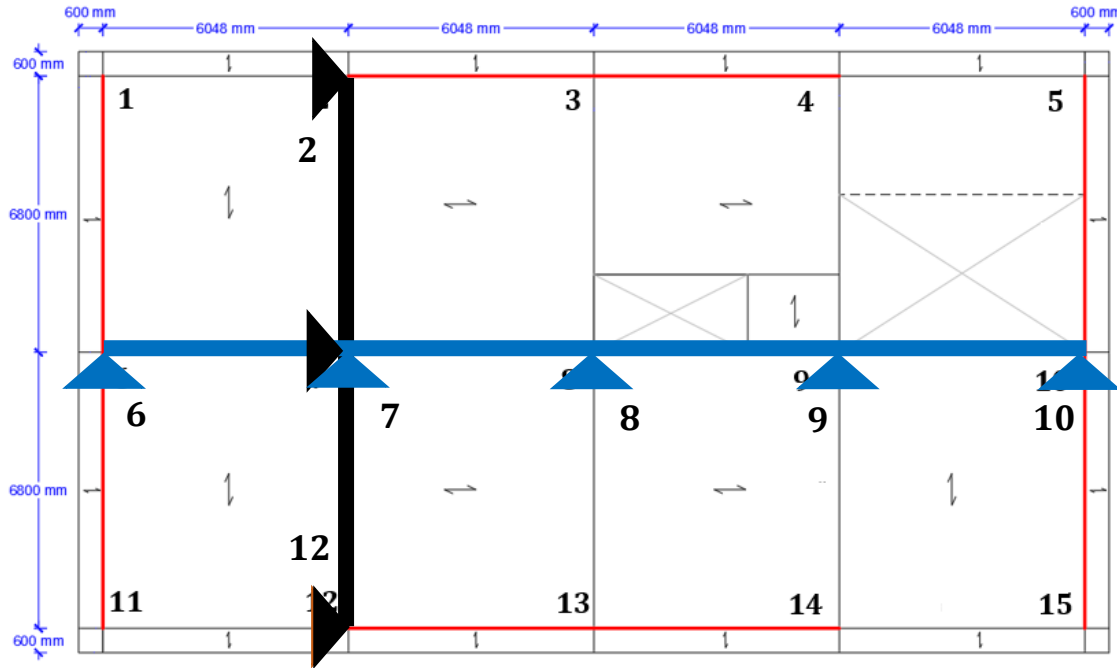


3) Sizing of sections and preliminary structural verification

The axial force acting at the base of a column is given by the sum of the reactions at node 7 in the continuous beam schemes 2-7-12 and 6-7-8-9-10, along with the self-weight of the column. This simple assessment is useful during the preliminary design phase.



3) Sizing of sections and preliminary structural verification



Floor 1: column 40x50

Floor 2: column 40x50

Floor 3: column 40x40

$$P_{7,ground} = \gamma_{cls} b h L = 54.2 \text{ kN}$$

$$\nu = 0.65 \quad b_{7,ground} = 400 \text{ mm}$$

$$CC_x = 1.10 \quad CC_y = 1.25$$

$$N_{7,ground} = 2 \left(q_{6-7,floors1-2} \frac{L_{6-7}}{2} CC_x + q_{7-8,floors1-2} \frac{L_{7-8}}{2} CC_x + q_{2-7,floors1-2} \frac{L_{2-7}}{2} CC_y + q_{7-12,floors1-2} \frac{L_{7-12}}{2} CC_y \right) +$$

$$\left(q_{6-7,floor3} \frac{L_{6-7}}{2} CC_x + q_{7-8,floor3} \frac{L_{7-8}}{2} CC_x + q_{2-7,floor3} \frac{L_{2-7}}{2} CC_y + q_{7-12,floor3} \frac{L_{7-12}}{2} CC_y \right) +$$

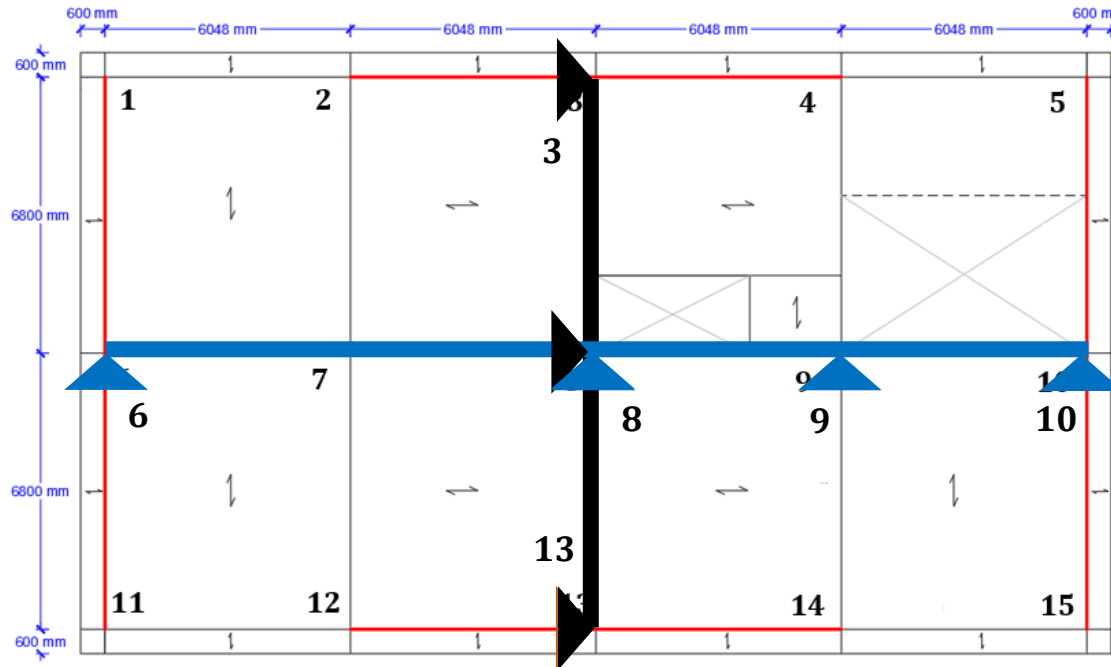
$$+ 1.3 P_{7,ground} = 2484.7 \text{ kN}$$

$$\nu = \frac{N_{7,ground}}{b_{7,ground} h_{7,ground} f_{cd}}$$

$$h_{7,ground} = \frac{N_{7,ground}}{b_{7,ground} \nu f_{cd}} = 482 \text{ mm}$$

$$h_{7,ground} = 50 \text{ cm}$$

3) Sizing of sections and preliminary structural verification



Floor 1: column 40x70

Floor 2: column 40x70

Floor 3: column 40x50

$$P_{7,ground} = \gamma_{cls} b h L = 73.3 \text{ kN}$$

$$\nu = 0.65 \quad b_{8,ground} = 400 \text{ mm}$$

$$CC_x = 1.10 \quad CC_y = 1.25$$

$$N_{8,ground} = 2 \left(q_{8-7,floors1-2} \frac{L_{8-7}}{2} CC_x + q_{8-9,floors1-2} \frac{L_{8-9}}{2} CC_x + q_{8-3,floors1-2} \frac{L_{8-3}}{2} CC_y + q_{8-13,floors1-2} \frac{L_{8-13}}{2} CC_y \right) +$$

$$\left(q_{8-7,floor3} \frac{L_{8-7}}{2} CC_x + q_{8-9,floor3} \frac{L_{8-9}}{2} CC_x + q_{8-3,floor3} \frac{L_{8-3}}{2} CC_y + q_{8-13,floor3} \frac{L_{8-13}}{2} CC_y \right) +$$

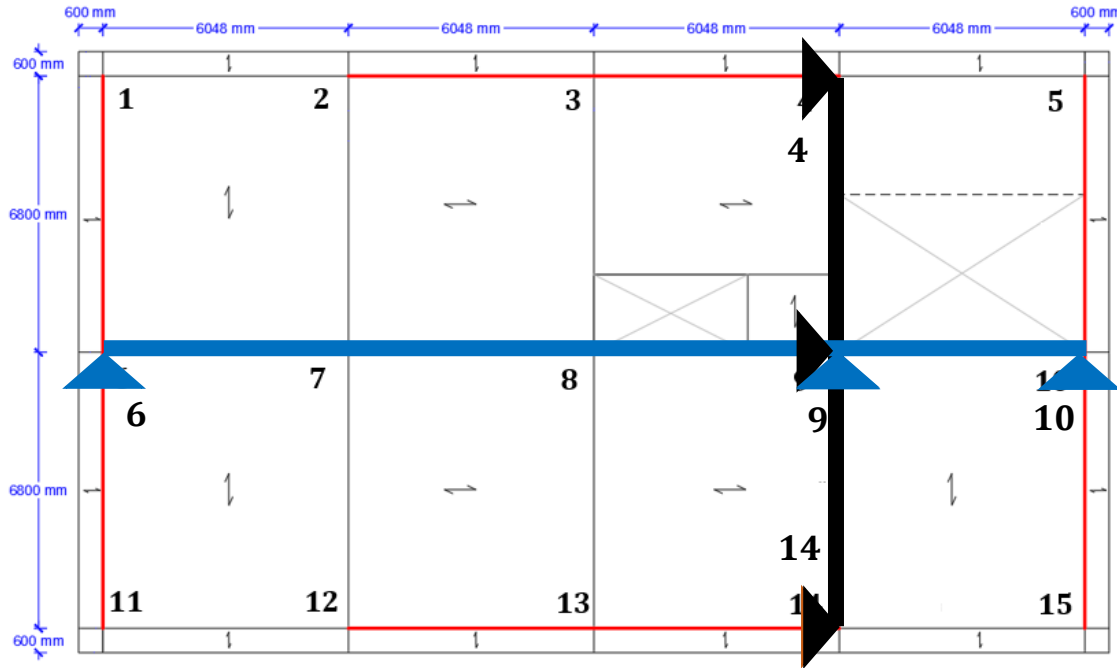
$$+ 1.3 P_{8,ground} = 2825.9 \text{ kN}$$

$$\nu = \frac{N_{8,ground}}{b_{8,ground} h_{8,ground} f_{cd}}$$

$$h_{8,ground} = \frac{N_{8,ground}}{b_{8,ground} \nu f_{cd}} = 548 \text{ mm} \quad \mathbf{h_{8,ground} = 70 \text{ cm}}$$

The choice to adopt a column depth of 70 cm instead of 60 cm stems from geometric considerations. This is because the hidden beams 3-8-13 have a width of 100 cm. According to the Eurocode, the difference between the width of the hidden beam and the column should not exceed the depth of the hidden beam.

3) Sizing of sections and preliminary structural verification



Floor 1: column 40x50

Floor 2: column 40x50

Floor 3: column 40x40

$$P_{7,ground} = \gamma_{cls} b h L = 54.2 \text{ kN}$$

$$\nu = 0.65 \quad b_{7,ground} = 400 \text{ mm}$$

$$CC_x = 1.10 \quad CC_y = 1.25$$

$$N_{9,ground} = 2 \left(q_{9-8,floors1-2} \frac{L_{9-8}}{2} CC_x + q_{9-10,floors1-2} \frac{L_{9-10}}{2} CC_x + q_{9-4,floors1-2} \frac{L_{9-4}}{2} CC_y + q_{9-14,floors1-2} \frac{L_{9-14}}{2} CC_y \right) +$$

$$\left(q_{9-8,floor3} \frac{L_{9-8}}{2} CC_x + q_{9-10,floor3} \frac{L_{9-10}}{2} CC_x + q_{9-4,floor3} \frac{L_{9-4}}{2} CC_y + q_{9-14,floor3} \frac{L_{9-14}}{2} CC_y \right) +$$

$$+ 1.3 P_{9,ground} = 1951.8 \text{ kN}$$

$$\nu = \frac{N_{9,ground}}{b_{9,ground} h_{9,ground} f_{cd}}$$

$$h_{9,ground} = \frac{N_{9,ground}}{b_{9,ground} \nu f_{cd}} = 379 \text{ mm}$$

$$h_{9,ground} = 50 \text{ cm}$$

The decision to use a column depth of 50 cm instead of 40 cm is based on geometric considerations, aiming to achieve symmetry in the structure. As a result, the depth of column 9 has been set equal to that of column 7.

3) Sizing of sections and preliminary structural verification

*Seismic resistant part
(primary beams and columns)*

3) Sizing of sections and preliminary structural verification

The structure is **symmetric and regular** in plan view. At this preliminary stage, it can be assumed that the **center of mass coincides with the center of stiffness on each floor**. In this context, the orientation of the columns in the plan view is illustrated below. Additionally, it is assumed that seismic forces at each floor are withstood by two frames in the x-direction (highlighted in red) and two frames in the y-direction (highlighted in blue).

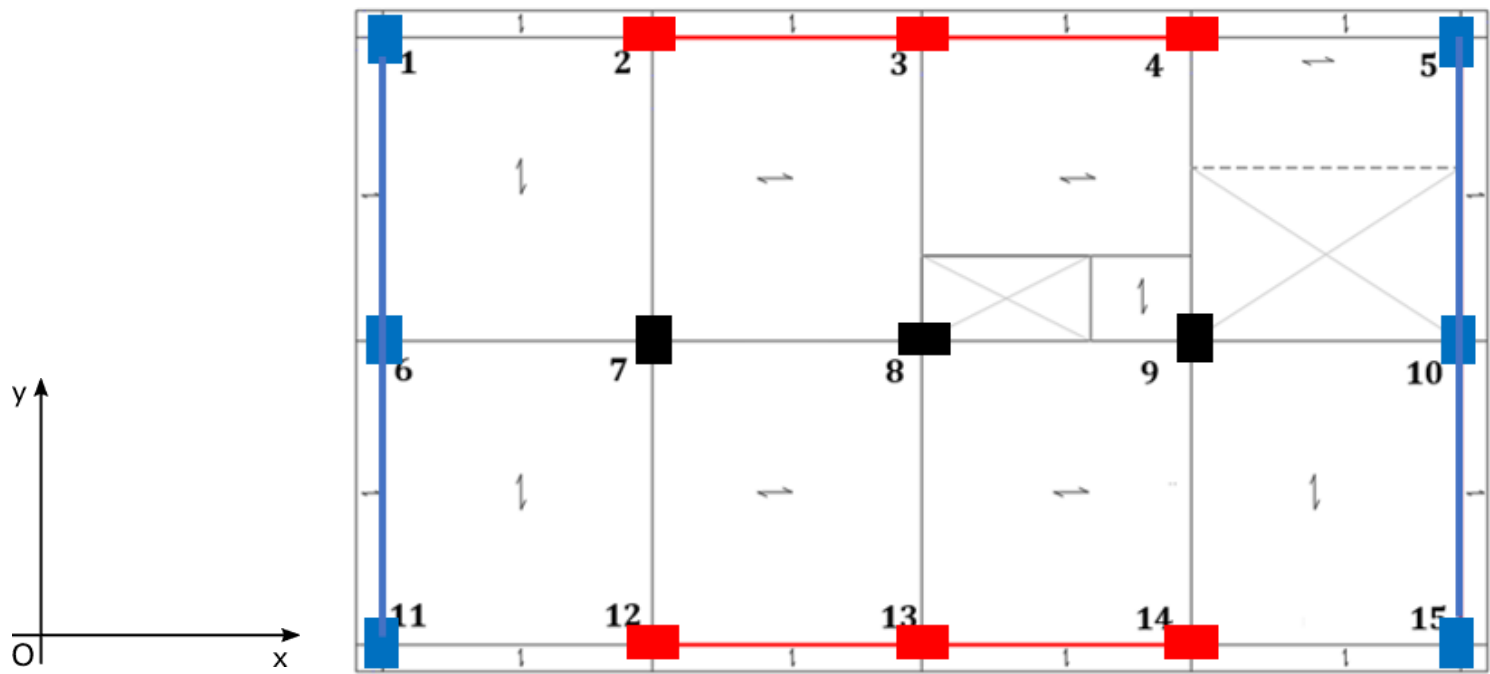
**Seismic frame in x-direction
(dropped beams)**

**Seismic frame in y-direction
(dropped beams)**

**Column in seismic frame
in x-direction**

**Column in seismic frame
in y-direction**

**Column in pendular
frame**
Pendular frame (hidden beams)

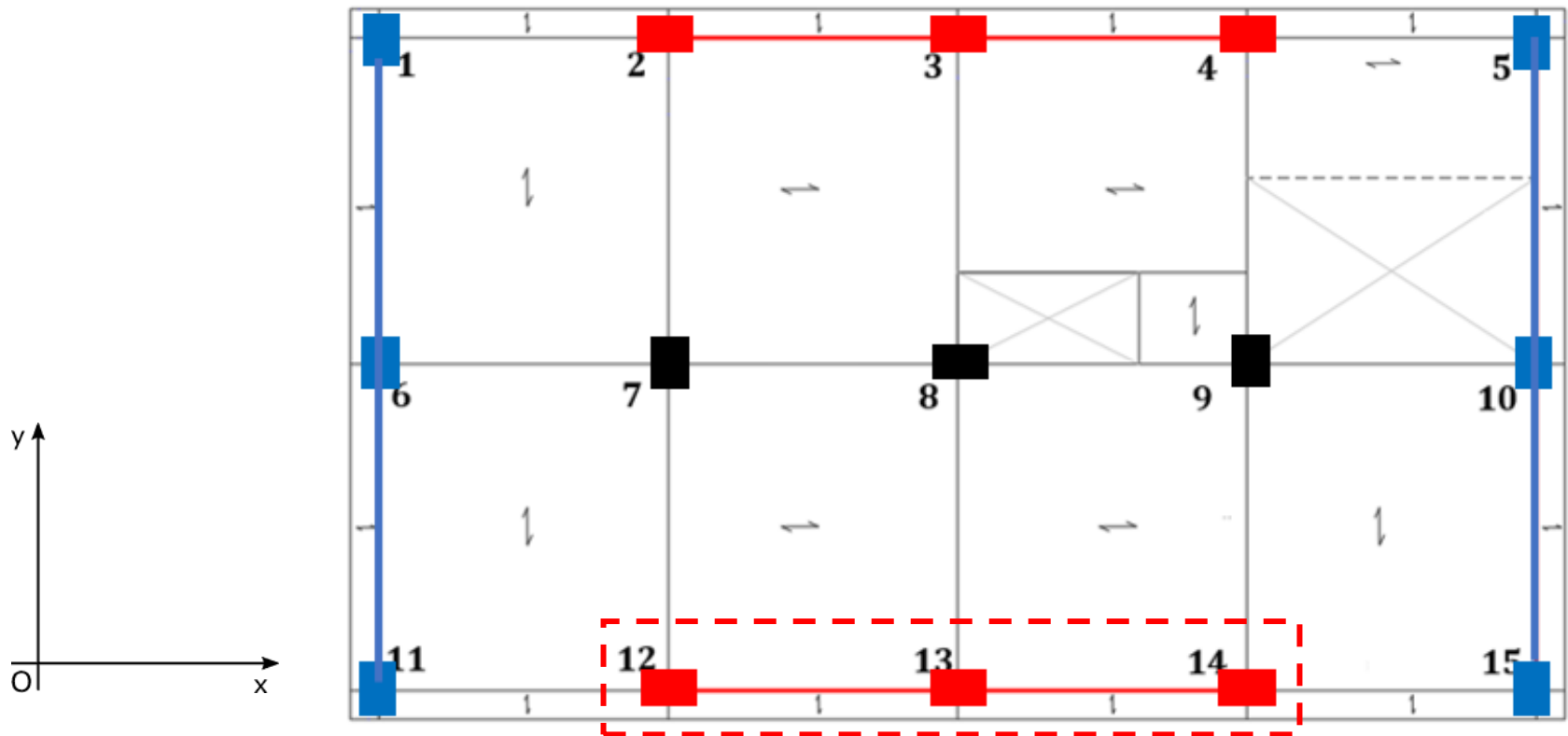


3) Sizing of sections and preliminary structural verification

Design of the frames considering the seismic action along the x-direction

 **Column in seismic frame in x-direction**

Seismic frame in x-direction (dropped beams)



3) Sizing of sections and preliminary structural verification

- 3.2.1) Floor masses
- 3.2.2) Design spectra
- 3.2.3) Evaluation of forces for static analysis
- 3.2.4) Shear distribution among columns
- 3.2.5) Estimation of bending moments in columns
- 3.2.6) Estimation of bending moments in beams
- 3.2.7) Section sizing of the beams
- 3.2.8) Section sizing of the columns
- 3.2.9) Stiffness evaluation and revised period estimation

3) Sizing of sections and preliminary structural verification

3.2.1) Floor masses

Loads (floor 3)

$$G_1 = 3.25 \text{ kN/m}^2$$

$$G_2 = 2.20 \text{ kN/m}^2$$

$$Q_1 = 0.50 \text{ kN/m}^2$$

$$g_{2,\text{claddings}} = 1.12 \text{ kN/m}$$

$$L_y = 14.80 \text{ m}$$

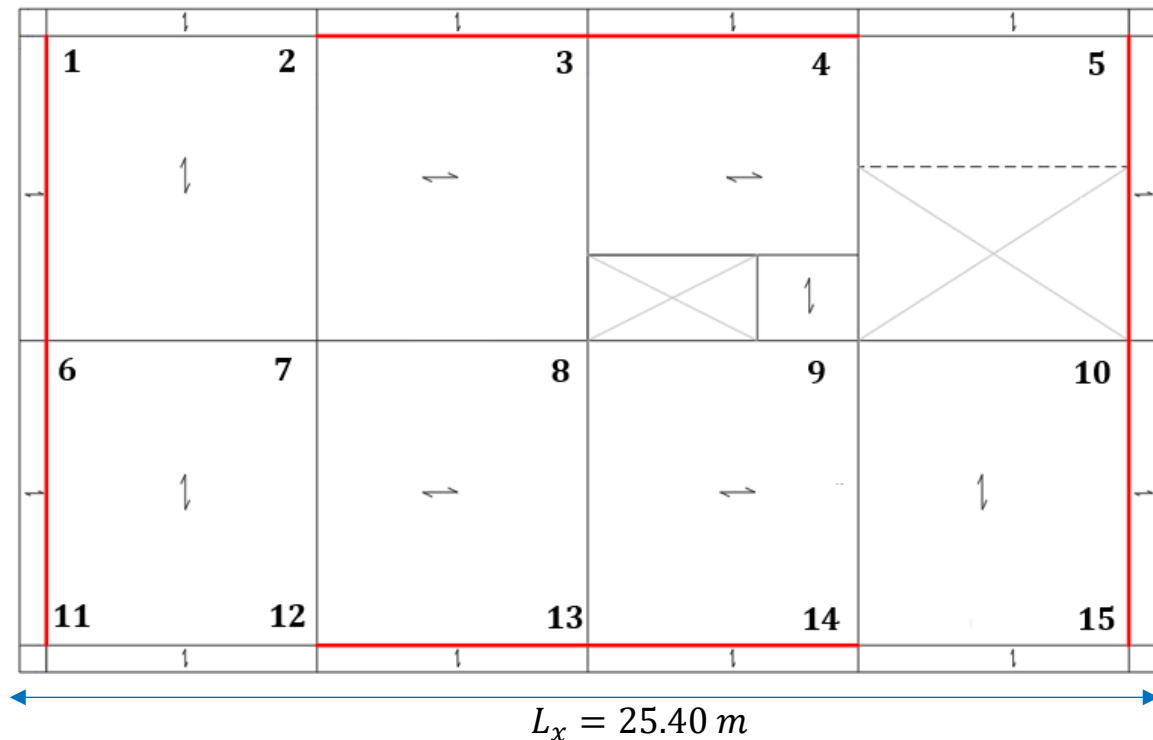
Loads (floors 1,2)

$$G_1 = 3.25 \text{ kN/m}^2$$

$$G_2 = 1.85 \text{ kN/m}^2$$

$$Q_1 = 3.00 \text{ kN/m}^2$$

$$g_{2,\text{claddings}} = 4.50 \text{ kN/m}$$



- G_1 = permanent structural load;
 - G_2 = permanent non-structural load;
 - Q_1 = accidental load;
- $$G_1 + G_2 + \psi_{21} Q_1$$

$$m_{\text{floor } 1} = 432167 \text{ kg}$$

$$m_{\text{floor } 2} = 432167 \text{ kg}$$

$$m_{\text{floor } 3} = 268324 \text{ kg}$$

3) Sizing of sections and preliminary structural verification

3.2.2) Design spectra

$$T_1 = C_t \cdot H^{\frac{3}{4}} = 0.075 \cdot 11.67^{\frac{3}{4}} = 0.47 \text{ s}$$

