CHAPITRE 18. Exemple de calcul d'ossature en portique en béton armé.

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Note[1] !!! In this example, notation « dot .» stands for "coma ,". Example: 1.2 means 1,2
Note[2] !!! The example is used with γ_c = 1,5 and γ_s =1,15. The values are different in each country: France: γ_c = 1,3 and γ_s =1,0; Belgium: γ_c = 1,5 and γ_s =1,00.

I. INTRODUCTION

The design example of a reinforced concrete building which is presented hereafter aims at two main goals:

- To present the partially designing procedures of a reinforced concrete frame under a given seismic excitation according to Eurocode 8 and Eurocode 2
- To check the behaviours of the reinforced concrete frame which is correspondingly designed and detailed to Eurocode 8 under un-given seismic excitations by using Pushover analysis.

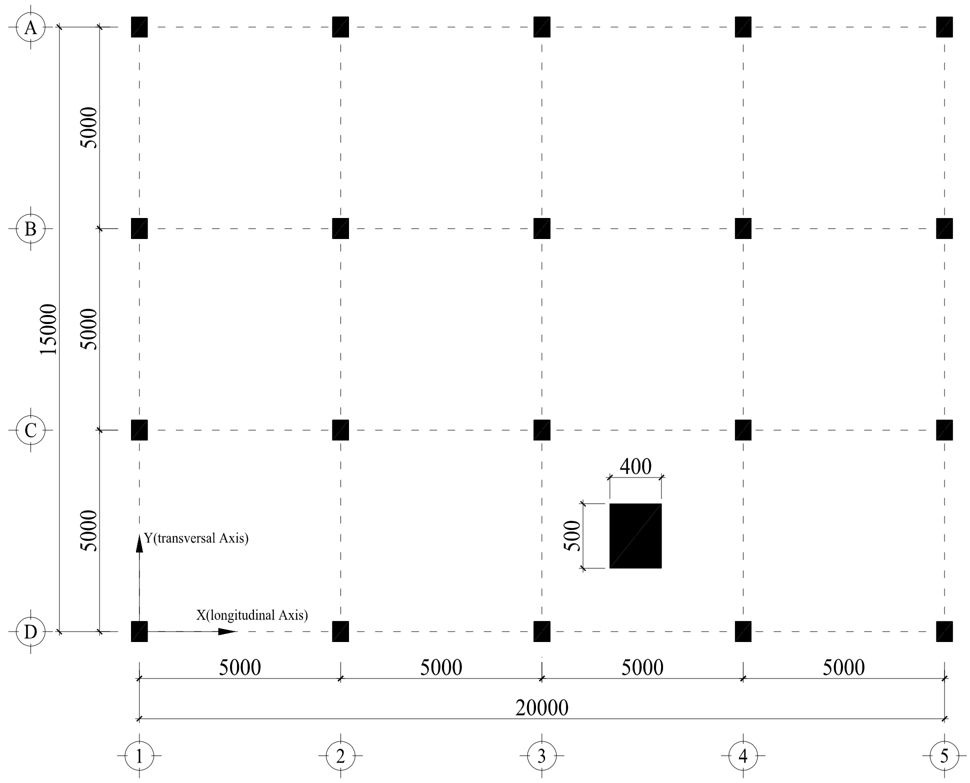
In order to get a fully designed and detailed reinforced concrete frame, there are several preliminary designing steps. The final drawings which are used on the sites are the results of series of calculations. To choose the best results, that is, the sectional dimensions, material properties, reinforcement areas, etc, the designing iterations must be carried out. The following presentation of the total design procedures is just some parts of the completely iterative processing.
The issues which are presented in the design example are:

- To describe the building architecture and properties such as materials, loads, …
- To check the chosen cross sectional dimensions in pre-design.
- To analyze the structure under a given seismic excitation.
- To verify the structural elements.
- To check the building under other seismic excitations.

II. GENERAL DESCRIPTION

The building which is chosen to design is an office and flat building. The building has 6 stories. It is a six-story reinforced concrete two-way frame. The floor plan is presented in Figure 1.

Figure 1. BUILDING PLAN



II.1. Main geometry descriptions:

There are 3 bays of 5m and 4 bays of 5m. The area of current floor is about 300m2 ($20 \times 15 = 300m^2$).

The structure has in-plane and elevation regularity.

The story height is 3m, except the ground story height is 3.5m.

The cross sectional dimensions for all columns are 400mmx500mm.

The slab thickness is 150mm; the dimensions of all beams are 250x500mm (slab included).

II.2. Exterior and partitioning walls:

The perimeter walls are glass and masonry ones. They do not affect the free displacement of the frame during earthquakes.

II.3. Loads:

The characteristic values for the loads are:

II.3.1 For the intermediate floor:

- Slab weight → 3.75kN/m^2
- Flooring → 1.92 kN/m^2
- Live load → 3 kN/m^2

II.3.2 For the roof floor:

- Slab weight → 3.75kN/m^2
- Flooring → 1.92 kN/m^2
- Live load → 0.75 kN/m^2
- snow load → 0.4 kN/m^2

II.4. Preliminary Considerations:

- Subsoil Class: C
- Ductility Level: DCM – Medium level.
- Important category of the building is "II" → ordinary building and $\gamma_I = 1$.
- The non-structural elements of the building are fixed in a manner as not to interfere with structural deformations.
- The structure is rigid fixed in non-deformable foundations.
- The relative design ground acceleration for the reference return period is $a_{gR} = 0.15g$.

II.5. Materials

- Concrete class: → C25/30 $E_{cm} = 31GPa = 31KN/mm^2$
- Longitudinal ribbed reinforcing steel bars → S500 was chosen.
- Transverse ribbed reinforcing steel bars → S500.

II.6. Design Procedures:

For the R/C multi-story flexible frame buildings, the inter-story drift control governs the design. → So, the pre-design procedures of the cross sectional dimensions of the frame members are the checks of horizontal displacements induced by the earthquakes.

III. PREDESIGN

III.1. Weights of Masses: - 3.2.4 – EC8

In accordance to 3.2.4 - EC8 [3], the inertial effects of the design seismic actions shall be evaluated by taking into account the presence of the masses associated to all gravity loads appearing in the following combination of actions:

$$\sum G_{kj} "+" \sum \psi_{E,i} Q_{k,i}$$

Where:

$\psi_{E,i}$: combination coefficients.

$\psi_{E,I}$ is determined as following 4.2.4 – EC8 $\psi_{E,i} = \varphi.\psi_2$

The value of φ is to be from the table 4.2 – 4.2.4 – EC8

$\varphi = 1.0 \rightarrow$ for the top story.

$\varphi = 0.8 \rightarrow$ for the correlated occupancies.

$\varphi = 0.5 \rightarrow$ for the roof story.

(EC8 table 4.2 for categories A-C* - domestic and residential and for stories independently occupied)

$\psi_{2,i}$: combination coefficients. Determining from the Annex A1:1990:2002, table A.1.1

$\psi_{2,i} = 0.3 \rightarrow$ for the occupancy (category A).

$\psi_{2,i} = 0 \rightarrow$ for the snow and wind loads.

So, the results are:

- For the intermediate stories:
  $W_{floori} = 3.75*300 + 1.92*300 + 0.8*0.3*3*300 = 1917KN$
- For the roof story:
  $W_{roof} = 3.75*300 + 1.92*300 + 1.0*0.3*0.75*300 + 1.0*0*0.4*300 = 1768.5KN$
  All weights of masses are calculated from 4.3.1(10P) – EC8 [3]
- Weight of the beams on floors:
  $W_b = 0.25*(0.5-0.15)*25*31(beams)*5m = 339KN*6 = 2034.4KN$ for all floors.
- Weight of the columns on floors:
  $W_c = (0.4*0.5*(3-0.5)*25*20(columns))*5\,floors + (0.4*0.5*(3.5-0.5)*25*20(columns)) = 1550KN$ for all floors.
- Total weights of the building:
  $W = \sum W_{floori} + W_{roof\ floor} + W_b + W_c = 1917*5 + 1768.5 + 2034.4 + 1550 = 14938KN$

III.2. Base shear force: (4.3.3.2.2 – EC8 [3])

- According to 4.3.3.2.2 – EC8 [3], Base shear force induced by an earthquake is determined as the following expression:

$$F_b = S_d(T_1)*m*\lambda \quad (4.5 - \text{EC8 [3]})$$

  Where:
  - $S_d(T_1)$ – the ordinate of the design spectrum at period T_1
  - T_1: The fundamental period of vibration of the building for lateral motion in direction considered.
  - m: is the total mass of the building. $m = \dfrac{W}{g}$
  - λ: is correction factor.

    λ = 0.85 if $T_1 \le 2Tc$ and the stories of the buildings ≥ 2 stories

    λ = 1 if otherwise
- According to 4.3.3.2 (3) – EC8 [3], for the buildings with heights up to 40m, the value of $T_1(s)$ may be approximated by the following expression:
  $T_1 = C_t * H^{3/4}$ (4.6 – EC8 [3])
  Where:
  - C_t =0.75 for the concrete frames.
  - H – is the height of the building (m); $H = 3.5*+5*3 = 18.5m$

    So, we have:

    $T_1 = 0.75*18.5^{3/4} = 0.67(s)$
- $S_d(T_1)$ – the ordinate of the design spectrum at period T_1 is determined from 3.2.2.5 – EC8.
  With subsoil class "C", we have, for the Type 1 spectrum:

$$\begin{cases} S = 1.15 \\ T_B(s) = 0.2(s) \\ T_C(s) = 0.6(s) \\ T_D(s) = 2(s) \end{cases}$$ Table 3.2 – Type 1 – 3.2.2.2 – EC8 [3]

- For $\begin{cases} S = 1.15 \\ T_B(s) = 0.2(s) \\ T_C(s) = 0.6(s) \\ T_D(s) = 2(s) \end{cases}$ and T_1 = 0.67(s) → so we have $T_C < T_1 < T_D$ and $S_d(T_1)$ is calculated from the expression 3.14 (3.2.2.5 – EC8 [3]).

$$S_d(T_1) = \begin{cases} a_g * S * \frac{2.5}{q} * \left(\frac{T_C}{T}\right) \\ \geq \beta * a_g \end{cases}$$

Where:

- $a_g = \gamma_I * a_{gR} = 1*0.15g$ – 3.2.1(3)
- β - is the lower bound factor for the horizontal design spectrum, β=0.2.
- S=1.15
- q – is the behaviour factor calculated from 5.2.2.2 – EC8. In accordance to 5.2.2.2 – EC8 [3], q is to be calculated from the following expression: $q = q_0 * k_w \geq 1.5$.
    - q_0 is the basic value of the behaviour factor determined from table 5.1 – 5.2.2.2 - EC8. For the concrete frames with the "DCM" ductility class, q_0 is calculated by the following expression: $q_0 = 3.0 * \frac{\alpha_u}{\alpha_1}$, in which the ratio, $\frac{\alpha_u}{\alpha_1} = 1,3$, is calculated from 5.2.2.2 (5a) – EC8. So the value of q_0 is 3.0*1.3=3.9
    - k_w is the factor reflecting the prevailing failure mode in structural systems with walls. k_w is calculated from 5.2.2.2 (11). So the value of k_w is 1.00

  So, the value of q is determined as below: q=3.9*1=3.9

$$\Rightarrow S_d(T_1) = \begin{cases} 0.15g * 1.15 * \frac{2.5}{3.9} * \left(\frac{0.6}{0.67}\right) = 0.099g \\ \geq 0.2 * 0.2g = 0.04g \end{cases} = 0.099g$$

$$\Rightarrow F_b = S_d(T_1) * m * \lambda = 0.099g * \frac{W}{g} * 0.85 = 0.08415W$$

$$\Rightarrow F_b = 1257KN$$

III.3. Torsion Effects

- The torsion effect is taken into account for the transverse current frame in a simplified manner. (4.3.3.2.4 – EC8 [3]).
- According to 4.3.3.2.4 – EC8 [3], if the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity of 4.3.2(1)P is taken into account by a more exact method, the accidental torsion effects may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of 4.3.3.2.3(4) by a factor δ given by: $\delta = 1 + 0.6 * \frac{x}{L_e}$. If the building is distributed symmetrically in plan and elevation, it can be divided into 2 plane models and the factor δ is determined by $\delta = 1 + 1.2 * \frac{x}{L_e}$.

- According to the building plan, we can determine the values of x and L_e as follow: x=5m for frame at line 2, x=10 for frame at line 5; L_e=20m.
- So, the value of δ is calculated by the following expression: $\delta = 1 + 0.6 * \frac{x}{L_e} = 1 + 0.6 * \frac{5}{20} = 1.15$ and

  δ=1.3 for line 5. However, we have seen in (Plumier, Construction en zone sismique,[9]) that in fact a realistic δ for such building is rather δ=1.15. In this pre-design step the value of δ = 1.15 will be used. It should be checked at the final design state.

III.4. <u>Seismic force distribution:</u>

- Seismic forces distributed to all frames of the building depend on both their stiffness and their positions in plan, due to torsion.
- Force distribution along the height of the building using the simplified formula 4.11 or 4.10 – EC8 [3].
- In accordance to 4.3.3.2.3 (1) – EC8 [3], the fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.
- According to 4.3.3.2.3(2) – EC8 [3], the seismic action effects shall be determined by applying, to the two planar models, horizontal forces, F_i, to all stories.

$$F_i = F_b * \frac{s_i * m_i}{\sum s_j * m_j} \quad (4.10)$$

  Where:
  - F_i – is the horizontal force acting on the story i.
  - F_b – is the seismic base shear.
  - s_i, s_j – displacements of masses m_i, m_j in the fundamental mode shape.
  - m_i, m_j - are the story masses.
- According to 4.3.3.2.3(3) – EC8 [3], when the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i are given by:

$$F_i = F_b * \frac{z_i * m_i}{\sum z_j * m_j} \quad (4.11 – EC8 [3])$$

  Where:
  - F_i – is the horizontal force acting on the story i.
  - F_b – is the seismic base shear.
  - z_i, z_j – heights of the masses m_i, m_j above the level of application of the seismic action..
  - m_i, m_j - are the story masses.
- So, the values of F_i can be calculated as the following table:

Table III.1 Horizontal seismic force Distribution

Story	Height (z_i) m	Weight – w_i	z_i*w_i	$\frac{z_i * w_i}{\sum z_j * w_j}$	F_i(KN)
6	18.5	2250.375	41631.94	0.267369	321.3376
5	15.5	2398.875	37182.56	0.238794	286.995
4	12.5	2398.875	29985.94	0.192576	231.4475
3	9.5	2398.875	22789.31	0.146358	175.9001
2	6.5	2398.875	15592.69	0.100139	120.3527
1	3.5	2436.375	8527.313	0.054764	65.81837

Fb = 1257 [KN].

- The above seismic forces, F_i, are total seismic forces acting at each story for the whome building and all frames. According to the stiffness of each frame, we will distribute the seismic forces to each frame linearly including torsion effects.

- In the action direction of the earthquake (direction Y or transverse direction), there are 5 portal frames. We will distribute the seismic forces to the transverse frame at line 2 as following:
  - $F_{2-6} = F_6 * \frac{1}{5} * \delta = 321.34 * \frac{1}{5} * 1.15 = 73.9KN$. $F_{2-5} = F_5 * \frac{1}{5} * \delta = 287.0 * \frac{1}{5} * 1.15 = 66.0KN$.
  - $F_{2-4} = F_4 * \frac{1}{5} * \delta = 231.5 * \frac{1}{5} * 1.15 = 53.23KN$. $F_{2-3} = F_3 * \frac{1}{5} * \delta = 175.9 * \frac{1}{5} * 1.15 = 40.5KN$.
  - $F_{2-2} = F_2 * \frac{1}{5} * \delta = 120.4 * \frac{1}{5} * 1.15 = 27.7KN$. $F_{2-1} = F_1 * \frac{1}{5} * \delta = 65.8 * \frac{1}{5} * 1.15 = 15.14KN$.

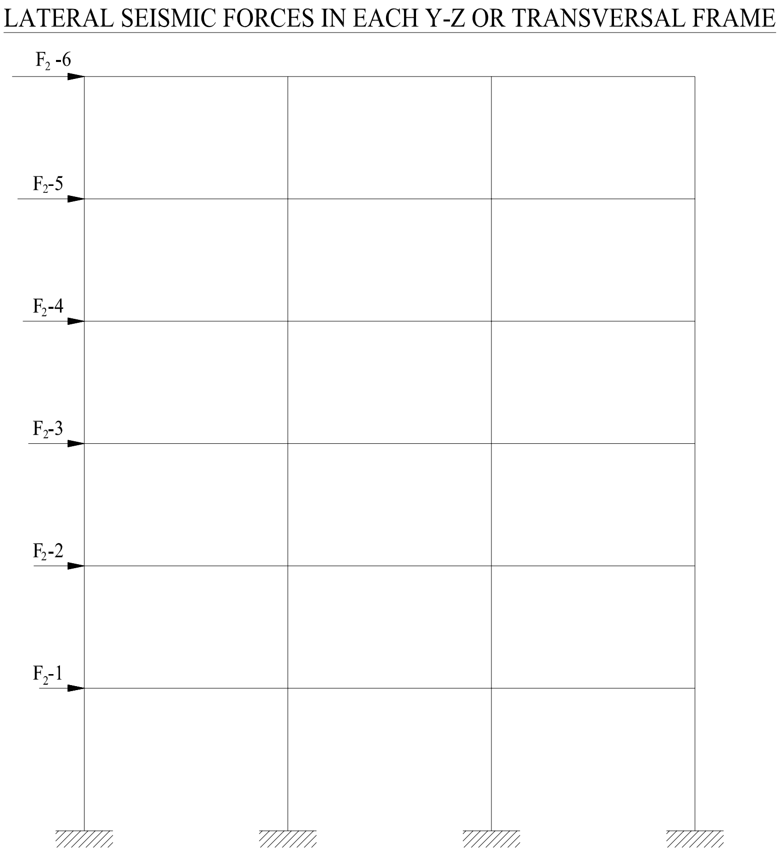


Figure 2 – Lateral seismic forces

III.5. The limitation of the inter-story drifts:

- A plane frame is analysed and the displacements of the frame subjected to the applied forces F_{2-1} to F_{2-6} , which are computed above, will be determined by using SAP 2000. Version 9.0.3. According to 4.3.1(7) EC8[3], the elastic modulus of Concrete E=E/2= 15,5KN/mm^2.
- According to 4.4.3.2 – EC8 [3], the following limits shall be observed: For the buildings having non-structural elements of ductile materials attached to the structure:

$$d_r * v \leq 0.0075 * h$$

  Where:
  - h: is the story height → h=3m and h=3.5m
  - d_r: is design inter-story drift as defined in 4.4.2.2 (2) – EC8 [3], evaluated as the difference of the average lateral displacements d_s at the top and bottom of the story under consideration and calculated according to 4.3.4 → $d_r = d_{s_i} - d_{s_{i-1}}$.

- According to 4.3.4 – EC8 [3], for displacement analysis, if linear analysis is performed the displacements induced by the design seismic action shall be calculated on the elastic deformations of the structural system by means of the following simplified expression:

$$d_s = q_d * d_e$$

Where:

+ d_s: is the displacement of a point of the structural system induced by the seismic action.

+ d_e: is the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum according to 3.2.2.5 – EC8.

+ q_d: is the displacement behaviour factor, assumed equal to q unless otherwise specified. So $q_d = q = 3.9$.

- ν: is the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement. The value of ν also depends on the important class of the building. The important class of the building is “II” so the value of ν is 0.5 (according to 4.4.3.2(2) – EC8 [3]).

- The value of d_s must be smaller than the value derived from the elastic spectrum.
- When determining d_e, the torsion effects of the seismic actions shall be accounted for.
- Table of drifts:

Table III.2 – Story Drifts

Story	Story elastic displacements. (d_e - mm)	Behaviour factor of Displacement	ds_i	$d_r=ds_i-ds_{i-1}$	$d_r*\nu$	Drift from EC8 0.0075h mm)
6	46.373	3.9	180.8547	14.1297	7.06485	22.5
5	42.75		166.725	23.4429	11.72145	22.5
4	36.739		143.2821	31.7733	15.88665	22.5
3	28.592		111.5088	37.8339	18.91695	22.5
2	18.891		73.6749	40.2597	20.12985	22.5
1	8.568		33.4152	33.4152	16.7076	26.25

So, the condition $d_r * \nu \leq 0.0075 * h$ is met.

The section dimensions and material properties which are chosen are satisfied with pre-designed steps according to EC8.

IV. **PRELIMINARY STEPS:** According to EC8 – 4.

IV.1. Structural regularity.

IV.1.1 Regularity in plan:

According to 4.2.3.2 – EC8 [3], criteria for the regularity in plan are:

- The building structure is symmetrical in plan with respect to two orthogonal directions.
- The plan configuration is compact.
- The in-plane stiffness of the floors is sufficiently large to distribute seismic forces among the vertical structural elements.
- The slenderness $\lambda = \frac{L_{\max}}{L_{\min}}$ of the building is $\lambda = \frac{L_{\max}}{L_{\min}} = \frac{20}{15} < 4$.
- In accordance to 4.2.3.2 (5) – EC8 [3], at each level for each direction of analysis x or y, the structural eccentricity e_0 and the torsional radius r verify the two conditions below, which are expressed for the direction of analysis y:

$$\begin{Bmatrix} e_{0X} \leq 0.3 * r_X \\ r_X > l_s \end{Bmatrix} \text{ (4.1-EC8 [3])}$$

Where:

- e_{0X}: is the distance between the centre of stiffness and the centre of mass, measured along x direction, which is normal to the direction of analysis considered.
- r_X: is square root of the ratio between torsional effects and lateral stiffness in y direction (torsional radius)
- l_s: is the radius of gyration of the floor mass in plan (square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the centre of mass of the floor to (b) the floor mass).

- There are not setbacks in the plan.

IV.1.2 Regularity in elevation:

According to 4.2.3.3 – EC8 [3], criteria for regularity in elevation are:

- All the lateral resisting systems run without interruption from their foundations to the top of the building.
- Both lateral stiffness and the mass remain constant or reduce gradually without abrupt changes, from the base to top.
- There are not setbacks.

IV.2. Structural Analysis: (4.3 – EC8 [3])

IV.2.1 Modelling:

- Because of the in-plan and in-elevation regularity, in accordance to 4.3.1(5) and to the table 4.1(4.2.3.1) the allowed simplifications are:
  - The analytical model: Planar.
  - The method of analysis: Using simplified method → Lateral force method of analysis can be used because all the conditions of 4.3.3.2.1 are met:
    - The building has fundamental period of vibration T_1 in the two main directions smaller than the following values:

$$T_1 \leq \begin{cases} 4 * T_C \\ 2.0s \end{cases} \Leftrightarrow T_1 = 0.67s \leq \begin{cases} 4 * 0.6 = 2.4s \\ 2.0s \end{cases}$$

    - All the criteria for regularity in elevation given in 4.2.3.3 – EC8 are met.
  - Behaviour factor: is the reference value.
- The building will be analysed with two planar frames using the lateral force method and comparison with two planar frames using the response spectrum analysis.

IV.2.2 Natural Periods:

According to 4.3.3.2 – EC8 [2], the natural period can be determined by Reileight Method or approximated method. The first period T_1 defined by approximated formula is equal to the value of 0.67.

IV.2.3 Local effects of infill

There is no infill for the current transverse and longitudinal frames.

IV.3. Verification of structural type:

IV.3.1 Torsional Rigidity:

In accordance to 5.2.2.1 (4)P – EC8 [3], the first four types of systems (i.e. frame, dual and wall systems of both types) shall possess a minimum torsional rigidity that satisfies expression (4.1b) in both horizontal directions.

But in accordance to 5.2.2.1(5), for frame or wall systems with vertical elements that are well distributed in plan, the requirement specified in (4)P above may be considered as being satisfied without analytical verification.

IV.4. Selection of ductility class

The chosen ductility class for design is "DCM". So, designing, dimensioning and detailing must ensure a ductile behaviour of the elements meaning that ductile modes of failure should precede failure modes with sufficient reliability. The plastic hinges which are developed in response to the seismic excitation must be able to dissipate a medium amount of energy in a stable manner.

IV.5. Material checks

IV.5.1 Concrete

In accordance to 5.4.1.1 – EC8 [3], for ductility class "DCM" the use of concrete class which is lower than C16/20 is not allowed in primary seismic elements. So, we choose the concrete class C25/30.

IV.5.2 Flexural reinforcement steel

In accordance to 5.4.1.1 – EC8 [3], only ribbed bars are allowed as reinforcing steel in critical sections of primary seismic elements. The reinforcing steel class S500, the high ductility steel that satisfies the additional requirements in critical regions concerned in table C.1, annex C – EC2 [2], is chosen.

IV.5.3 Shear reinforcement steel

In accordance to 5.4.1.1 (2P) – EC8 [3], except for the closed stirrups or cross-ties, only ribbed bars are allowed as reinforcing steel in critical of primary seismic elements. The reinforcing steel class S500 for flexural reinforcement steel was chosen. So, we also choose S500 for shear reinforcement steel.

IV.6. Second order Effects(P-Δ)

According to 4.2.2.2 (2) - EC8 [3], the second-order effects (P-Δ effects) need not to be taken into account if the following condition is fulfilled in all stories:

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \leq 0.1 (4.28 - \text{EC8 [3]})$$

Where:

- θ – is the inter-story drift sensitivity coefficient.
- P_{tot} – is the total gravity load at and above the story considered in the seismic design situation.
- $d_r = d_{s_i} - d_{s_{i-1}}$ - is design inter-story drift, evaluated as the difference of the average lateral displacements d_s at the top and the bottom of the story under consideration.
- V_{tot} – is the total seismic shear at the considered level.

At the ground story: $\theta = \frac{P_{tot} * d_r}{V_{tot} * h} = \frac{P_{tot}}{V_{tot}} * \frac{d_r}{h} = \frac{14282}{1257} * \frac{34}{3500} = 0.1104 > 0.1$

At the intermediate story: $\theta = \frac{P_{tot} * d_r}{V_{tot} * h} = \frac{11846}{1191} * \frac{40.3}{3000} = 0.134 > 0.1$

So, the second-order effects cannot be neglected.

V. BEAM DESIGN AND VERIFICATION: According to EC8 [3]– 4 and EC2[2].

V.1. TRANSVERSE CURRENT FRAME OR DIRECTION Y CURRENT FRAME:

V.1.1 Action effects: (According to 5.4.2 – EC8 [3])

- In accordance to 5.4.2 – EC8 [3], for the beams with ductility DCM, the design values for the building moments shall be obtained from the analysis of the structures for the seismic situation according to 6.4.3.4 – EN 1990. Combination of actions for seismic design situations is calculated as following expression: $\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} A_{Ed} \text{"+"} \sum_{i\geq 1} \psi_{2,i} * Q_{k,i}$

  Where:
  - $G_{k,j}$ – is the permanent or persistent action j.
  - P – is the pre-stressing action.
  - A_{Ed} – is the design value of seismic actions for the reference return period (design spectrum)
- The loads were uniformly distributed along the length of the beam. No redistribution of the bending moments was made.
- The design value of the shear forces shall be determined in accordance with the capacity design rules, EC 8 – 5.4.2.2 – 1(P), considering the equilibrium of the beam under: a) the transverse load acting on it with the seismic design situation and b) end moment $M_{i,d}$ (with i=1,2 – denoting the end sections of the beam), corresponding, for each sense of seismic action, to plastic hinge formation at the end of either of the beam or of the vertical elements, which ever takes place first – which are connected to the joint where the beam end and i frame into.
- The analysis is performed using two planar models, one for each main direction.
- The torsion effects were determined separately by these two dimensions according to 4.3.3.2.4(2) – EC8: If the analysis is performed using two planar models, one for each main horizontal direction, torsion effects may be determined by doubling the accidental eccentricity e_{ai} of the expression $e_{ai} = \pm 0.05 L_i$ and applying the rules of 4.3.3.2.4 (1) – EC8 with the factor δ, $\delta = 1 + 0.6 * \frac{x}{L_e}$, replaced by the factor δ, $\delta = 1 + 1.2 * \frac{x}{L_e}$.
- Because of the symmetry, the actual eccentricity between stiffness centre S and the nominal mass centre M, $e_0 = 0$, is equal to 0, and the additional eccentricity, e_2, taking into account of the dynamic effect of simultaneous transitional and torsional vibration, can not be computed.

  → So, the only eccentricity taken into account is accidental torsional effect. $e_1 = e_{ai} = \pm 0.05 L_i$ (When using the Response Spectrum Analysis)

  Where:
  - e_{ai} – is the accidental eccentricity of the storey mass from its nominal location, applied in the same direction at all floors.
  - L_i – is floor dimension perpendicular to the direction of the seismic action.

V.1.2 Action Summary:

V.1.2.1 Gravity actions:

- DEAD Load: The self-weight load.
- DL slab: The dead loads induced by the floor and coating weight.
- LL slab: The live loads induced by the variable actions
- LL roof slab: The live loads induced by the variable roof actions
- Snow load: The loads induced by the snow.
- Joint load: The loads acting to the joints of the transverse frame induced by perpendicular frames.
- WIND load.

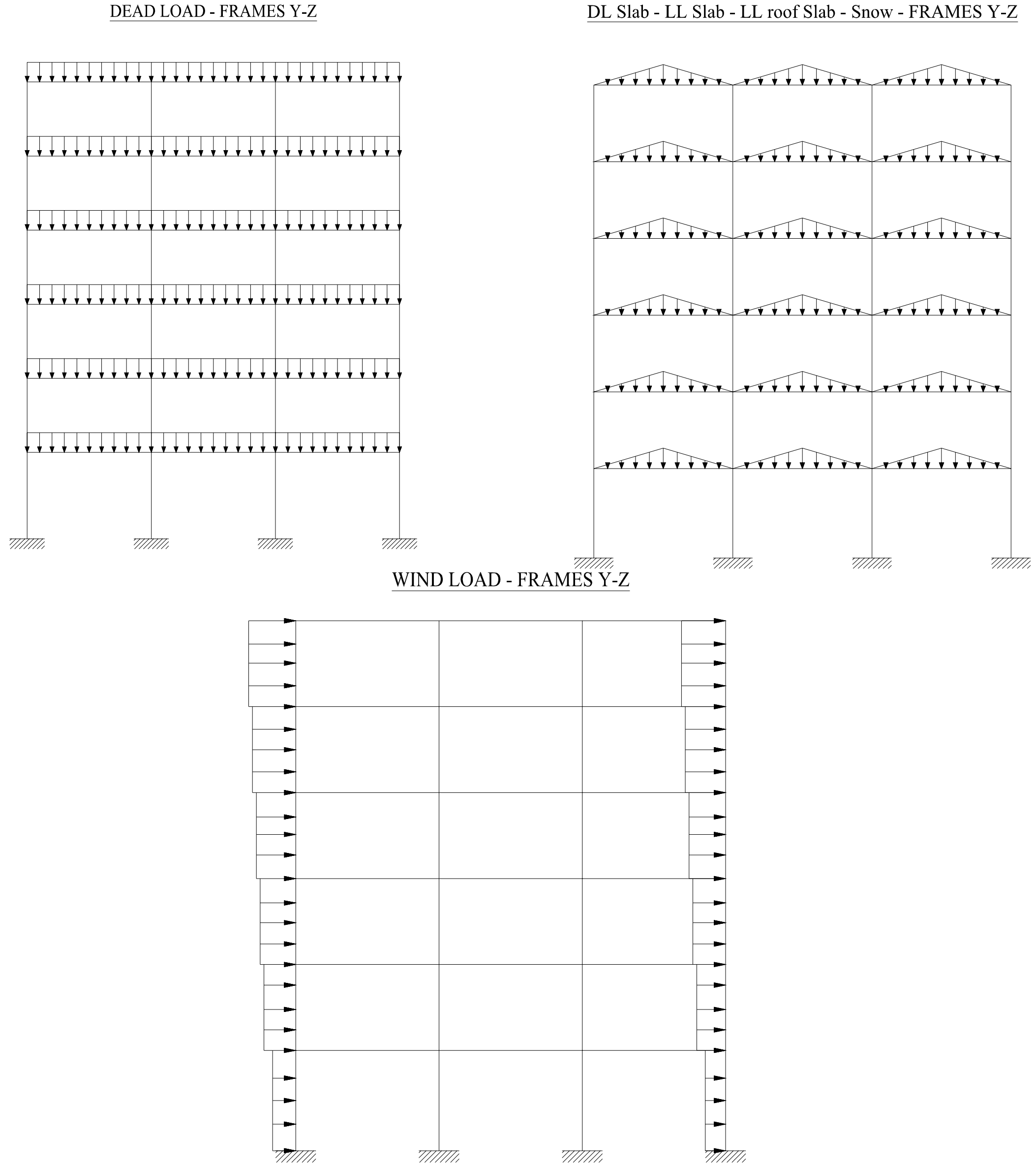


Figure 3 – Gravity Loads

V.1.2.2 <u>Seismic actions</u>

- Seismic actions used to analyse in the frames will be determined by two methods of analysis: Lateral Force Analysis and Response Spectrum Analysis.
- The analysis is performed using two planar models, one for each main direction.
- Lateral force Analysis:
  - The torsion effects were determined separately by these two dimensions according to 4.3.3.2.4(2) – EC8: If the analysis is performed using two planar models, one for each main horizontal direction, torsion effects may be determined by doubling the accidental eccentricity e_{ai} of the expression $e_{ai} = \pm 0.05 L_i$ and applying the rules of 4.3.3.2.4 (1) – EC8 with the factor δ, $\delta = 1 + 0.6 * \frac{x}{L_e}$, replaced by the factor δ, $\delta = 1 + 1.2 * \frac{x}{L_e}$. So we have δ = 1.3
  - The above seismic forces are total seismic acting in all of the building or all frames. According to the stiffness of each frame, we will distribute the seismic forces to each frame linearly including torsion effects.
  - In the action direction of the earthquake, there are 5 portal frames. When distributing seismic forces to all floors of the frame, the torsion effects will be taken into account by the factor δ which is calculated from 4.3.3.2.4 (1) – EC8. Factor δ, here, accounts for the analysis model

with two planar directions. We will distribute the seismic forces to the current transverse frame as following:

- $F_{2-6} = F_6 * \frac{1}{5} * \delta = 321.34 * \frac{1}{5} * 1.3 = 83.54KN$.
- $F_{2-5} = F_5 * \frac{1}{5} * \delta = 287.0 * \frac{1}{5} * 1.3 = 74.6KN$.
- $F_{2-4} = F_4 * \frac{1}{5} * \delta = 231.5 * \frac{1}{5} * 1.3 = 60.2KN$.
- $F_{2-3} = F_3 * \frac{1}{5} * \delta = 175.9 * \frac{1}{5} * 1.3 = 45.8KN$.
- $F_{2-2} = F_2 * \frac{1}{5} * \delta = 120.4 * \frac{1}{5} * 1.3 = 31.3KN$.
- $F_{2-1} = F_1 * \frac{1}{5} * \delta = 65.8 * \frac{1}{5} * 1.3 = 17.11KN$.

- Response Spectrum Analysis:
  - According to 4.3.3.3.1 – EC8 [2], the response of all modes of vibration contributing significantly to the global response shall be taken into account. The requirements may be deemed to be satisfied if either of the following can be demonstrated:
    - the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
    - All modes with effective modal masses greater than 5% of the total mass are taken into account.
  - When using a spatial model, the above conditions should be verified for each relevant direction.
  - According to 4.3.3.3.2 – EC8 [2], whenever all relevant modal responses may be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as: $E_E = \sqrt{\sum E_{Ei}^2}$.
  - According to 4.3.3.3.3 – EC8 [2], whenever a spatial model is used for the analysis, the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments
    - M_{ai} about the vertical axis of each storey i: $M_{ai} = e_{ai} \cdot F_i$

    Where:

    M_{ai} is the torsional moment applied at storey i about its vertical axis.

    e_{ai} is the accidental eccentricity of storey mass i

    F_i is the horizontal force acting on storey i, as derived in **4.3.3.2.3** for all relevant directions.
  - According to 4.3.3.3.3 (2) - EC8 [2], the effects of the torsional loadings should be taken into account with positive and negative signs (the same sign for all storeys). Whenever two separate planar models are used for the analysis, the torsional effects may be accounted for by applying the rules of **4.3.3.2.4(2)** to the action effects computed in accordance with **4.3.3.3.2**. → From all things mentioned above, SAP2000 will be used to analyse the structure with Modal Response Spectrum. The analysis will run accordingly to EC8.

V.1.3 COMBINATIONS:

According to EC8 [3], there are 3 combinations determined from all above actions:

- Combination 1: DEAD Load + DL Slab + Joint Load + Seismic Load + $\sum \psi_{2,i} * Q_k, i$

- Combination 2(in opposite direction): DEAD Load + DL Slab + Joint Load - Seismic Load + $\sum \psi_{2,i} * Q_k, i$
- Combination3: Envelope of Combination 1 and Combination2.

V.1.4 INTERNAL FORCES:

The internal forces will be determined by SAP2000, version 9.0. The internal forces, which are used to design the section reinforcement of the frames, are determined by Lateral Force Analysis method.

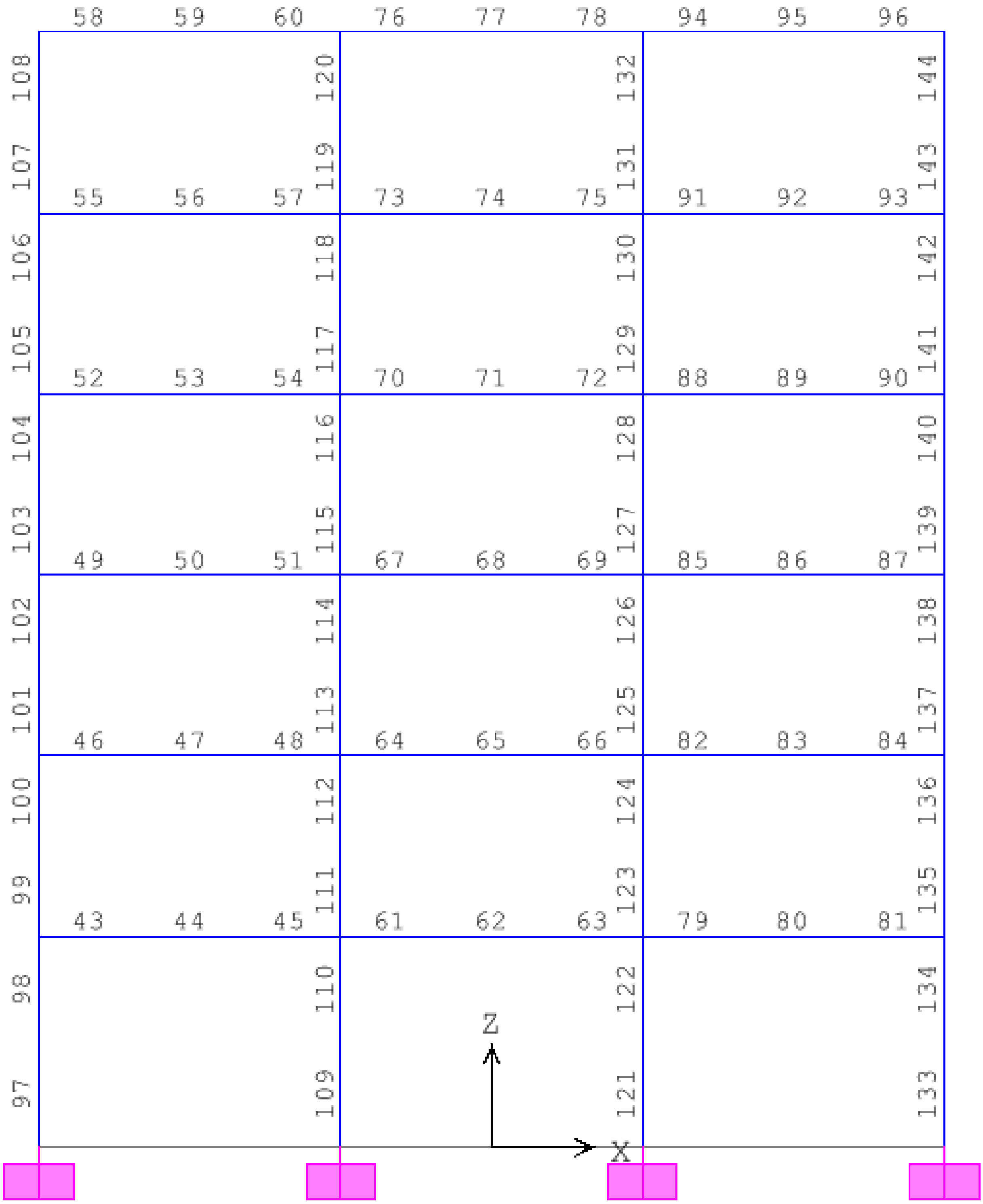


Figure 4 - Transverse or Direction Y Frame Line 2

TABLE: Element Forces – Transverse Y current Frames – First Story Beams							
Frame	**Section**	**OutputCase**	**CaseType**	**StepType**	**P=M_{Ed} Axial**	**V2=V_{Ed} Shear**	**M3=M_{Ed} Moment**
Text	m	Text	Text	Text	KN	KN	KN-m
43	0	envelopeofseism	Combination	Max	9.864	5.006	92.5335
43	0	envelopeofseism	Combination	Min	9.256	-102.682	-186.834
44	0.83333	envelopeofseism	Combination	Max	9.864	53.883	36.0313
44	0.83333	envelopeofseism	Combination	Min	9.256	-53.805	25.8847
45	1.66667	envelopeofseism	Combination	Max	9.864	102.761	82.1908
45	1.66667	envelopeofseism	Combination	Min	9.256	-4.927	-176.8834
61	0	envelopeofseism	Combination	Max	18.518	0.107	73.302
61	0	envelopeofseism	Combination	Min	1.907	-97.862	-171.6978
62	0.83333	envelopeofseism	Combination	Max	18.518	48.985	29.0468
62	0.83333	envelopeofseism	Combination	Min	1.907	-48.985	28.9698
63	1.66667	envelopeofseism	Combination	Max	18.518	97.862	73.225
63	1.66667	envelopeofseism	Combination	Min	1.907	-0.107	-171.6209
79	0	envelopeofseism	Combination	Max	25.951	4.71	81.7629
79	0	envelopeofseism	Combination	Min	-6.831	-102.543	-176.4554
80	0.83333	envelopeofseism	Combination	Max	25.951	53.587	35.9144
80	0.83333	envelopeofseism	Combination	Min	-6.831	-53.665	26.0016
81	1.66667	envelopeofseism	Combination	Max	25.951	102.465	91.8717
81	1.66667	envelopeofseism	Combination	Min	-6.831	-4.788	-186.1723

V.1.5 DESIGN OF BEAMS OF THE FIRST STORY IN FRAME LINE 2(members 43-45, 61-63 and 79-81)

V.1.5.1 Geometrical Restraints

- Effective flange width:

According to 5.4.3.1.1(3) – EC8 [3], the effective flange width b_{eff} may be assumed as follows:

- For primary seismic beams framing into exterior columns, the effective flange width, b_{eff}, is taken to the width b_c of the column in the absence of the transverse beam, or equal to this width increased by $2h_f$ on each side of the beam if there is transverse beam of similar depth.
- For the primary seismic beams framing into interior columns the above length may be increased by $2h_f$ on each side of the beam.