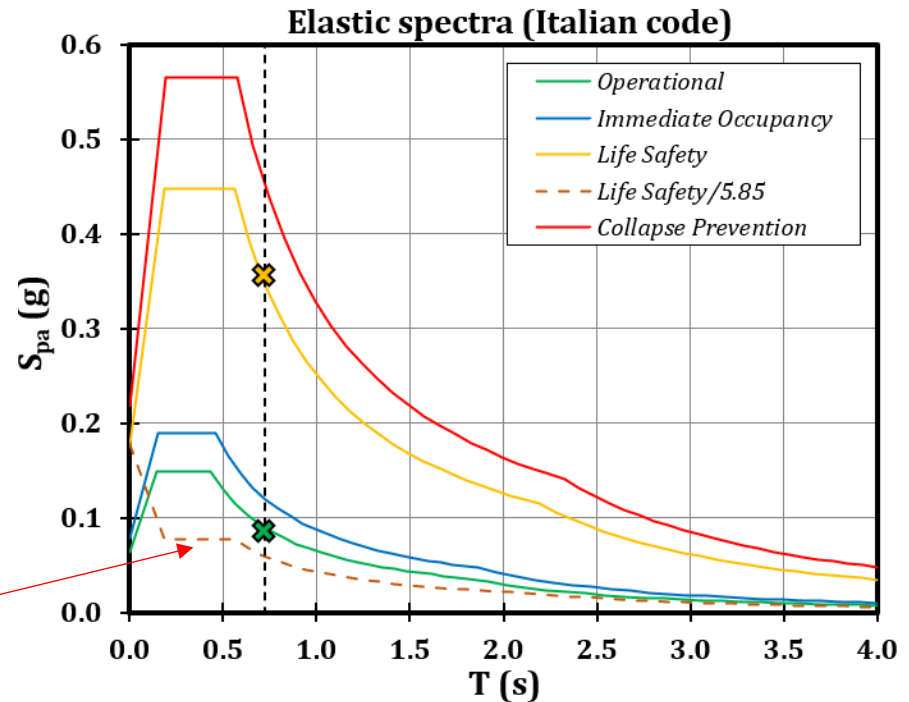
*This is a simplified equation for estimating the structure's period. A more accurate value is determined after designing the structural elements, requiring an iterative process. Following this process, the period of the structure is found to be 0.67 seconds, which is the value used in the following calculations.

$$V_b = \frac{S_d(T_1)}{q_{LS}} \cdot m \cdot \lambda = \frac{0.376}{3.88} \cdot \frac{9.81 \text{ m}}{\text{s}^2} \cdot (2 \cdot 432.2 + 268.3) \text{ tons} = 916.1 \text{ kN}$$

$$F_{b,1 \text{ frame}} = \frac{V_b}{2} = 458.1 \text{ kN}$$

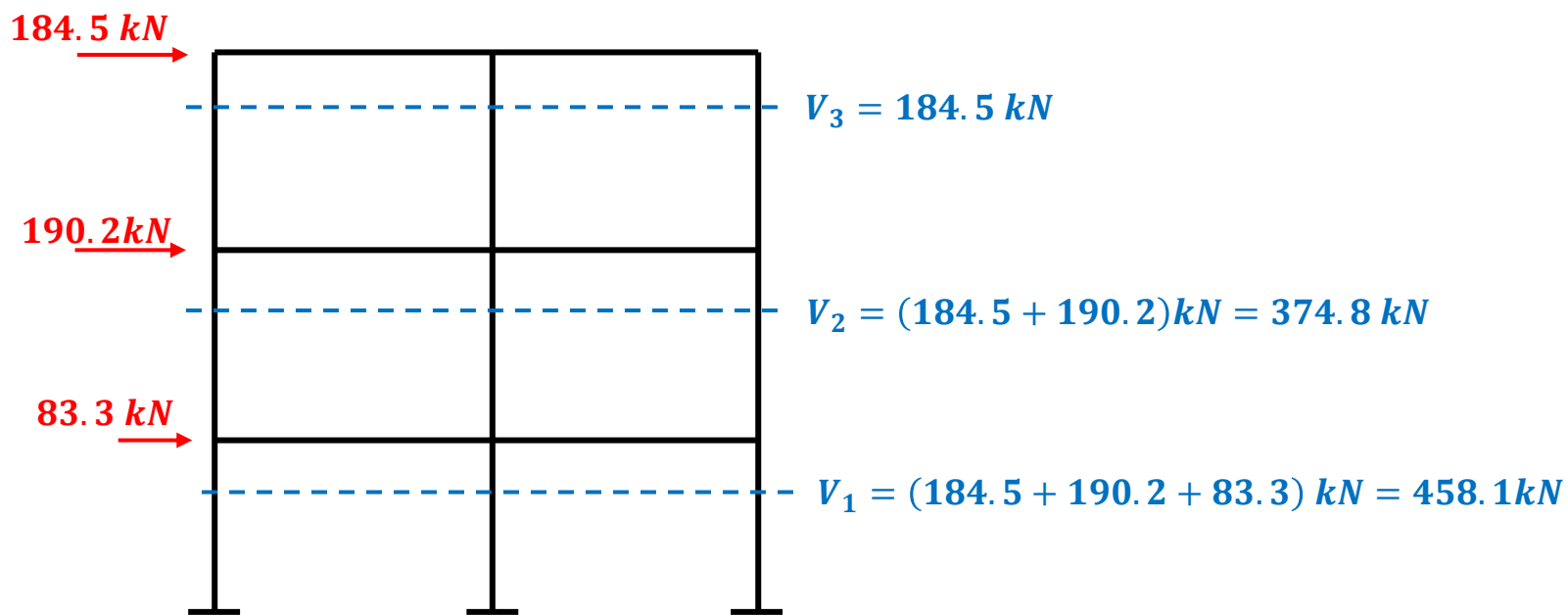
$$1.5 < q_{LS} \leq \min \left(\begin{array}{l} 4.5 \frac{\alpha_u}{\alpha_1} = 5.85; \\ \frac{S_{pa, \text{Life Safety}}}{S_{pa, \text{Operational}}} = \frac{0.376 \text{ g}}{0.097 \text{ g}} = 3.88 \end{array} \right) = 3.88$$

To ensure that the design spectrum does not fall below the Operational spectrum



3) Sizing of sections and preliminary structural verification

3.2.3) Evaluation of forces for static analysis



$$F_1 = F_{b,1 \text{ frame}} \frac{z_1 m_1}{\sum_{i=1}^3 z_i m_i} = 83.3 \text{ kN}$$

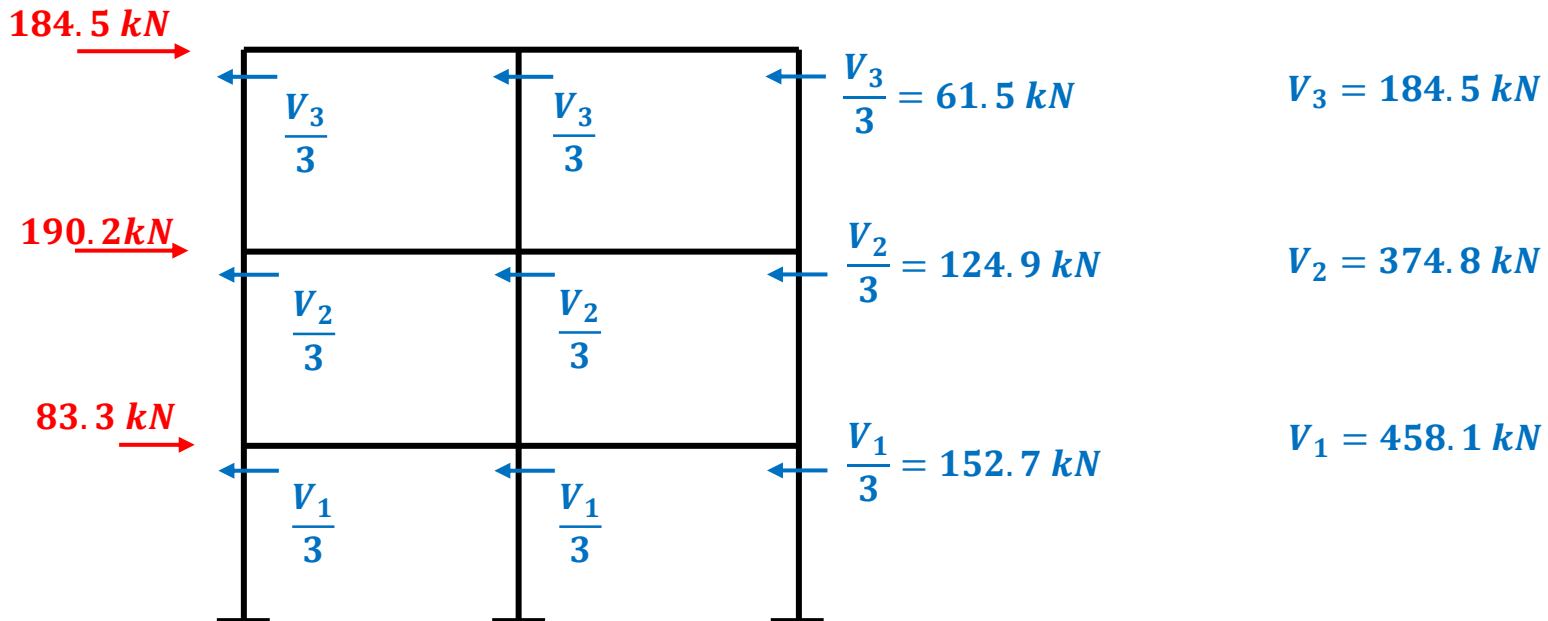
$$F_2 = F_{b,1 \text{ frame}} \frac{z_2 m_2}{\sum_{i=1}^3 z_i m_i} = 190.2 \text{ kN}$$

$$F_3 = F_{b,1 \text{ frame}} \frac{z_3 m_3}{\sum_{i=1}^3 z_i m_i} = 184.5 \text{ kN}$$

3) Sizing of sections and preliminary structural verification

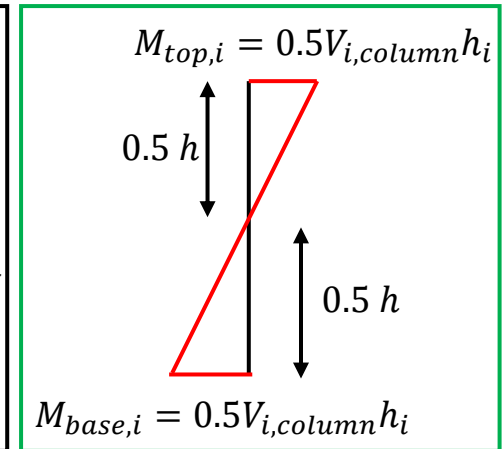
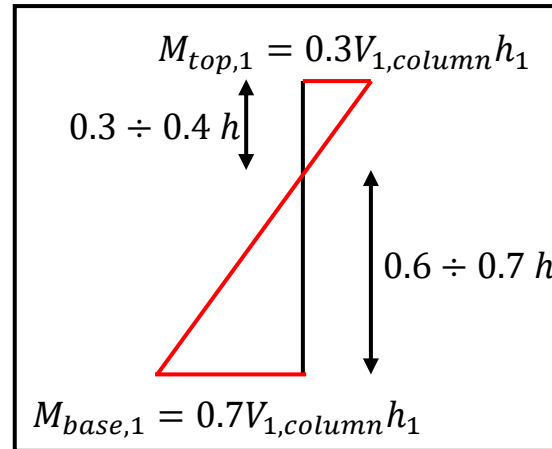
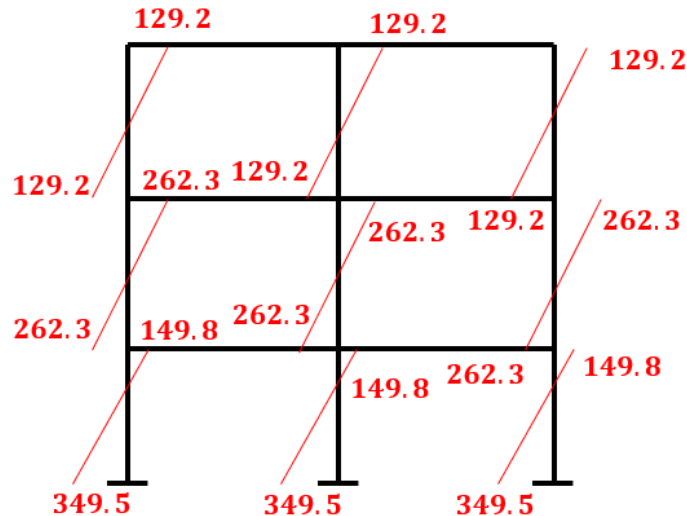
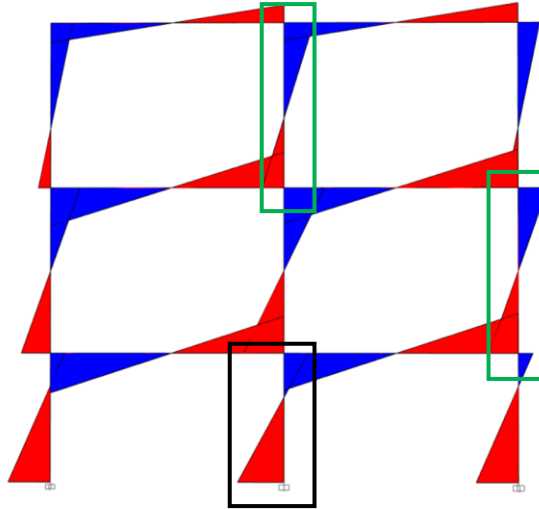
3.2.4) Shear distribution among columns

In the initial sizing phase, it is generally advisable to assign the same cross-section to all columns on the same floor. In later stages, once the beam and column sections have been sized, the stiffness of individual columns can be estimated more accurately, allowing for a more precise shear distribution based on their stiffness



3) Sizing of sections and preliminary structural verification

3.2.5) Estimation of bending moments in columns



$$M_{top,1} = 0.3V_{1,column}h_1 = 0.3 \cdot 152.7 \text{ kN} \cdot 3.27 \text{ m} = 149.8 \text{ kNm}$$

$$M_{top,2} = 0.5V_{2,column}h_2 = 0.5 \cdot 124.9 \text{ kN} \cdot 4.2 \text{ m} = 262.3 \text{ kNm}$$

$$M_{top,3} = 0.5V_{3,column}h_3 = 0.5 \cdot 61.5 \text{ kN} \cdot 4.2 \text{ m} = 129.2 \text{ kNm}$$

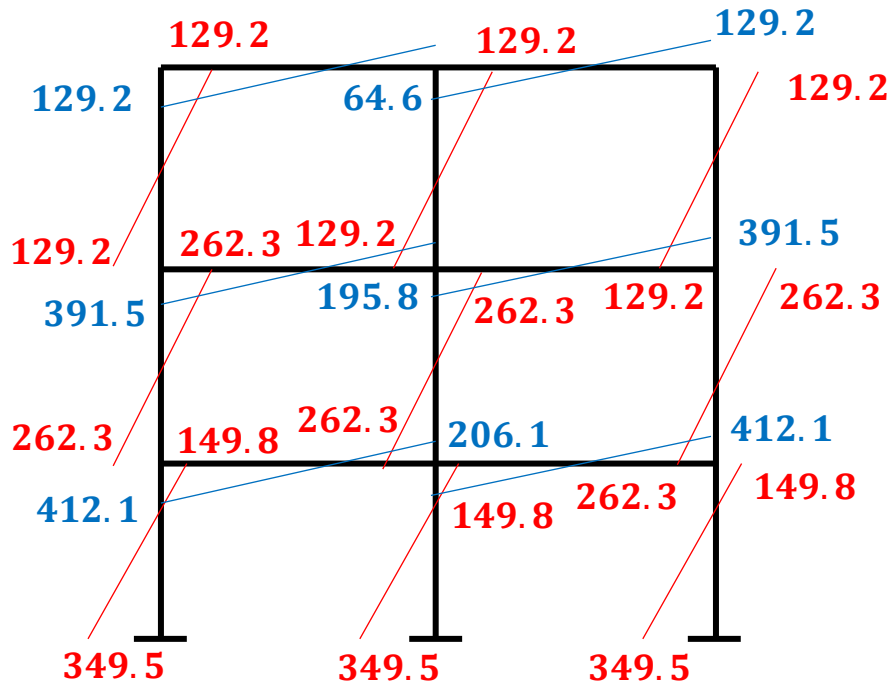
$$M_{base,1} = 0.7V_{1,column}h_1 = 0.7 \cdot 152.7 \text{ kN} \cdot 3.27 \text{ m} = 349.5 \text{ kNm}$$

$$M_{base,2} = 0.5V_{2,column}h_2 = 0.5 \cdot 124.9 \text{ kN} \cdot 4.2 \text{ m} = 262.3 \text{ kNm}$$

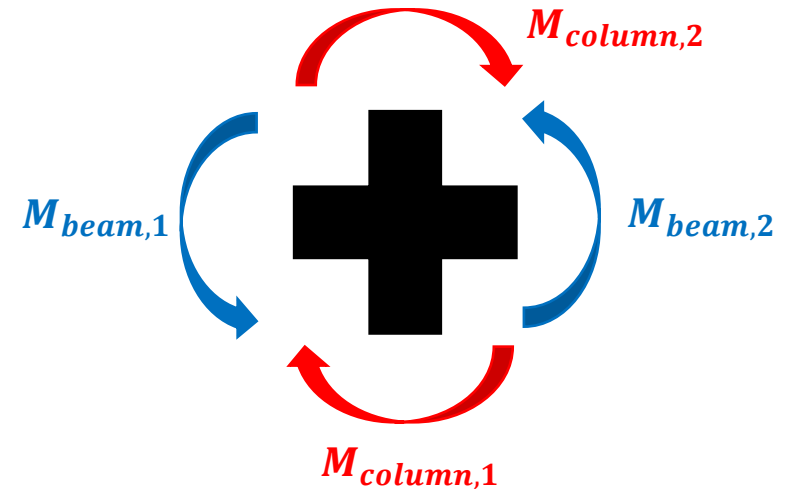
$$M_{base,3} = 0.5V_{3,column}h_3 = 0.5 \cdot 61.5 \text{ kN} \cdot 4.2 \text{ m} = 129.2 \text{ kNm}$$

3) Sizing of sections and preliminary structural verification

3.2.6) Estimation of bending moments in beams



Rotational equilibrium (internal node)

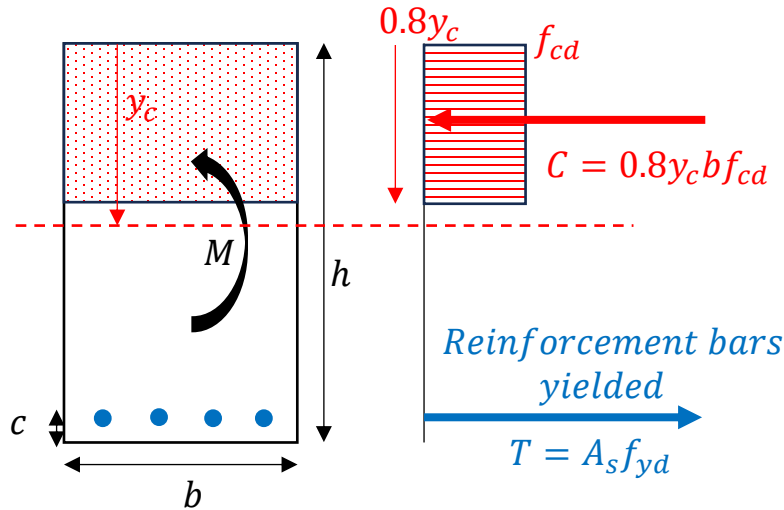


$$M_{beam,1} = M_{beam,2} = \frac{M_{column,1} + M_{column,2}}{2}$$

* In the examined case, the gravity loads associated with the seismic actions are not taken into account because the slabs do not transfer their loads to the beams of the examined frame. This is a simplification used in the design phase. A more accurate evaluation will be done with the structural model implemented in a structural software.

3) Sizing of sections and preliminary structural verification

3.2.7) Section sizing of the beams



Design assumptions

Steel bars in compression are neglected

$$b = 400 \text{ mm}$$

$$c = 30 \text{ mm} \Rightarrow \delta = \frac{c}{h} = 0.05$$

$$\xi = \frac{y_c}{h} = 0.15 \text{ for ductility reasons}$$

Rotational equilibrium around the bars

$$b \cdot 0.8y_c \cdot f_{cd} \cdot (h - c - 0.4y_c) = M_{beam}$$

Divide the first and second part of the rotational equilibrium by the quantity $bh^2 f_{cd}$

$$\frac{b \cdot 0.8y_c \cdot f_{cd} \cdot (h - c - 0.4y_c)}{bh^2 f_{cd}} = \frac{M_{beam}}{bh^2 f_{cd}}$$

Rotational equilibrium in dimensionless form

$$0.8\xi \cdot (1 - \delta - 0.4\xi) = \frac{M_{beam}}{bh^2 f_{cd}}$$

$$h_{min} = \sqrt{\frac{1}{0.8\xi \cdot (1 - \delta - 0.4\xi)} \cdot \frac{M_{beam}}{b \cdot f_{cd}}}$$

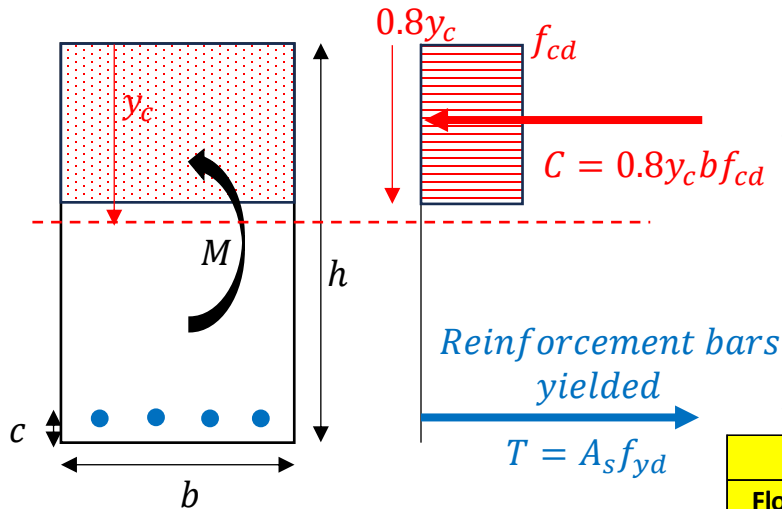
Beam	M (kNm)	b (mm)	h _{min} (mm)	h (mm)
Floor 1	412.1	400.0	697.4	800.0
Floor 2	391.5	400.0	679.8	800.0
Floor 3	129.2	400.0	390.5	800.0

All the beams of the seismic frames are 40x80

3) Sizing of sections and preliminary structural verification

3.2.7) Section sizing of the beams

In this phase, a preliminary estimation of the bars needed for the reinforcement of the concrete beams can be made.