So, the effective flange width of beams framing to the exterior columns is:

$$b_c = 400mm \ ; \ h_f = 150mm \ \rightarrow \ b_{eff} = b_c + 2*2*h_f = 1000mm$$

The effective flange width of beams framing to the interior columns is:

$$b_c = 400mm \ ; \ h_f = 150mm \ \rightarrow \ b_{eff} = b_c + 2*4*h_f = 1600mm$$

- Beam – Column centroidal axis distance :

The beam framing symmetrically into the exterior columns has the eccentricity as following expression: e = 0.00mm

- Minimum width of the beams :
  - In accordance to 5.4.1.2.1 – EC8 [3], the effective transfer of cyclic moments from a primary seismic beam to a column shall be achieved, by limiting the eccentricity of the beam axis relative to that of the column into which it frames.
  - A deemed to satisfy rule for 5.4.1.2.1 – EC8 is to limit the distance between the centroidal axes of the two members to less than $b_c/4$. $\rightarrow \begin{cases} b_w \le \min\{b_c + h_w; 2b_c\} \\ b_w \ge 200mm \end{cases}$

- Width to height ratio of the web of the beam:

In accordance to 4.3.5.7 – EC2 [2], lateral buckling of the slender beams:

- (P1), where the safety of beams against lateral buckling is in doubt, it shall be checked by an appropriate method.
- (P2), the width to height ratio of the beam's web must be ensure to the following condition: $h_w < 2.5 * b_w$, so $\left.\begin{array}{l} h_w = 500mm \\ b_w = 250 \end{array}\right\} \rightarrow 350 < 2.5*250 = 625$

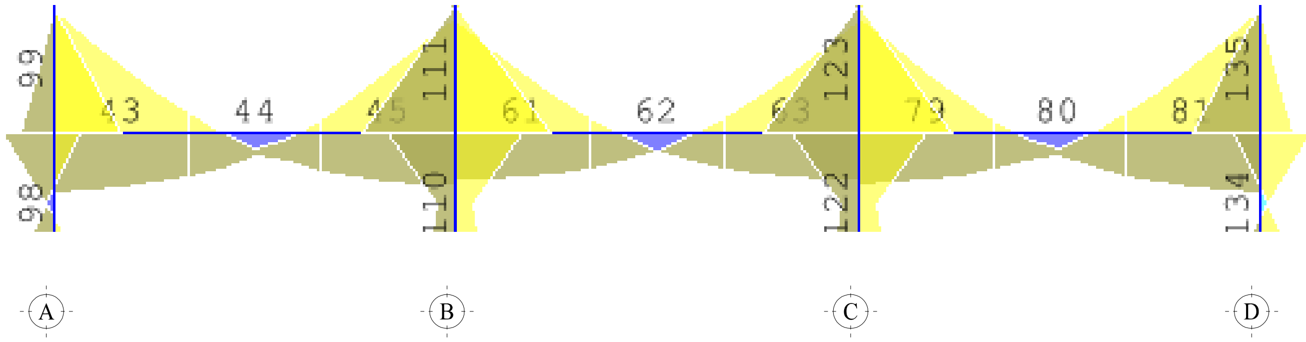
- Limitation of the beam width:

According to 5.4.1.2.1 – EC8 [3], the beam width has to be checked as following condition:

$b_w \le \min(b_c + h_w; 2*b_c) \;\rightarrow\; b_w = 250 \le \begin{cases} 250+500=750 \\ 2*250=500 \end{cases}$. This condition is met.

V.1.5.2 <u>Flexural reinforcement – Ultimate limit States</u>

- Bending moment envelope diagram is presented as following:



- The reinforcement of the sections will be calculated by using EC2.
- The actual strength of the materials:
  - Concrete:
    As chosen above, the concrete class is C25/30, according to 3.1.2.4 – EC2 [2], concrete material properties are:

$$f_{ck} = 25MPa = 25N/mm^2;\; f_{cm} = 33MPa = 33N/mm^2;\; f_{ctm} = 2.6MPa = 2.6N/mm^2$$

$$f_{cd} = \frac{\alpha_c * f_{ck}}{\gamma_c} = 16.67MPa = 16.67N/mm^2; \alpha_c = 1; \gamma_c = 1.5$$

(Note: In Belgium γ_C = 1.5; In France γ_C = 1.3)

  - Reinforcing steel:
    As chosen above, the steel class is S500, according to EC2 and EC3, steel material properties are:

$$f_{yk} = 500MPa = 500N/mm^2$$

$$f_{yd} = \frac{\alpha * f_{yk}}{\gamma_s} = \frac{1*500}{1.15} = 434.8MPa = 434.8N/mm^2$$

(Note: In Belgium γ_S = 1.0; In France γ_S = 1.0)

  - In accordance to ultimate limit states of bending plus axial force design procedure of 6.1 - EC 2 and 3.1.5 – EC2 [2], we will use a rectangular diagram for compressed concrete block. In this case, the value of η is 1, so $\eta * f_{cd} = 1 * f_{cd} = 16.7$ MPa.

- Flexural Reinforcing steel of Left – side of first span of the beam (Axis A)

$$M^+ = 92534Nm$$

$$M^- = -186834Nm$$

  - For M^+:
    - Beam's dimensions: b=250mm; h=500mm.
    - Cover:

$c_{nom} = c_{min} + \Delta c_{dev}$ (Expression 4.1 – EC2 [2])

Where:

$c_{min} = \max[c_{min,b}; c_{min,dur}]$. (According to Expression 4.2 – EC2[2])

$c_{min,b}$ = diameter of bar. Assume 20mm bars and 6 mm hoops - Table 4.2 – EC2[2].

$c_{min, dur}$ = minimum cover due to environmental conditions. Assuming that Exposure class is XC1 and Structural Class is S4 $\rightarrow c_{min,dur}$ = 15mm

Δc_{dev} = 10mm

$\rightarrow c_{nom}$ = 20 +10 =30 mm

- Concrete cover thickness + ½ reinforcement diameter = 30 + 6 + 7 = 43 mm
- The effective height d: d = h – concrete cover thickness – stirrup diameter – ½ reinforcement diameter = 500 – 43 = 457 mm
- The effective width of the beam : b_{eff} = 1450mm
- According to Appendix A1 – Concise EC2 [5], and to How to design concrete using EC2 [6]:

$$K = \frac{M_{Ed}}{b_{eff} * d^2 * f_{ck}} = \frac{92534*10^3}{1600*457^2*25} = 0.01$$

To restrict the ratio x/d < 0.45

$\rightarrow \delta$ = 0.85 and K' = 0.168

$$\rightarrow z = \frac{d}{2}*\left[1+\sqrt{1-3.53K}\right] = \frac{457}{2}*\left[1+\sqrt{1-3.53*0.0122}\right] = 452.03 \leq 0.95*d=434.15$$

$\rightarrow$ x = 2.5*(d-z) = 57.125mm <1.25*h_f = 1.25*150=187.5mm

- On the other hand we have the relationship: $M_{Rd} = A_s * f_{yd} * z$, So the area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd} * z} = \frac{92534*10^3}{434.8*434.15} = 490.2mm^2 = 4.9cm^2$$

- We choose 4 ϕ 14 (A_s = 6.2 cm^2) for flexural reinforcements $\Rightarrow$ The resistance of the section is $M^+_{Rd,b,A} = +119kNm$ and the over-strength factor is $\frac{119}{92.5} = 1.29$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{6.2}{25*(50-3-0.6-\frac{1.4}{2})} = 0.005$$

- Checking for spacing of the bars

  Clear spacing of the bars: (250-2*30-2*6-4*14)/3 = 40.7mm

  According to 8.2(2) – EC2 [2], minimum clear distance between bars:

  = max [bar diameter, aggregate +5mm]

  = max [14, 20+5] = 25mm < 40.7

- For M^-:
  - Beam's dimensions: b=250mm; h=500mm
  - Cover:

$c_{nom} = c_{min} + \Delta c_{dev}$ (Expression 4.1 – EC2 [2])

Where:

$c_{min} = \max[c_{min,b}; c_{min,dur}]$. (According to Expression 4.2 – EC2[2])

$c_{min,b}$ = diameter of bar. Assume 20mm bars and 6 mm hoops - Table 4.2 – EC2[2].

$c_{min, dur}$ = minimum cover due to environmental conditions. Assuming that Exposure class is XC1 and Structural Class is S4 $\rightarrow c_{min,dur}$ = 15mm

Δc_{dev} = 10mm $\rightarrow c_{nom}$ = 20 +10 =30 mm

- Concrete cover thickness + ½ reinforcement diameter + stirrup diameter = 60mm

- Working height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-60=440mm

$$M_{Rd} \geq M_{Ed}; M_{Rd} = \mu * b * d^2 * \eta * f_{cd}; M_{Ed} = -186834Nm$$

- The ULS condition: $\Rightarrow \mu \geq \dfrac{M_{Ed}}{b * d^2 * \eta * f_{cd}} = \dfrac{186834 * 10^3}{250 * 440^2 * 1 * 16.7} = 0.231$

$$\Rightarrow \frac{x}{d} = 0.33 < 0.45; \Rightarrow \frac{z}{d} = 0.85 \Rightarrow z = 0.86 * d = 378.4mm;$$

- On the other hand we have the relationship: $M_{Rd} = A_s * f_{yd} * z$, So the area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd} * z} = \frac{186834 * 10^3}{434.8 * 378.4} = 1134mm^2 = 11.3cm^2$$

- We choose 3 ϕ 20+2*2ϕ10 or 4 ϕ20 (A_s = 12.6 cm^2) for flexural reinforcements ⇒ The resistance of the section is $M^{-}_{Rd,b,A} = -218kNm$ and the over-strength factor is $\dfrac{218}{186.8} = 1.17$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{12.6}{25 * (50 - 2.0 - 0.6 - \frac{2.0}{2})} = 0.011$$

- Checking for spacing of the bars
  Clear spacing of the bars: (250-2*30-2*6-4*20)/3 = 32.7mm
  According to 8.2(2) – EC2 [2], minimum clear distance between bars:
  = max [bar diameter, aggregate +5mm]
  = max [14, 20+5] = 25mm < 32.7 → so this condition is met

- Check for the ratio between negative reinforcement and positive reinforcement: According to 5.4.3.1.2 (4a) – EC8 [3], at the compression zone, reinforcement is not less than half of the reinforcement provided at the tension zone. The compression reinforcement area is 620mm^2 and the tension reinforcement area is 1260 mm^2→ so this condition is met

- Flexural Reinforcing steel of right – side of first span of beam (Axis B):

$$M^{+} = 82191Nm$$

$$M^{-} = -176883Nm$$

- For M^+:
  - Beam's dimensions: b_{eff} =1600 mm; h_w =500 mm
  - Concrete cover thickness + ½ reinforcement diameter =50mm
  - Working height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-50=450mm
  - The ULS condition: $M_{Rd} \geq M_{Ed}; M_{Rd} = \mu * b_{eff} * d^2 * \eta * f_{cd}; M_{Ed} = 82191Nm$

$$\Rightarrow \mu \geq \frac{M_{Ed}}{b * d^2 * \eta * f_{cd}} = \frac{82191 * 10^3}{1600 * 450^2 * 1 * 16.7} = 0.02 \Rightarrow \frac{x}{d} = 0.066 < 0.45$$

$$\rightarrow x = 29 < h_f = 150; \Rightarrow \frac{z}{d} = 0.977 \Rightarrow z = 0.977 * d = 440mm$$

  - Area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd} * z} = \frac{82191 * 10^3}{434.8 * 440} = 429mm^2 = 4.3cm^2$$

- We choose 4 ϕ 14 (A_s = 6.2 cm^2) for flexural reinforcements. ⇒ The resistance of the section is $M^{+}_{Rd,b,B} = 119kNm$ and the over-strength factor is $\frac{119}{82} = 1.45$.

- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{6.2}{25*(50-3.0-0.6-\frac{1.4}{2})} = 0.005$$

- Checking for spacing of the bars
  Clear spacing of the bars: (250-2*30-2*6-4*14)/3 = 40.7mm. According to 8.2(2) – EC2 [2], minimum clear distance between bars:= max [bar diameter, aggregate +5mm]
  = max [14, 20+5] = 25mm < 40.7 → so this condition is met

- For M$^-$:
  - Beam's dimensions: b=250mm; h=500mm
  - Concrete cover thickness + ½ reinforcement diameter =60mm
  - Working height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-60=440mm
  - The ULS condition:

$$M_{Rd} \geq M_{sd}; M_{Rd} = \mu * b * d^2 * \eta * f_{cd}$$

$$M_{sd} = -176883Nm \Rightarrow \mu \geq \frac{M_{sd}}{b*d^2*\eta*f_{cd}} = \frac{176883*10^3}{250*440^2*1*16.7} = 0.218$$

$$\Rightarrow \frac{x}{d} = 0.312 < 0.45 \Rightarrow \frac{z}{d} = 0.87 \Rightarrow z = 0.87*d = 382.8mm$$

  - Area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{sd}}{f_{yd}*z} = \frac{176883*10^3}{434.8*382.8} = 1062.7mm^2 = 11cm^2$$

  - We choose 2 ϕ 20 + 1 ϕ 18 + 2ϕ10(A_s = 11.9 cm^2) or 2ϕ20+2ϕ18 for flexural reinforcements. ⇒ The resistance of the section is $M^{-}_{Rd,b,B} = -208kNm$ and the over-strength factor is $\frac{208}{176} = 1.18$.
  - The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{11.9}{25*(50-3.0-0.6-\frac{2.0}{2})} = 0.01$$

  - Checking for spacing of the bars
    Clear spacing of the bars: (250-2*30-2*6-2*18-2*20)/3 = 34mm
    According to 8.2(2) – EC2 [2], minimum clear distance between bars:
    = max [bar diameter, aggregate +5mm]
    = max [14, 20+5] = 25mm < 34mm → so this condition is met

- Check for the ratio between negative reinforcement and positive reinforcement: According to 5.4.3.1.2 (4a) – EC8 [3], at the compression zone, reinforcement is not less than half of the reinforcement provided at the tension zone. The compression reinforcement area is 620mm^2 and the tension reinforcement area is 1190 mm^2→ so this condition is met

- Check for the deflection: According to 15.7 – Concise EC2[5], the SLS state of deflection may be checked by using the span to effective depth approach. To use the span – to effective – depth approach, verify that:
  Allowable l/d = N*K*F1*F2*F3 ≥ actual l/d

Where:

N = Basic l/d: check whether $\rho > \rho_0$: $\rho = \rho' = 0.005$; $\rho_0 = f_{ck}^{0.5}/1000 = 0.005 \rightarrow$ use the Exp (7.16a) – Concise EC2 [5] : $N = 11+1.5*f_{ck}^{0.5}\rho/\rho_0 + 3.2*f_{ck}^{0.5}*(\rho/\rho_0 - 1)^{1.5} = 18.5$

K=1.3 (end span) table 15.11 – Concise EC2

F1 = 1; F2=1

$F3=310/\sigma_s$

Where: $\sigma_s = (A_{s,pro}/A_{s,req}) = 182/500 <= 1.5$

l/d = 36.04

Actual l/d = 5000/457=10.9. So this condition is met.

V.1.5.3 Specific measures for the flexural reinforcement.

- Min/max reinforcing steel
  - In accordance to 5.4.3.1.2 – EC8 [3], minimum tension reinforcement ratio shall not exceed the value: $\rho_{\min} = 0.5 * \frac{f_{ctm}}{f_{yk}} = 0.5 * \frac{2.6}{500} = 0.0026$ (5.12 - EC8 [3]).

    → **The reinforcement content is satisfactory.**
  - According to 5.4.3.1.2 – EC8 [3], within the critical regions, the tension reinforcement ratio shall not exceed the value below: $\rho_{\max} = \rho' + \frac{0.018}{\mu_\phi * \varepsilon_{sy,d}} * \frac{f_{cd}}{f_{yd}}$

    Where:
    - μ_ϕ - Curvature ductility, $T_1 = 0.67s > T_C = 0.6s \rightarrow \mu_\phi = 2*q_0 - 1 = 6.8$.
    - $\varepsilon_{sy,d} = \frac{f_{yd}}{E_s}$

    So, $\rho_{\max} = 0.00532 + \frac{0.018}{6.8 * \frac{434.8}{200000}} * \frac{16.7}{434.8} = 0.052 > 0.01$

    → **The reinforcement content is satisfactory.**
- Longitudinal bar diameters:

According to 5.6.2.2 – EC8 [3], to prevent the bond failure, the diameter of longitudinal bars of the beams is limited as the following conditions:

- For interior beam – column joints:

$$\frac{d_{bL}}{h_C} \le \frac{7.5 * f_{ctm}}{\gamma_{Rd} * f_{yd}} * \frac{1+0.8\nu_d}{1+0.75*k_D*\frac{\rho'}{\rho_{\max}}} \leftrightarrow \frac{d_{bL}}{h_c} \le 4.0 * \frac{f_{ctm}}{f_{yd}} * (1+0.8*\nu_d) \quad (2.7.2.2.1 - \text{ENV8.})$$

Where:
  - h_c – is the width of the column parallel to the bars, so h_c = 500mm.
  - f_{ctm}: is the mean value of the tensile strength of concrete → $f_{ctm} = 2.6 N/mm^2$.
  - F_{yd} = 434.8 MPa.
  - ν_d – is the normalised design axial force in column, taken with its minimum value for seismic design situation. $\nu_d = \frac{N_{Ed}}{f_{cd} * A_C}$

    N_{Ed} = -1295000N; f_{cd} = 16.67MPa; $A_c = 400x500 = 200000mm^2$.

$$\rightarrow \nu_d = \frac{N_{Ed}}{f_{cd} * A_C} = \frac{1295000}{16.7 * 200000} = 0.387$$

  - k_D – is the factor reflecting the ductility class equal to 1 for DCH, to 2/3 for DCM.
  - ρ' – compression steel ratio → ρ' = 0.00532
  - $\rho_{\max}$ = 0.052.

- $\gamma_{Rd} = 1$.

So: $\frac{d_{bL}}{h_c} \leq \frac{7.5*2.6}{1*434.8} * \frac{1+0.8*0.387}{1+0.75*\frac{2}{3}*\frac{0.00532}{0.052}} = 0.056$ → d_{bL} = 500*0.056=28mm

→ **The chosen reinforcement is satisfactory.**

- For exterior beam – column joints:

$$\frac{d_{bL}}{h_C} \leq \frac{7.5*f_{ctm}}{\gamma_{Rd}*f_{yd}}*1+0.8\nu_d \leftrightarrow \frac{d_{bL}}{h_c} \leq 4.0*\frac{f_{ctm}}{f_{yd}}*(1+0.8*\nu_d) \text{ (2.7.2.2.1 – ENV8)}$$

Where:

- h_c – is the width of the column parallel to the bars, so h_c = 500mm.
- f_{ctm}: is the mean value of the tensile strength of concrete → f_{ctm} = 2.6N/mm^2.
- F_{yd} = 434.8 MPa.
- ν_d – is the normalised design axial force in column, taken with its minimum value for seismic design situation. $\nu_d = \frac{N_{Ed}}{f_{cd}*A_C}$

  N_{Ed} = -940000N; f_{cd} = 16.67MPa; A_c = 400x500=200000mm^2.

  $$\rightarrow \nu_d = \frac{N_{Ed}}{f_{cd}*A_C} = \frac{940000}{16.7*200000} = 0.28$$

- k_D – is the factor reflecting the ductility class equal to 1 for DCH, to 2/3 for DCM.
- ρ' – compression steel ratio → ρ' = 0.00532
- ρ_{max} = 0.052.
- γ_{Rd} = 1.

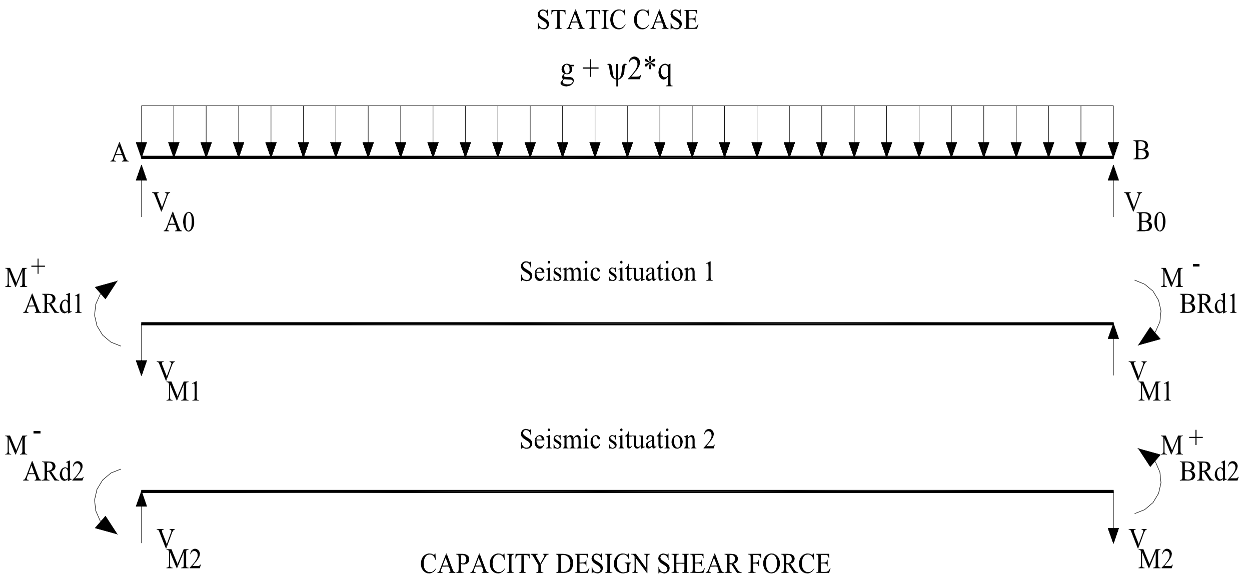
So: $\frac{d_{bL}}{h_c} \leq \frac{7.5*2.6}{1*434.8}*(1+0.8*0.28) = 0.055$ → d_{bL} = 500*0.055=27.5mm

→ **The chosen reinforcement is satisfactory.**

- Top reinforcement of the beam.
  - In accordance to 2.7.3.4 part 1-3 – ENV8, one fourth of the maximum top reinforcement shall run along the entire beam length.
  - Two ϕ20 bars will run along the entire span.

V.1.5.4 Shear resistance

- Design shear forces computed in accordance to the capacity design criterion:
  - According to 5.4.2.2 – EC8 [3], in the primary seismic beams shear forces shall be calculated in accordance with the capacity design rule, considering the equilibrium of the beam under: a) the transverse load acting on it in seismic design situation and b) end moments $M_{i,d}$ (with i=1,2 denoting the end sections of the beam), corresponding for each sense of the seismic action, to plastic hinge formation at the ends either of the beams or of the vertical elements – which are connected to the joint where beam end i frames into.
  - The calculation of shear forces as following the sketch below:



CAPACITY DESIGN SHEAR FORCE

- Determining M^{+}_{ARd1}, M^{-}_{BRd1}, M^{-}_{ARd2}, M^{+}_{BRd2}, V_{A0}, V_{B0}.
  - M^{+}_{ARd1} and M^{+}_{BRd2}

    The bottom reinforcement area of longitudinal bars is $6.2cm^2$ (4 ϕ 14), we determine the value of $M^{+}_{ARd1} = M^{+}_{BRd2}$ as following:

    $$x = \frac{f_{yd} * A_s}{\eta * f_{cd} * b} = \frac{434.8*6.2*100}{1*16.7*250} = 64.6mm$$

    $$d = 500 - 30 - 6 - \frac{14}{2} = 455mm$$

    $$\frac{x}{d} = 0.139 \rightarrow \frac{z}{d} = 0.95$$

    $$\rightarrow M^{+}_{ARd1} = z * f_{yd} * A_s = 0.95*465*434.8*620 = 119085198Nmm \rightarrow M^{+}_{ARd1} = 119KNm$$

    $M^{+}_{BRd2} = 119$kNm.
  - M^{-}_{ARd2} : The top reinforcement area of longitudinal bars is 4 ϕ 20 (A_s = 12.6 cm^2), we determine the value of M^{-}_{ARd2} as following:

    $$F_{as} = f_{yd} * A_s = 434.8*1260 = 547848Nmm$$

    $$F_c = \eta * f_{cd} * b * (h - z) = 16.7*250*(500 - z)$$

    $$= 4175*(500 - z)$$

    $$F_{as} = F_c \rightarrow 547848 = 4175*(500 - z)$$

    $$\rightarrow z = 368.8mm$$

    $$M^{-}_{ARd2} = F_{as} * b_{as} + F_c * b_c$$

    $$b_{as} = z - coating - \phi stirrup - \frac{1}{2}\phi bars$$

    $$b_{as} = 368.8 - 20 - 6 - 10 = 332.8mm$$

    $$b_c = \frac{h - z}{2} = \frac{500 - 368.8}{2} = 65.6mm$$

    $$\rightarrow M^{-}_{ARd2} = 547848*332.8 + 4175*(500 - 368.8)*65.6 = 218256870Nmm \rightarrow M^{-}_{ARd2} = 218KNm$$

  - M^{-}_{BRd1} : The top reinforcement area of longitudinal bars is 2ϕ20 + 2ϕ18 (A_s = 11.9 cm^2), we determine the value of M^{-}_{BRd1} as follows:

$$F_{as} = f_{yd} * A_s = 434.8 * 1190 = 517412Nmm$$
$$F_c = \eta * f_{cd} * b * (h - z) = 16.7 * 250 * (500 - z)$$
$$= 4175 * (500 - z)$$
$$F_{as} = F_c \rightarrow 517412 = 4175 * (500 - z)$$
$$\rightarrow z = 376.1mm$$
$$M^-_{BRd1} = F_{as} * b_{as} + F_c * b_c$$
$$b_{as} = z - coating - \phi stirrup - \frac{1}{2}\phi bars$$
$$b_{as} = 376.1 - 20 - 6 - 10 = 340.1mm$$
$$b_c = \frac{h - z}{2} = \frac{500 - 376.1}{2} = 62mm$$
$$\rightarrow M^-_{BRd1} = 517412 * 340.1 + 4175 * (500 - 376.1) * 62 = 208043336Nmm \rightarrow M^-_{BRd1} = 208KNm$$

- Determining V_{B0} and V_{A0}:

$$V_{B0} = V_{A0} = \frac{3750 * 5}{2} + \frac{32850 * 5}{2 * 2} = 50437.5N = 50.4KN$$

- So we have:

$$V_{M1} = -\gamma_{Rd} * \frac{(M^+_{ARd1} + M^-_{BRd1})}{l} = -1 * \frac{119 + 208}{5} = -65$$
$$V_{M2} = \gamma_{Rd} * \frac{(M^-_{ARd2} + M^+_{BRd2})}{l} = 1 * \frac{218 + 119}{5} = 67 \; ; \; V_{B0} = V_{A0} = 50.4KN$$

And so:

- At support A:
  $V_{min} = V_{M1} + V_{A0}$ = -65+50.4=-14.6 ; $V_{man} = V_{M2} + V_{A0}$ = 67+50.4=117.4
- At support B:
  $V_{min} = -V_{M2} + V_{B0}$ = -67+50.4=-16.6 ; $V_{man} = V_{M1} + V_{B0}$ = 65+50.4=115.4

- V_{cd} and V_{Rd} Computations :
  - In the critical sections: $V_{cd} = 0$
  - Outside the critical sections: $V_{cd} = V_{Rd1}$.
  - In accordance with EC(2) 4.3.2.3 and neglecting the axial force influence, the value of $V_{Rd,c}$

EN 1992
$$V_{Rd,c}=\left[\frac{0.18}{\gamma_c}*k*(100*\rho_l*f_{ck})^{1/3}-0.15*\sigma_{cp}\right]*b_w*d$$ • f_{ck} – is compressed strength of the concrete at the age of 28 days. γ_c = 1.5 • $k=1+\sqrt{\frac{200}{d}}\leq 2.0$ d – mm; $\rho_l=\frac{A_{sl}}{b_w*d}\leq 0.02$ where: ▪ A_{sl} – is the area of tension reinforcements. ▪ b_w – is the minimum width. • $\sigma_{cp}=\frac{N_{sd}}{A_c}$; N_{sd} – is the longitudinal force. MPa • Replacing with the value of f_{ck} is 25MPa, reinforcing steel percentage is $\frac{620}{250*(500-20-6-7)}=0.00532$, so we have: $V_{Rd,ct}=\left[\frac{0.18}{1.5}*1*(100*0.00532*25)^{1/3}\right]*250*467=32942N=32.9KN$

- Computations
  - The computations shall run in accordance to 6.2.1(2) – EN1992 and the specific rules shall get along with truss model (EN1998)
  - According to 6.2.1(2) – EC2 [2], the shear resistance of a member with shear reinforcement is equal to $V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}$. V_{td} is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord, so $V_{td} = 0$ and $V_{ccd} = 0$.

    So: $V_{Rd} = V_{Rd,s} + V_{ccd} \rightarrow V_{Rds}=\frac{A_{sw}}{s}*z*f_{ywd}*\cos\theta$
  - In critical regions (2*height of the beams) shear force will be carried out only by the stirrups. We choose stirrups with 2 legs, of 6mm in diameter and 80mm spacing. Shear force capacity is: $V_{Rd}=V_{Rds}=\frac{57}{80}*467*434.8=144674.4N=145KN >> V_{max}$.
  - Outside critical regions, shear forces are carried out by stirrups with 2 legs, 6mm in diameter and 120mm spacing. Shear force capacity is: $V_{Rd}=V_{Rd,c}+V_{Rds}=32900+\frac{57}{120}*467*434.8=129400N=129.4KN$
- The difference between the shear force values at support A and B is small enough to neglect the possibility of modifying the shear reinforcement. Along the whole beam length, shear force must be less than the value of $V_{Rd,max}$ which is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

  $V_{Rd,max} = \alpha_{cw} * b_w * z * \nu_1 * f_{cd} / (\cot\theta + \tan\theta)$

  Where: ν_1 is a strength reduction factor for concrete cracked in shear

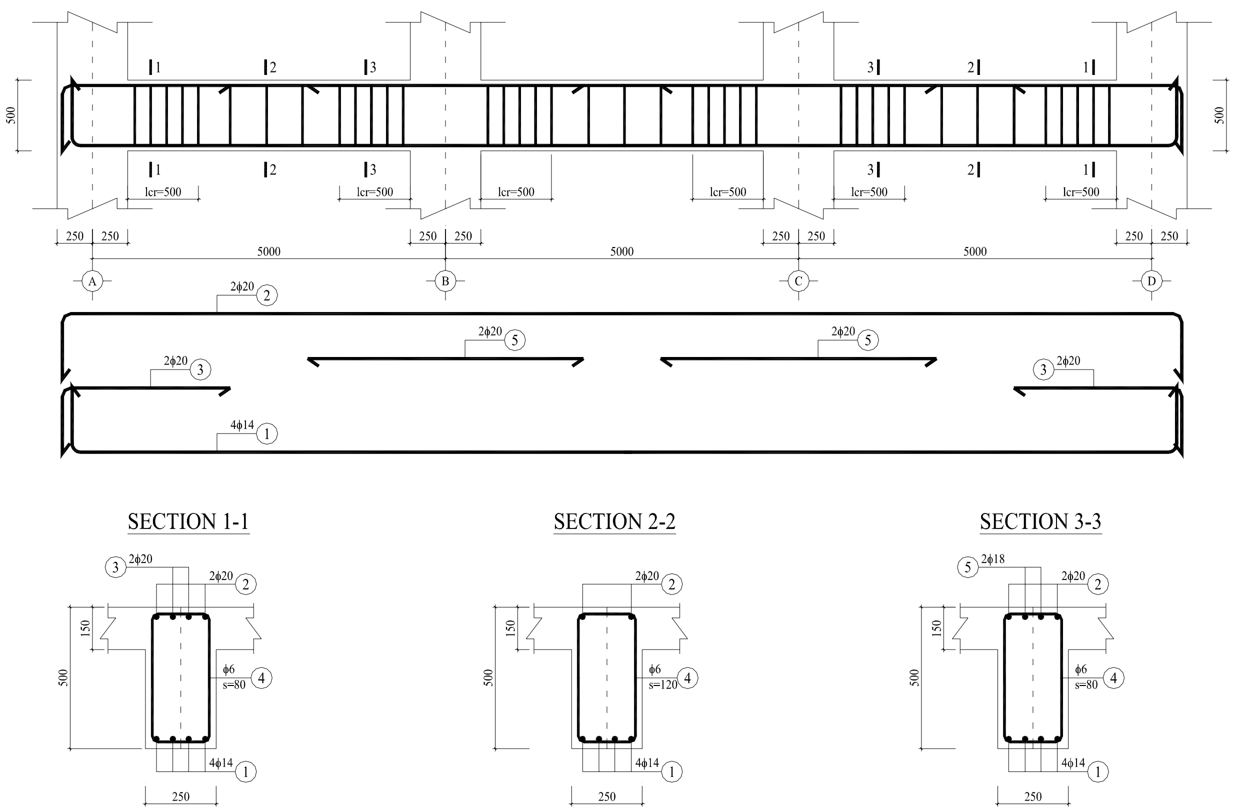
  α_{cw} is a coefficient taking account of the state of the stress in the compression chord. So: $\upsilon_l=0.6*\left[1-\frac{f_{ck}}{250}\right]=0.6*0.9=0.54$ and $\alpha_{cw} = 1$.

  $\rightarrow V_{Rd,max}$ = 1*250*467*16.7/(1+1)=526000N=526KN>V_{max}
  - The computations shall run in accordance to 6.2.1(2) – EN1992 and the specific rules shall get along with truss model (EN1998)

V.1.5.5 Specific measures

- Detailing:
  - In accordance to 5.4.3.1.2(6P) - EC8 [3], the stirrup minimum diameter within the critical regions is 6mm – this requirement is met.
  - The first hoop is placed not more than 50mm from the end cross section of the beam – this requirement is met.
  - Within the critical regions, the spacing of the hoops is not greater than: $h_w / 4 = 125mm; 24 * d_{bw} = 24 * 6 = 144mm$ $225mm; 8 * d_{bl} = 8 * 14 = 112mm$ So, the condition is satisfactory.
- Casting and Placing for beam: All requirements are met.

DETAILING FOR THE FIRST FLOOR BEAM



V.1.6 REINFORCEMENT OF OTHER BEAMS

The reinforcement of other beams of the transverse frame are determined by using the similar ways as the beams on the first floor. They are summarized in the following tables.

Table V.1- Properties of the section and seismic actions in transverse frame

Floor Level	Position of column	Sections of the beams	b_w (mm)	b_c (mm)	h_w (mm)	b_{eff} (mm)	Δ_{nom} (mm)	M^+_{Ed} (kNm)	M^+_{Rd} (kNm)	M^-_{Ed} (kNm)	M^-_{Rd} (kNm)
1-2	External	End	250	400	500	1600	30	92.5	119	-186.8	-218
1-2	Internal	End	250	400	500	1600	30	82.2	119	-176.9	-208
1-2	Internal	Middle	250	400	500	1600	30	36.0	119		
3-4	External	End	250	400	500	1600	30	68.1		-178.5	-198.5
3-4	Internal	End	250	400	500	1600	30	65.5		-162.9	-185.1
3-4	Internal	Middle	250	400	500	1600	30	34.3			
5-6	External	End	250	400	500	1600	30	2.6		-122.3	
5-6	Internal	End	250	400	500	1600	30	13.7		-109.7	
5-6	Internal	Middle	250	400	500	1600	30	32.0			

Table V.2 - Designed Longitudinal Reinforcement and specific measures in transverse frame

Beams of Floor	Position of column	Sections of the beams	Top Reinforc (mm^2)	Bottom Reinforc (mm^2)	ρ (%)	ρ' (%)	ρ_{max} (%)	ρ_{min} (%)	d_{max} (mm)	M_{Rd} (KNm)
1-2	External	End	1260	620(4ϕ14)	0.5	1.1	5.2	0.26	28	-218
1-2	Internal	End	1190	620(4ϕ14)	0.5	1	5.2	0.26	28	-208
1-2	Internal	Middle	628	620(4ϕ14)	0.5	0.5	5.2	0.26	26	+119
3-4	External	End	910	620(4ϕ14)	0.5	0.8	5.2	0.26	26	-198.5
3-4	Internal	End	816	620(4ϕ14)	0.5	0.7	5.2	0.26	26	-185.1
3-4	Internal	Middle	508	620(4ϕ14)	0.5	0.45	5.2	0.26	26	+119
5	External	End	804	462(3ϕ14)	0.4	0.7	5.2	0.26	26	
5	Internal	End	804	462(3ϕ14)	0.4	0.7	5.2	0.26	26	
5	Internal	Middle	402	462(3ϕ14)	0.4	0.35	5.2	0.26	26	+119
6	External	End	462	462(3ϕ14)	0.4	0.4	5.2	0.26	26	
6	Internal	End	462	462(3ϕ14)	0.4	0.4	5.2	0.26	26	
6	Internal	Middle	462	462(3ϕ14)	0.4	0.4	5.2	0.26	26	+119

Table V.3 - Designed Stirrup Reinforcement and specific measures of transverse frame

Beams of Floor	Sections of the beams	V_{max} (KN)	ϕ - stirrup (mm)	Number of legs	Spacing	V_{Rd} (KN)	V_{Rdmax} (KN)
1-2	Critical region	102.7	6	2	80	145	1053
1-2	Outside critical region	82	6	2	120	129	1053
3-4	Critical region	99.7	6	2	80	145	1053
3-4	Outside critical region	76.2	6	2	120	129	1053
5-6	Critical region	77.8	6	2	80	145	1053
5-6	Outside critical region	54.4	6	2	120	129	1053

V.2. LONGITUDINAL OR DIRECTION X CURRENT FRAME:

V.2.1 Action effects: (According to 5.4.2 – EC8 [3])

- All the steps are the same as transverse frames.
- The loads were uniformly distributed along the length of the beam. No distribution of the bending moments was made.
- The design value of the shear forces shall be determined in accordance with the capacity design rules, EC 8 – 5.4.2.2 – 1(P).
- The torsion effects were determined separately by these two dimensions according to 4.3.3.2.4(2) – EC8: If the analysis is performed using two planar models, one for each main horizontal direction, torsion effects may be determined by doubling the accidental eccentricity e_{ai} of the expression $e_{ai} = \pm 0.05 L_i$ and applying the rules of 4.3.3.2.4 (1) – EC8 with the factor δ, $\delta = 1 + 0.6 * \frac{x}{L_e}$, replaced by the factor δ, $\delta = 1 + 1.2 * \frac{x}{L_e}$.

V.2.2 Action Summary:

V.2.2.1 Gravity actions:

- DEAD Load: The self-weight load.
- DL slab: The dead loads induced by the floor and coating weight.
- LL slab: The live loads induced by the variable actions
- LL roof slab: The live loads induced by the variable roof actions
- Snow load: The loads induced by the snow.
- Joint load: The loads acting to the joints of the longitudinal frame induced by perpendicular frames.
- WIND load.

V.2.2.2 Seismic actions:

- The analysis is performed using two planar models, one for each main direction.
- The torsion effects were determined separately by these two dimensions according to 4.3.3.2.4(2) – EC8: $\delta = 1 + 1.2 * \frac{x}{L_e} = 1 + 1.2 * \frac{2.5}{15} = 1.2$. So we have δ = 1.2
- The above seismic forces are total seismic acting in all of the building or all frames. According to the stiffness of each frame, we will distribute the seismic forces to each frame linearly including torsion effects.
- In the action direction of the earthquake, there are 4 portal frames. When distributing seismic forces to all floors of the frame, the torsion effects will be taken into account by the factor δ which is calculated from 4.3.3.2.4 (1) – EC8. Factor δ, here, accounts for the analysis model with two planar directions. We will distribute the seismic forces to the current transverse frame as following:
  - $F_{B-6} = F_6 * \frac{1}{4} * \delta = 321.34 * \frac{1}{4} * 1.2 = 96.4 KN$. $F_{B-5} = F_5 * \frac{1}{4} * \delta = 287.0 * \frac{1}{4} * 1.2 = 86.1 KN$.
  - $F_{B-4} = F_4 * \frac{1}{4} * \delta = 231.5 * \frac{1}{4} * 1.2 = 69.5 KN$. $F_{B-3} = F_3 * \frac{1}{4} * \delta = 175.9 * \frac{1}{4} * 1.2 = 52.8 KN$.
  - $F_{B-2} = F_2 * \frac{1}{4} * \delta = 120.4 * \frac{1}{4} * 1.2 = 36.1 KN$. $F_{B-1} = F_1 * \frac{1}{4} * \delta = 65.8 * \frac{1}{4} * 1.2 = 19.7 KN$.

- Response Spectrum Analysis: Response Spectrum Analysis will be carried out the same as the transverse frame.