Assumption that the lever arm between the reinforcement bars and the centroid of the compressed section is equal to $0.9(h - c)$

$$A_{s,min} = \frac{M_{beam}}{0.9(h - c)f_{yd}}$$

$$n_{min,\varphi 20} = \frac{A_{s,min}}{\pi \frac{\varphi^2}{4}}$$

The number of bars is rounded up to n

$$M_{Rd,beam,est.} = 0.9(h - c)f_{yd} \cdot n\pi \frac{\varphi^2}{4}$$

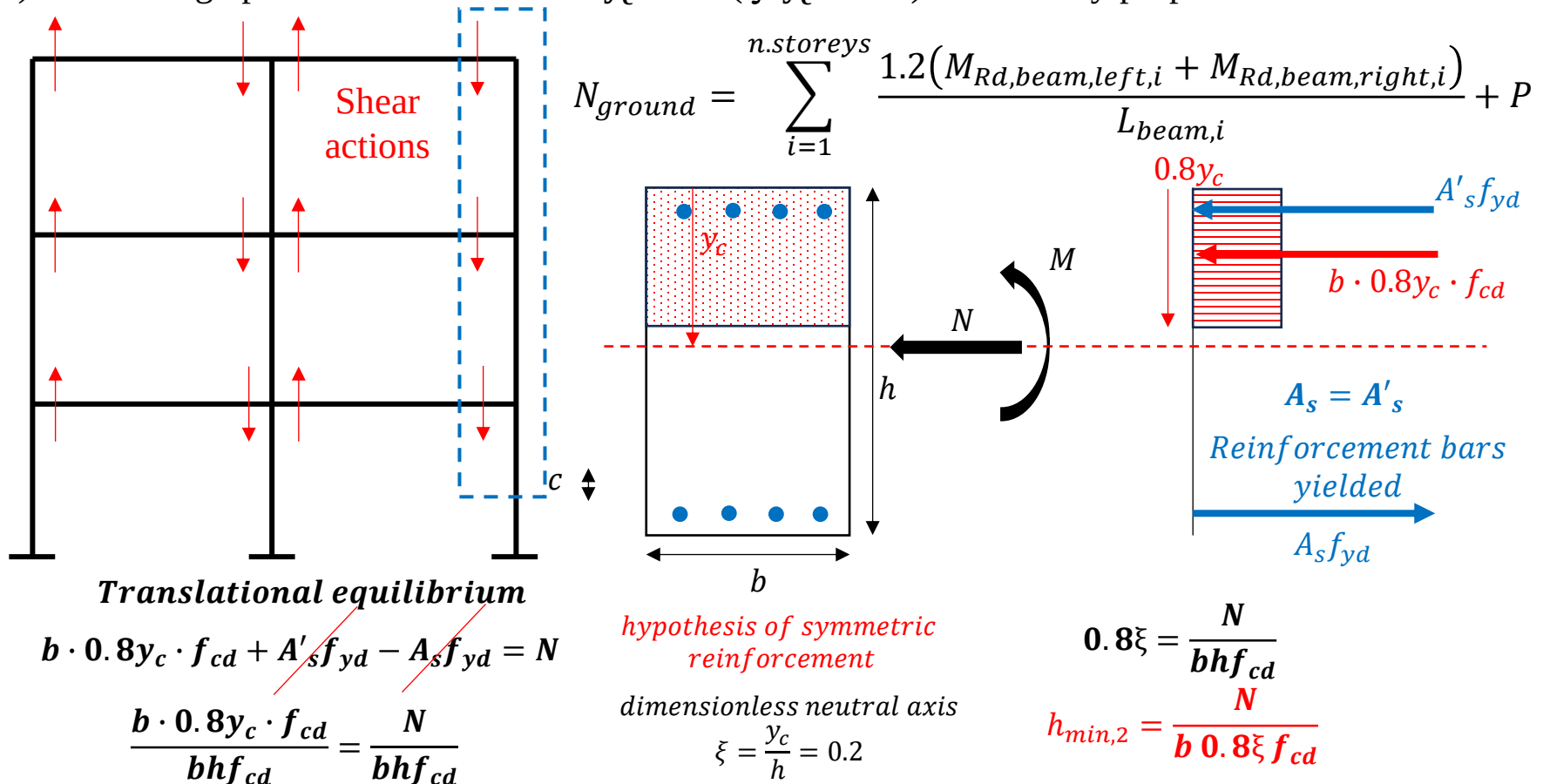
Beam	M (kNm)	b (mm)	h (mm)	$A_{s,min}$ (mm ²)	n $\varphi 20$	$M_{Rd,est.}$ (kNm)
Floor 1 beam 1 left	412.14	400.00	800.00	1367.84	5	473.05
Floor 1 beam 1 right	206.07	400.00	800.00	683.92	3	283.83
Floor 1 beam 2 left	206.07	400.00	800.00	683.92	3	283.83
Floor 1 beam 2 right	412.14	400.00	800.00	1367.84	5	473.05
Floor 2 beam 1 left	391.52	400.00	800.00	1299.43	5	473.05
Floor 2 beam 1 right	195.76	400.00	800.00	649.72	3	283.83
Floor 2 beam 2 left	195.76	400.00	800.00	649.72	3	283.83
Floor 2 beam 2 right	391.52	400.00	800.00	1299.43	5	473.05
Floor 3 beam 1 left	129.18	400.00	800.00	428.72	3	283.83
Floor 3 beam 1 right	64.59	400.00	800.00	214.36	3	283.83
Floor 3 beam 2 left	64.59	400.00	800.00	214.36	3	283.83
Floor 3 beam 2 right	129.18	400.00	800.00	428.72	3	283.83

3) Sizing of sections and preliminary structural verification

3.2.8) Section sizing of the columns

The column width is fixed at 400 mm. Three design conditions are considered for the columns.

- 1) The column depth must not be less than the depth of the connected beams. $h_{min,1} = \max\{h_{beams}\}$
- 2) In the design phase it is assumed that $y_c = 0.2h$ ($\xi = y_c/h = 0.2$) for ductility purposes.



3) Sizing of sections and preliminary structural verification

3.2.8) Section sizing of the columns

The column width is fixed at 400 mm. Two design conditions are considered for the columns.

- 1) The column depth must not be less than the depth of the connected beams.
- 2) In the design phase it is assumed that $y_c = 0.2h$ ($x = y_c/h = 0.2$) for ductility purposes.

$$h_{min,1} = \max\{h_{beams}\}$$

$$h_{min,2} = \frac{N}{b \cdot 0.8\xi \cdot f_{cd}}$$

Column floor	M (kNm)	N (kN)	b (mm)	$h_{min,1}$ (mm)	$h_{min,2}$ (mm)	h (mm)
1	349.5	827.7	400.0	800.0	652.0	800.00
2	262.3	532.3	400.0	800.0	419.4	800.00
3	129.2	229.5	400.0	800.0	180.8	800.00

3) Sizing of sections and preliminary structural verification

Floors 1 and 2

Seismic frame

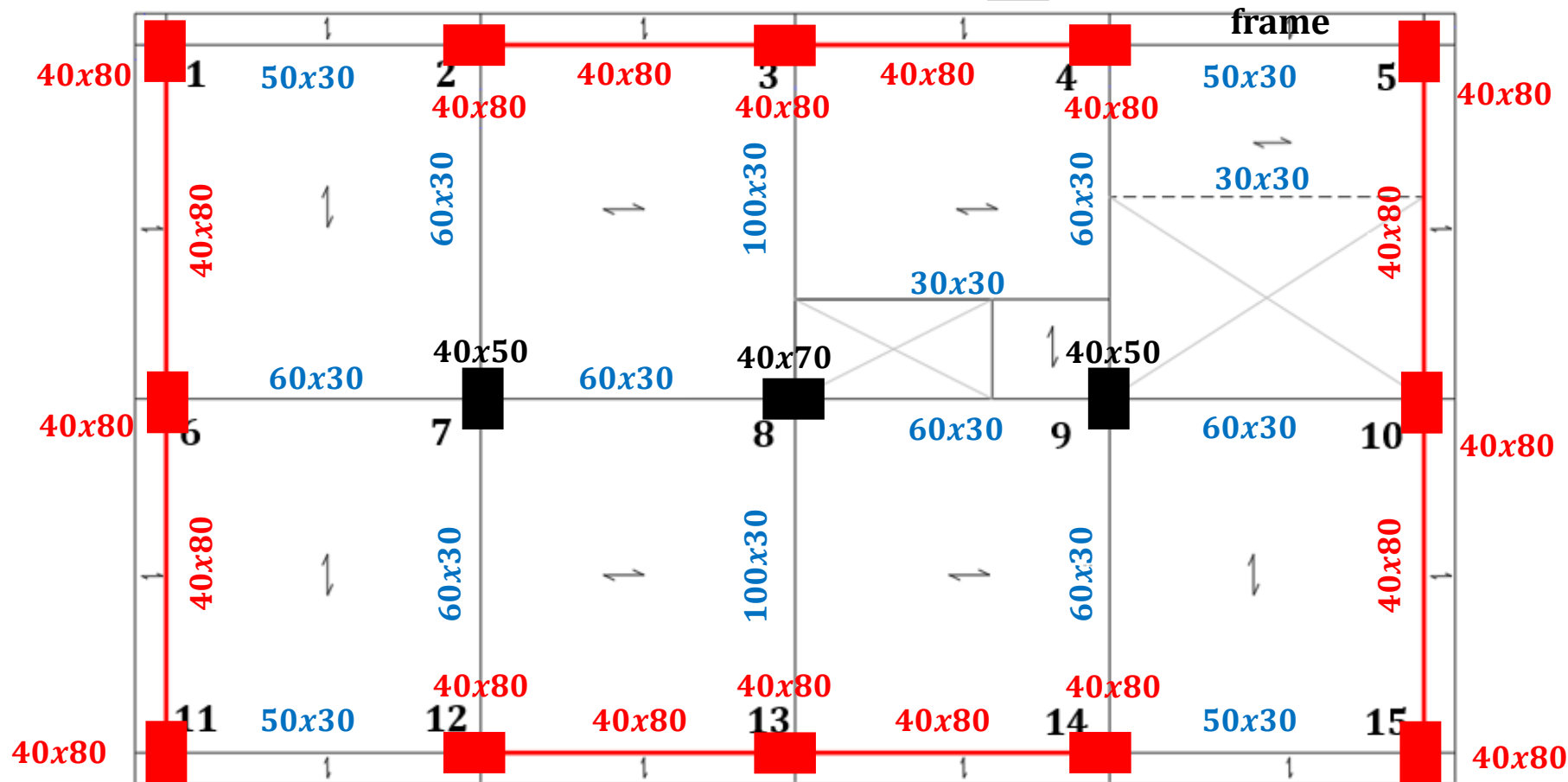


Column in seismic frame

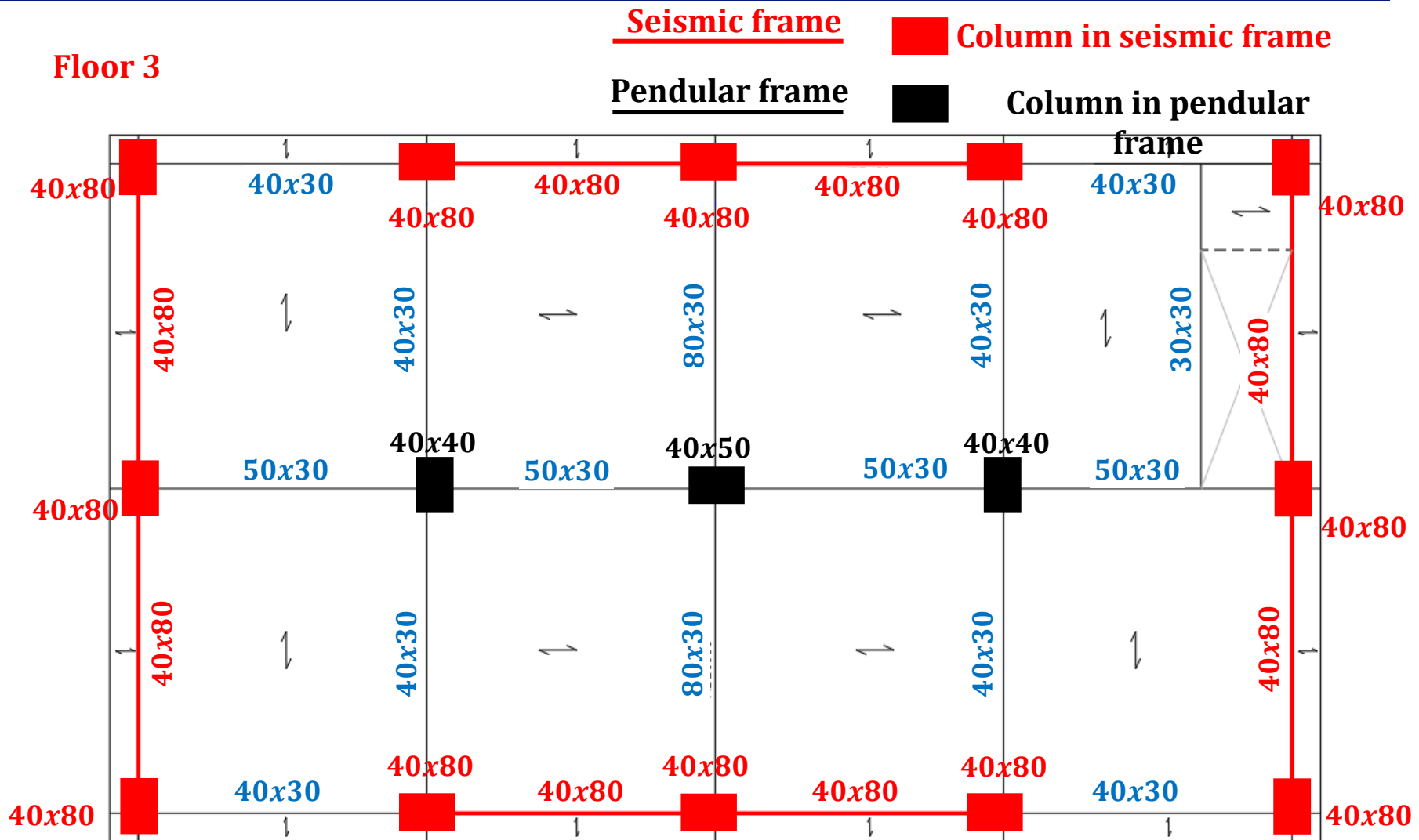
Pendular frame



Column in pendular frame



3) Sizing of sections and preliminary structural verification



3) Sizing of sections and preliminary structural verification

3.2.9) Stiffness evaluation and revised period estimation

4.2.2 Primary and secondary seismic members

(4) The total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members.

$$T_{model\ 1} = 2\pi \sqrt{\frac{m}{k_{model\ 1}}} \quad T_{model\ 2} = 2\pi \sqrt{\frac{m}{k_{model\ 2}}}$$

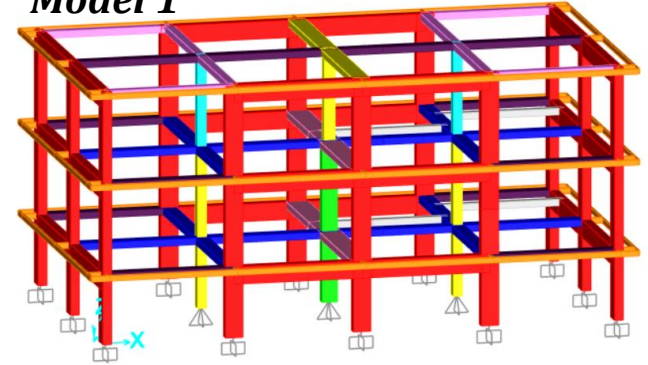
$$\frac{T_{model\ 1}}{T_{model\ 2}} = \frac{2\pi \sqrt{\frac{m}{k_{model\ 1}}}}{2\pi \sqrt{\frac{m}{k_{model\ 2}}}} = \sqrt{\frac{k_{model\ 2}}{k_{model\ 1}}}$$

$$\frac{k_{model\ 1}}{k_{model\ 2}} = \left(\frac{T_{model\ 2}}{T_{model\ 1}} \right)^2$$

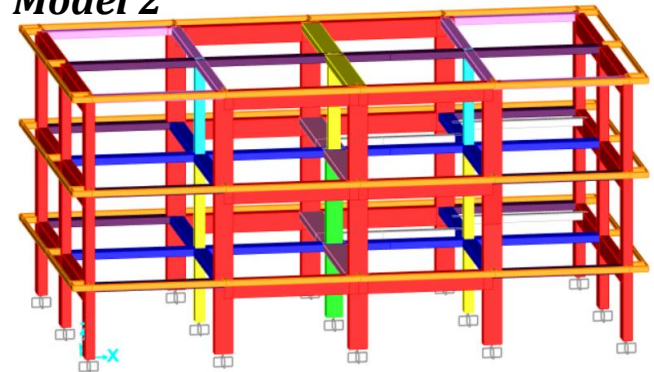
First vibration period in line with the initial prevision

	SAP2000 model with hinges at inner column bases and hidden beams (model 1)	SAP2000 model with fixed constraints (model 2)	$k_{model\ 1}/k_{model\ 2}$
T_1 (s)	0.668	0.617	0.853
T_2 (s)	0.659	0.616	0.874

Model 1



Model 2

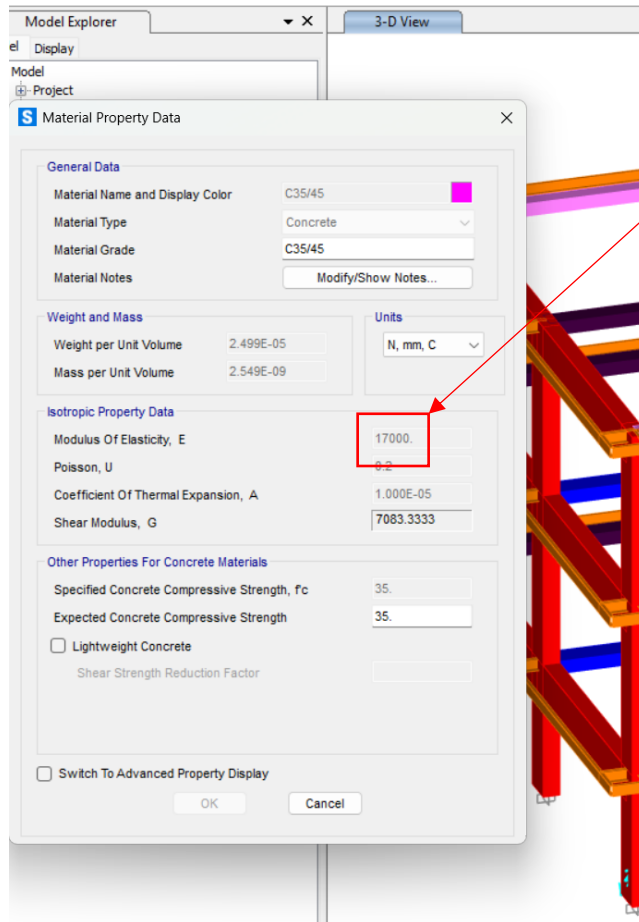


$$\frac{k_{model\ 1}}{k_{model\ 2}} > 0.85$$

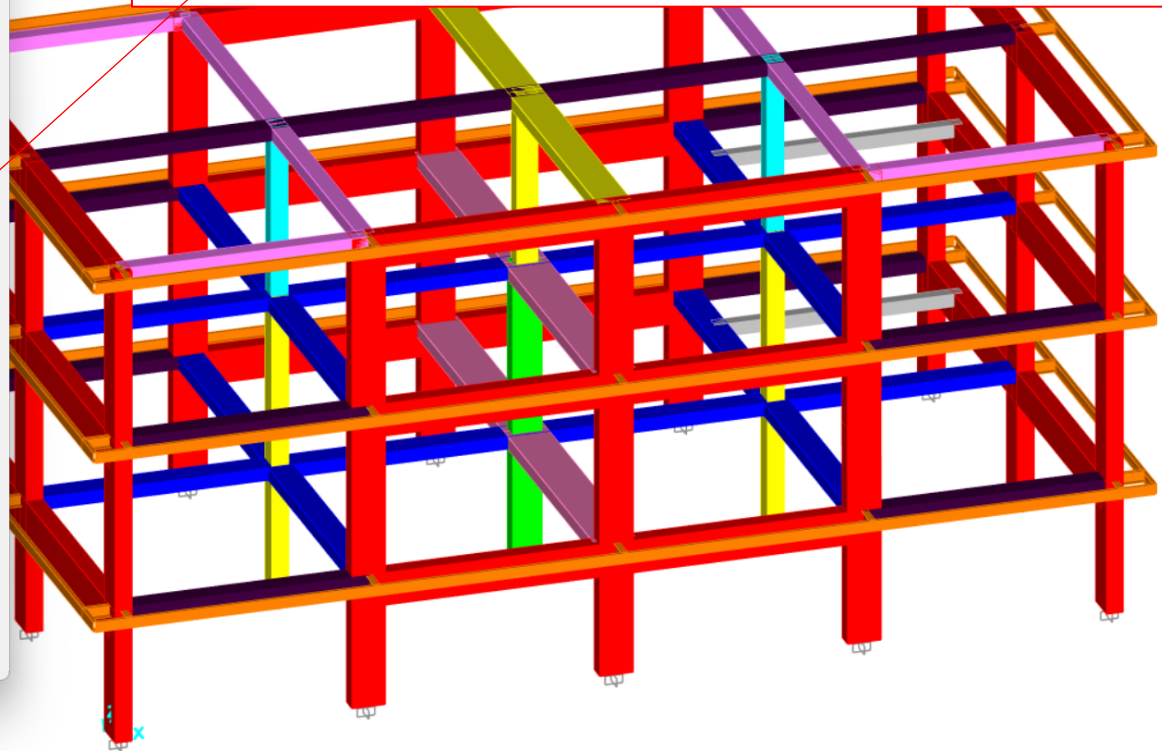


4) Detailed structural analysis

SAP2000 model (material properties)



Please note that the use of a simplified cracked stiffness is required for the analysis (50% reduction)

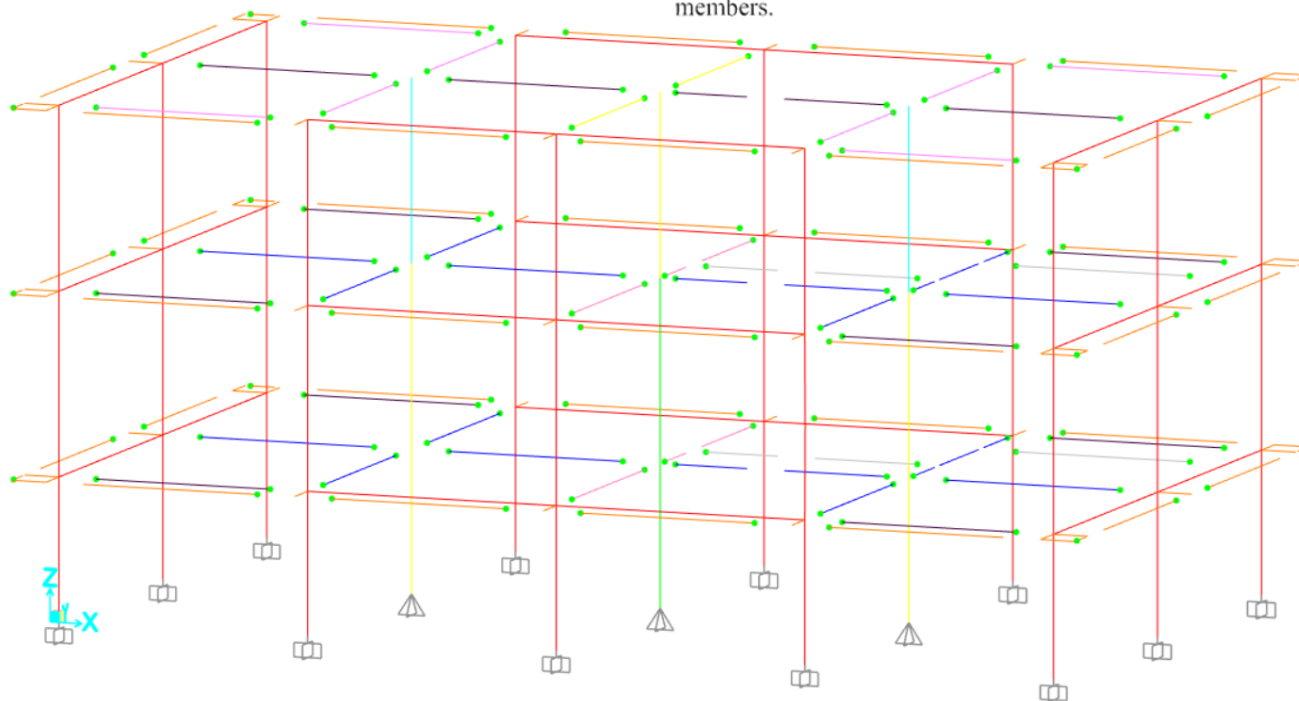


4) Detailed structural analysis

SAP2000 model (bending releases applied at the ends of secondary beams and at the base of secondary columns)

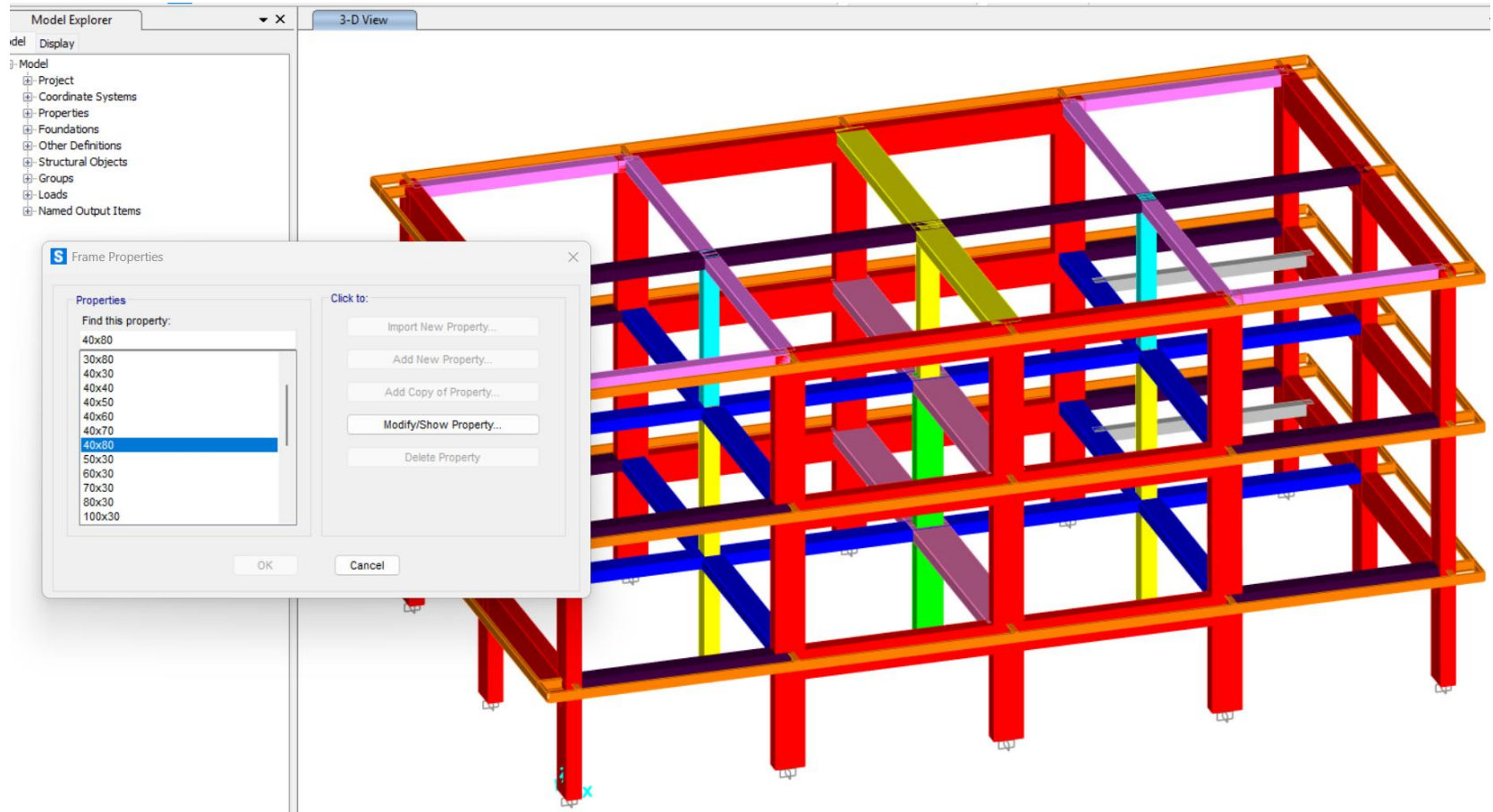
4.2.2 Primary and secondary seismic members

(1)P A certain number of structural members (e.g. beams and/or columns) may be designated as “secondary” seismic members (or elements), not forming part of the seismic action resisting system of the building. The strength and stiffness of these elements against seismic actions shall be neglected. They do not need to conform to the requirements of Sections 5 to 9. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition. Due allowance of 2nd order effects (P-Δ effects) should be made in the design of these members.