V.2.3 <u>COMBINATIONS:</u>

According to EC8 [3], there are 3 combinations determined from all actions above:

- Combination 1: DEAD Load + DL Slab + Joint Load + Seismic Load + $\sum \psi_{2,i} * Q_k, i$
- Combination 2(in opposite direction): DEAD Load + DL Slab + Joint Load - Seismic Load + $\sum \psi_{2,i} * Q_k, i$
- Combination3: Envelope of Combination 1 and Combination2.

V.2.4 <u>INTERNAL FORCES:</u>

The internal forces will be determined by SAP2000, version 9.0. The internal forces, which are used to design the section reinforcements of the frames, will be determined by Lateral Force Analysis method.

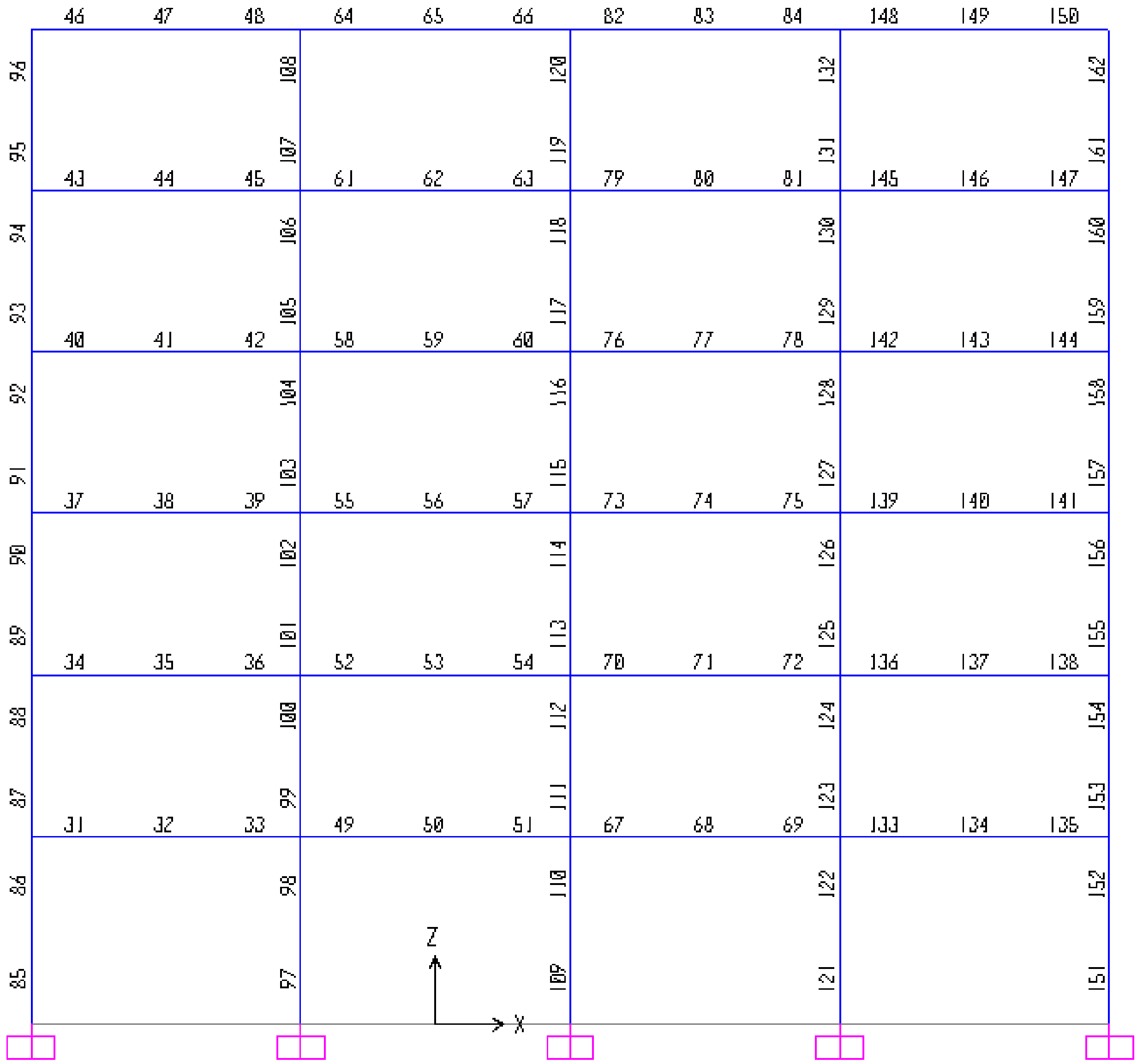


Figure 5 - Longitudinal Frame or Direction X Frame in line B

TABLE V.4 : Element Forces – Longitudinal or Direction X Frames							
Frame	**Section**	**OutputCase**	**CaseType**	**StepType**	**P – N_{Ed} Axial**	**V2 – V_{Ed} Shear**	**M3 – M_{Ed} Moment**
Text	m	Text	Text	Text	KN	KN	KN-m
31	0	envelopeofseism	Combination	Max	12.245	4.213	93.3445
31	0	envelopeofseism	Combination	Min	6.247	-100.021	-181.1328
32	0.83333	envelopeofseism	Combination	Max	12.245	53.09	38.8253
32	0.83333	envelopeofseism	Combination	Min	6.247	-51.143	24.9314
33	1.66667	envelopeofseism	Combination	Max	12.245	101.968	74.583
33	1.66667	envelopeofseism	Combination	Min	6.247	-2.266	-172.1064
49	0	envelopeofseism	Combination	Max	17.308	-3.372	64.0981
49	0	envelopeofseism	Combination	Min	1.643	-94.623	-163.0272
50	0.83333	envelopeofseism	Combination	Max	17.308	45.506	29.5425
50	0.83333	envelopeofseism	Combination	Min	1.643	-45.745	28.5401
51	1.66667	envelopeofseism	Combination	Max	17.308	94.383	65.6996
51	1.66667	envelopeofseism	Combination	Min	1.643	3.132	-163.4305
67	0	envelopeofseism	Combination	Max	20.169	-2.257	67.7165
67	0	envelopeofseism	Combination	Min	-2.661	-93.334	-161.125
68	0.83333	envelopeofseism	Combination	Max	20.169	46.62	29.3726
68	0.83333	envelopeofseism	Combination	Min	-2.661	-44.456	28.2213
69	1.66667	envelopeofseism	Combination	Max	20.169	95.498	61.1551
69	1.66667	envelopeofseism	Combination	Min	-2.661	4.421	-165.3839
133	0	envelopeofseism	Combination	Max	24.831	0.968	71.3445
133	0	envelopeofseism	Combination	Min	-7.835	-102.606	-173.9815
134	0.83333	envelopeofseism	Combination	Max	24.831	49.845	38.545
134	0.83333	envelopeofseism	Combination	Min	-7.835	-53.728	24.9379
135	1.66667	envelopeofseism	Combination	Max	24.831	98.723	94.6589
135	1.66667	envelopeofseism	Combination	Min	-7.835	-4.851	-177.8811

V.2.5 DESIGN OF BEAMS OF THE FIRST STORY (members 31-33, 49-51, 67-69 and 133-135)

V.2.5.1 Geometrical Restraints

- Effective flange width:

According to 5.4.3.1.1(3) – EC8 [3], the effective flange width b_{eff} may be assumed as follows:

- For primary seismic beams framing into exterior columns, the effective flange width, b_{eff}, is taken to the width b_c of the column in the absence of the transverse beam, or equal to this width increased by $2h_f$ on each side of the beam if there is transverse beam of similar depth.
- For the primary seismic beams framing into interior columns the above length may be increased by $2h_f$ on each side of the beam.

So, the effective flange width of beams framing to the exterior columns is:

$$b_c = 500mm \ ; \ h_f = 150mm \rightarrow b_{eff} = b_c + 2*2*h_f = 1100mm$$

The effective flange width of beams framing to the interior columns is:

$$b_c = 500mm \ ; \ h_f = 150mm \rightarrow b_{eff} = b_c + 2*4*h_f = 1700mm$$

- Beam – Column centroidal axis distance :

The beam framing symmetrically into the exterior columns has the eccentricity as following expression: e = 0.00mm

- Minimum width of the beams :

- In accordance to 5.4.1.2.1, the effective transfer of cyclic moments from a primary seismic beam to a column shall be achieved, by limiting the eccentricity of the beam axis relative to that of the column into which it frames.
- A deemed to satisfy rule for 5.4.1.2.1 is to limit the distance between the centroidal axes of the two members to less than $b_c/4$. $\rightarrow \begin{cases} b_w \le \min\{b_c + h_w; 2b_c\} \\ b_w \ge 200mm \end{cases}$

- Width to height ratio of the web of the beam:

In accordance to 2.7.2.4 – ENV 1998 and 4.3.5.7 – EC2 [2], lateral buckling of the slender beams: (P2)

- (P1), where the safety of beams against lateral buckling is in doubt, it shall be checked by an appropriate method.
- (2), the width to height ratio of the beam's web must be ensure to the following condition:

$$h_w < 2.5 * b_w \text{, so } \left.\begin{matrix} h_w = 500mm \\ b_w = 250 \end{matrix}\right\} \rightarrow 350 < 2.5 * 250 = 625$$

- Limitation of the beam width:

According to 5.4.1.2.1 – EC8 [3], the beam width has to be checked as following condition:

$b_w \le \min(b_c + h_w; 2 * b_c) \;\rightarrow\; b_w = 250 \le \begin{cases} 250 + 500 = 750 \\ 2 * 250 = 500 \end{cases}$. This condition is met.

V.2.5.2 <u>Flexural reinforcement:</u>

- In accordance to 2.5.3.3(5) – EC2 and 2.5.3.4.2 (7), a reduction of the design bending moments is made to the column margins.
- The frame is a sway frame, so, according to the principle of 2.5.3.4.2 – EC2 [2], no redistribution of the bending moment is made.
- The actual strength of the materials:
  - Concrete: As chosen above, the concrete class is C25/30, according to 3.1.2.4 – EC2 [2], concrete material properties are:

$$f_{ck} = 25MPa = 25N/mm^2;\; f_{cm} = 33MPa = 33N/mm^2;\; f_{ctm} = 2.6MPa = 2.6N/mm^2$$

$$f_{cd} = \frac{\alpha_c * f_{ck}}{\gamma_c} = 16.67MPa = 16.67N/mm^2; \alpha_c = 1; \gamma_c = 1.5$$

(Note: In Belgium γ_C = 1.5; In France γ_C = 1.3)

  - Reinforcing steel: As chosen above, the steel class is S500, according to EC2 and EC3, steel material properties are:

$$f_{yk} = 500MPa = 500N/mm^2$$

$$f_{yd} = \frac{\alpha * f_{yk}}{\gamma_s} = \frac{1*500}{1.15} = 434.8MPa = 434.8N/mm^2$$

(Note: In Belgium γ_S = 1.0; In France γ_S = 1.0)

  - In accordance to ultimate limit states of bending plus axial force design procedure of 6.1 – EC 2 and 3.1.5 – EC2 [2], we will use a rectangular diagram for compressed concrete block. In this case $\eta * f_{cd} = 1 * f_{cd} = 16.7$ Mpa.

- Flexural Reinforcing steel of Left – side of beam (Beam number 31-33): $\begin{matrix} M^+ = 93304Nm \\ M^- = -181173Nm \end{matrix}$

  - For M^+:
    - Beam's dimensions : b=250mm ; h=500mm
    - Concrete cover thickness + ½ reinforcement diameter =50mm
    - Working height d: d = h – concrete cover thickness – stirrup diameter – ½ reinforcement diameter = 500 – 50 = 450mm

$$M_{Rd} \geq M_{Ed}$$
$$M_{Rd} = \mu * b * d^2 * \eta * f_{cd}$$
$$M_{Ed} = 93304Nm$$

- The ULS condition: $\Rightarrow \mu \geq \dfrac{M_{Ed}}{1700 * d^2 * \eta * f_{cd}} = \dfrac{93304 * 10^3}{1700 * 450^2 * 1 * 16.7} = 0.02$

$$\Rightarrow \frac{x}{d} = 0.066 < 0.45$$
$$\Rightarrow \frac{z}{d} = 0.977 \Rightarrow z = 0.977 * d = 440mm$$

- On the other hand we have the relationship: $M_{Rd} = A_s * f_{yd} * z$, So the area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd} * z} = \frac{93304 * 10^3}{434.8 * 440} = 490mm^2 = 4.9cm^2$$

- We choose 2 ϕ 14 + 1 ϕ 16 (A_s = 5.1 cm^2) for flexural reinforcements. $\Rightarrow$ The resistance of the section is $M^{+}_{Rd,b} = +98kNm$ and the over-strength factor is $\dfrac{98}{93.3} = 1.05$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{5.1}{25 * (50 - 3.0 - 0.6 - \frac{1.6}{2})} = 0.004$$

- For M^-:
  - Beam's dimensions : b=250mm ; h=500mm
  - Concrete cover thickness + ½ reinforcement diameter + stirrup diameter = 60mm
  - Working height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-60=440mm

$$M_{Rd} \geq M_{Ed}$$
$$M_{Rd} = \mu * b * d^2 * \eta * f_{cd}$$
$$M_{Ed} = 181173Nm$$

- The ULS condition: $\Rightarrow \mu \geq \dfrac{M_{Ed}}{b * d^2 * \eta * f_{cd}} = \dfrac{181173 * 10^3}{250 * 440^2 * 1 * 16.7} = 0.247$

$$\Rightarrow \frac{x}{d} = 355 < 0.45$$
$$\Rightarrow \frac{z}{d} = 0.85 \Rightarrow z = 0.85 * d = 374mm$$

- Area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{sd}}{f_{yd} * z} = \frac{181173 * 10^3}{434.8 * 374} = 1114mm^2 = 11cm^2$$

- We choose 2 ϕ 18 + 1 ϕ 20 +2ϕ10(A_s = 11.9 cm^2) or 2 ϕ 20 + 2 ϕ 18 for flexural reinforcements. $\Rightarrow$ The resistance of the section is $M^{-}_{Rd,b} = -208kNm$ and the over-strength factor is $\dfrac{208}{181.2} = 1.14$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{11.9}{25*(50-3.0-0.6-\frac{2.0}{2})} = 0.01$$

- Flexural Reinforcing steel of right – side of beam: $M^+ = 75000Nm$ $M^- = -172000Nm$
  - For M^+:
    - Beam's dimensions : b=250mm ; h=500mm
    - Concrete cover thickness + ½ reinforcement diameter =50mm
    - Effective height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-50=450mm

$$M_{Rd} \geq M_{Ed}; M_{Rd} = \mu * b * d^2 * \eta * f_{cd}; M_{Ed} = 75000Nm$$

- The ULS condition: $\Rightarrow \mu \geq \frac{M_{Ed}}{b*d^2*\eta*f_{cd}} = \frac{75000*10^3}{250*450^2*1*16.7} = 0.088$

$$\Rightarrow \frac{x}{d} = 0.152 < 0.45; \Rightarrow \frac{z}{d} = 0.94 \Rightarrow z = 0.94*d = 423mm$$

- Area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd}*z} = \frac{75000*10^3}{434.8*423} = 408mm^2 = 4.1cm^2$$

- We choose 3 ϕ 14 (A_s = 4.62 cm^2) for flexural reinforcements. ⇒ The resistance of the section is $M^+_{Rd,b} = +119kNm$ and the over-strength factor is $\frac{119}{75} = 1.59$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{4.62}{25*(50-2.0-0.6-\frac{1.4}{2})} = 0.004$$

- For M^-:
  - Beam's dimensions : b=250mm ; h=500mm
  - Concrete cover thickness + ½ reinforcement diameter =60mm
  - Working height d: d = h – concrete cover thickness – ½ reinforcement diameter = 500-60=440mm

$$M_{Rd} \geq M_{Ed}; M_{Rd} = \mu * b * d^2 * \eta * f_{cd}; M_{Ed} = -172000Nm$$

- The ULS condition: $\Rightarrow \mu \geq \frac{M_{Ed}}{b*d^2*\eta*f_{cd}} = \frac{172000*10^3}{250*440^2*1*16.7} = 0.212$

$$\Rightarrow \frac{x}{d} = 0.30 < 0.45; \Rightarrow \frac{z}{d} = 0.88 \Rightarrow z = 0.88*d = 387.2mm;$$

- Area of reinforcement steel can be determined as follows:

$$A_s \geq \frac{M_{Ed}}{f_{yd}*z} = \frac{172000*10^3}{434.8*387.2} = 1022mm^2 = 10.2cm^2$$

- We choose 3 ϕ 18+2ϕ10(A_s = 10.2 cm^2) for flexural reinforcements. ⇒ The resistance of the section is $M^-_{Rd,b} = -174kNm$ and the over-strength factor is $\frac{174}{172} = 1.01$.
- The normalised flexural reinforcements are :

$$\rho = \frac{A_s}{b_w * d} = \frac{10.2}{25*(50-2.0-0.6-\frac{1.8}{2})} = 0.01$$

V.2.5.3 <u>Specific measures for the flexural reinforcement.</u>

- Min/max reinforcing steel
  - In accordance to 5.4.3.1.2 – EC8 [3], minimum tension reinforcement ratio shall not exceed the value: $\rho_{\min} = 0.5 * \frac{f_{ctm}}{f_{yk}} = 0.5 * \frac{0.6}{500} = 0.0006$ (5.12 – EC8 [3]).

    → The reinforcement content is satisfactory.
  - According to 5.4.3.1.2 – EC8 [3], Within the critical regions, the tension reinforcement ratio shall not exceed the value below: $\rho_{\max} = \rho^{'} + \frac{0.018}{\mu_{\phi} * \varepsilon_{sy,d}} * \frac{f_{cd}}{f_{yd}}$

    Where:
    - μ_ϕ - Curvature ductility, $T_1 = 0.67s > T_C = 0.6s \rightarrow \mu_\phi = 2*q_0 - 1 = 6.8$.
    - $\rho' = 0.004$
    - $\varepsilon_{sd,y} = \frac{f_{yd}}{E_s} = \frac{434.8}{200000} = 0.00217 \rightarrow \rho_{\max} = 0.004 + \frac{0.018}{6.7*0.00217} * \frac{16.7}{434.8} = 0.05 > 0.01$

    → The reinforcement content is satisfactory.
- Longitudinal bar diameters:

According to 5.6.2.2 – EC8 [3], to prevent the bond failure, the diameter of longitudinal bars of the beams is limited as the following conditions:

- For interior beam – column joints:

$$\frac{d_{bL}}{h_C} \le \frac{7.5 * f_{ctm}}{\gamma_{Rd} * f_{yd}} * \frac{1 + 0.8\nu_d}{1 + 0.75 * k_D * \frac{\rho'}{\rho_{\max}}}$$

Where:
- h_c – is the width of the column parallel to the bars, so $h_c = 250$mm.
- f_{ctm}: is the mean value of the tensile strength of concrete → $f_{ctm} = 2.6N/mm^2$.
- $F_{yd} = 434.8$ Mpa.
- ν_d – is the normalised design axial force in column, taken with its minimum value for seismic design situation. $\nu_d = \frac{N_{Ed}}{f_{cd} * A_C}$

  $N_{Ed} = -1288000N$; $f_{cd} = 16.67Mpa$; $A_c = 400x500=200000mm^2$.

  $$\rightarrow \nu_d = \frac{N_{Ed}}{f_{cd} * A_C} = \frac{1288000}{16.7 * 200000} = 0.385$$
- k_D – is the factor reflecting the ductility class equal to 1 for DCH, to 2/3 for DCM.
- ρ' – compression steel ratio → $\rho' = 0.004$
- $\rho_{\max} = 0.05$.
- $\gamma_{Rd} = 1$.

So: $\frac{d_{bL}}{h_c} \le \frac{7.5*2.6}{1*434.8} * \frac{1 + 0.8*0.385}{1 + 0.75 * \frac{2}{3} * \frac{0.004}{0.05}} = 0.056 \rightarrow d_{bL} = 400*0.056 = 22.4mm$

→ The chosen reinforcement is satisfactory.

- For exterior beam – column joints:

$$\frac{d_{bL}}{h_C} \le \frac{7.5 * f_{ctm}}{\gamma_{Rd} * f_{yd}} * 1 + 0.8\nu_d \leftrightarrow \frac{d_{bL}}{h_c} \le 4.0 * \frac{f_{ctm}}{f_{yd}} * (1 + 0.8 * \nu_d)$$

Where:
- h_c – is the width of the column parallel to the bars, so $h_c = 600$mm.
- f_{ctm}: is the mean value of the tensile strength of concrete → $f_{ctm} = 2.6N/mm^2$.
- $F_{yd} = 434.8$ Mpa.

- ν_d – is the normalised design axial force in column, taken with its minimum value for seismic design situation. $\nu_d = \frac{N_{Ed}}{f_{cd} * A_C}$

  N_{Ed} = -896000N; f_{cd} = 16.67Mpa; A_c = 400x500=200000mm^2.

$$\rightarrow \nu_d = \frac{N_{Ed}}{f_{cd} * A_C} = \frac{896000}{16.7 * 200000} = 0.268$$

- k_D – is the factor reflecting the ductility class equal to 1 for DCH, to 2/3 for DCM.
- ρ' – compression steel ratio → ρ' = 0.004
- ρ_{max} = 0.05.
- γ_{Rd} = 1.

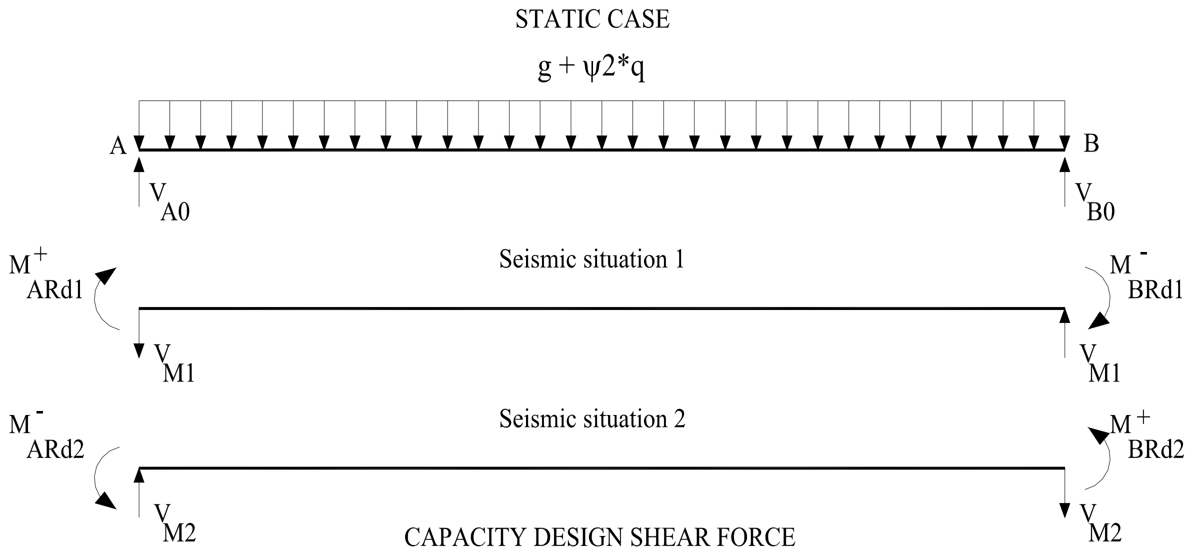
So: $\frac{d_{bL}}{h_c} \le \frac{7.5 * 2.6}{1 * 434.8} * (1 + 0.8 * 0.268) = 0.054$ Or: $\frac{d_{bL}}{h_c} \le \frac{7.5 * 2.6}{1 * 434.8} * \frac{1 + 0.8 * 0.268}{1 + 0.75 * \frac{2}{3} * \frac{0.0047}{0.05}} = 0.052$

→ d_{bL} = 400*0.054=21.6mm → **The chosen reinforcement is satisfactory.**

- Top reinforcement of the beam.
  - In accordance to 2.7.3.4 part 1-3 – ENV8, one fourth of the maximum top reinforcement shall run along the entire beam length.
  - Two ϕ18 bars will run along the entire span.

V.2.5.4 Shear resistance

- Design shear forces computed in accordance to the capacity design criterion:
  - According to 5.4.2.2 – EC8 [3], in the primary seismic beams shear forces shall be calculated in accordance with the capacity design rule
  - The calculation of shear forces as following the sketch below:



- Determining M^+_{Ard1}, M^-_{BRd1}, M^-_{Ard2}, M^+_{BRd2}, V_{A0}, V_{B0}.
  - M^+_{Ard1} : The bottom reinforcement area of longitudinal bars is 2ϕ14+1ϕ16 (5.1cm^2), we determine the value of M_{Ard1} as following:

$$F_{as1} = f_{yd} * A_{s1} = 434.8 * 510 = 221748 Nmm$$

$$F_c = \eta * f_{cd} * b * (h - z) = 16.7 * 250 * (500 - z)$$
$$= 4175 * (500 - z)$$

$$\sum F_{asi} = F_c \rightarrow 221748 = 4175 * (500 - z)$$

$$\rightarrow z = 446.9 mm$$

$$M^{+}_{ARd1} = \sum F_{asi} * b_{asi} + F_c * b_c$$

$$b_{as} = z - coating - \phi stirrup - \frac{1}{2}\phi bars$$

$$b_{as1} = b_{as2} = 446.9 - 20 - 6 - 7 = 413.9 mm$$

$$b_c = \frac{h - z}{2} = \frac{500 - 446.9}{2} = 26.6 mm$$

$$\rightarrow M^{+}_{ARd1} = 221748 * 413.9 + 4175 * (500 - 446.9) * 26.6 = 97678517 Nmm \rightarrow M^{+}_{ARd1} = 98 KNm$$

- M^{-}_{Ard2} : The top reinforcement area of longitudinal bars is 2ϕ18 + 2ϕ20(A_s = 11.9 cm^2), we determine the value of M_{Ard2} as following:

$$\sum F_{asi} = f_{yd} * A_{si} = 434.8 * 1190 = 517412 Nmm$$

$$F_c = \eta * f_{cd} * b * (h - z) = 16.7 * 250 * (500 - z) = 4175 * (500 - z)$$

$$\sum F_{asi} = F_c \rightarrow 517412 = 4175 * (500 - z) \rightarrow z = 376 mm$$

$$M^{-}_{ARd2} = \sum F_{asi} * b_{asi} + F_c * b_c$$

$$b_{asi} = z - coating - \phi stirrup - \phi transverse \frac{1}{2}\phi bars$$

$$b_{as1} = 376 - 20 - 6 - 10 = 340 mm$$

$$b_c = \frac{h - z}{2} = \frac{500 - 376}{2} = 62 mm$$

$$\rightarrow M^{-}_{ARd2} = 434.8 * 1190 * 340 + 4175 * (500 - 376) * 62 = 208017480 Nmm \rightarrow M^{-}_{ARd2} = 208 KNm$$

- M^{-}_{BRd1}:
  The top reinforcement area of longitudinal bars is 4 ϕ 18(A_s = 10.2 cm^2), we determine the value of M_{BRd1} as following:

$$\sum F_{asi} = f_{yd} * A_{si} = 434.8 * 1020 = 443496 Nmm$$

$$F_c = \eta * f_{cd} * b * (h - z) = 16.7 * 250 * (500 - z)$$
$$= 4175 * (500 - z)$$

$$\sum F_{asi} = F_c \rightarrow 443496 = 4175 * (500 - z)$$

$$\rightarrow z = 394 mm$$

$$M^{-}_{BRd1} = \sum F_{asi} * b_{asi} + F_c * b_c$$

$$b_{asi} = z - coating - \phi stirrup - \phi transverse \frac{1}{2}\phi bars$$

$$b_{as1} = 394 - 20 - 6 - 20 - 9 = 339 mm$$

$$b_c = \frac{h - z}{2} = \frac{500 - 394}{2} = 53 mm$$

$$\rightarrow M^{-}_{BRd1} = 434.8 * 1020 * 339 + 4175 * (500 - 394) * 53 = 173800294 Nmm \rightarrow M^{-}_{BRd1} = 174 KN$$

- M^{+}_{BRd2}:
  The bottom reinforcement area of longitudinal bars is 3ϕ14 (4.62cm^2), we determine the value of M_{BRd2} as following:
  $$F_{as1} = f_{yd} * A_{s1} = 434.8 * 462 = 200877.6Nmm$$
  $$F_c = \eta * f_{cd} * b * (h - z) = 16.7 * 250 * (500 - z) = 4175 * (500 - z)$$
  $$\sum F_{asi} = F_c \rightarrow 200877.6 = 4175 * (500 - z)$$
  $$\rightarrow z = 452mm$$
  $$M^{+}_{BRd2} = \sum F_{asi} * b_{asi} + F_c * b_c$$
  $$b_{as} = z - coating - \phi stirrup - \frac{1}{2}\phi bars$$
  $$b_{as1} = b_{as2} = 452 - 20 - 6 - 7 = 419mm$$
  $$b_c = \frac{h - z}{2} = \frac{500 - 452}{2} = 24mm$$
  $$\rightarrow M^{+}_{BRd2} = 200877.6 * 419 + 4175 * (500 - 452) * 24 = 88977314Nmm \rightarrow M^{+}_{BRd2} = 89KNm$$
- Determining V_{B0} and V_{A0}:
$$V_{B0} = V_{A0} = \frac{3750 * 5}{2} + \frac{32850 * 5}{2 * 2} = 50437.5N = 50.4KN$$
- So we have:
$$V_{M1} = -\gamma_{Rd} * \frac{(M_{ARd1} + M_{BRd1})}{l} = -1 * \frac{98 + 174}{5} = -54.4$$
$$V_{M2} = \gamma_{Rd} * \frac{(M_{ARd2} + M_{BRd2})}{l} = 1 * \frac{208 + 89}{5} = 59.4$$
$$V_{B0} = V_{A0} = 50.4KN$$

And so:
- At support A:
  $V_{min} = V_{M1} + V_{A0}$ = -54.4+50.4=-4 ;$V_{max} = V_{M2} + V_{A0}$ = 49.4+50.4=109.8
- At support B:
  $V_{min} = -V_{M2} + V_{B0}$ = -59.4+50.4=-9;$V_{max} = V_{M1} + V_{B0}$ = 54.4+50.4=104.8

- V_{ccd} and V_{Rd} Computations :
  - In the critical sections: $V_{ccd} = 0$
  - Outside the critical sections: $V_{ccd} = V_{Rd,c}$.
  - In accordance with EC(2) 4.3.2.3 and neglecting the axial force influence, the value of $V_{Rd,c}$

Table V.5 – Shear resistance of concrete section

EN 1992
$V_{Rd,c} = \left[\frac{0.18}{\gamma_c} * k * (100 * \rho_l * f_{ck})^{1/3} - 0.15 * \sigma_{cp} \right] * b_w * d$ • f_{ck} – is compressed strength of the concrete at the age of 28 days. $\gamma_c = 1.5$ • $k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ d – mm.   • $\rho_l = \frac{A_{sl}}{b_w * d} \leq 0.02$ where: ▪ A_{sl} – is the area of tension reinforcements.   ▪ b_w – is the minimum width.   • $\sigma_{cp} = \frac{N_{sd}}{A_c}$; N_{sd} – is the longitudinal force. MPa
• Replacing with the value of f_{ck} is 25MPa, reinforcing steel percentage is 0.01, so we have: $V_{Rd,c} = \left[\frac{0.18}{1.5} * 1 * (100 * 0.01 * 25)^{1/3} \right] * 250 * 464$ $= 40703N = 40.7KN$

- Computations
  - The computations shall run in accordance to 6.2.1(2) – EN1992 and the specific rules shall get along with truss model (EN1998)
  - According to 6.2.1(2) – EN 1992, the shear resistance of a member with shear reinforcement is equal to $V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}$. V_{td} is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord, so $V_{td} = 0$ and $V_{cc,d} = 0$.
    So:
    $V_{Rd} = V_{Rd,s} + V_{ccd}$
    $$V_{Rds} = \frac{A_{sw}}{s} * z * f_{ywd} * \cos\theta$$
  - In critical regions (2*height of the beams) shear force will be carried out only by the stirrups. We choose stirrups with 2 legs, of 6mm in diameter and 80mm spacing. Shear force capacity is:
    $$V_{Rd} = V_{Rds} = \frac{57}{80} * 450 * 434.8 = 139408N = 139.4KN >> V_{max}.$$
  - Outside critical regions, shear forces are carried out by stirrups with 2 legs, 6mm in diameter and 150mm spacing. Shear force capacity is:
    $$V_{Rd} = V_{Rd,c} + V_{Rds} = \frac{57}{120} * 464 * 434.8 = 95KN$$
- Along the whole beam length, shear force must be less than the value of $V_{Rd,max}$ which is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.
  $V_{Rd,max} = \alpha_{cw} * b_w * z * \nu_1 * f_{cd} / (\cot\theta + \tan\theta)$
  Where:
  ν_1 is a strength reduction factor for concrete cracked in shear
  α_{cw} is a coefficient taking account of the state of the stress in the compression chord.

So: $\upsilon_l = 0.6*\left[1-\frac{f_{ck}}{250}\right] = 0.6*0.9 = 0.54$ and $\alpha_{cw} = 1$.

$\rightarrow V_{Rd,max}$ = 1*250*464*16.7/(1+1)=968KN>V_{max}

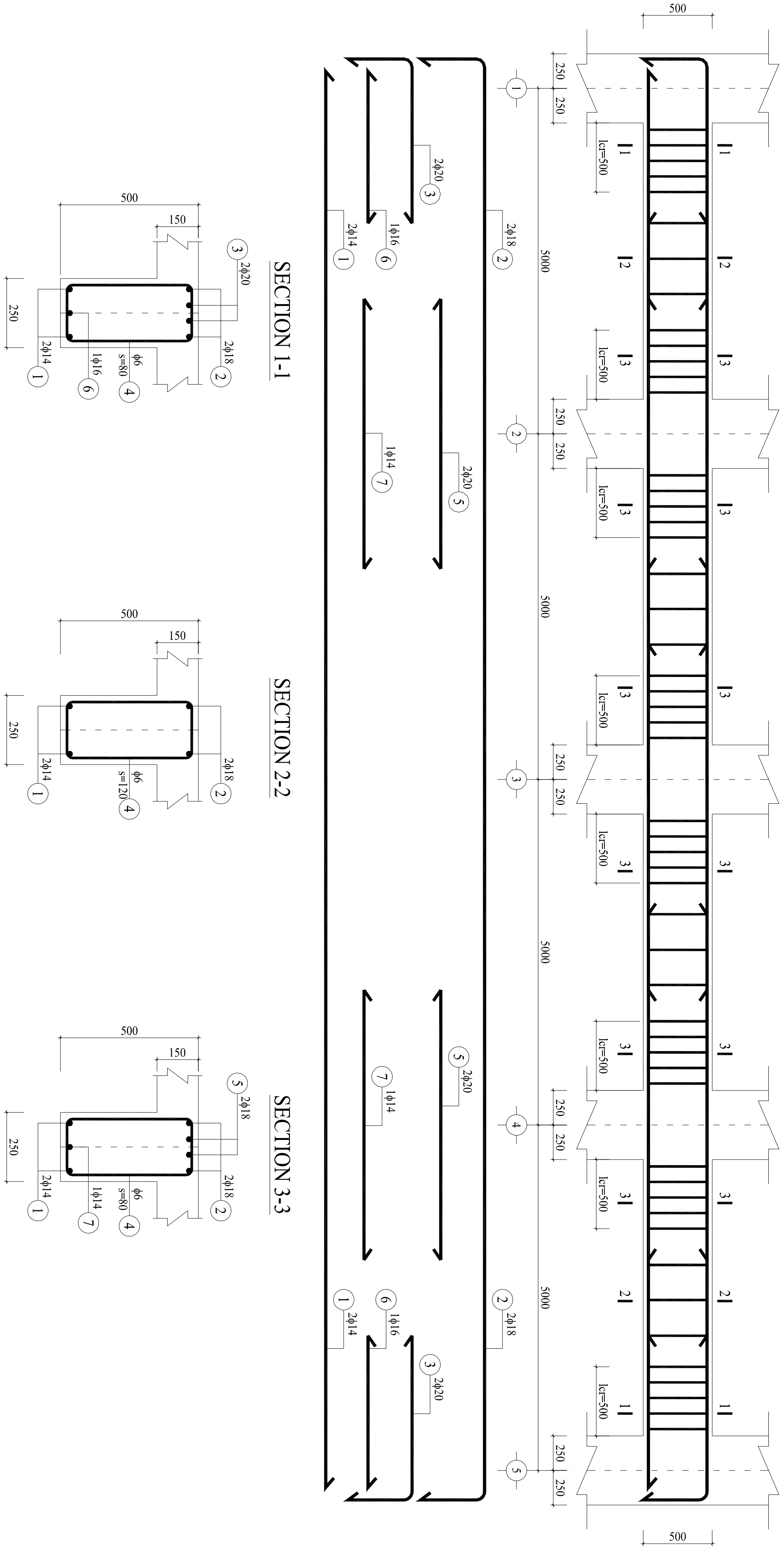
V.2.5.5 **Specific measures**

- Detailing:
  - In accordance to 5.4.3.1.2(6P) - EC8 [3], the stirrup minimum diameter within the critical regions is 6mm – this requirement is met.
  - The first hoop is placed not more than 50mm from the end cross section of the beam – this requirement is met.
  - Within the critical regions, the spacing of the hoops is not greater than:

$$h_w / 4 = 125mm; 24*d_{bw} = 24*6 = 144mm; 225mm$$

$$8*d_{bl} = 8*14 = 112mm$$

- Casting and Placing for beam : All requirements are met.



DETAILING FOR THE FIRST FLOOR BEAM - LONGITUDINAL FRAME

V.2.6 REINFORCEMENT OF OTHER BEAMS

The reinforcement of other beams of the transverse frame will be determined by using the similar ways as the beams on the first floor. They are summarized as following tables.

Table V.6: Properties of the section and seismic actions of longitudinal or direction X frame

Floor level	Position of column	Sections of beams	b_w (mm)	b_c (mm)	h_w (mm)	b_{eff} (mm)	Δ_{nom} (mm)	M^+_{Ed} (kNm)	M^+_{Rd} (kN m)	M^-_{Ed} (kNm)	M^-_{Ed} (kN m)
1-2	External	End	250	500	500	1700	30	93.3	98	-181.1	-208
1-2	Internal	End	250	500	500	1700	30	75.6	89	-172.1	-174
1-2	Internal	Middle	250	500	500	1700	30	38.8			
3-4	External	End	250	500	500	1700	30	60.1		-164.3	
3-4	Internal	End	250	500	500	1700	30	60.9		-144.5	
3-4	Internal	Middle	250	500	500	1700	30	36.0			
5	External	End	250	500	500	1700	30			-112	
5	Internal	End	250	500	500	1700	30	12.4		-88.8	
5	Internal	Middle	250	500	500	1700	30	35.7			
6	External	End	250	500	500	1700	30			-68.3	
6	Internal	End	250	500	500	1700	30			-68.8	
6	Internal	Middle	250	500	500	1700	30	31.7			

Table V.7: Designed Longitudinal Reinforcement and specific measures in beams of longitudinal frame

Beams of Floor	Position of column	Sections of the beams	Top Reinforc (mm^2)	Bottom Reinforcement (mm^2)	ρ (%)	ρ' (%)	ρ_{max} (%)	ρ_{min} (%)	d_{max} (mm)	M_{Rd} (KNm)
1-2	External	End	1190	510(2ϕ14+1ϕ16)	0.45	1	5.2	0.26	28	-208
1-2	Internal	End	1020	462(3ϕ14)	0.4	0.9	5.2	0.26	28	-174
1-2	Internal	Middle	508	308(2ϕ14)	0.27	0.45	5.2	0.26	26	
3-4	External	End	1020	462(3ϕ14)	0.4	0.9	5.2	0.26	26	
3-4	Internal	End	816	462(3ϕ14)	0.4	0.72	5.2	0.26	26	
3-4	Internal	Middle	508	462(3ϕ14)	0.4	0.45	5.2	0.26	26	
5	External	End	620	308(2ϕ14)	0.5	0.27	5.2	0.26	26	
5	Internal	End	620	308(2ϕ14)	0.5	0.27	5.2	0.26	26	
5	Internal	Middle	620	308(2ϕ14)	0.5	0.27	5.2	0.26	26	
6	External	End	462	308(2ϕ14)	0.4	0.27	5.2	0.26	26	
6	Internal	End	462	308(2ϕ14)	0.4	0.27	5.2	0.26	26	
6	Internal	Middle	462	308(2ϕ14)	0.4	0.27	5.2	0.26	26	

Table V.8: Designed Stirrups and specific measures in beams of longitudinal frame

Beams of Floor	Sections of the beams	V_{max} (KN)	ϕ - stirrup (mm)	Number of legs	Spacing	V_{Rd} (KN)	V_{Rdmax} (KN)
1-2	Critical region	100	6	2	80	145	1053
1-2	Outside critical region	76.6	6	2	120	129	1053
3-4	Critical region	93.9	6	2	80	145	1053
3-4	Outside critical region	70.4	6	2	120	129	1053
5-6	Critical region	73.8	6	2	80	145	1053
5-6	Outside critical region	50.3	6	2	120	129	1053

VI COLUMN DESIGN

VI.1 Ground story column in axis B2 (members 121-122 transverse frame or members 97-98 longitudinal frame)

VI.1.1 Geometrical restraint

- Cross section dimensions: In accordance to 5.5.1.2.2 (1P) – EC8 [3], the minimum cross-sectional dimension of primary seismic columns shall be not less than 250 mm →Because the cross section dimensions for the ground–level columns are 400x500, so this condition is met.
- According to 5.4.1.2.2 (1), unless $\theta \leq 0.1$ (4.4.2.2(2) – EC8 [3]), the cross-sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contra-flexure and the ends of the column, for bending within a plane parallel to the column dimension considered → Because of $\theta \leq 0.1$, so this condition must not be checked.
-

VI.1.2 Action effects due to the analysis for the seismic combination

- In order to design the member 121-122, we will determine the actions from two members: 121-122 and 123-124 (see figures 1 and 2)
- According to result from SAP2000 – version 9.0, we have action effects for members 121-122 and 123-124 as following tables:

Table VI.1 Calculated Action effects for members 121-122 and 123-124

Seismic directions	Top for member 122			Bottom for member 123		
	N (axial force) – KN	V shear force KN	M (bending) - KNm	N Axialforce KN	V shear force KN	M bending KNm
Transverse Seism Y →	-1255	87.5	-121.5	-1056	96	144
Longitudinal Seism X →	-1288	79.0	-110	-1075	83.6	126
Transverse Seism Y ←	-1291	-87.3	121.1	-1076	-94.7	-143
Longitudinal Seism X ←	-1255	-79.5	111	-1054	-84.6	-127

Table VI.2 Action effects for the base of members 121-122.