4) Detailed structural analysis

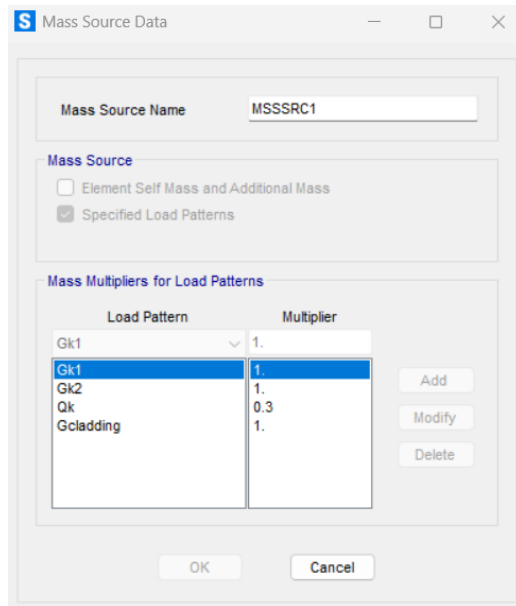
SAP2000 model (frame sections)



4) Detailed structural analysis

SAP2000 model (masses and loads)

Masses

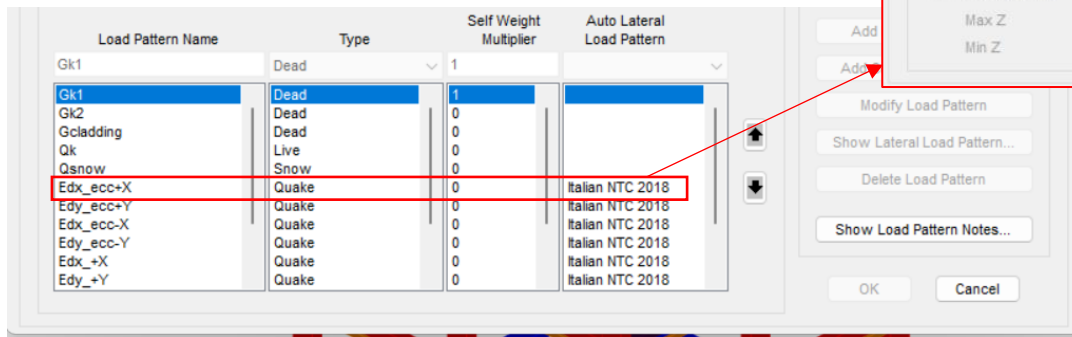


Mass Source Data dialog box. Mass Source Name: MSSSRC1. Mass Source: ☐ Element Self Mass and Additional Mass, ☒ Specified Load Patterns. Mass Multipliers for Load Patterns:

Load Pattern	Multiplier
Gk1	1.
Gk2	1.
Qk	0.3
Gcladding	1.

Buttons: Add, Modify, Delete, OK, Cancel.

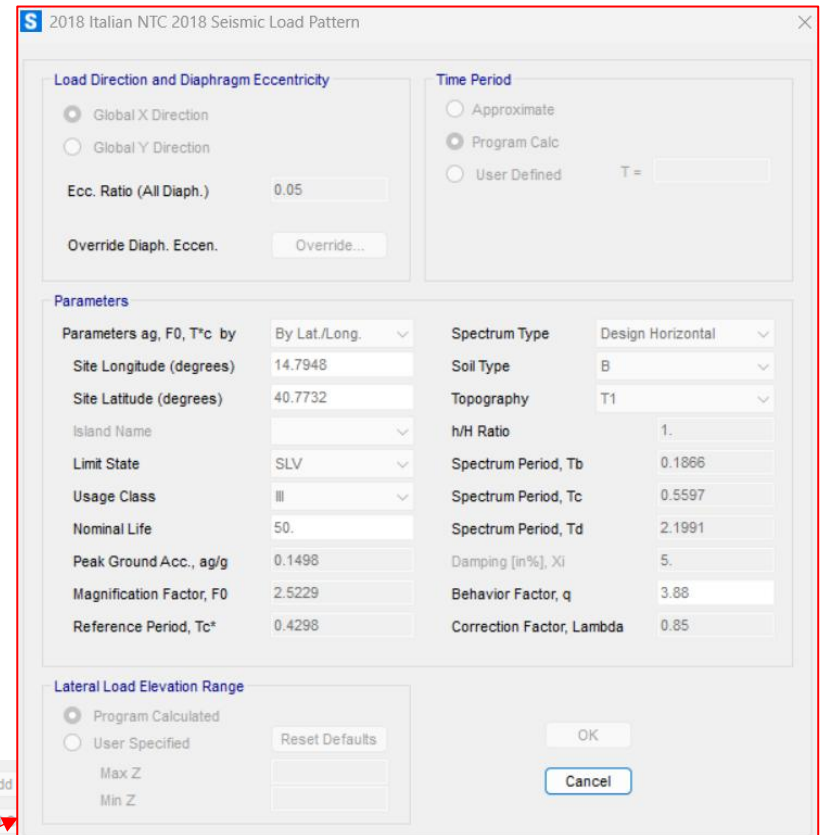
Load cases



Load Pattern List dialog box. Table:

Load Pattern Name	Type	Self Weight Multiplier	Auto Lateral Load Pattern
Gk1	Dead	1	
Gk1	Dead	0	
Gk2	Dead	0	
Gcladding	Dead	0	
Qk	Live	0	
Qsnow	Snow	0	
Edx_ecc+X	Quake	0	Italian NTC 2018
Edy_ecc+Y	Quake	0	Italian NTC 2018
Edx_ecc-X	Quake	0	Italian NTC 2018
Edy_ecc-Y	Quake	0	Italian NTC 2018
Edx_+X	Quake	0	Italian NTC 2018
Edy_+Y	Quake	0	Italian NTC 2018

Buttons: Add, Modify Load Pattern, Show Lateral Load Pattern..., Delete Load Pattern, Show Load Pattern Notes..., OK, Cancel.

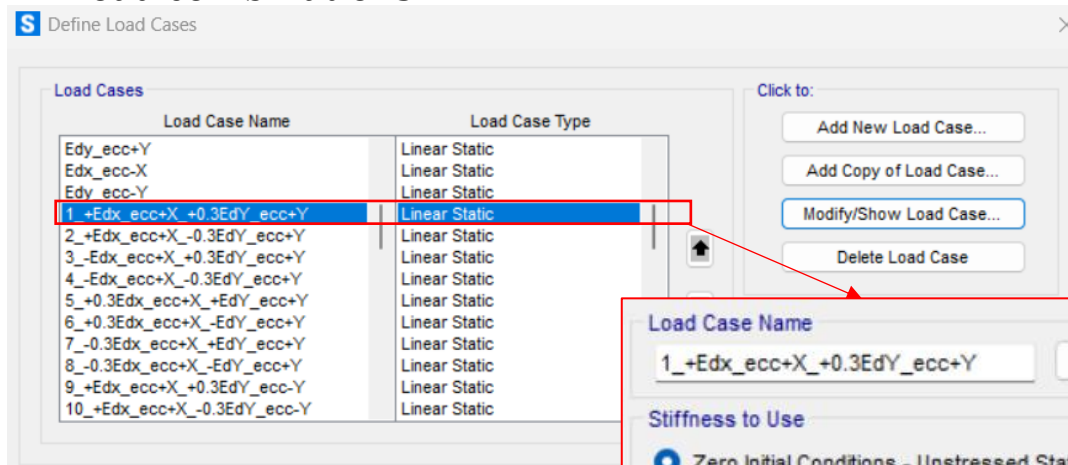


2018 Italian NTC 2018 Seismic Load Pattern dialog box. Load Direction and Diaphragm Eccentricity: ☒ Global X Direction, ☐ Global Y Direction. Ecc. Ratio (All Diaph.): 0.05. Override Diaph. Eccen.: Override... Time Period: ☐ Approximate, ☒ Program Calc, ☐ User Defined. T = Parameters: Parameters ag, F0, T*c by: By Lat./Long. Site Longitude (degrees): 14.7948, Site Latitude (degrees): 40.7732, Island Name: Limit State: SLV, Usage Class: III, Nominal Life: 50, Peak Ground Acc., ag/g: 0.1498, Magnification Factor, F0: 2.5229, Reference Period, Tc*: 0.4298. Spectrum Type: Design Horizontal, Soil Type: B, Topography: T1, h/H Ratio: 1, Spectrum Period, Tb: 0.1866, Spectrum Period, Tc: 0.5597, Spectrum Period, Td: 2.1991, Damping [in%], Xi: 5, Behavior Factor, q: 3.88, Correction Factor, Lambda: 0.85. Lateral Load Elevation Range: ☒ Program Calculated, ☐ User Specified. Max Z, Min Z. Buttons: OK, Cancel.

4) Detailed structural analysis

SAP2000 model (load combinations)

Load combinations



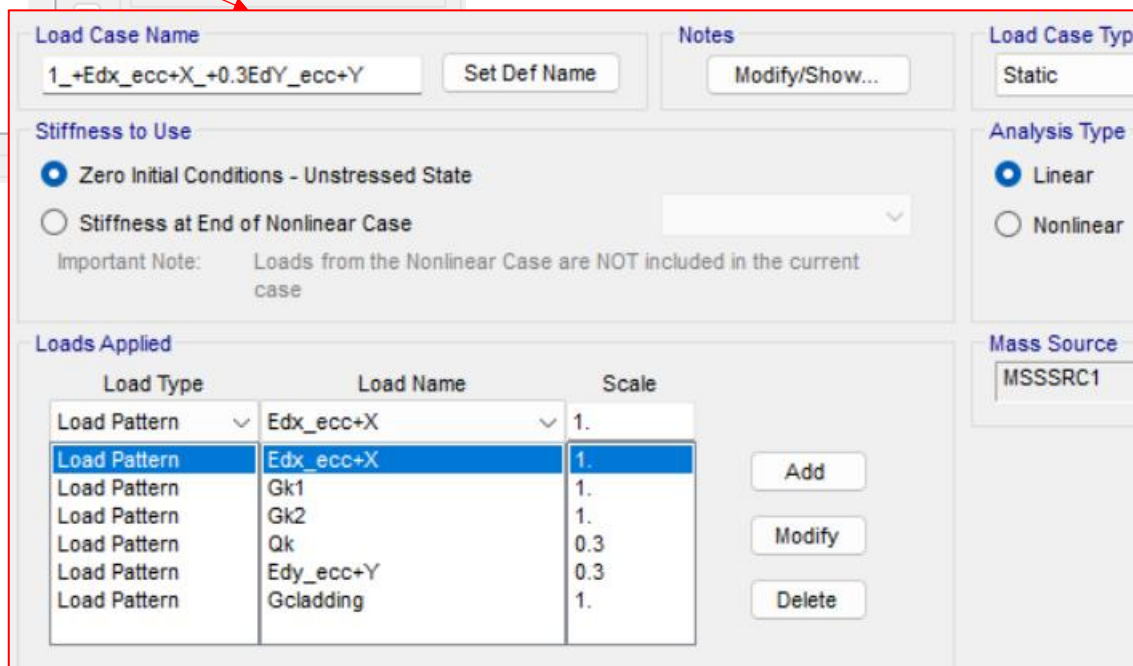
ULS Combinations

$$\gamma_{G1} G_1 + \gamma_{G2} G_2 + \gamma_Q Q_1$$

$$G_1 + G_2 + \psi_{21} Q_1 + E_X + 0.3 E_Y$$

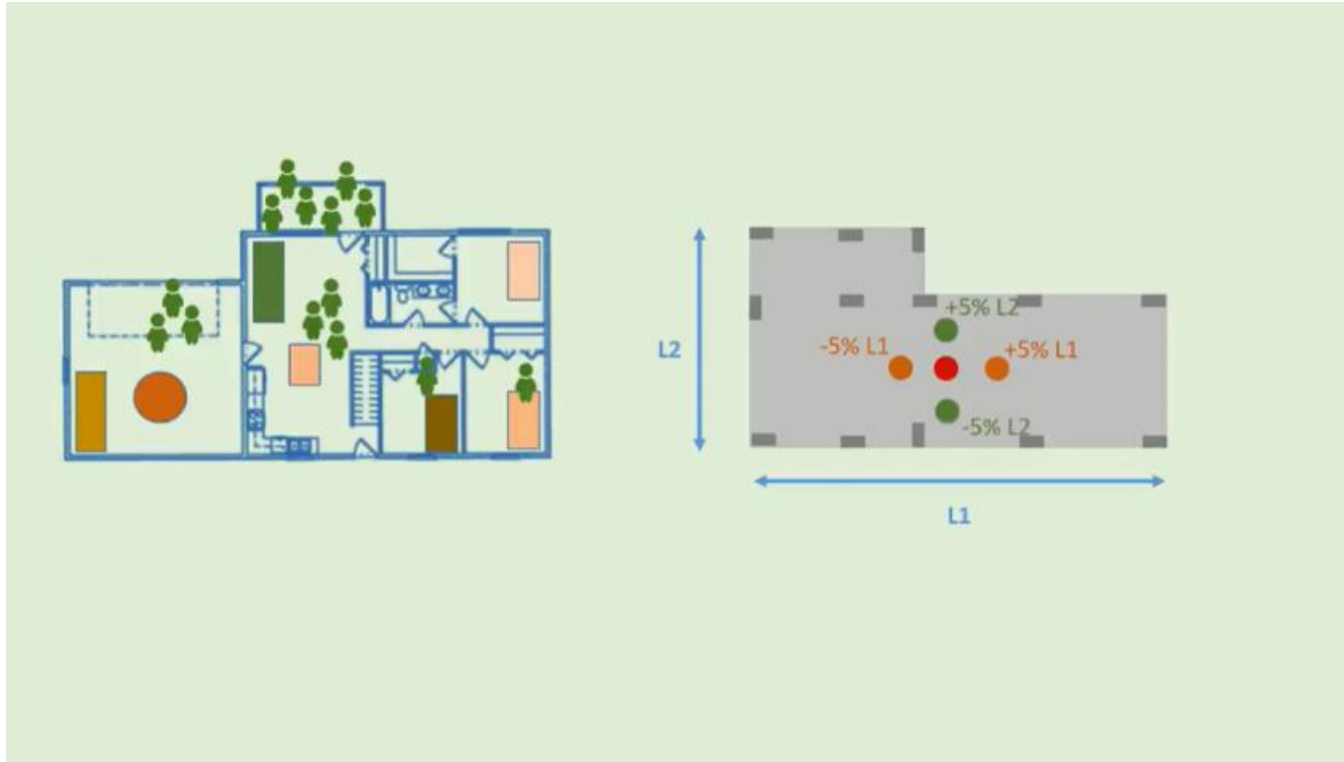
$$G_1 + G_2 + \psi_{21} Q_1 + 0.3 E_X + E_Y$$

32 seismic combinations



4) Detailed structural analysis

Load combinations

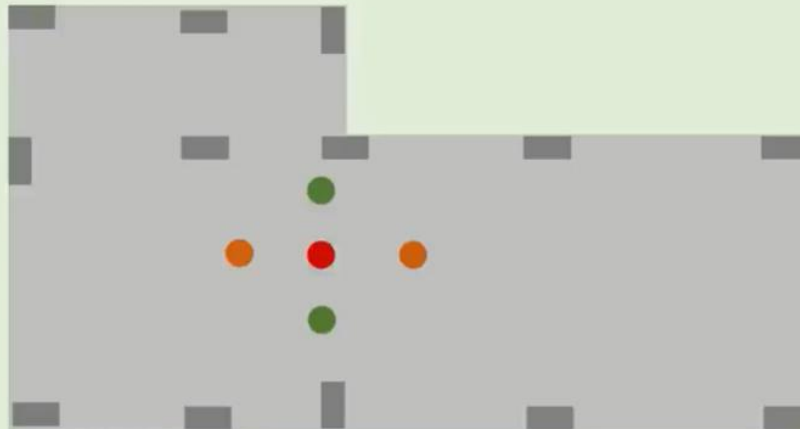


Video created by Marco De Pisapia: <https://www.youtube.com/watch?v=0dvvCym-NY8>

4) Detailed structural analysis

Load combinations

[Ex; 30% Ey]



Video created by Marco De Pisapia: <https://www.youtube.com/watch?v=0dvbCym-NY8>

4) Detailed structural analysis

Load combinations

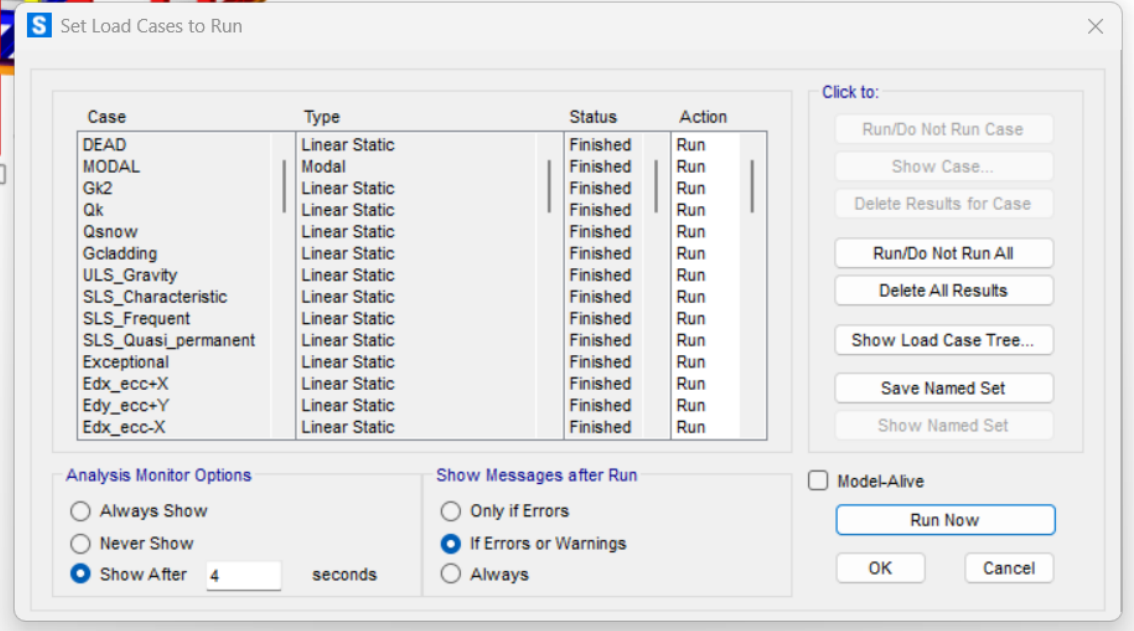
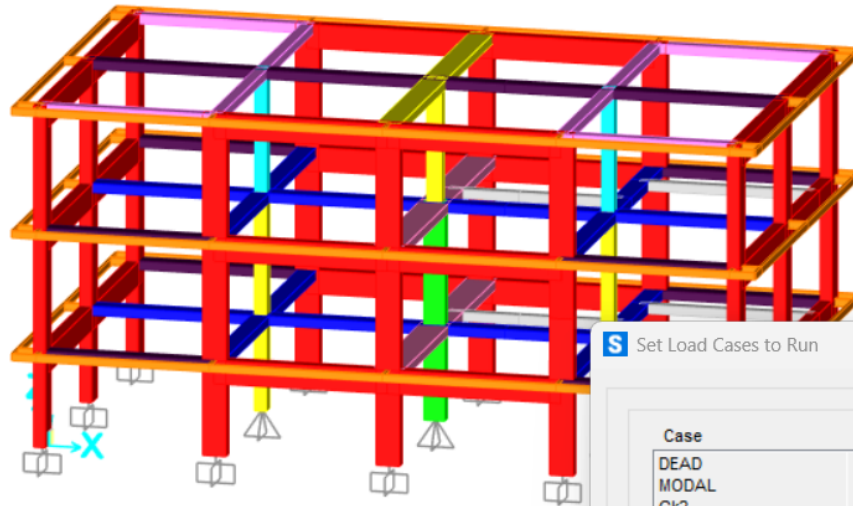
E_x : seismic action along x direction
 E_y : seismic action along y direction
 e_x : 5% eccentricity along x direction
 e_y : 5% eccentricity along y direction

Azione sismica principale	segno	eccentricità	Azione sismica secondaria	eccentricità	N° comb.
E_x	+	$+ e_y$	$+ 0.3 E_y$	$+ e_x$	1
				$- e_x$	2
			$- 0.3 E_y$	$+ e_x$	3
				$- e_x$	4
		$- e_y$	$+ 0.3 E_y$	$+ e_x$	5
				$- e_x$	6
			$- 0.3 E_y$	$+ e_x$	7
				$- e_x$	8
	-	$+ e_y$	$+ 0.3 E_y$	$+ e_x$	9
				$- e_x$	10
			$- 0.3 E_y$	$+ e_x$	11
				$- e_x$	12
		$- e_y$	$+ 0.3 E_y$	$+ e_x$	13
				$- e_x$	14
			$- 0.3 E_y$	$+ e_x$	15
				$- e_x$	16
E_y	+	$+ e_x$	$+ 0.3 E_x$	$+ e_y$	17
				$- e_y$	18
			$- 0.3 E_x$	$+ e_y$	19
				$- e_y$	20
		$- e_x$	$+ 0.3 E_x$	$+ e_y$	21
				$- e_y$	22
			$- 0.3 E_x$	$+ e_y$	23
				$- e_y$	24
	-	$+ e_x$	$+ 0.3 E_x$	$+ e_y$	25
				$- e_y$	26
			$- 0.3 E_x$	$+ e_y$	27
				$- e_y$	28
		$- e_x$	$+ 0.3 E_x$	$+ e_y$	29
				$- e_y$	30
			$- 0.3 E_x$	$+ e_y$	31
				$- e_y$	32

Table created by prof. Gherzi

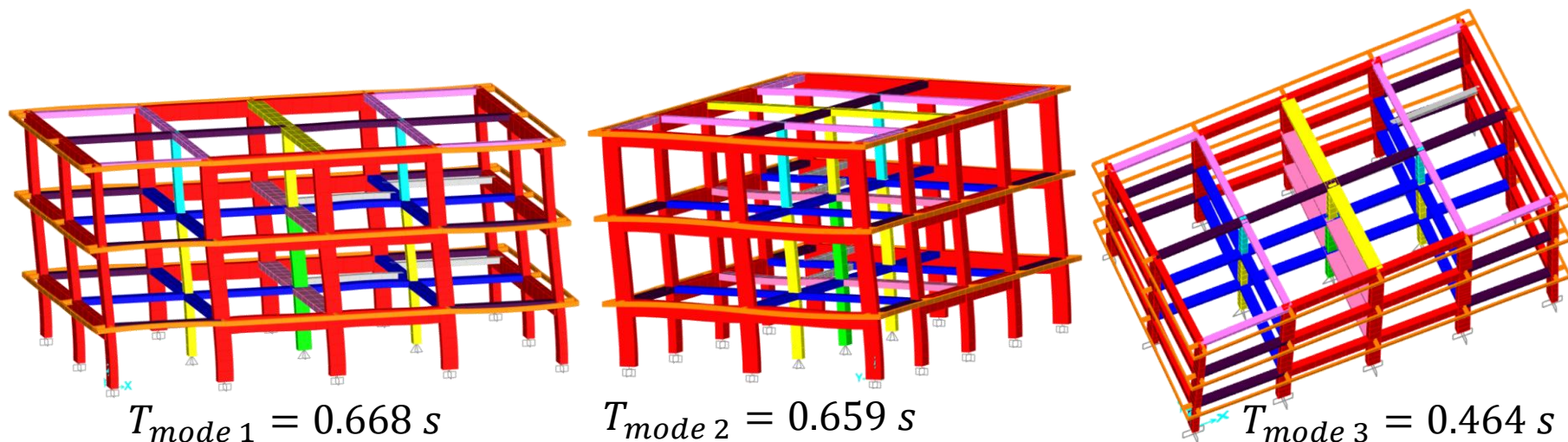
4) Detailed structural analysis

SAP2000 model (run the simulations)



4) Detailed structural analysis

SAP2000 model (modal properties)



Modal participating mass ratios

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless	UY Unitless	UZ Unitless	SumUX Unitless	SumUY Unitless	SumUZ Unitless	RX Unitless	RY Unitless	RZ Unitless	SumRX Unitless	SumRY Unitless	SumRZ Unitless
MODAL	Mode	1	0.668177	0.78077	0.0009	7.432E-10	0.78077	0.0009	7.432E-10	9.464E-05	0.04081	3.377E-05	9.464E-05	0.04081	3.377E-05
MODAL	Mode	2	0.659453	0.00094	0.76831	1.786E-05	0.7817	0.76921	1.786E-05	0.07895	3.906E-05	0.00973	0.07904	0.04084	0.00976
MODAL	Mode	3	0.464136	3.743E-06	0.00967	4.699E-07	0.78171	0.77888	1.833E-05	0.00185	2.7E-05	0.77769	0.08089	0.04087	0.78745
MODAL	Mode	4	0.254357	2.372E-05	0.00105	0.0087	0.78173	0.77993	0.00872	0.0012	0.00406	8.594E-05	0.08209	0.04493	0.78754
MODAL	Mode	5	0.251494	1.389E-05	0.00178	0.01478	0.78175	0.78172	0.0235	0.00481	0.00016	2.741E-08	0.08689	0.04509	0.78754
MODAL	Mode	6	0.249366	6.936E-07	4.518E-05	0.03182	0.78175	0.78176	0.05532	0.00141	0.00034	6.312E-06	0.0883	0.04543	0.78755
MODAL	Mode	7	0.219212	0.00051	1.4E-05	0.01128	0.78226	0.78178	0.0666	0.00473	0.01105	1.574E-06	0.09303	0.05647	0.78755
MODAL	Mode	8	0.218129	2.62E-05	1.703E-05	0.00789	0.78228	0.78179	0.07449	0.00368	0.00479	7.71E-08	0.09671	0.06127	0.78755
MODAL	Mode	9	0.20814	1.588E-05	0.13348	1.988E-05	0.7823	0.91528	0.07451	0.1737	5.068E-08	0.00049	0.27041	0.06127	0.78804
MODAL	Mode	10	0.207	0.13564	2.03E-05	2.066E-05	0.91794	0.9153	0.07453	2.394E-06	0.08787	3.013E-06	0.27041	0.14914	0.78804
MODAL	Mode	11	0.149834	4.065E-06	0.0003	2.396E-06	0.91794	0.9156	0.07453	0.00057	1.039E-06	0.13048	0.27099	0.14914	0.91852
MODAL	Mode	12	0.119176	0.00011	0.08213	3.36E-05	0.91805	0.99773	0.07456	0.05696	8.06E-05	0.00037	0.32795	0.14922	0.9189

4) Detailed structural analysis (P-Δ effects check)

(2) Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0,10 \quad (4.28)$$

where

θ is the interstorey drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;

d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with 4.3.4;

V_{tot} is the total seismic storey shear; and

h is the interstorey height.

(3) If $0,1 < \theta \leq 0,2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

(4)P The value of the coefficient θ shall not exceed 0,3.

(5) If design action effects E_d are obtained through a nonlinear method of analysis (see 4.3.3.4), (1)P of this subclause should be applied in terms of forces only for brittle elements. For dissipative zones, which are designed and detailed for ductility, the resistance condition, expression (4.27), should be satisfied in terms of member deformations (e.g. plastic hinge or chord rotations), with appropriate material partial factors applied on member deformation capacities (see also EN 1992-1-1:2004, 5.7(2); 5.7(4)P).

	$\theta_{level 1}$	$\theta_{level 2}$	$\theta_{level 3}$
1_+Edx_ecc+X_+0.3EdY_ecc+Y	0.0488	0.0478	0.0262
2_+Edx_ecc+X_-0.3EdY_ecc+Y	0.0475	0.0519	0.0261
3_-Edx_ecc+X_+0.3EdY_ecc+Y	0.0487	0.0477	0.0261
4_-Edx_ecc+X_-0.3EdY_ecc+Y	0.0475	0.0522	0.0261
9_+Edx_ecc+X_+0.3EdY_ecc-Y	0.0489	0.0479	0.0261
10_+Edx_ecc+X_-0.3EdY_ecc-Y	0.0475	0.0522	0.0261
11_-Edx_ecc+X_+0.3EdY_ecc-Y	0.0488	0.0477	0.0261
12_-Edx_ecc+X_-0.3EdY_ecc-Y	0.0475	0.0519	0.0261
17_+Edx_ecc-X_+0.3EdY_ecc-Y	0.0488	0.0477	0.0262
18_+Edx_ecc-X_-0.3EdY_ecc-Y	0.0476	0.0525	0.0260
19_-Edx_ecc-X_+0.3EdY_ecc-Y	0.0489	0.0477	0.0262
20_-Edx_ecc-X_-0.3EdY_ecc-Y	0.0474	0.0523	0.0260
25_+Edx_ecc-X_+0.3EdY_ecc+Y	0.0487	0.0476	0.0262
26_+Edx_ecc-X_-0.3EdY_ecc+Y	0.0475	0.0524	0.0261
27_-Edx_ecc-X_+0.3EdY_ecc+Y	0.0488	0.0477	0.0262
28_-Edx_ecc-X_-0.3EdY_ecc+Y	0.0474	0.0523	0.0260

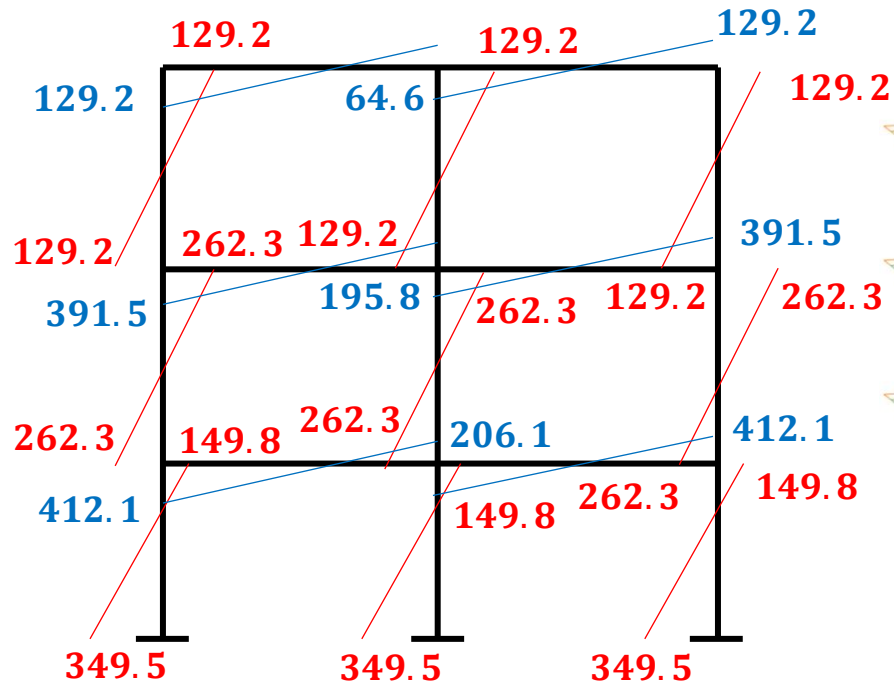
$$\vartheta = \frac{P_{tot} d_r}{V_{tot} h} < 0.1$$

$$\vartheta_{max} = 0.0525 < 0.1 \quad \checkmark$$

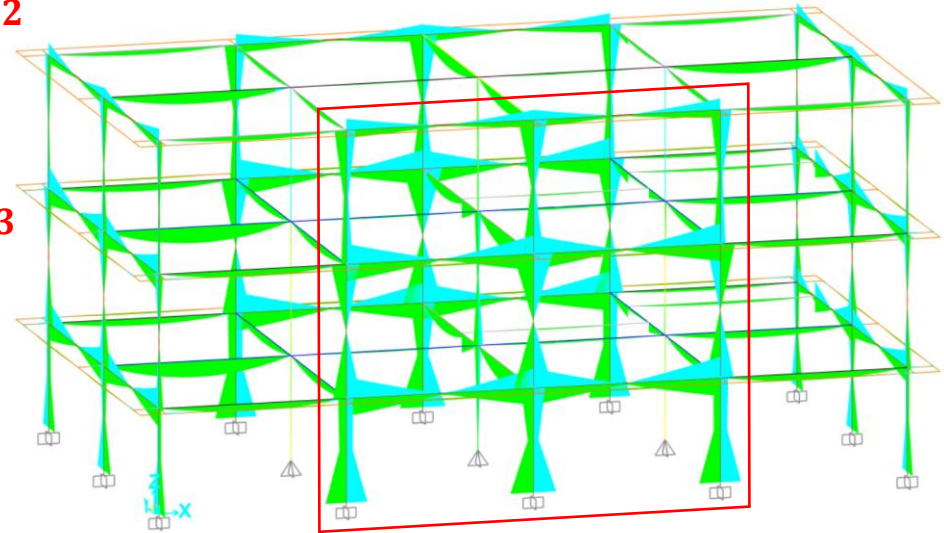
Second – order effects not taken into account

4) Detailed structural analysis

Simplified calculations

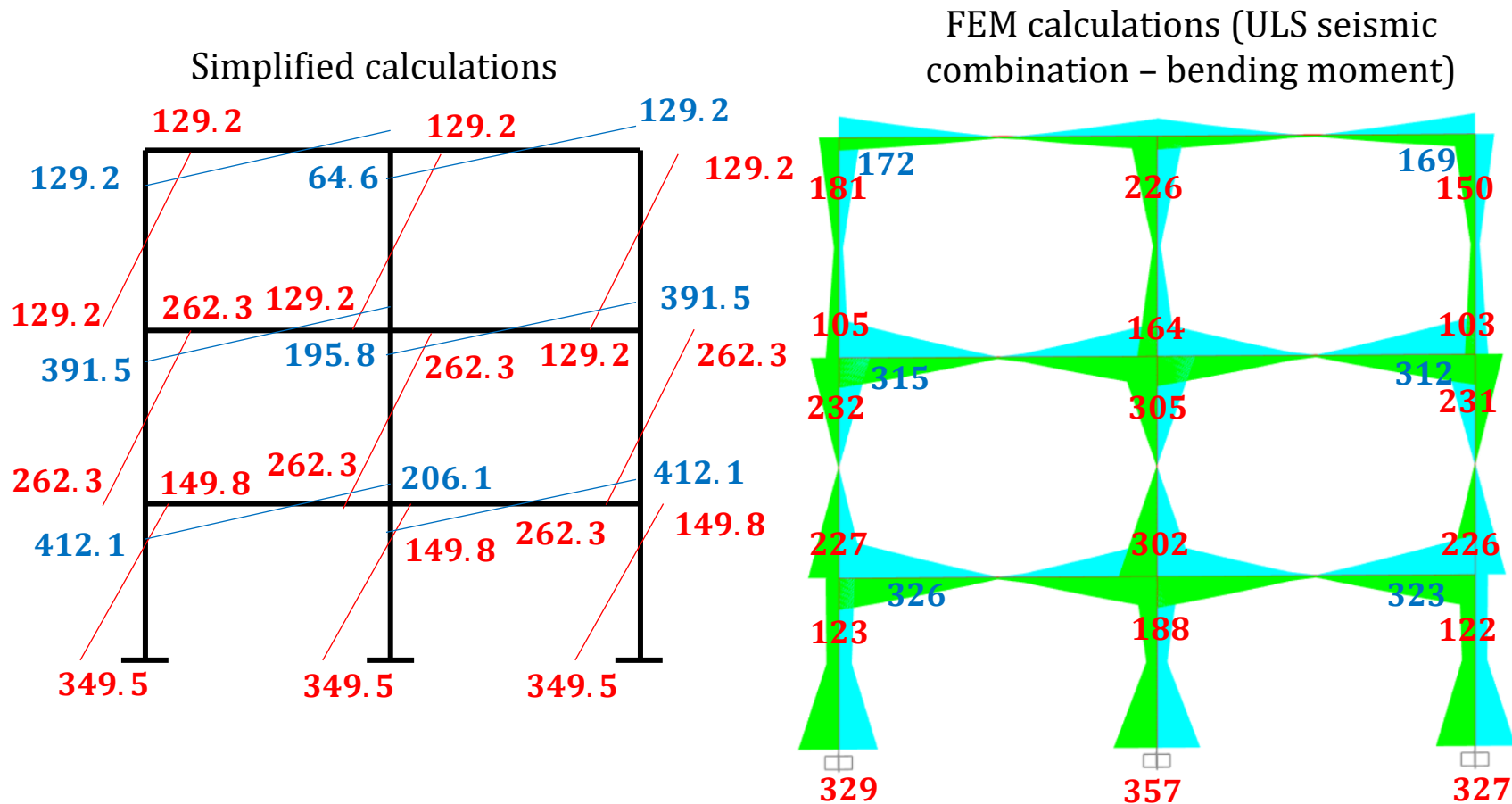


FEM calculations (ULS seismic combination – bending moment)



Focus the attention on this frame

4) Detailed structural analysis



*The observed variations are attributed to the limitations of the simplified calculations, as well as to the fact that a small portion of the shear is also transferred to the columns of the frames acting in the y-direction.

5) Detailed design (beams)

TITOLO : _____

N° figure elementari **Zoom**

N°	b [cm]	h [cm]
1	40	80

Sollecitazioni
S.L.U. Metodo n

N_{Ed} kN
M_{Ed} kNm
M_{yEd} kNm

Materiali
B500C C35/45
E_{cu} % E_{c2} %
f_{yd} N/mm² E_{cu} %
E_s N/mm² f_{cd} %
E_s/E_c f_{cc}/f_{cd} ?
E_{syd} % G_{c,adm} %
G_{s,adm} N/mm² τ_{co} %
τ_{c1} %

P.to applicazione N
☒ Centro ☐ Baricentro cls
☐ Coord.[cm] xN
yN

Tipo rottura
Lato calcestruzzo - Acciaio snervato

M_{xRd} kNm
σ_c N/mm²
σ_s N/mm²
ε_s %
ε_s %
d cm
x x/d
δ

Tipo Sezione
☒ Rettang.re ☐ Trapezi
☐ a T ☐ Circolare
☐ Rettangoli ☐ Coord.
☐ DXF

Metodo di calcolo
☒ S.L.U. ☐ S.L.U. ☐ Metodo n

Tipo flessione
☒ Retta ☐ Deviata

N° rett.
Calcola MRd Dominio M-N
L₀ cm Col. modello
M-curvatura

☐ Precompresso

TITOLO : _____

N° figure elementari **Zoom**

N°	b [cm]	h [cm]
1	40	80

Sollecitazioni
S.L.U. Metodo n

N_{Ed} kN
M_{Ed} kNm
M_{yEd} kNm

Materiali
B500C C35/45
E_{cu} % E_{c2} %
f_{yd} N/mm² E_{cu} %
E_s N/mm² f_{cd} %
E_s/E_c f_{cc}/f_{cd} ?
E_{syd} % G_{c,adm} %
G_{s,adm} N/mm² τ_{co} %
τ_{c1} %

P.to applicazione N
☒ Centro ☐ Baricentro cls
☐ Coord.[cm] xN
yN

Tipo rottura
Lato calcestruzzo - Acciaio snervato

M_{xRd} kNm
σ_c N/mm²
σ_s N/mm²
ε_s %
ε_s %
d cm
x x/d
δ

Tipo Sezione
☒ Rettang.re ☐ Trapezi
☐ a T ☐ Circolare
☐ Rettangoli ☐ Coord.
☐ DXF

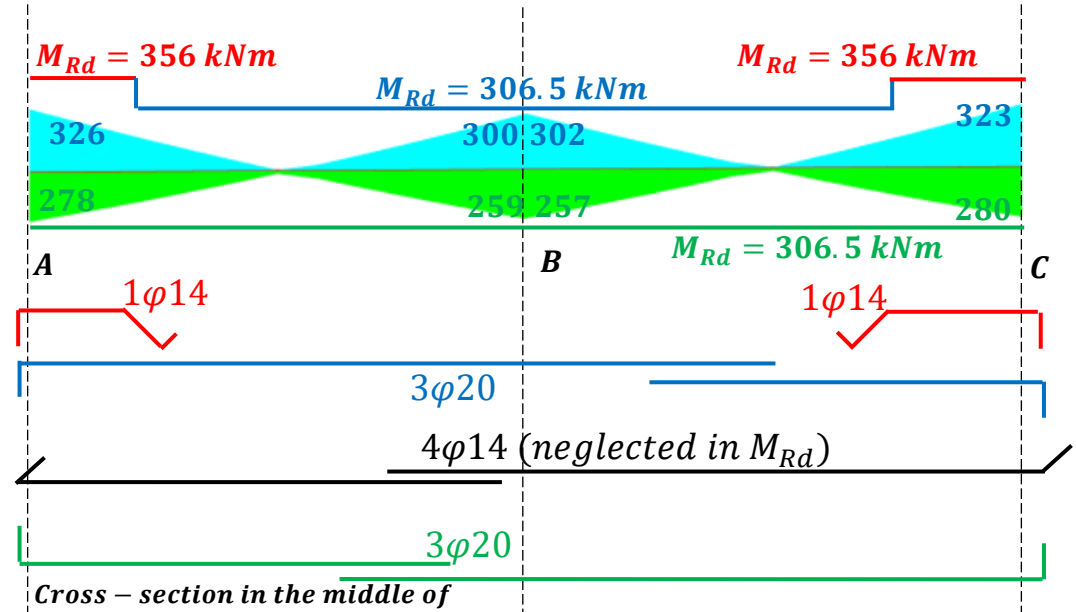
Metodo di calcolo
☒ S.L.U. ☐ S.L.U. ☐ Metodo n

Tipo flessione
☒ Retta ☐ Deviata

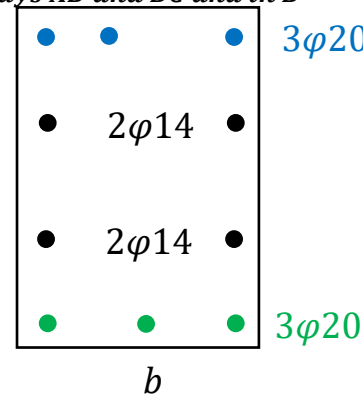
N° rett.
Calcola MRd Dominio M-N
L₀ cm Col. modello
M-curvatura

☐ Precompresso

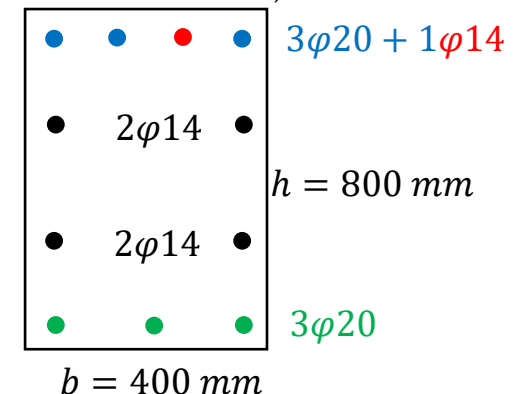
Detailed design of the beam at storey 1 (longitudinal bars)



**Cross – section in the middle of
bays AB and BC and in B**



Cross – section in A, C



5) Detailed design (beams)

Detailed design of the beam at storey 3 (longitudinal bars)

Titolo : _____

N° figure elementari 1 Zoom N° strati barre 2 Zoom

N°	b [cm]	h [cm]	N°	A _s [cm²]	d [cm]
1	40	80	1	9.42	3
			2	9.42	77

Sollecitazioni S.L.U. Metodo n

N_{Ed} 0 kN
M_{Ed} 0 kNm
M_{yEd} 0

P.to applicazione N
Centro Baricentro cls
Coord.[cm] xN 0 yN 0

Tipo rottura
Lato calcestruzzo - Acciaio snervato

M_{xRd} 306.5 kNm

Materiali B500C C35/45

ε_{su} 67.5 ‰ ε_{c2} 2 ‰
f_{yd} 434.8 N/mm² ε_{cu} 3.5 ‰
E_s 210 000 N/mm² f_{cd} 19.83 ‰
E_c/E_s 15 f_{cc}/f_{cd} 1 ‰
ε_{syd} 2.07 ‰ σ_{c,adm} 13.5 N/mm²
σ_{s,adm} 255 N/mm² τ_{co} 0.8
τ_{c1} 2.257

σ_c -19.83 N/mm²
σ_s 434.8 N/mm²
ε_c 3.5 ‰
ε_s 65.82 ‰
d 77 cm
x 3.888 x/d 0.05049
δ 0.7

Metodo di calcolo S.L.U. + S.L.U. - Metodo n

Tipo flessione Retta Deviata

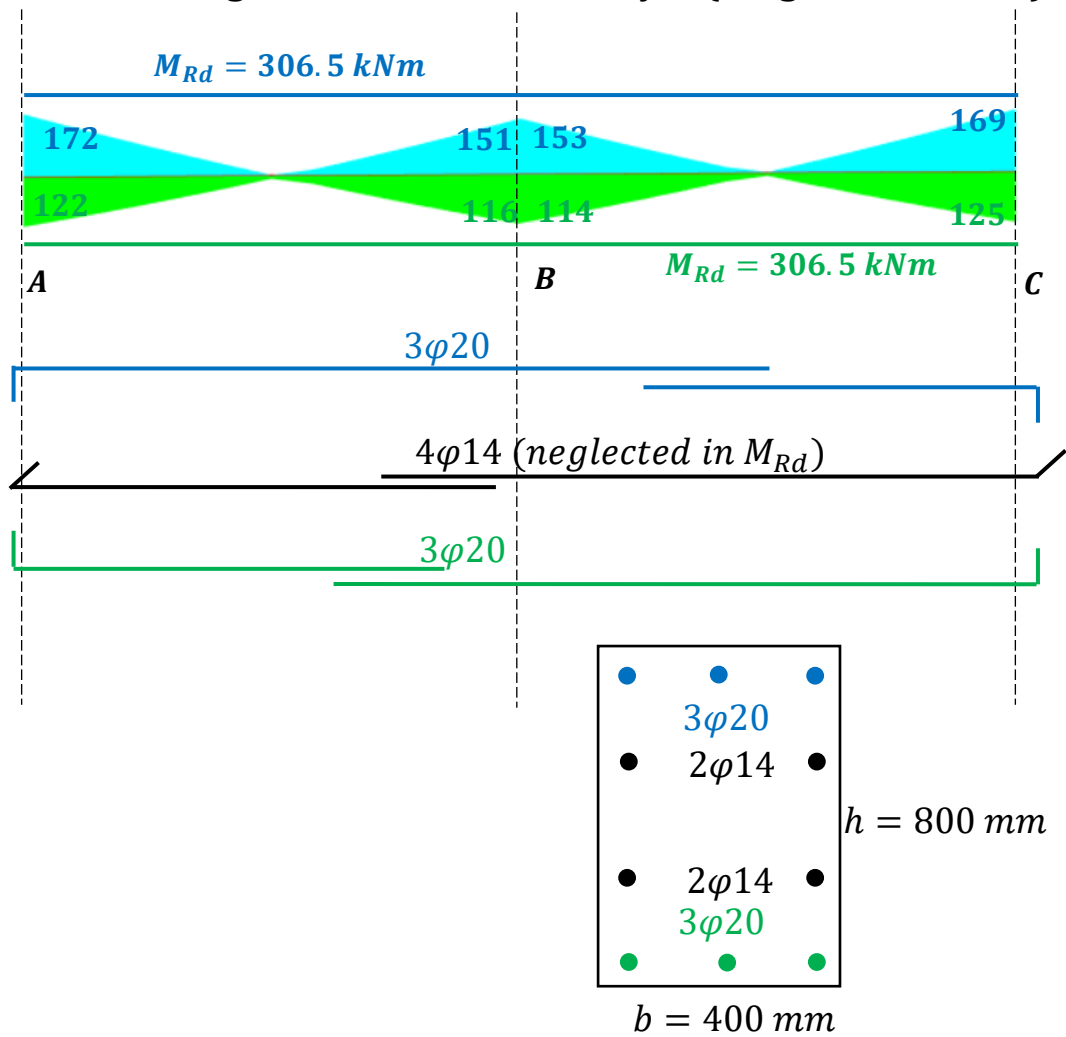
N° rett. 100

Calcola MRd Dominio M-N

L₀ 0 cm Col. modello

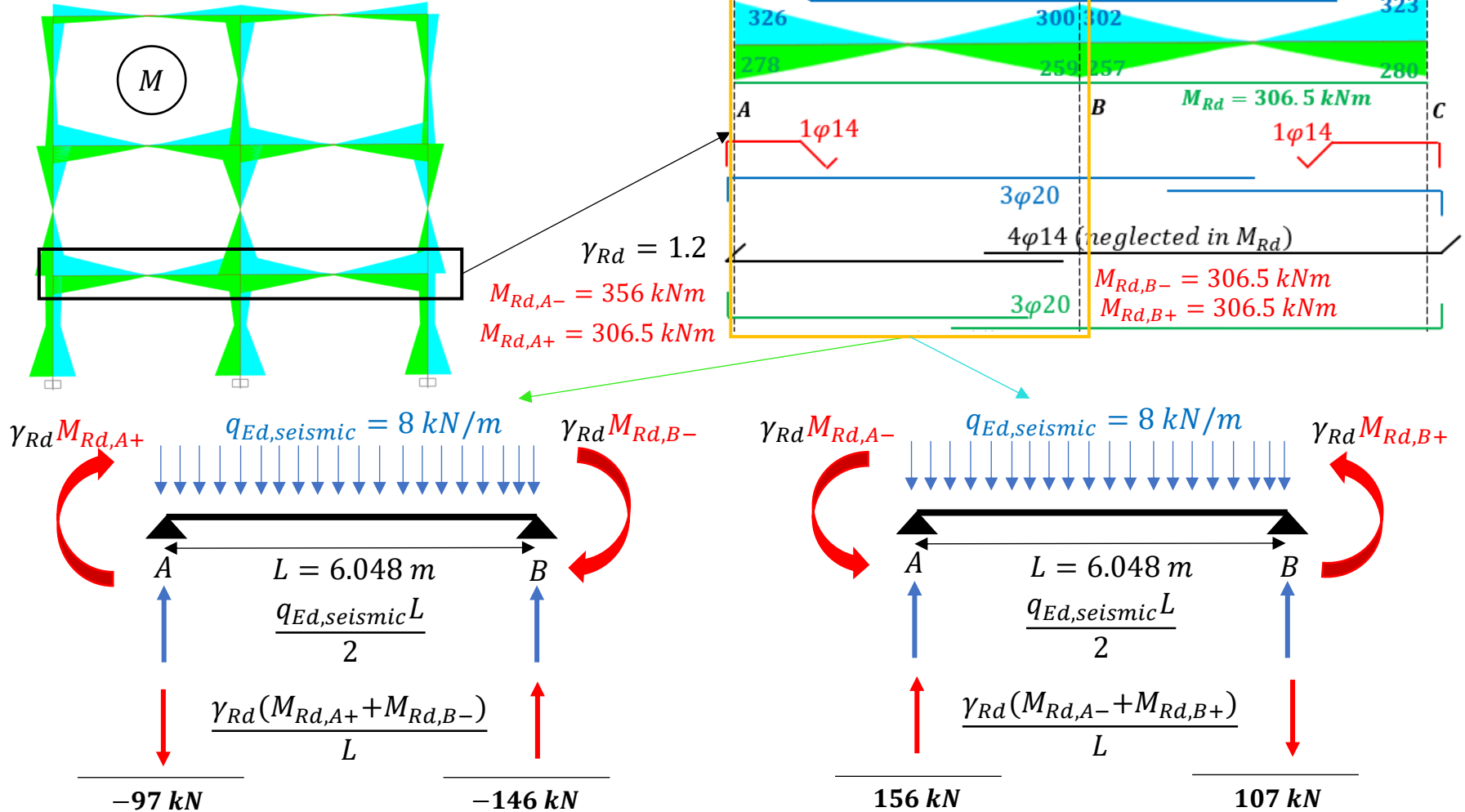
M-curvatura

Precompresso



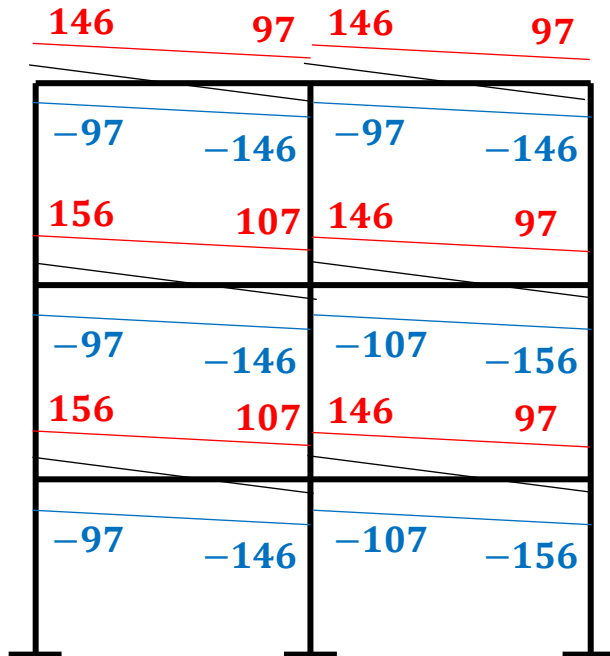
5) Detailed design (beams)

Design of transverse reinforcement (bending-shear hierarchy)

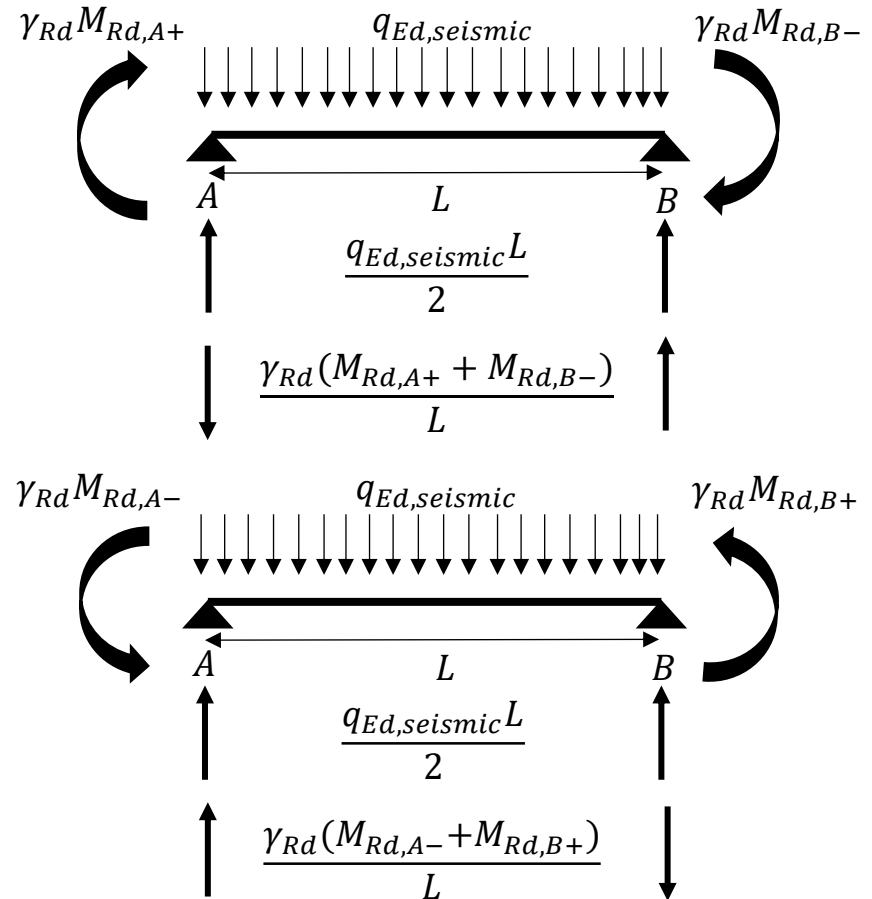


5) Detailed design (beams)

Design of transverse reinforcement
(bending-shear hierarchy)

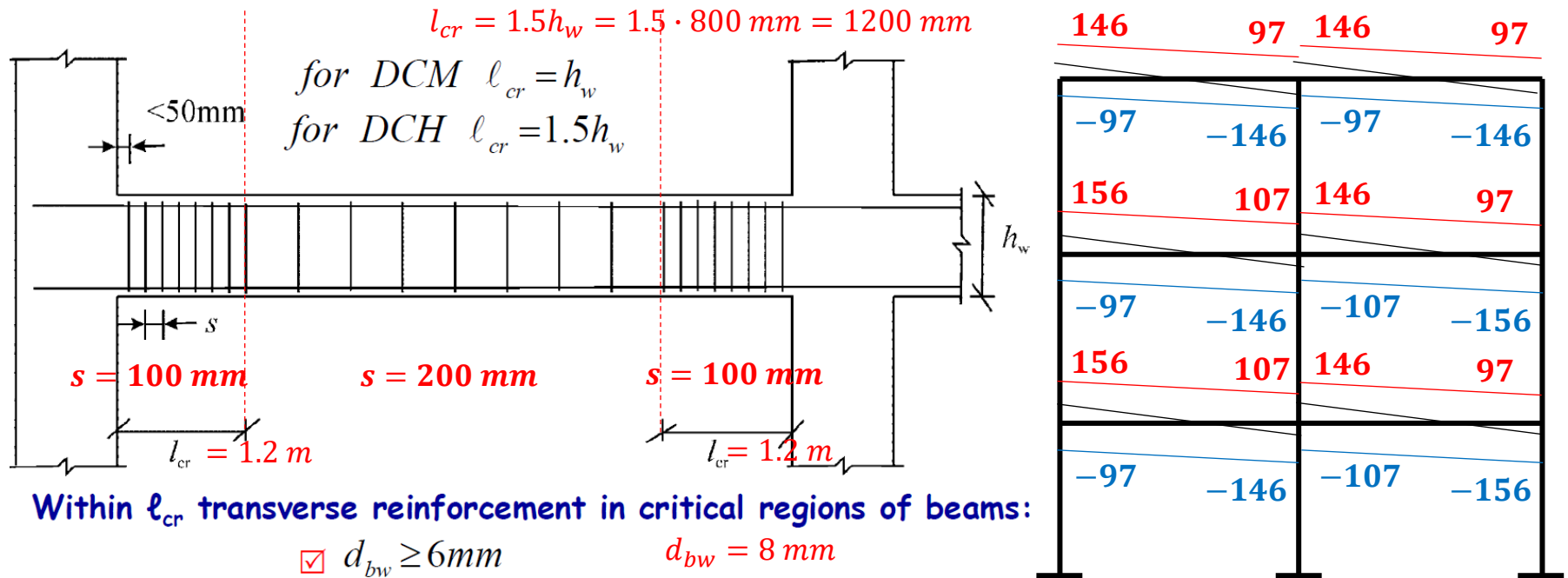


Shear actions in the beams



- ULS gravity combination (from structural analysis)
- ULS seismic combination along direction +X (application of hierarchy criterium)
- ULS seismic combination along direction -X (application of hierarchy criterium)

5) Detailed design (beams)



- ☒ $d_{bw} \geq 6 \text{ mm}$ $d_{bw} = 8 \text{ mm}$
- ☒ $s \leq$
 - $h_w / 4 = 200 \text{ mm}$
 - $24 d_{bw} = 24 \cdot 8 \text{ mm} = 192 \text{ mm}$
 - $8 d_{bL} \text{ (DCM) or } 6 d_{bL} \text{ (DCH)} = 6 \cdot 20 \text{ mm} = 120 \text{ mm}$
 - $225 \text{ mm (DCM) or } 175 \text{ mm (DCH)} = 175 \text{ mm}$

$s_{limits, EC8} = 120 \text{ mm}$
 $s_{theoretical} = 435 \text{ mm}$

$s = 100 \text{ mm in critical regions}$
 $s = 200 \text{ mm in non - critical regions}$

5) Detailed design (beams)

Ductility demand of a rc beam

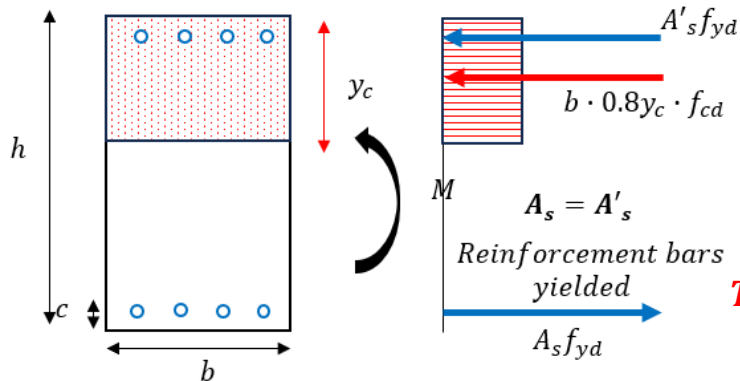
According to the Eurocode 8:

$$\mu_{\phi, demand} = 2q - 1 \quad \text{if } T_1 \geq T_c \quad \rightarrow \mu_{\phi, demand} = 1.5 \cdot (2 \cdot 3.88 - 1) = 10.1$$

$$\mu_{\phi, demand} = 1 + 2(q - 1) \frac{T_c}{T_1} \quad \text{if } T_1 < T_c$$

For steel class B μ_{ϕ} demand increases by 50%

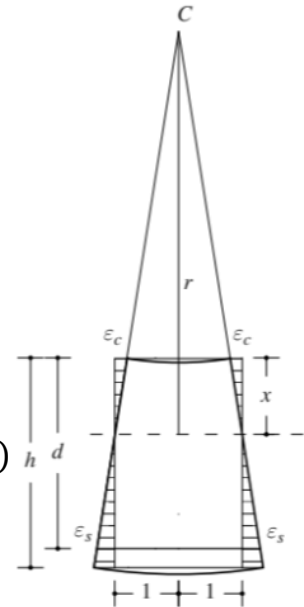
Ductility capacity of a rc beam



$$\left(\frac{1}{r}\right)_y = \frac{\varepsilon_{sy}}{d - y_{ce}} = \frac{\varepsilon_{sy}}{d(1 - y_{ce}/d)}$$

$$\left(\frac{1}{r}\right)_u = \frac{\varepsilon_{cu}}{y_c} = 0.8 \frac{\varepsilon_{cu}}{h} \frac{1}{\omega - \omega'}$$

Typically: $y_{ce} \cong 0.25 \div 0.3d = 0.25 \div 0.3(h - c)$
 $c/h \cong 0.1 \quad d/h \cong 0.9$



Translational equilibrium at failure

$$0.8y_c b f_{cd} + A' s f_{yd} - A_s f_{yd} = 0$$

$$y_c = h \frac{(\omega - \omega')}{0.8}$$

$$\omega = \frac{A_s f_{yd}}{b h f_{cd}} \quad \omega' = \frac{A' s f_{yd}}{b h f_{cd}}$$

$$\mu_{\phi, capacity} = \frac{\left(\frac{1}{r}\right)_u}{\left(\frac{1}{r}\right)_y} = \frac{0.8 d \varepsilon_{cu}}{1.4 h \varepsilon_{sy}} \frac{1}{\omega - \omega'} \cong 0.51 \frac{\varepsilon_{cu}}{\varepsilon_{sy}} \frac{1}{\omega - \omega'} = 81.8 > 10.1 = \mu_{\phi, demand}$$



5) Detailed design (beams)

Table 3.4.3 EN 1998 rules for detailing and dimensioning of primary beams (secondary beams as in DCL)

	DC H	DCM	DCL
"critical region" length	$1.5h_w$	h_w	
Longitudinal bars (L):			
ρ_{min} , tension side	$0.5f_{ctm}/f_{yk}$	$0.26f_{ctm}/f_{yk}$, $0.13\%^{(d)}$	
ρ_{max} , critical regions ⁽¹⁾	$\rho'+0.0018f_{cd}/(\mu_{\phi}\varepsilon_{sy,d}f_{yd})^{(1)}$		0.04
$A_{s,min}$, top & bottom	$2\Phi 14$ (308mm ²)	-	
$A_{s,min}$, top-span	$A_{s,top-supports}/4$	-	
$A_{s,min}$, critical regions bottom	$0.5A_{s,top}^{(2)}$		-
$A_{s,min}$, supports bottom	$A_{s,bottom-span}/4^{(2)}$		
d_{BL}/h_c - bar crossing interior joint ⁽³⁾	$\leq \frac{6.25(1+0.8v_d)}{(1+0.75\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$	$\leq \frac{7.5(1+0.8v_d)}{(1+0.5\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$	-
d_{BL}/h_c - bar anchored at exterior joint ⁽³⁾	$\leq 6.25(1+0.8v_d) \frac{f_{ctm}}{f_{yd}}$	$\leq 7.5(1+0.8v_d) \frac{f_{ctm}}{f_{yd}}$	-
Transverse bars (w):			
(i) outside critical regions			
spacing $s_w \leq$		0.75d	
$\rho_w \geq$		$0.08\sqrt{f_{tk}(MPa)/f_{yk}(MPa)}^{(2)}$	
(ii) in critical regions:			
$d_{BL} \geq$		6mm	
spacing $s_w \leq$	$6d_{BL}, \frac{h_w}{4}, 24d_{BL}, 175mm$	$8d_{BL}, \frac{h_w}{4}, 24d_{BL}, 225mm$	-
Shear design:			
V_{Ed} , seismic ⁽⁴⁾	$1.2 \frac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+v_d,g}^{(4)}$	$\frac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+v_d,g}^{(4)}$	from analysis for design seismic action plus gravity
$V_{Rd,max}$ seismic ⁽⁵⁾	As in EC2: $V_{Rd,max}=0.3(1-f_{ctd}(MPa)/250)b_w z f_{ctd} \sin 2\delta^{(5)}$, $1 \leq \cot \delta \leq 2.5$		
$V_{Rd,s}$, outside critical regions ⁽⁵⁾	As in EC2: $V_{Rd,s}=b_w z \rho_w f_{yw} \cot \delta^{(5)}$, $1 \leq \cot \delta \leq 2.5$		
$V_{Rd,s}$, critical regions ⁽⁵⁾	$V_{Rd,s}=b_w z \rho_w f_{yw} \cot \delta$ ($\delta=45^\circ$)	As in EC2: $V_{Rd,s}=b_w z \rho_w f_{yw} \cot \delta$, $1 \leq \cot \delta \leq 2.5$	
If $\zeta \equiv V_{Ed}/V_{Ed,max}^{(6)} < 0.5$: inclined bars at angle $\pm \alpha$ to beam axis, with cross-section A_s /direction	If $V_{Ed,max}/(2+\zeta)f_{ctd}b_w d > 1$: $A_s=0.5V_{Ed,max}/f_{yd} \sin \alpha$ & stirrups for $0.5V_{Ed,max}$		

$$l_{cr} = 1.5h_w = 1.5 \cdot 800 \text{ mm} = 1200 \text{ mm}$$

$$\rho_{min,limit} = 0.5 \frac{f_{ctm}}{f_{yk}} = 0.00225 < 0.00294 = \rho_{min}$$

$$\rho_{max,limit} = \rho' + \frac{0.0018 f_{cd}}{\mu_\phi \varepsilon_{sy,d} f_{yd}} = 0.00696 > 0.00342 = \rho_{max}$$

Worked examples here!

https://eurocodes.jrc.ec.europa.eu/sites/default/files/2022-06/EC8_Seismic_Design_of_Buildings-Worked_examples-annex_only.pdf

5) Detailed design (columns)

The Eurocode 8 recommends amplifying the bending moments of the columns by a factor of $1/0.7$ (which is approximately **1.43**).

This amplification can be done to simplify the check process (Conservative assumption). Instead of analyzing the structure in terms of both bending moments (M_x and M_y) and axial forces (N), the design can be done using axial forces (N) and the amplified bending moment.

Capacity design check at beam-column joints: ⁽¹⁰⁾	$1.3 \sum M_{Rd} \leq \sum M_{Rc}$ No moment in transverse direction of column		-
Verification for M_x - M_y - N :	Truly biaxial, or uniaxial with $(M_z/0.7, N)$, $(M_y/0.7, N)$		
Axial load ratio $\nu_d = N_{Ed}/A_c f_{cd}$	≤ 0.55	≤ 0.65	-
Shear design:			
V_{Ed} seismic ⁽¹¹⁾	$1.3 \frac{\sum M_{Rc}^{endz}}{l_{cl}}$ ⁽¹¹⁾	$1.1 \frac{\sum M_{Rc}^{endz}}{l_{cl}}$ ⁽¹¹⁾	from analysis for design seismic action plus gravity
$V_{Rd,max}$ seismic ^{(12), (13)}	As in EC2: $V_{Rd,max} = 0.3(1 - f_{ck}/250)b_w z f_{cd} \sin 2\delta$, $1 \leq \cot \delta \leq 2.5$		
$V_{Rd,s}$ seismic ^{(12), (13), (14)}	As in EC2: $V_{Rd,s} = b_w z \rho_w f_{yd} \cot \delta + N_{Ed}(h-x)/l_{cl}$ ⁽¹³⁾ , $1 \leq \cot \delta \leq 2.5$		

Check for columns

5) Detailed design (columns)

For ductility considerations, Eurocode 8 requires designing the columns with a 30% increase in the bending resistance transferred by the beams. In this context, the previously calculated bending moments of the columns are amplified by the coefficient α , determined through rotational equilibrium analysis.

(4) Unless otherwise specified in Sections 5 to 8, to satisfy the requirement of (3)P, in frame buildings, including frame-equivalent ones as defined in 5.1.2(1), with two or more storeys, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns:

$$\sum M_{Rc} \geq 1,3 \sum M_{Rb} \quad (4.29)$$

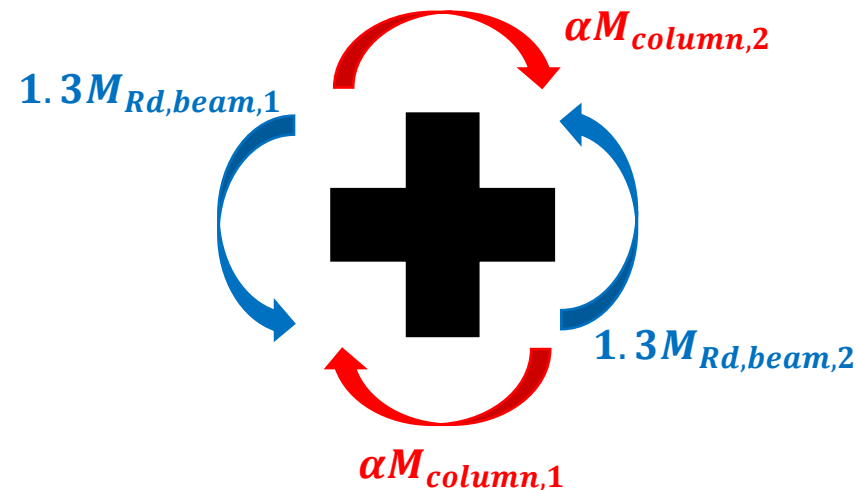
where

$\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in expression (4.29); and

$\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{Rb}$.

NOTE A rigorous interpretation of expression (4.29) requires calculation of the moments at the centre of the joint. These moments correspond to development of the design values of the moments of resistance of the columns or beams at the outside faces of the joint, plus a suitable allowance for moments due to shears at the joint faces. However, the loss in accuracy is minor and the simplification achieved is considerable if the shear allowance is neglected. This approximation is then deemed to be acceptable.

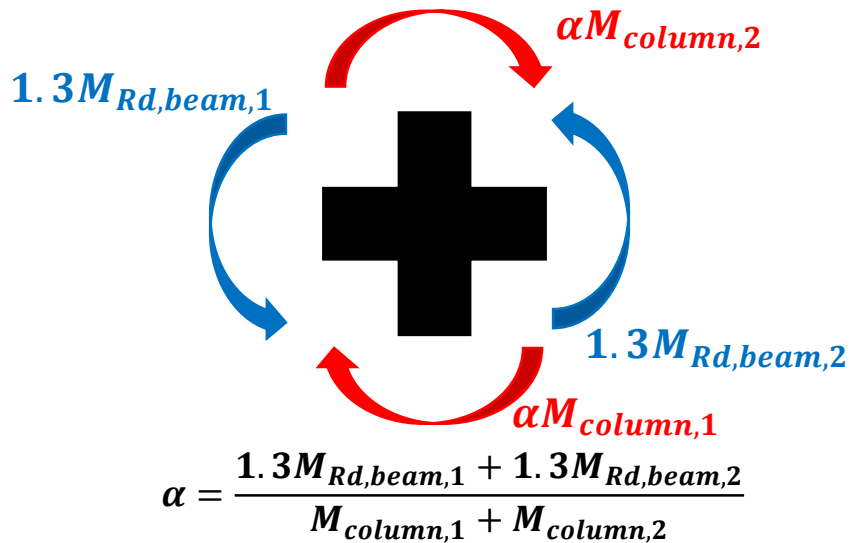
Beam-to-column hierarchy criterion



$$\alpha = \frac{1.3 M_{Rd,beam,1} + 1.3 M_{Rd,beam,2}}{M_{column,1} + M_{column,2}}$$

Increased bending
moments in the columns

5) Detailed design (columns)

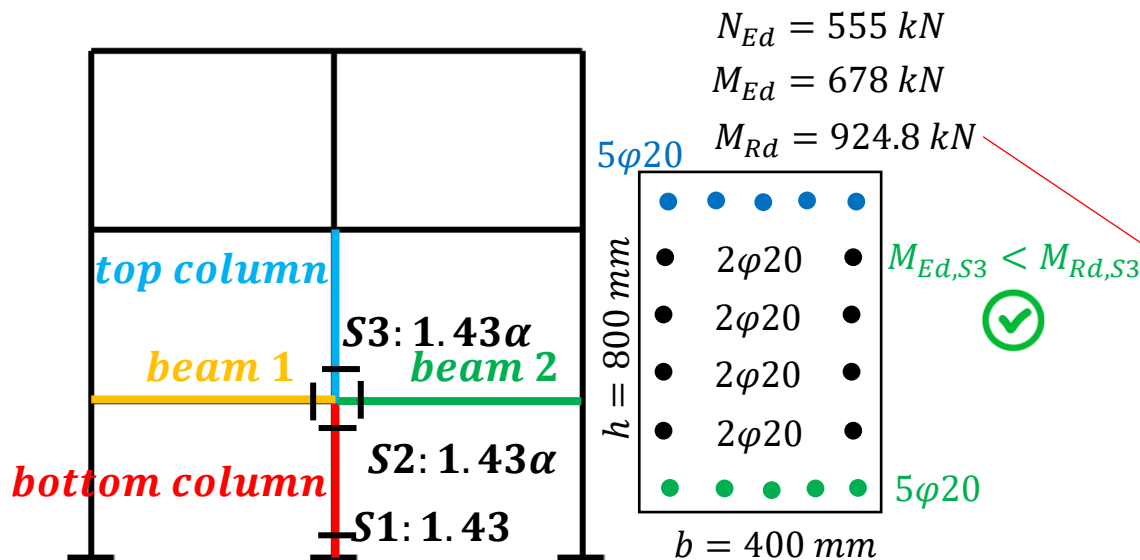


Capacity design check at beam-column joints: ⁽¹⁰⁾	$1.3\sum M_{Rd} \leq \sum M_{Rc}$		-
Verification for M_x-M_y-N :	Truly biaxial, or uniaxial with $(M_z/0.7, N)$, $(M_y/0.7, N)$		-
Axial load ratio $v_d = N_{Ed}/A_c f_{cd}$	≤ 0.55	≤ 0.65	-
Shear design:			
V_{Ed} seismic ⁽¹¹⁾	$1.3 \frac{\sum M_{Rc}^{ends}}{l_d}$ ⁽¹¹⁾	$1.1 \frac{\sum M_{Rc}^{ends}}{l_d}$ ⁽¹¹⁾	from analysis for design seismic action plus gravity
$V_{Rd,max}$ seismic ^{(12), (13)}	As in EC2: $V_{Rd,max} = 0.3(1-f_{ck}/MPa)/250 b_w z f_{cd} \sin 2\delta$, $1 \leq \cot \delta \leq 2.5$		
$V_{Rd,s}$ seismic ^{(12), (13), (14)}	As in EC2: $V_{Rd,s} = b_w z p_{wf} y_{wd} \cot \delta + N_{Ed}(h-x)/l_d$ ⁽¹³⁾ , $1 \leq \cot \delta \leq 2.5$		

1.43	1.43	1.43
1.43α	1.43α	1.43α
1.43α	1.43α	1.43α
1.43	1.43	1.43

5) Detailed design (columns)

Combination	$N_{\text{bottom_column}}$ (kN)	$N_{\text{top_column}}$ (kN)	$M_{\text{bottom_column}}$ (kNm)	$M_{\text{top_column}}$ (kNm)	$M_{Rd, \text{left, beam}}$ (kNm)	$M_{Rd, \text{right, beam}}$ (kNm)	α	$1.43 \cdot \alpha \cdot M_{\text{bottom_column}}$ (kNm)	$1.43 \cdot \alpha \cdot M_{\text{top_column}}$ (kNm)
1_+Edx_ecc+X_+0.3EdY_ecc+Y	844	555	-173	276	356.0	306.5	1.71	-424	676
2_+Edx_ecc+X_-0.3EdY_ecc+Y	844	555	-172	276	356.0	306.5	1.72	-422	678
3_-Edx_ecc+X_+0.3EdY_ecc+Y	844	555	173	-278	306.5	356.0	1.67	414	-665
4_-Edx_ecc+X_-0.3EdY_ecc+Y	844	555	175	-278	306.5	356.0	1.67	416	-663
9_+Edx_ecc+X_+0.3EdY_ecc-Y	844	555	-168	268	356.0	306.5	1.77	-424	676
10_+Edx_ecc+X_-0.3EdY_ecc-Y	844	555	-177	284	356.0	306.5	1.67	-422	678
11_-Edx_ecc+X_+0.3EdY_ecc-Y	844	555	178	-286	306.5	356.0	1.62	414	-665
12_-Edx_ecc+X_-0.3EdY_ecc-Y	844	555	169	-270	306.5	356.0	1.72	416	-663
17_+Edx_ecc-X_+0.3EdY_ecc-Y	844	555	-178	283	356.0	306.5	1.67	-424	676
18_+Edx_ecc-X_-0.3EdY_ecc-Y	844	555	-187	300	356.0	306.5	1.58	-422	678
19_-Edx_ecc-X_+0.3EdY_ecc-Y	844	555	188	-302	306.5	356.0	1.54	414	-665
20_-Edx_ecc-X_-0.3EdY_ecc-Y	844	555	179	-285	306.5	356.0	1.62	416	-663
25_+Edx_ecc-X_+0.3EdY_ecc+Y	844	555	-183	291	356.0	306.5	1.62	-424	676
26_+Edx_ecc-X_-0.3EdY_ecc+Y	844	555	-181	292	356.0	306.5	1.63	-422	678
27_-Edx_ecc-X_+0.3EdY_ecc+Y	844	555	183	-294	306.5	356.0	1.58	414	-665
28_-Edx_ecc-X_-0.3EdY_ecc+Y	844	555	184	-294	306.5	356.0	1.58	416	-663



Titolo: _____

N° figure elementari: 1 Zoom N° strati barre: 5 Zoom

N°	b [cm]	h [cm]	N°	As [cm²]	d [cm]
1	40	80	1	15.71	3
			2	6.28	20
			3	6.28	40
			4	6.28	60
			5	15.71	77

Sollecitazioni: S.L.U. Metodo n

N_{Ed}: 555 kN
M_{Ed}: 678 kNm
M_{Rd}: 924.8 kNm

P.to applicazione N: Centro Baricentro cls
Coord. [cm]: xN 0, yN 0

Tipo rottura: Lato calcestruzzo - Acciaio snervato

Materiali: B500 C35/45

ε_{cu}: 67.5 ‰ ε_{c2}: 2 ‰
f_{yd}: 434.8 N/mm² ε_{cu}: 3.5 ‰
E_s: 210 000 N/mm² f_{cd}: 19.83 ‰
E_s/E_c: 15 f_{cc}/f_{cd}: 0.8 [7]
ε_{syd}: 2.07 ‰ σ_{c,adm}: 13.5
σ_{s,adm}: 255 N/mm² τ_{co}: 0.8
τ_{ct}: 2.257

σ_c: -19.83 N/mm²
σ_s: 434.8 N/mm²
ε_s: 3.5 ‰
ε_s: 11.51 ‰
d: 77 cm
x: 17.96 x/d: 0.2332
ξ: 0.7315

Metodo di calcolo: S.L.U. + S.L. Metodo n

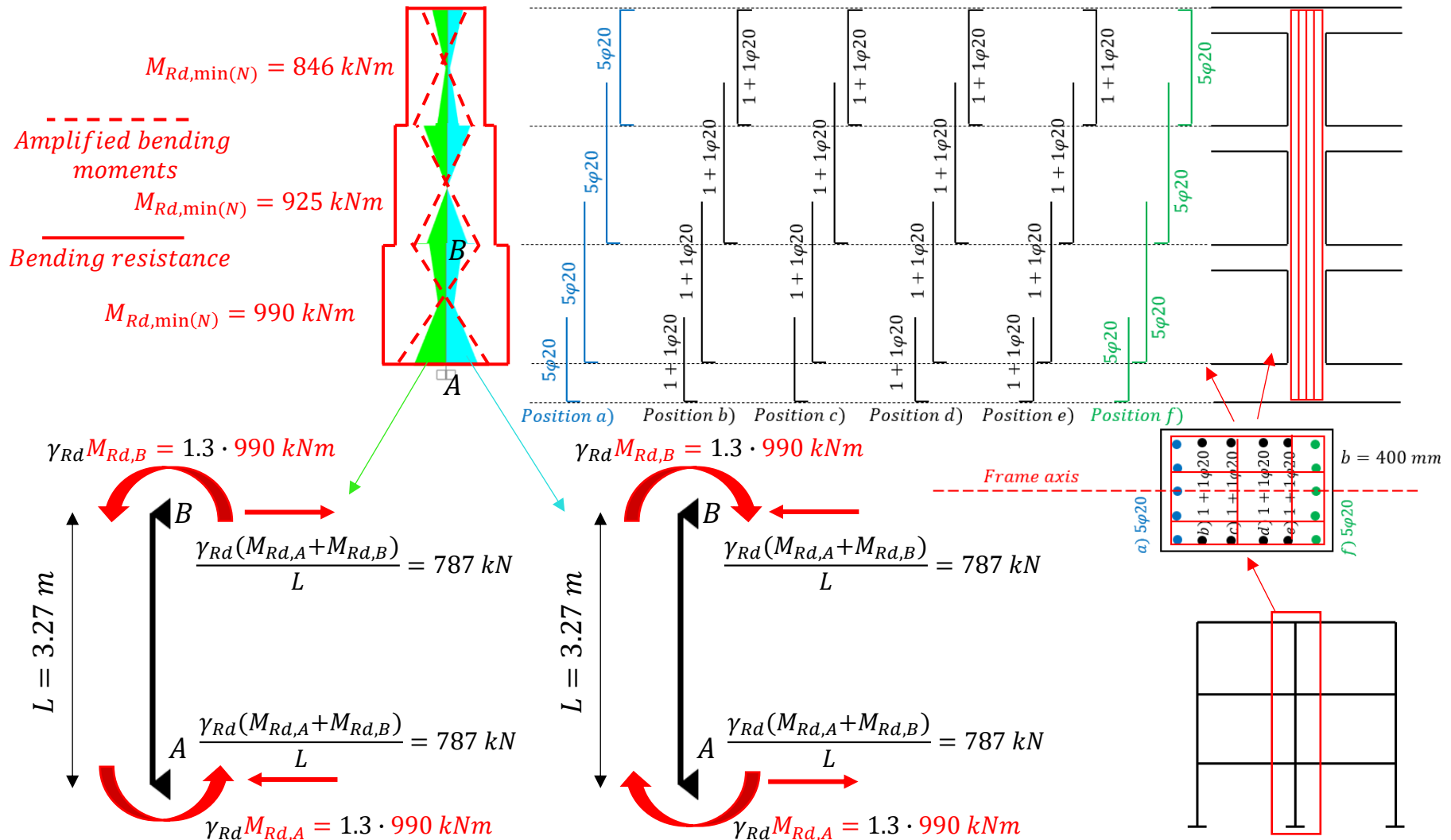
Tipo flessione: Retta Deviat

N° rett.: 1 Calcola MRd Dominio I
L₀: 0 cm Col. mod
M-curva

Precompresso

5) Detailed design (columns)

Design of transverse reinforcement (bending-shear hierarchy)



5) Detailed design (columns)

Detailing of primary seismic columns for local ductility

EN 1998-1:2004 (E) § 5.4.3.2.2

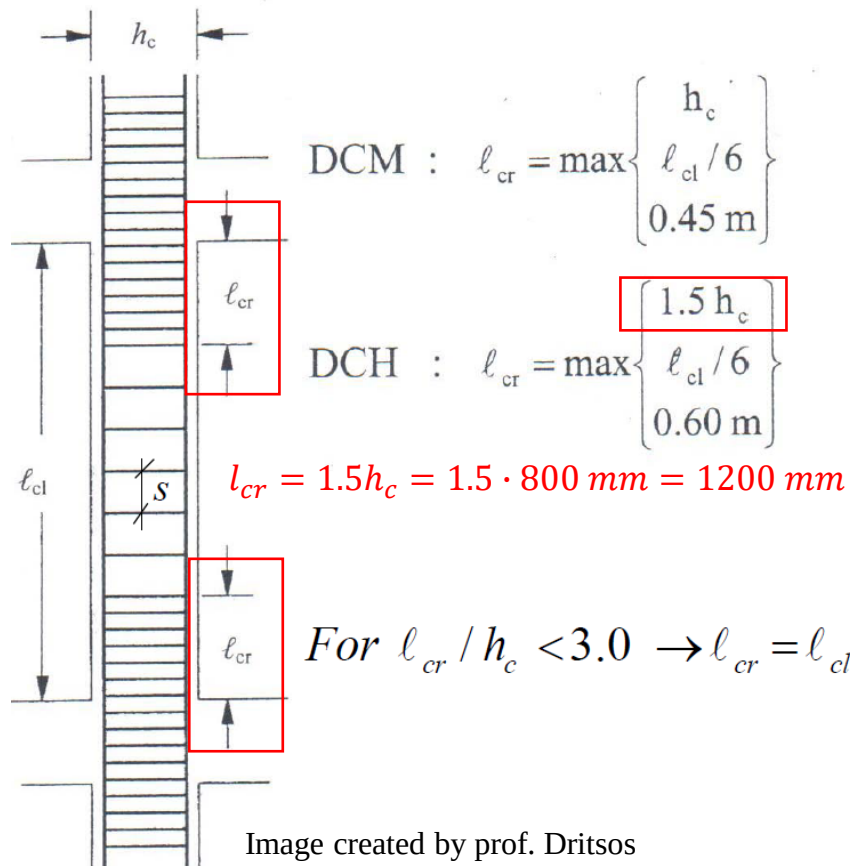


Image created by prof. Dritsos
(March 2015)

Everywhere

$$d_{bw} \geq \begin{cases} 6 \text{ mm} \\ \frac{1}{4} d_{bL, \max} \end{cases} \quad d_{bw} = 8 \text{ mm}$$

- more restrictions for DCH critical regions

In critical regions

for DCM

$$s \leq \begin{cases} b_o / 2 \\ 8 d_{bL, \min} \\ 175 \text{ mm} \end{cases}$$

for DCH

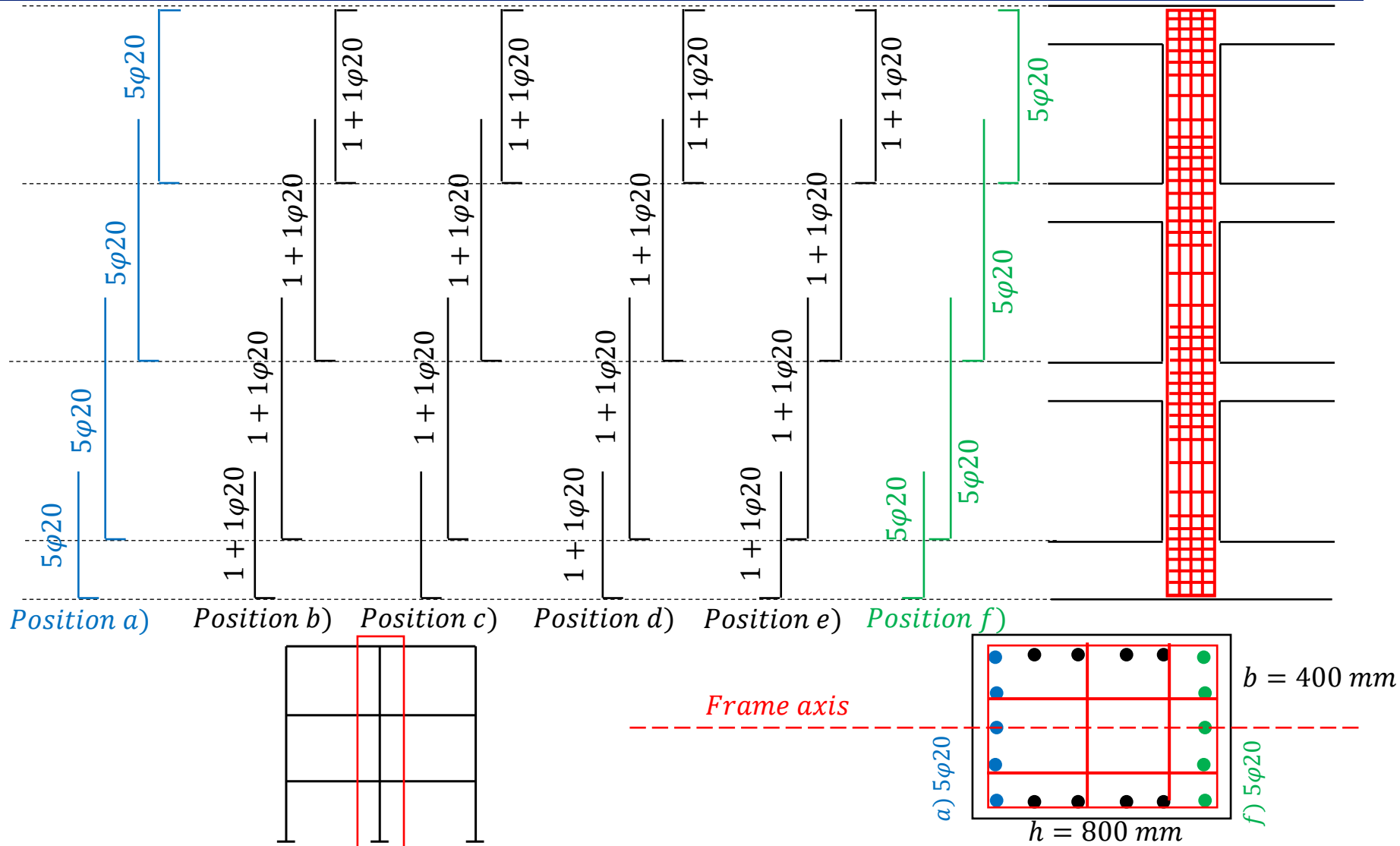
$$s \leq \begin{cases} b_o / 3 = 320 / 3 \text{ mm} = 107 \text{ mm} \\ 6 d_{bL, \min} = 6 \cdot 20 \text{ mm} = 120 \text{ mm} \\ 125 \text{ mm} \end{cases}$$

$$s_{\text{limits, EC8}} = 107 \text{ mm}$$

$$s_{\text{theoretical}} = 111 \text{ mm}$$

$$\left. \begin{matrix} s_{\text{limits, EC8}} = 107 \text{ mm} \\ s_{\text{theoretical}} = 111 \text{ mm} \end{matrix} \right\} \begin{matrix} s = 100 \text{ mm in critical regions} \\ s = 200 \text{ mm in non-critical regions} \end{matrix}$$

5) Detailed design (columns)

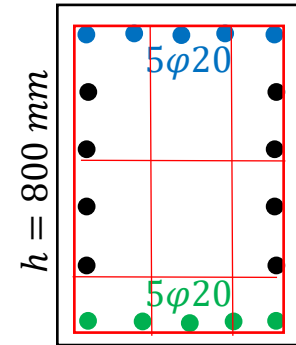


5) Detailed design (columns)

Table 3.4.4 EN 1998 rules for detailing and dimensioning of primary columns (secondary ones as DCL)

	DCH	DCM	DCL
Cross-section sides, $h_c, b_c \geq$	0.25m; $h_c/10$ if $\theta = P\delta/Vh > 0.1^{(1)}$	-	-
"critical region" length $^{(1)} \geq$	$1.5h_c, 1.5b_c, 0.6m, l_c/5$	$h_c, b_c, 0.45m, l_c/6$	h_c, b_c
Longitudinal bars (L):			
ρ_{min}	1%	$0.1N_d/A_c f_{yd}, 0.2\%^{(0)}$	
ρ_{max}	4%	4% ⁽⁰⁾	
$d_{bL} \geq$	8mm		
bars per side $\geq$	3	2	
Spacing between restrained bars	$\leq 150mm$	$\leq 200mm$	-
Distance of unrestrained bar from nearest restrained nearest bar	$\leq 150mm$		
Transverse bars (w):			
Outside critical regions:			
$d_{bw} \geq$	6mm, $d_{bL}/4$		
spacing $s_w \leq$	$20d_{bL}, h_c, b_c, 400mm$	$12d_{bL}, 0.6h_c, 0.6b_c, 240mm$	
at lap splices, if $d_{bL} > 14mm$: $s_w \leq$	$12d_{bL}, 0.6h_c, 0.6b_c, 240mm$		
Within critical regions: ⁽²⁾			
$d_{bw} \geq$ ⁽³⁾	$6mm, 0.4(f_{yd}/f_{ywd})^{1/2} d_{bL}$	6mm, $d_{bL}/4$	
$s_w \leq$ ^{(3),(4)}	$6d_{bL}, b_c/3, 125mm$	$8d_{bL}, b_c/2, 175mm$	-
$\omega_{wd} \geq$ ⁽⁵⁾	0.08	-	
$\alpha \omega_{wd} \geq$ ^{(4),(5),(6),(7)}	$30\mu_y V_d \varepsilon_{sy,d} b_c / b_o - 0.035$	-	
In critical region at column base:			
$\omega_{wd} \geq$	0.12	0.08	-
$\alpha \omega_{wd} \geq$ ^{(4),(5),(6),(8),(9)}	$30\mu_y V_d \varepsilon_{sy,d} b_c / b_o - 0.035$	-	
Capacity design check at beam-column joints: ⁽¹⁰⁾			
	$1.3 \sum M_{Rd} \leq \sum M_{Rc}$ No moment in transverse direction of column		-
Verification for M_x-M_y-N :	Truly biaxial, or uniaxial with $(M_x/0.7, N), (M_y/0.7, N)$		
Axial load ratio $v_d = N_{Ed}/A_c f_{cd}$	≤ 0.55	≤ 0.65	-
Shear design:			
V_{Ed} seismic ⁽¹¹⁾	$1.3 \frac{\sum M_{Rd}^{endz}}{l_{cl}}$ ⁽¹¹⁾	$1.1 \frac{\sum M_{Rd}^{endz}}{l_{cl}}$ ⁽¹¹⁾	from analysis for design seismic action plus gravity
$V_{Rd,max}$ seismic ^{(12), (13)}	As in EC2: $V_{Rd,max} = 0.3(1-f_{ck}(MPa)/250)b_w z f_{ctd} \sin 2\theta, 1 \leq \cot \theta \leq 2.5$		
$V_{Rd,s}$ seismic ^{(12), (13), (14)}	As in EC2: $V_{Rd,s} = b_w z \rho_w f_{ywd} \cot \theta + N_{Ed}(h-x)/l_{cl}$ ⁽¹³⁾ , $1 \leq \cot \theta \leq 2.5$		

$$1\% < \rho = \frac{A_s}{bh} = 1.57\% < 4\%$$



$b = 400 \text{ mm}$

$$d_{bL} = 20 \text{ mm}$$

$$\text{Bars per side} = 4, 5 \text{ or } 6$$

$$\text{Spacing between restrained bars} = 148 \text{ mm}$$

$$d_{bw} = 8 \text{ mm} > \max \left\{ 6 \text{ mm}; 0.4 \left(\frac{f_{yd}}{f_{ywd}} \right)^{0.5} d_{bw} \right\} = 8 \text{ mm}$$

$$s = 100 \text{ mm in critical regions}$$

$$s = 200 \text{ mm in non - critical regions}$$

5) Detailed design (base of primary column)

(6)P In the critical region at the base of primary seismic columns a value of the curvature ductility factor, μ_ϕ , should be provided, at least equal to that given in 5.2.3.4(3).

(7)P If for the specified value of μ_ϕ a concrete strain larger than $\varepsilon_{cu2}=0.0035$ is needed anywhere in the cross-section, compensation for the loss of resistance due to spalling of the concrete shall be achieved by means of adequate confinement of the concrete core, on the basis of the properties of confined concrete in EN 1992-1-1:2004, 3.1.9.

(8) The requirements specified in (6)P and (7)P of this subclause are deemed to be satisfied if:

$$\alpha \omega_{wd} \geq 30 \mu_\phi v_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0.035 \quad (5.15)$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions

$$\left[\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} \right];$$

μ_ϕ is the required value of the curvature ductility factor;

v_d is the normalised design axial force ($v_d = N_{Ed}/A_c f_{cd}$);

$\varepsilon_{sy,d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth (parallel to the horizontal direction in which the value of μ_ϕ used in (6)P of this subclause applies);

h_o is the depth of confined core (to the centreline of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centreline of the hoops);

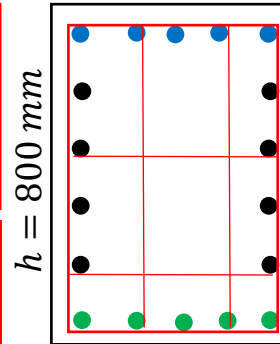
α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:

a) For rectangular cross-sections:

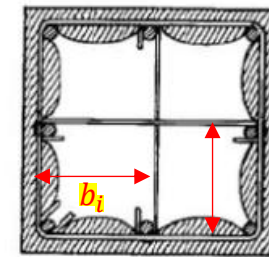
$$\alpha_n = 1 - \sum_i b_i^2 / 6 b_o h_o \quad (5.16a)$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_o) \quad (5.17a)$$

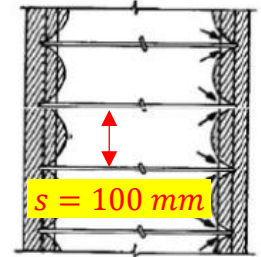
where



$$b = 400 \text{ mm}$$



(b) Rectangular hoops with cross ties



(d) Confinement by transverse bars

$$\mu_{\phi,demand} = 1.5 \cdot (2q - 1) = 1.5 \cdot (2 \cdot 3.88 - 1) = 10.1$$

$$b_{stirrup} = b - 2c + \varphi_{stirrup} + \varphi_{long.bar} = 400 - 2 \cdot 30 + 8 + 20 = 368 \text{ mm}$$

$$h_{stirrup} = h - 2c + \varphi_{stirrup} + \varphi_{long.bar} = 800 - 2 \cdot 30 + 8 + 20 = 768 \text{ mm}$$

$$s = 100 \text{ mm}$$

Stirrups with 4 legs are adopted in order to fulfill the following requirement

	DCH	DCM	DCL
In critical region at column base:			
$\omega_{wd} \geq$	0.12	0.08	-
$\alpha \omega_{wd} \geq (4),(5),(6),(8),(9)$	$30 \mu_\phi v_d \varepsilon_{sy,d} b_o / b_o - 0.035$		
			-

$$\omega_{wd} = \frac{\pi \frac{\varphi_{stirrups}^2}{4} 4 (b_{stirrup} + h_{stirrup}) f_{yd}}{b_{stirrup} \cdot h_{stirrup} \cdot s} \frac{f_{yd}}{f_{cd}} = 0.177$$

5) Detailed design (base of primary column)

(6)P In the critical region at the base of primary seismic columns a value of the curvature ductility factor, μ_ϕ , should be provided, at least equal to that given in 5.2.3.4(3).

(7)P If for the specified value of μ_ϕ a concrete strain larger than $\varepsilon_{cu2}=0.0035$ is needed anywhere in the cross-section, compensation for the loss of resistance due to spalling of the concrete shall be achieved by means of adequate confinement of the concrete core, on the basis of the properties of confined concrete in EN 1992-1-1:2004, 3.1.9.

(8) The requirements specified in (6)P and (7)P of this subclause are deemed to be satisfied if:

$$\alpha \omega_{wd} \geq 30 \mu_\phi v_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0.035 \quad (5.15)$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions

$$\left[\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} \right]$$

μ_ϕ is the required value of the curvature ductility factor;

v_d is the normalised design axial force ($v_d = N_{Ed}/A_c f_{cd}$);

$\varepsilon_{sy,d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth (parallel to the horizontal direction in which the value of μ_ϕ used in (6)P of this subclause applies);

h_o is the depth of confined core (to the centreline of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centreline of the hoops);

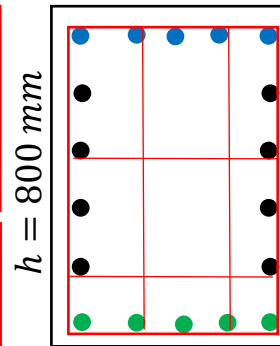
α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:

a) For rectangular cross-sections:

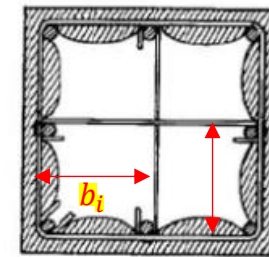
$$\alpha_n = 1 - \sum \frac{b_i^2}{6 b_o h_o}$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_o)$$

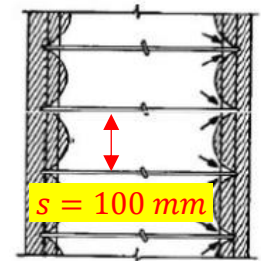
where



$$b = 400 \text{ mm}$$



(b) Rectangular hoops with cross ties



(d) Confinement by transverse bars

$$s = 100 \text{ mm}$$

$$b_{stirrup} = b - 2c + \varphi_{stirrup} + \varphi_{long.bar} = 400 - 2 \cdot 30 + 8 + 20 = 368 \text{ mm}$$

$$h_{stirrup} = h - 2c + \varphi_{stirrup} + \varphi_{long.bar} = 800 - 2 \cdot 30 + 8 + 20 = 768 \text{ mm}$$

$$\alpha_s = \left(1 - \frac{s}{2b_{stirrup}}\right) \left(1 - \frac{s}{2h_{stirrup}}\right) = 0.81 \quad \alpha_n = 1 - \frac{1}{6b_{stirrup}h_{stirrup}} \sum b_i^2 = 0.66$$

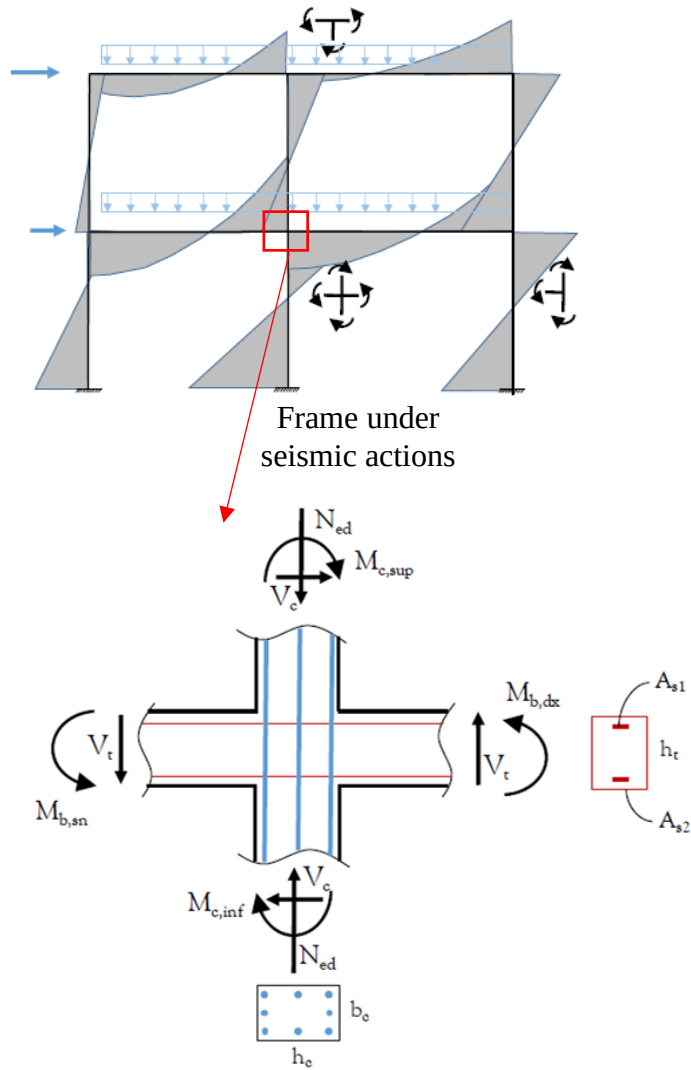
$$\varepsilon_{sy,d} = \frac{f_{yd}}{E} = \frac{434.8 \text{ MPa}}{210000 \text{ MPa}} = 0.00207$$

$$v_d = \frac{N_{Ed,max}}{A_c f_{cd}} = \frac{844000 \text{ N}}{400 \text{ mm} \cdot 800 \text{ mm} \cdot 19.83 \text{ MPa}} = 0.13$$

$$\alpha \omega_{wd} = \alpha_n \alpha_s \omega_{wd} = 0.094 > 0.055 = 30 \mu_\phi v_d \varepsilon_{sy,d} \frac{b}{b_{stirrup}} - 0.035$$



5) Detailed design (joints)



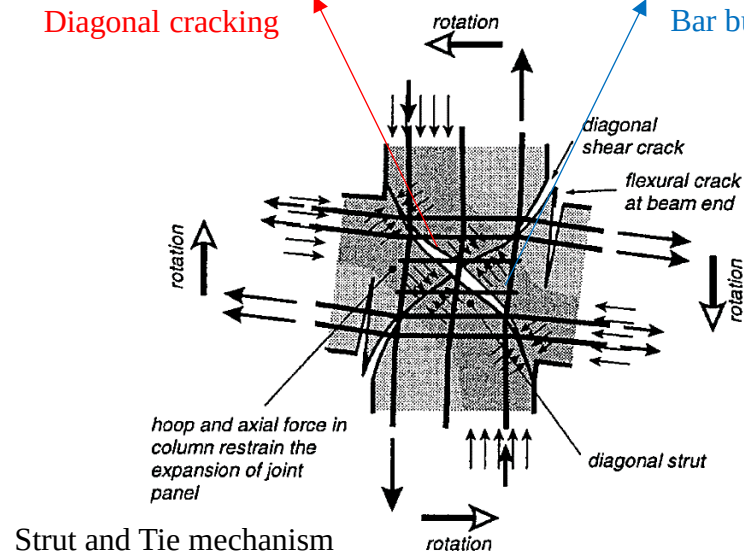
Possible collapse mechanisms



Diagonal cracking



Bar buckling



5) Detailed design (joints)

5.5.2.3 Beam-column joints

(1)P The horizontal shear acting on the core of a joint between primary seismic beams and columns shall be determined taking into account the most adverse conditions under seismic actions, i.e. capacity design conditions for the beams framing into the joint and the lowest compatible values of shear forces in the other framing elements.

(2) Simplified expressions for the horizontal shear force acting on the concrete core of the joints may be used as follows:

a) for interior beam-column joints:

$$V_{jhd} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_C$$

b) for exterior beam-column joints:

$$V_{jhd} = \gamma_{Rd} \cdot A_{s1} \cdot f_{yd} - V_C$$

where **Shear action in the joint**

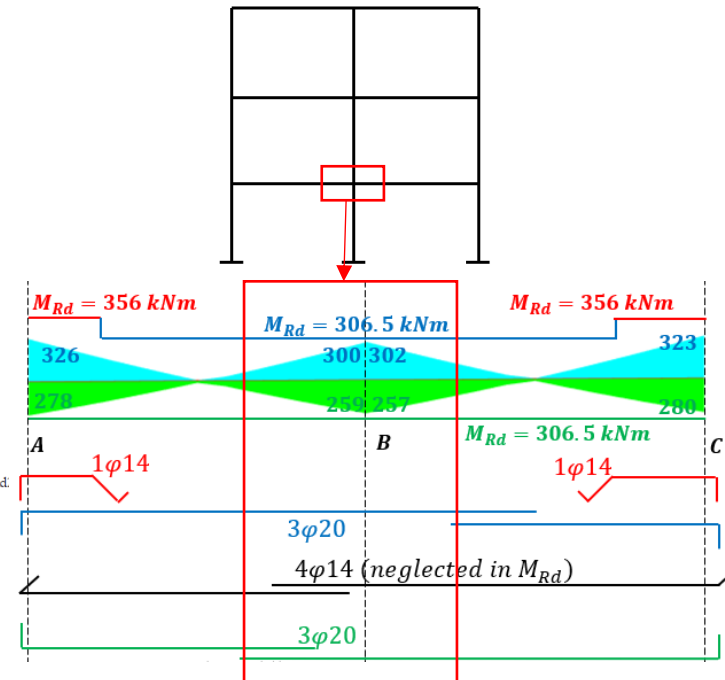
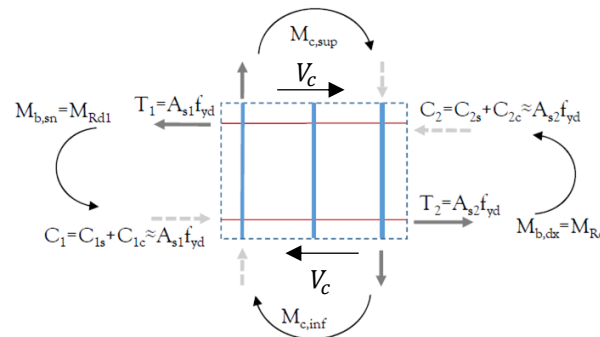
A_{s1} is the area of the beam top reinforcement;

A_{s2} is the area of the beam bottom reinforcement;

V_C is the shear force in the column above the joint, from the analysis in the seismic design situation;

γ_{Rd} is a factor to account for overstrength due to steel strain-hardening and should be not less than 1.2.

(3) The shear forces acting on the joints shall correspond to the most adverse sense of the seismic action influencing the values A_{s1} , A_{s2} and V_C to be used in expressions (5.22) and (5.23).



$$V_{j,Ed} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_C = 855 \text{ kN}$$

1.2
Area 3φ20
Area 3φ20
434.8 MPa

Minimum shear 128 kN

5) Detailed design (joints)

Joints' check

(4) As an alternative to the rule specified in (3) of this subclause, integrity of the joint after diagonal cracking may be ensured by horizontal hoop reinforcement. To this end the following total area of horizontal hoops should be provided in the joint.

a) In interior joints:

$$A_{sh}f_{ywd} \geq \gamma_{Rd}(A_{s1} + A_{s2})f_{yd}(1 - 0,8 v_d) \quad (5.36a)$$

b) In exterior joints:

$$A_{sh}f_{ywd} \geq \gamma_{Rd}A_{s2}f_{yd}(1 - 0,8 v_d) \quad (5.36b)$$

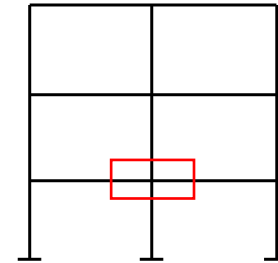
where γ_{Rd} is equal to 1,2 (cf 5.5.2.3(2)) and the normalised axial force v_d refers to the column above the joint in expression (5.36a), or to the column below the joint in expression (5.36b).

(5) The horizontal hoops calculated as in (3) and (4) of this subclause should be uniformly distributed within the depth h_{jw} between the top and bottom bars of the beam. In exterior joints they should enclose the ends of beam bars bent toward the joint.

$$A_{sh,min} \geq \frac{\gamma_{Rd}(A_{s1} + A_{s2})f_{yd}(1 - 0,8 v_d)}{f_{ywd}} = 2103 \text{ mm}^2$$

Area 3φ20 (pointing to A_{s1})
Area 3φ20 (pointing to A_{s2})
 1.2 (pointing to γ_{Rd})

$$v_d = \frac{N_{Ed}}{A_c f_{cd}} = \frac{555000 \text{ N}}{400\text{mm} \cdot 800\text{mm} \cdot 19.83\text{MPa}} = 0.09$$



This is a very conservative approach. Its main limitation is the exclusion of the beneficial contribution of the shear in the column, which helps reduce the term on the right-hand side of the equation.

This approach will change in new Eurocode 8.

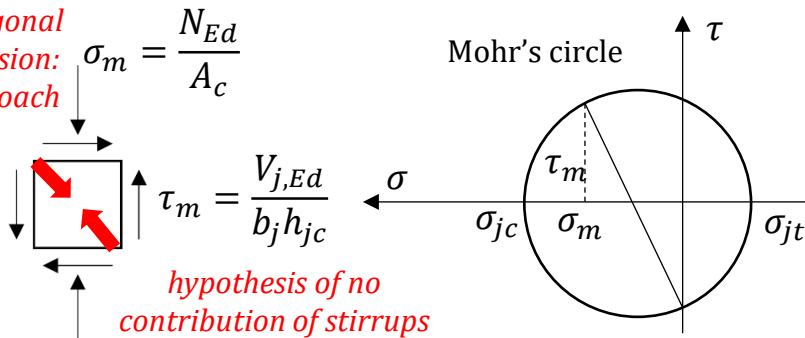
$A_{sh} = 2211 \text{ mm}^2$
 (area of 11 stirrups with 4 legs in the joint) 

$$A_{sh}f_{ywd} = 961 \text{ kN} > 914 \text{ kN}$$

$$= \gamma_{Rd}(A_{s1} + A_{s2})f_{yd}(1 - 0.8 v_d)$$

5) Detailed design (joints)

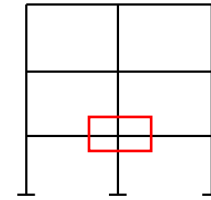
Check of the diagonal
strut in compression:
theoretical approach



$$\sigma_{jc} = \sqrt{\left(\frac{\sigma_m}{2}\right)^2 + \tau_m^2} + \frac{\sigma_m}{2} \leq \eta f_{cd} \rightarrow \sqrt{\left(\frac{\sigma_m}{2}\right)^2 + \tau_m^2} \leq \eta f_{cd} - \frac{\sigma_m}{2}$$

$$\tau_m \leq \eta^2 f_{cd}^2 - \eta f_{cd} \sigma_m = \eta f_{cd} \sqrt{1 - \frac{\sigma_m}{\eta f_{cd}}} = \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}}$$

$$V_{j,Ed} \leq V_{j,Rd} = \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_{jc}$$



Example

$$\eta = 0.6 \left(1 - \frac{f_{ck}}{250}\right) = 0.6 \left(1 - \frac{35}{250}\right) = 0.52$$

$$f_{cd} = 19.83 \text{ MPa}$$

$$v_d = \frac{N_{Ed}}{A_c f_{cd}} = \frac{555000 \text{ N}}{400 \text{ mm} \cdot 800 \text{ mm} \cdot 19.83 \text{ MPa}} = 0.09$$

$$b_j = 400 \text{ mm}$$

$$h_{jc} = h_{jc} - 2c = 800 \text{ mm} - 60 \text{ mm} = 740 \text{ mm}$$

$$V_{j,Ed} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_c = 855 \text{ kN}$$

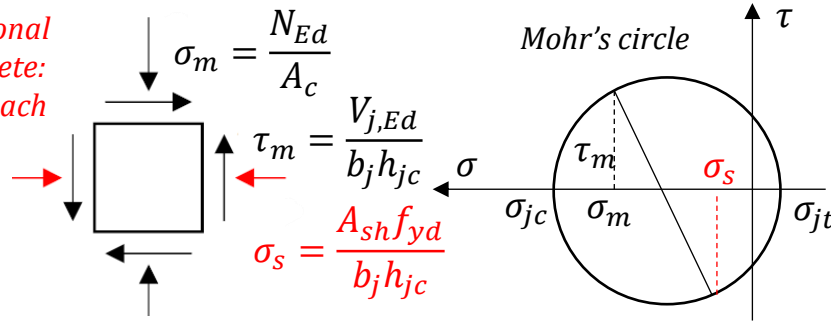
$$V_{j,Rd} = \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_{jc} = 2761 \text{ kN}$$

$$V_{j,Ed} \leq V_{j,Rd} \quad \checkmark$$

No crisis provided by the diagonal strut in
compression

5) Detailed design (joints)

Check of the diagonal cracking of concrete: theoretical approach



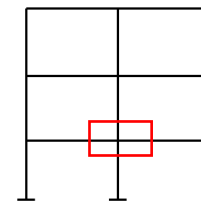
$$\sigma_{jt} = \sqrt{\left(\frac{\sigma_m - \sigma_s}{2}\right)^2 + \tau_m^2} - \frac{\sigma_m + \sigma_s}{2} \leq f_{ctd}$$

$$\sigma_s \geq \frac{\tau_m^2}{(\sigma_m + f_{ctd})} - f_{ctd}$$

$$\frac{A_{sh} f_{yd}}{b_j h_{jc}} \geq \frac{\left(\frac{V_{j,Ed}}{b_j h_{jc}}\right)^2}{(v_d f_{cd} + f_{ctd})} - f_{ctd} \quad \text{EC8 requirement}$$

$$A_{sh,min} \geq \frac{b_j h_{jc}}{f_{yd}} \left[\frac{\left(\frac{V_{j,Ed}}{b_j h_{jc}}\right)^2}{(v_d f_{cd} + f_{ctd})} - f_{ctd} \right] \quad \text{Design formulation}$$

Example



$$f_{yd} = 434.8 \text{ MPa}$$

$$f_{cd} = 19.83 \text{ MPa}$$

$$f_{ctd} = \frac{0.7 \cdot 0.3 \cdot f_{ck}^{2/3}}{1.5} = 1.5 \text{ MPa}$$

$$b_j = 400 \text{ mm}$$

$$h_{jc} = h_{jc} - 2c = 800 \text{ mm} - 60 \text{ mm} = 740 \text{ mm}$$

$$\eta = 0.6 \left(1 - \frac{f_{ck}}{250}\right) = 0.6 \left(1 - \frac{35}{250}\right) = 0.52$$

$$v_d = \frac{N_{Ed}}{A_c f_{cd}} = \frac{555000 \text{ N}}{400 \text{ mm} \cdot 800 \text{ mm} \cdot 19.83 \text{ MPa}} = 0.09$$

$$V_{j,Ed} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_c = 855 \text{ kN}$$

$$A_{sh,min} \geq \frac{b_j h_{jc}}{f_{yd}} \left[\frac{\left(\frac{V_{j,Ed}}{b_j h_{jc}}\right)^2}{(v_d f_{cd} + f_{ctd})} - f_{ctd} \right] = 737 \text{ mm}^2$$

$$A_{sh} = 804 \text{ mm}^2$$

(area of 8 stirrups with 2 legs in the joint)

$$\frac{A_{sh} f_{yd}}{b_j h_{jc}} = 1.18 > 1.08 = \frac{\left(\frac{V_{j,Ed}}{b_j h_{jc}}\right)^2}{(v_d f_{cd} + f_{ctd})} - f_{ctd}$$

5) Detailed design (joints)



5) Detailed design (damage limitation check)

4.4.3 Damage limitation

4.4.3.1 General

(1) The “damage limitation requirement” is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the “no-collapse requirement” in accordance with 2.1(1)P and 3.2.1(3), the interstorey drifts are limited in accordance with 4.4.3.2.

(2) Additional damage limitation verifications might be required in the case of buildings important for civil protection or containing sensitive equipment.

4.4.3.2 Limitation of interstorey drift

(1) Unless otherwise specified in Sections 5 to 9, the following limits shall be observed:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r v \leq 0,005 h; \quad (4.31)$$

b) for buildings having ductile non-structural elements:

$$d_r v \leq 0,0075 h; \quad (4.32)$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

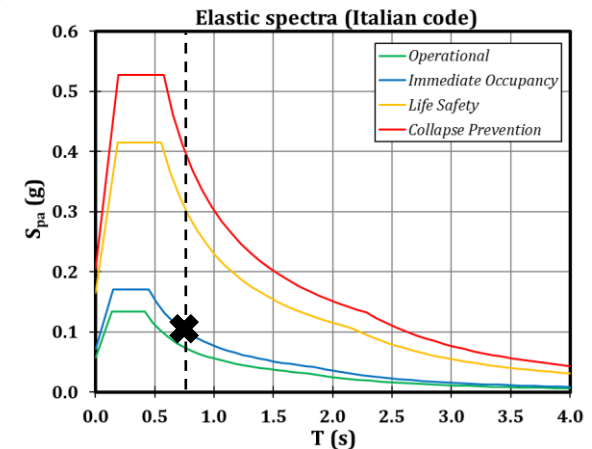
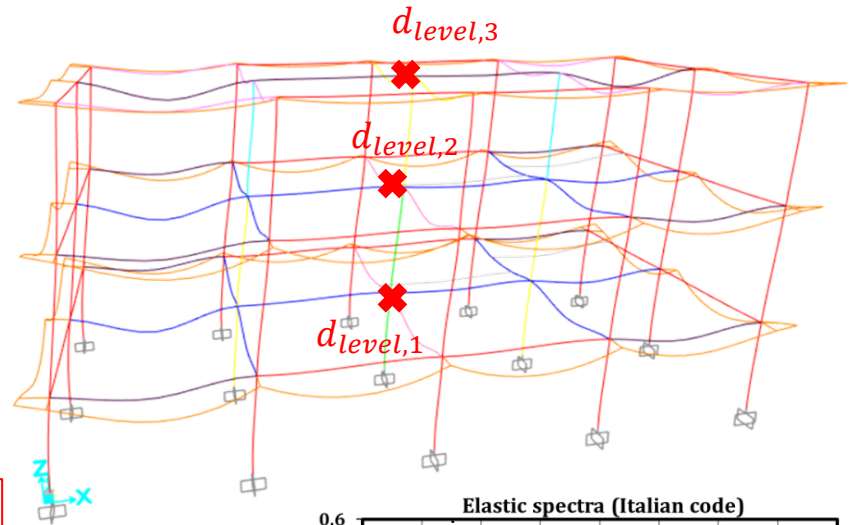
$$d_r v \leq 0,010 h \quad (4.33)$$

where

d_r is the design interstorey drift as defined in 4.4.2.2(2);

h is the storey height;

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.



$$q = 3.88 \quad v = \frac{S_{pa,IO}}{S_{pa,LS}} = \frac{0.130g}{0.376g} = 0.35 \quad \checkmark$$

$$q \cdot v \cdot \max\{d_{level,i+1} - d_{level,i}\} = 0.0025h_i < 0.005h_i$$

UZ PRIEKSU INŽENIERIJĀ

Paldies par uzmanību un sadarbību!

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

PhD. Eng. Massimo Latour

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