Seismic directions	Base of member 121					
	Seismic Direction Y →			Seismic Direction X ←		
	N axial force KN	V shear force KN	M Bending KNm	N axial force KN	V shear force KN	M Bending KNm
Transverse Y	-1273	87.5	185	-1308	-87.3	-184.4
Longitudinal X	-1306	79.0	166.5	-1272	-79.5	-167.4

VI.1.3 <u>Flexural reinforcement for columns</u>

VI.1.3.1 <u>Design bending moment</u>

For columns DCM, the design bending moments are determined in accordance to the capacity design criterion 5.4.2.3 – EC8. So, the sketch of designing column is the following pictures:

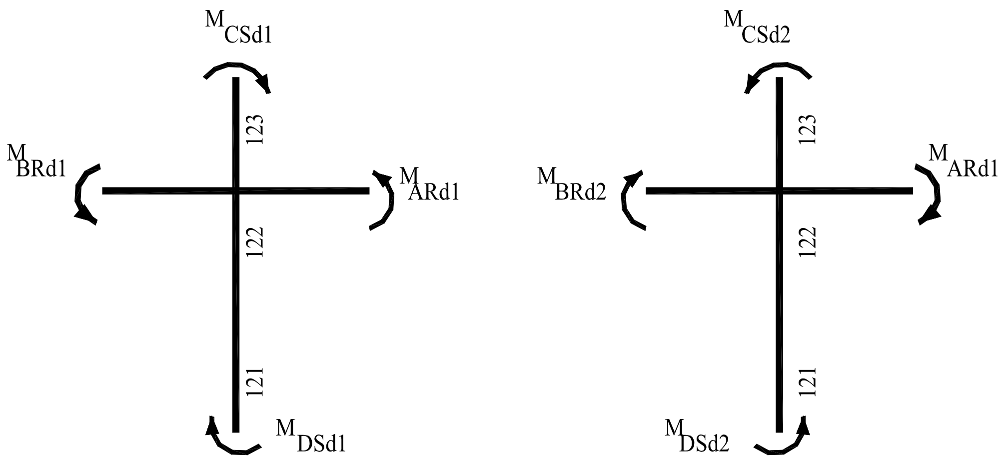


Table VI.3: Applied bending moments for column member 121-122 in transverse frame as derived from the analysis of the structure.

Transverse frame (Direction 1 →)			Transverse frame (Direction 2 ←)		
Bending Resistance of beam	Over-strength factor	Average	Bending Resistance of beam	Over-strength factor	Average
M^{-}_{BRd1} = -208 KNm	1.18	1.32	M^{+}_{BRd2} = +119 KNm	1.18	1.32
M^{+}_{ARd1} = +119 KNm	1.45		M^{-}_{ARd2} = -208 KNm	1.45	
Calculated action effects of columns					
M_{CSd1} = 144.0 KNm			M_{CSd2} = 143.1 KNm		
M_{DSd1} = 121.4 KNm			M_{DSd2} = 121.1 KNm		

Table VI.4: Applied bending moments for column member 121-122 in longitudinal frame as derived from the analysis of the structure.

Longitudinal frame (Direction 1 →)			Longitudinal frame (Direction 2 ←)		
Bending Resistance of beam	Over-strength factor	Average	Bending Resistance of beam	Over-strength factor	Average
M^{-}_{BRd1} = -174 KNm	1.01	1.09	M^{+}_{BRd2} = +89 KNm	1.17	1.09
M^{+}_{ARd1} = +89 KNm	1.17		M^{-}_{ARd2} = -174 KNm	1.01	
Calculated action effects of columns					
M_{CSd1} = 126.1 KNm			M_{CSd2} = 126.6 KNm		
M_{DSd1} = 110.1 KNm			M_{DSd2} = 111 KNm		

- According to 4.4.2.3 – EC8, in multi-storey buildings formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey. In frame buildings, with two or more storeys,

the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns: $\Sigma M_{Rc} \geq 1,3\Sigma M_{Rb}$. – 4.29
where

- ΣM_{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
- ΣM_{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 ΣM_{Rb}.

So, once the beam resistance in bending M_{BRd1} and M_{BRd2} have been defined, the required moment resistance in columns are computed as:

$M_{Sd1,CD}$ = 1,3 x ($M_{BRd1} + M_{BRd2}$)/

(Beam number 31-33): $M^{+} = 93304Nm$ $M^{-} = -181173Nm$

And so, we have: For the top section of column member 121-122 in the transverse frame:

Direction Y 1 →	Direction Y 2 ←
$M_{tR,b} = 1.3\sum\left(M_{Rb,i}\right) = 1.3(208+119) = 425$ $M_{Sd1,CD} = \dfrac{121.4}{121.4+144} * M_{tR,b} = 0.45*425 = 194KNm$	$M_{tR,b} = 1.3\sum\left(M_{Rb,i}\right) = 1.3(208+119) = 425$ $M_{Sd2,CD} = \dfrac{121.1}{121.1+143.1} * M_{tR,b} = 0.46*425 = 195.5KNm$

For the top section of column member 121-122 in the longitudinal frame:

Direction X 1 →	Direction X 2 ←
$M_{tR,b} = 1.3\sum\left(M_{Rb,i}\right) = 1.3(174+89) = 342kNm$ $M_{Sd1,CD} = \dfrac{110.1}{110.1+126} * M_{tR,b} = 0.46*342 = 157KNm$	$M_{tR,b} = 1.3\sum\left(M_{Rb,i}\right) = 1.3(174+89) = 342kNm$ $M_{Sd2,CD} = \dfrac{111}{111+126.6} * M_{tR,b} = 0.47*342 = 161KNm$

For the bottom section of column member 121-122 in the transverse frame:

Direction Y 1 →	Direction Y 2 ←
$M_{Sd1,CD} = 1.3*1.32*121.4 = 208.3KNm$ Safe side: $M_{Sd1,CD} = 1.3*1.45*121.4 = 229KNm$	$M_{Sd2,CD} = 1.3*1.32*121.1 = 208KNm$ Safe side: $M_{Sd2,CD} = 1.3*1.45*121.1 = 228.3KNm$

For the bottom section of column member 121-122 in the longitudinal frame:

Direction X 1 →	Direction X 2 ←
$M_{Sd1,CD} = 1.3*1.09*110.1 = 156KNm$ Safe side: $M_{Sd1,CD} = 1.3*1.17*110.1 = 167KNm$	$M_{Sd2,CD} = 1.3*1.09*111 = 157KNm$ Safe side: $M_{Sd1,CD} = 1.3*1.17*111 = 169KNm$

- In accordance to 4.3.3.5.1 – EC8 [3], the computations of the flexural reinforcement takes into account the bi-directional character of the seismic action effects. We take into account two seismic action directions:
  - i. Along X axis
  - ii. Along Y axis
- According to 4.3.3.5.1 (3) – EC8 [3], the combination of the horizontal components of the seismic action may be computed using two following combinations:

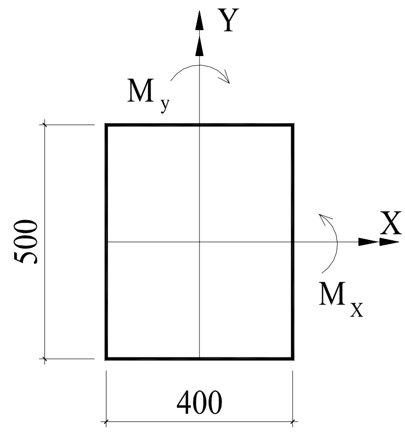
a. $E_{Edx} + 0.30\ E_{Edy}$
b. $0.30E_{Edx} + E_{Edy}$.

So, we have two combinations of efforts:

a. M_x = 229KNm, M_y = 0.3*167=50.1 KNm, N = -1309.3KN
b. M_x = 229*0.3=69KNm, M_y = 167KNm, N=-1306KN

VI.1.3.2 <u>Column flexural reinforcement:</u>

The designing follows Concise Eurocode 2 [5] and "How to design reinforced concrete using EC2" [6]:



- Check the slenderness:
  - Effective length l_0:

$$l_0 = 0.5 * l * \sqrt{\left(1+\frac{k_1}{0.45+k_1}\right)*\left(1+\frac{k_2}{0.45+k_2}\right)} \qquad \text{(Exp (5.15 – EC2 [2])}$$

Where: k1,k2 are the relative flexibilities of rotational restraints at end 1 and 2.
We can compute l_0 as following expression:
$l_0 = 0.85*l_{clearance}$.
$L_{clearance} = H-h_w$ = 3500-500=3000mm
→ l_0 = 0.85*3000=2550

- The column slenderness: $\lambda = \frac{l_0}{i}$

  Where: i – radius of gyration,

$$i = \sqrt{\frac{I}{F}} = \frac{h}{\sqrt{12}} or \frac{b}{\sqrt{12}} \Rightarrow i = \frac{500}{\sqrt{12}} = 144.3\, and\, i = \frac{400}{\sqrt{12}} = 115.5$$

  So, λ = 17.7 and λ = 22.1
- The limiting slenderness, λ_{lim}

  $\lambda_{\lim} = 20 * A * B * C / \sqrt{n}$

  Where:
  - B=1.1 A=0.7 C=1.7-r_m = 1.7-(-M_{01}/M_{01})=2.7
  - $\eta = \frac{N_{Ed}}{A_c * f_{cd}} = \frac{1309000}{400*500*16.7} = 0.392$

$\lambda_{\lim} = 20*0.7*1.1*12.7/\sqrt{0.392} = 68$

$\rightarrow$ Column is not slender

- Cover:

  $c_{nom} = c_{min} + \Delta c_{dev}$ (Exp 4.1 – EC2 [2])

  Where:

  - $c_{min,b}$ = diameter of bar. Assume 25mm bars and 8mm hoops
  - $c_{min,\ dur}$ = minimum cover due to environmental conditions. Assume XC1 $\rightarrow$ $c_{min,dur}$ = 15mm

  $\rightarrow c_{nom}$ = 25mm

- Design moments:

According to 5.8.8.2(1) – EC2 [2], we calculate the design moments for the flexural reinforcements:

$M_{Ed} = \max\left[M_{02}, M_{Ed} + M_2, M_{01} + 0.5*M_2\right]$

Where:

- $M_{02} = M + e_i*N_{Ed} \geq e_0*N_{Ed}$
- $e_i = l_0/400$
- e_{0x} = max(500/30,20)=20mm
- e_{0y} = max(400/30,20)=20mm
-

So, we have two cases for design moments:

* CASE 1:
$$M_{02x} = 229 + 1309.3*\frac{2.55}{400} = 235.4 > 0.02*1309.3$$
$$M_{02y} = 50.1 + 1309.3*\frac{2.55}{400} = 53.8 > 0.02*1309.3$$
and N = 1309.3

- CASE 2:
$$M_{02x} = 69 + 1306*\frac{2.55}{400} = 76.6 > 0.02*1306$$
$$M_{02y} = 167 + 1306*\frac{2.55}{400} = 160 > 0.02*1306$$
and N = 1306

- According to [4], we use charts to determine flexural reinforcements:

CASE 1:

a. $\beta = \dfrac{b - 1.5*d_b^{'}}{h - 1.5*d_h^{'}} = \dfrac{400 - 1.5*40}{500 - 1.5*40} = 0.8$

b. First estimate of $M_{ux} = M_x + \dfrac{M_y}{\beta} = 235.4 + \dfrac{53.8}{0.8} = 302.7kNm$

c. $\dfrac{N_{uy}}{N} = 1 + \dfrac{M_{ux}}{0.4*N*\left(h - 1.5*d_h^{'}\right)} = 1 + \dfrac{302.7}{0.4*1309.3*(0.5 - 1.5*0.004)} = 2.079 \rightarrow \dfrac{N}{N_{uy}} = 0.48$

d. From table 5.4 [4] $\rightarrow$ a=1.34

e. $\rightarrow$Corrected M_{ux}: $M_{ux} = \left[M_x^a + \left(\dfrac{M_y}{\beta}\right)^a\right]^{\frac{1}{a}} = \left[235.4^{1.34} + \left(\dfrac{53.8}{0.8}\right)^{1.34}\right]^{\frac{1}{1.34}} = 246.3$

f. So,
$$\frac{M_{ux}}{b*h^2*f_{ck}} = \frac{246.3*10^6}{400*500^2*25} = 0.100$$
$$\frac{N}{b*h*f_{ck}} = \frac{1309.3*1000}{400*500*25} = 0.262$$

With the ratios $d_h^{'}/h$, $\eta = \dfrac{N}{b*h*f_{ck}}$ and $\dfrac{M_{ux}}{b*h^2*f_{ck}}$ is equal to 0.08, 0.262 and 0.116, we use the chart number 5.16 and we have $\dfrac{A_s*f_{yk}}{b*h*f_{ck}} = 0.24 \rightarrow$ A_s = 2400mm^2. We choose 16ϕ16 – A_s = 3216mm^2.

CASE 2:

a. $\beta = \dfrac{h-1.5*d_h^{'}}{b-1.5*d_b^{'}} = \dfrac{500-1.5*40}{400-1.5*40} = 1.294$

b. First estimate of $M_{uy} = M_y + \dfrac{M_x}{\beta} = 160 + \dfrac{76.6}{1.294} = 219.2$

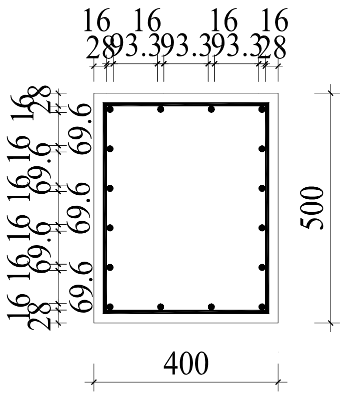
c. $\dfrac{N_{ux}}{N} = 1 + \dfrac{M_{uy}}{0.4*N*\left(b-1.5*d_b^{'}\right)} = 1 + \dfrac{219.2}{0.4*1306*(0.4-1.5*0.004)} = 2.065 \rightarrow \dfrac{N}{N_{ux}} = 0.484$

d. From table 5.4 [4] → a=1.34

e. →Corrected M_{ux}: $M_{ux} = \left[M_x^a + \left(\dfrac{M_y}{\beta}\right)^a\right]^{\frac{1}{a}} = \left[160^{1.34} + \left(\dfrac{76.6}{1.294}\right)^{1.34}\right]^{\frac{1}{1.34}} = 190.6$

f. So,
$$\frac{M_{ux}}{b^2*h*f_{ck}} = \frac{190.6*10^6}{500*400^2*25} = 0.100$$
$$\frac{N}{b*h*f_{ck}} = \frac{1306*1000}{400*500*25} = 0.261$$

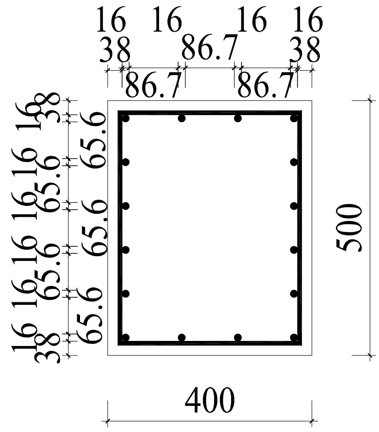
With the ratios $d_b^{'}/b$, $\eta = \dfrac{N}{b*h*f_{ck}}$ and $\dfrac{M_{ux}}{b*h^2*f_{ck}}$ is equal to 0.1, 0.261 and 0.119, we use the chart number 5.16 [4] and we have $\dfrac{A_s*f_{yk}}{b*h*f_{ck}} = 0.24 \rightarrow$ A_s = 2400mm^2. We choose 16ϕ16 – A_s = 3216mm^2.



COLUMN REINFROCEMENT SECTION

VI.1.3.3 <u>The effective bending moment capacities to uni-axial bending</u>

- Along X direction: $M_{x,cap}$
  - $A_{s,tot} = 16\phi16 = 3216mm^2$; N = 1295000N



COLUMN REINFORCEMENT SECTION

- We assume that $12\phi16$ are in the tensile area, $4\phi16$ are in the compression area and z is the distance taking from compressed fiber to the neutral axis

$$F_{as} = \sum F_{asi} = \sum A_{si} * f_{yd} = 12*201*434.8 = 1048737.6$$

$$F_{ai} = \sum F_{aii} = \sum A_{si} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_c = f_{cd} * b * (h - z) = 16.7*400*(500 - z) = 6680*(500 - z)$$

$$\rightarrow F_{as} + N = F_{ai} + F_c$$

$$\leftrightarrow 1048737.6 + 1295000 = 349579.2 + 6680(500 - z)$$

$$\rightarrow 500 - z = 298.5 \rightarrow z = 201.5mm$$

- The condition of z is that: (38+16)<500-z<(38+16+69.6)→ 54<298.5<123, so the value of z is not correct.
- We assume that the neutral axis will run through the right – third layer reinforcement; so there are over $6\phi16$ in compression zone and over 8 $\phi16$ in tension zone.

$$F_{as1} = A_{s1} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_{as2} = F_{as3} = A_{s2} * f_{yd} = 2*201*434.8 = 219789.6$$

$$F_{as4} = A_{s4} * f_{yd} * \frac{z - 282.8}{16}$$

$$F_{ai1} = A_{cs1} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_{ai2} = A_{cs2} * f_{yd} = 2*201*434.8 = 219789.6$$

$$F_{ai3} = A_{s4} * f_{yd} * (1 - \frac{z - 282.8}{16})$$

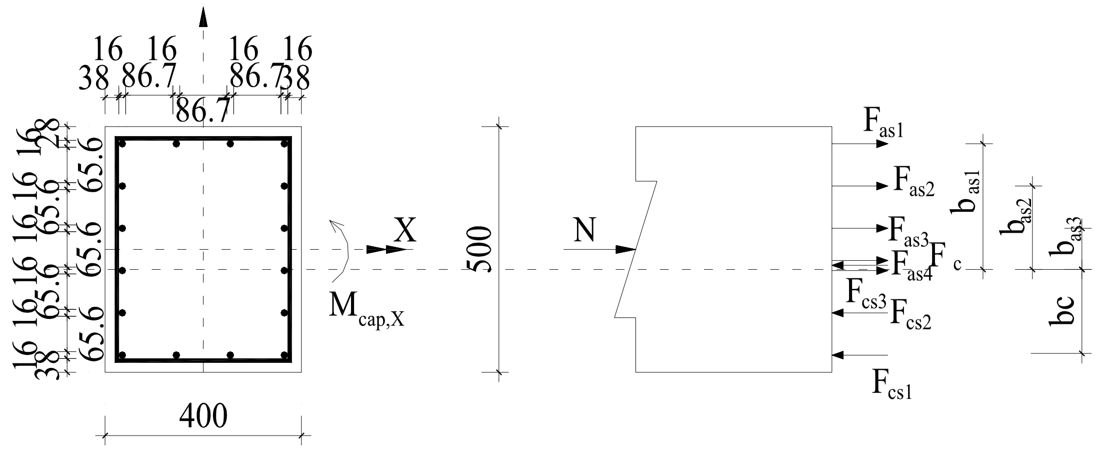
$$F_c = f_{cd} * b * (h - z) = 16.7*400*(500 - z) = 6680*(500 - z)$$

$$\rightarrow \sum F_{si} + N = \sum F_{ai} + F_c$$

$$\rightarrow z = 287.4mm$$

The condition of z is that 282.8 <=z<=282.8+16=298.8, so this condition is met

- Plastic moment will be calculated as following:



PLASTIC MOMENT CALCULATION

$$b_{as1} = z - a_{s1} = 287.4 - (38+8) = 241.4mm$$

$$b_{as2} = z - a_{s2} = 287.4 - (38+16+65.6+8) = 159.8mm$$

$$b_{as3} = z - a_{s3} = 287.4 - (38+16*2+65.6*2+8) = 78.2mm$$

$$b_{as4} = 287.4 - 282.8 = 4.6mm$$

$$b_{cs1} = h - z - a_{cs1} = 500 - 287.4 - (38+8) = 166.6mm$$

$$b_{cs2} = h - z - a_{cs2} = 500 - 287.4 - (38+16+65.6+8) = 85mm$$

$$b_{cs3} = 16 - b_{as4} = 16 - 4.6 = 11.4mm; b_c = \frac{500-z}{2} = 106.3mm$$

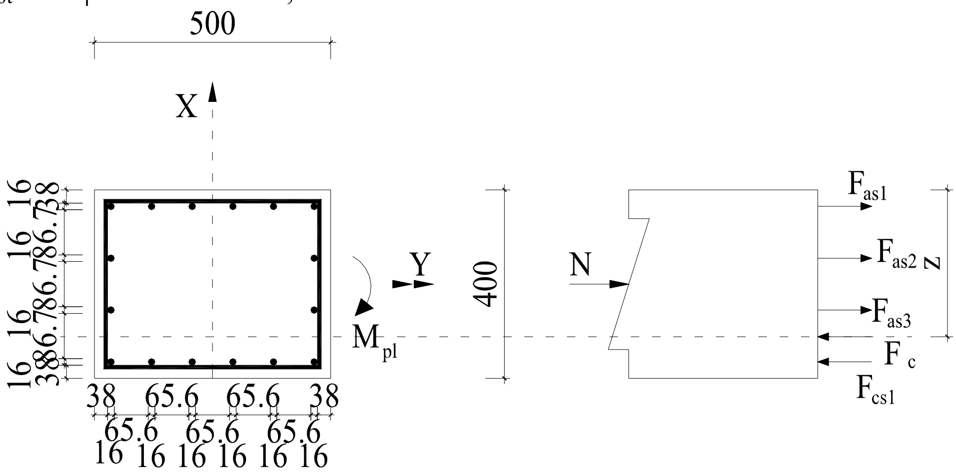
$$F_c = 16.7*(500-287.4)*400 = 1420168N$$

$$M_{x,cap} = \sum F_{asi}*b_{asi} + \sum F_{csi}*b_{csi} + F_c*b_c =$$

$$= 349579.2*(241.4+166.6) + 219789.6*(159.8+78.2+85) + 219789.6*\frac{4.6}{16}*4.6$$

$$+219789.6*\frac{11.4}{16}*11.4 + 1420168*106.3 = 366660126Nmm = 367KNm$$

- Along Y direction: $M_{y,cap}$
  - $A_{s,tot}$ = 16ϕ16 = 3216mm^2; N = 1306000N



PLASTIC MOMENT CALCULATION

- We assume that 10ϕ16 are in the tensile area, 6ϕ16 are in the compression area and z is the distance taking from compressed fiber to the neutral axis

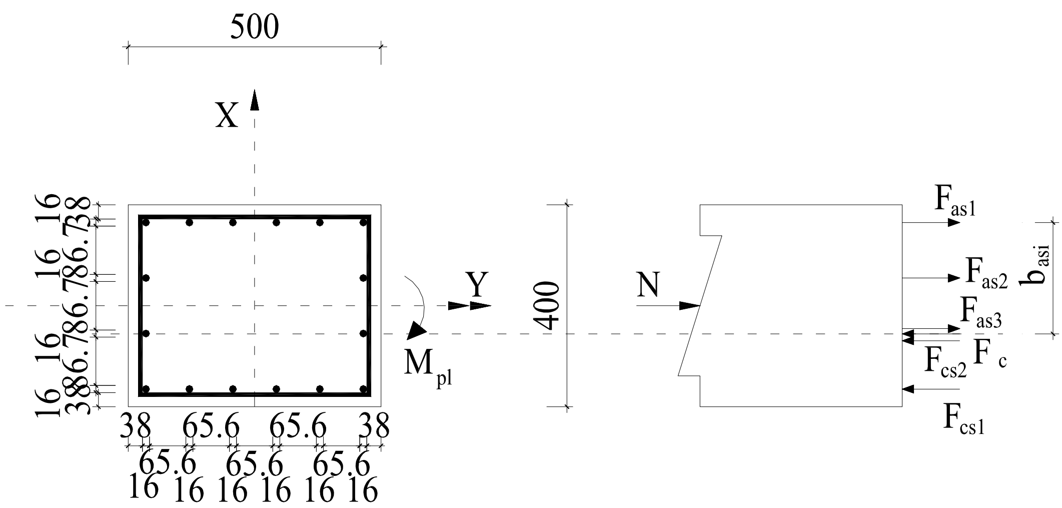
$$F_{as} = \sum F_{asi} = \sum A_{si} * f_{yd} = 10 * 201 * 434.8 = 873948.0$$
$$F_{ai} = \sum F_{aii} = \sum A_{si} * f_{yd} = 6 * 201 * 434.8 = 524368.8$$
$$F_c = f_{cd} * b * (h - z) = 16.7 * 500 * (400 - z) = 8350 * (400 - z)$$
$$\rightarrow F_{as} + N = F_{ai} + F_c$$
$$\leftrightarrow 873948.0 + 1306000 = 524368.8 + 8350(400 - z)$$
$$\rightarrow 400 - z = 198.3 \rightarrow z = 201.7mm$$

- The condition of z is that: (38+16)<400-z<(38+16+86.7)→ 54<198.3<140, so the value of z is not correct.
- We assume that the neutral axis will run through the first layer reinforcement; so there are less than 8ϕ16 in compression zone and over 8 ϕ16 in tension zone.

$$F_{as1} = A_{s1} * f_{yd} = 6 * 201 * 434.8 = 524368.8$$
$$F_{as2} = A_{s2} * f_{yd} = 2 * 201 * 434.8 = 174789.6$$
$$F_{as3} = A_{s3} * f_{yd} * \frac{z - 243.4}{16} = 2 * 201 * 434.8 * \frac{z - 243.4}{16} = 174789.6 * \frac{z - 243.4}{16}$$
$$F_{cs1} = 6 * 201 * 434.8 = 524368.8N$$
$$F_{cs2} = A_{as3} * f_{yd} * (1 - \frac{z - 243.4}{16}) = 174789.6 * (1 - \frac{z - 243.4}{16})$$
$$F_c = f_{cd} * b * (h - z) = 16.7 * 500 * (400 - z) = 8350 * (400 - z)$$
$$\rightarrow \sum F_{si} + N = \sum F_{ai} + F_c$$
$$\leftrightarrow 174789.6 + 174789.6 * \frac{z - 243.4}{16} + 1306000 = 174789.6 * (1 - \frac{z - 243.4}{16}) + 8350 * (400 - z)$$
$$\rightarrow z = 243.5mm$$

The condition of z is that 243.4 <z<=243.4+16, so this condition is met

- Plastic moment will be calculated as follows:



PLASTIC MOMENT CALCULATION

$b_{as1} = z - a_{s1} = 243.5 - (38+8) = 197.5mm; b_{as2} = z - a_{s2} = 243.5 - (38+16+86.7+8) = 94.8mm$

$b_{as3} = z - a_{s3} = 243.5 - 243.4 = 0.1mm; b_{cs1} = h - z - a_{cs1} = 400 - 243.5 - (38+8) = 110.5mm$

$b_{cs2} = 16 - b_{as3} = 16 - 0.1 = 15.9mm; b_c = \dfrac{400-z}{2} = 78.25mm$

$F_c = 16.7*(400-243.5)*500 = 1306775N$

$M_{x,cap} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c = 524368.8*(197.5+110.5) + 174789.6*94.8 +$

$174789.6*\dfrac{0.1}{16}*0.1 + 174789.6*\dfrac{15.9}{16}*15.9 + 1306775*78.25 = 283092682Nmm = 283KNm$

- The effective reinforcing steel percentages:

$\rho_x = \dfrac{A_s}{b*d} = \dfrac{12*2.01}{40*(50-5.25)} = 0.01684 = 1.684\%; \rho_x^{'} = \dfrac{A_s^{'}}{b*d} = \dfrac{4*2.01}{40*(50-5.25)} = 0.0045 = 0.45\%$

$So, \rho_{xtot} = 2.13\%;$

$$\rho_y = \frac{A_s}{b*d} = \frac{10*2.01 + 6*2.01*\dfrac{356.2-355.9}{16}}{50*(40-2.19)} = 0.011 = 1.1\%$$

$$\rho_x^{'} = \frac{A_s^{'}}{b*d} = \frac{6*2.01*\left(1-\dfrac{356.2-355.9}{16}\right)}{50*(40-2.19)} = 0.00626 = 0.63\%$$

$So, \rho_{xtot} = 1.73\%$

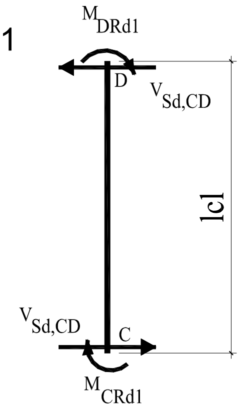
VI.1.3.4 Specific measures for the flexural reinforcement

- According to 5.4.3.2.1(3) – EC8 [3], in primary seismic columns the value of the normalised axial force v_d shall not exceed the value of 0.65.
  $\upsilon_d = \frac{N_{sd}}{A_c * f_{cd}} = \frac{1309.3*1000}{400*500*16.7} = 0.392 < \upsilon_{d,\max}$, so the condition is met
- According to 5.4.3.2.2(1P) – EC8 [3], The total longitudinal reinforcement ratio ρl shall be not less than 0,01 and not more than 0,04. In symmetrical cross-sections symmetrical reinforcement should be provided (ρ = ρ′) → The condition is met.
- According to 5.4.3.2.2(2) – EC8 [3], at least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints. → The condition is met.
-

VI.1.3.5 Shear Resistance

- In accordance to 5.4.2.3 – EC8 [3], the design values for the acting shear forces shall be determined in accordance with the capacity design criterion, as it follows:
  - According to 5.4.2.3(2), end moments $M_{i,d}$ may be determined from the following expression: $M_{i,d} = \gamma_{Rd} * M_{Rc,i} * \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right)$ and γ_{Rd} is equal to 1.1
  - $V_{Ed,CD} = \gamma_{Rd} * \frac{M_{DRd} + M_{CRd}}{l_{cl}}$

  Where:
  l_{cl} = 3500-500=3000mm; γ_{Rd} = 1.1.
  $M_{DRd} = M_{CRd} = M_{x,cap}$ = 283KNm. So $V_{Ed,CD}$ = 208KN



- $V_{Rd,c}$ Computations :
  - In accordance with EC(2) 4.3.2.3 and neglecting the axial force influence, the value of $V_{Rd,c}$ is calculated as below
  -

EN 1992

$$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{1/3} + k_1 * \sigma_{cp}\right] * b_w * d$$

- $C_{Rd,c} = 0.18/\gamma_c$
- f_{ck} – is compressed strength of the concrete at the age of 28 days. γ_c = 1.5
- $k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{378.1}} = 1.73 \leq 2.0$
- $\rho_l = \frac{A_{sl}}{b_w * d} \leq 0.02$ where:
  - A_{sl} – is the area of tension reinforcements.
  - b_w – is the minimum width.
- $\sigma_{cp} = \frac{N_{sd}}{A_c}$; N_{sd} – is the longitudinal force. So, $\sigma_{cp} = \frac{1306*1000}{500*400} = 6.53 N/mm^2$
- k_1 = 0.15

$$V_{Rd,c} = \left[\frac{0.18}{1.5} * 1 * (100 * 0.01 * 25)^{1/3}\right] * 500 * 378.1 + 0.15 * 6.53 * 500 * 378.1 = 251508.7N = 252KN$$

- Computations

- The computations shall run in accordance to 6.2.1(2) – EN1992 and the specific rules shall get along with truss model (EN1998)
- According to 6.2.1(2) – EN 1992, the shear resistance of a member with shear reinforcement is equal to $V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}$. V_{td} is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord, so $V_{td} = 0$. So: $V_{Rd} = V_{Rd,s} + V_{ccd}$; $V_{Rd,s} = \frac{A_{sw}}{s} * z * f_{ywd} * \cos\theta$
- Because of $V_{Rd,c} > V_{Ed,\,CD}$ so the shear resistance of the concrete of the column is enough to carry the shear forces. So we can choose the hoops with 8mm in diameter and 4 legs, the distance is 100mm.

VI.1.3.5.1 Specific Measures

- In accordance to 5.4.3.2.2 – EC8 [3], in the critical sections of the primary seismic columns the diameter of hoops or cross-tie is at least of 6mm. The diameter of the hoops is of 8mm, so this condition is satisfactory.
- In accordance to 5.4.3.2.2 (11)– EC8 [3], the maximum spacing of the hoops:
s_{max} = min (b_0/2; 175; 8d_{bL}) = min (344/2;175;8*16)=128mm;
s = 100mm, so this condition is met.

VI.1.3.6 <u>Local Ductility</u>

- The length of critical regions l_{cr}: According to 5.4.3.2.2 (4) – EC8 [3], l_{cr} may be computed as following expression: $l_{cr} = \max\{h_c ; l_{cl} / 6; 0{,}45\}$ = max {0.4, 3/6;0.45} = 0.5m
- According to 5.4.3.2.2 (6P and 7P) – EC8 [3], in the critical region at the base of primary seismic columns a value of the curvature ductility factor, μ_ϕ, should be provided, and if for the specified value of μ_ϕ a concrete strain larger than ε_{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. These conditions are met if the following conditions are satisfied: $\alpha * \omega_{wd} \geq 30 * \mu_\phi * \nu_d * \varepsilon_{sy,d} * \frac{b_c}{b_0} - 0.035$

Where:

- ω_{wd} - is the mechanical volumetric ratio of confining hoops within the critical regions. So, ω_{wd} is calculated as follows:

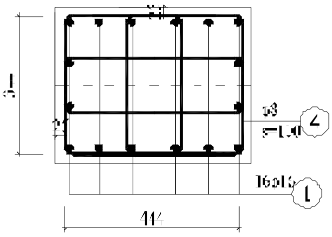
$$\omega_{wd} = \frac{volume\ of\ confining\ hoops}{volume\ of\ concrete\ core} * \frac{f_{yd}}{f_{cd}} = \frac{201*444}{100*444*344} * \frac{434.8}{16.7} = 0.15$$

- μ_ϕ - is the required value of the curvature ductility factor, μ_ϕ = 6.8
- $\nu_d = \frac{N}{b*h*f_{cd}} = \frac{1065.4*10^3}{400*500*16.7} = 0.319$
- $\varepsilon_{sy,d} = \frac{f_{yd}}{E} = \frac{434.8}{200000} = 0.00217$; b_c = 400;b_0 = 344
- α - is the confinement effectiveness factor, equal to $\alpha = \alpha_n * \alpha_s$, with:

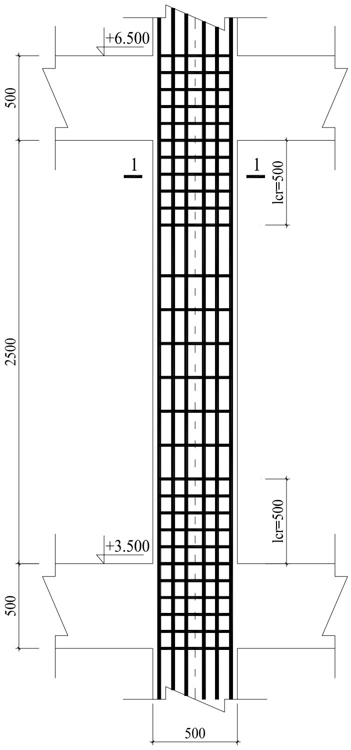
$$\alpha_n = 1 - \frac{\sum b_i^2}{6*b_0*h_0} = 1 - \frac{3*148^2}{6*344*444} = 0.93 \, ; \alpha_s = \frac{1-\frac{s}{2*b_0}}{1-\frac{s}{2*h_0}} = \frac{1-\frac{100}{2*344}}{1-\frac{100}{2*444}} = 0.96$$

$$\alpha = 0.9 \rightarrow 0.9*0.15 \geq 30*6.8*0.319*0.00217*\frac{400}{344} - 0.035 = 0.129$$

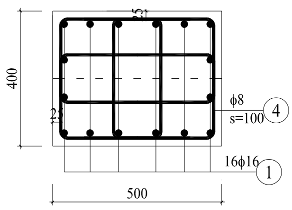
Concrete Core



DETAILING FOR COLUMN



SECTION 1-1



VI CHECK FOR RESISTANCE BETWEEN COLUMNS AND BEAMS

- According to 4.4.2.3 (3) – EC8 [3], in multi-storey buildings formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey.
- According to 4.4.2.3(4) – EC8 [3], the following condition should be satisfied at all joints of primary seismic beams and primary seismic columns:

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

Where:

  - ΣM_{Rc} – is the sum of the design values of the moment resistance of the columns framing to the joints.
  - ΣM_{Rb} – is the sum of the design values of the moment resistance of the beams framing to the joints

- Transverse Frames: It is just necessary to check for the first floor because the reinforcing areas of columns are not changed over their length and the reinforcing areas of the first floor beams are greater than the other beams.
  - ΣM_{Rc} = 366*2=732KNm
  - 1.3* ΣM_{Rb} = 1.3* (M_{BRd1} + M_{BRd2})= 1.3*(208 +119) = 425.1KNm

  $\rightarrow \Sigma M_{Rc}$ = 366*2=732KNm >1.3* ΣM_{Rb} = 422.5 KNm . The condition is met.

- Longitudinal Frames: It is just necessary to check for the first floor because the reinforcing areas of columns are not changed over their length and the reinforcing areas of the first floor beams are greater than the other beams.
  - ΣM_{Rc} = 283*2=566 KNm (from V.1.3.3)
  - 1.3* ΣM_{Rb} = 1.3* (M_{BRd1} + MB_{Rd2})= 1.3*(174 +89) = 341.9KNm

  → ΣM_{Rc} = 566KNm >1.3* ΣM_{Rb} = 341.9 KNm . The condition is met.

VII PROVISION CHECK FOR COLUMN-BEAM JOINTS

VII.1 Beam-Column Joint Ductility:

- The Beam-Column joints in the frames play a very important role during an earthquake. Under the horizontal force caused by earthquake the joints are immediately subjected to opposite moments from above and below columns and to similar moments from left and right beams. The internal forces acting on the joints are normally the shear forces. The magnitude of these internal shear forces is usually much greater than those of adjacent elements framing to the joints.
- Normally, the joints should consider as a part of the column and preferably respond within elastic range [4]. They also must have very high ductility to absorb the energy caused by the severe earthquake.
- The chosen ductility of beam-column joints is DCH.

VII.2 Design action effects:

- Action effects will be determined for the interior joint – intersection of column members 122-123 and beam members 63-79 in the transverse frame and intersection of column members 98-99 and beam members 33-49 in the longitudinal frame. It is the interior beam-column joint.
- In accordance to 5.5.2.3 – EC8 [2], 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.
- According to 5.5.2.3 – EC8 [2], simplified expression for the horizontal shear force acting on the concrete core of the joints may be used as follows:

$$V_{jhd} = \gamma_{Rd} * \left(A_{s1} + A_{s2}\right) * f_{yd} - V_c$$

  Where:
  - 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 about the joint, from the analysis in seismic design situation
  - γ_{Rd} – is a factor to account for over-strength due to steel strain hardening $\gamma_{Rd} \geq 1.2$.
- For the transverse frame:
  - $A_{s1} = 11.9cm^2$;
  - $A_{s2} = 6.2cm^2$;
  - V_C = 96.6KN
  - γ_{Rd} = 1.2

  So: $V_{jhd} = 1.2 * \left(1190 + 620\right) * 434.8 - 96600 = 847785.6N$ = 848 kN
- For the longitudinal frame:
  - $A_{s1} = 10.2cm^2$;
  - $A_{s2} = 4.62cm^2$;
  - V_C = 86.365KN
  - γ_{Rd} = 1.2

  So: $V_{jhd} = 1.2 * \left(1020 + 462\right) * 434.8 - 86365 = 558008.6N$ = 558 kN

→ The values of shear force taking from the transverse frame will be taken into account for maximum values to design the joints.

VII.3 Design resistance evaluation and verification

- Effective joint width: According to 5.5.3.3 – EC8 [3], effective joint width b_j is as follows:
  - If $b_c > b_w$: $b_j = \min\{b_c; (b_w + 0.5h_c)\}$ (5.34a)
  - If $b_c < b_w$: $b_j = \min\{b_w; (b_c + 0.5h_c)\}$ (5.34b)
- In the transverse frame, the value of column width is of b_c = 400mm and the value of beam width is of b_w = 250mm. So, we have the value of effective joint width is of $b_j = \min\{b_c; (b_w + 0.5h_c)\}$, so: b_j = min (400; 250+0.5*500) = 400mm.
- In accordance with 5.5.3.3 (1P) – EC8 [3], the diagonal compression induced in the joint by the diagonal strut mechanism shall not exceed the compressive strength of concrete in the presence of transverse tensile strains.
- In accordance with 5.5.3.3 (2) – EC8 [3], if there is no more precise model to compute the requirements of 5.5.3.3 (1P) may be satisfied by means of following rules: At interior beam-column joints the following expression should be satisfied:
  - $V_{jhd} \le \eta * f_{cd} * \sqrt{1 - \frac{\nu_d}{\eta}} * b_j * h_{jd}$

    Where:

    - $\eta = 0.6 * \left(1 - \frac{f_{ck}}{250}\right) So, \eta = 0.6 * \left(1 - \frac{25}{250}\right) = 0.54$
    - h_{jc} – is the distance between extreme layers of column reinforcement, so h_{jc} = 500-40-16=444mm.
    - b_j = 400.
    - υ_d – is the normalised axial force in the column about the joint. So, we have $\nu_d = \frac{N}{b * h * f_{cd}} = \frac{1065.4 * 10^3}{400 * 500 * 16.7} = 0.319$
    - f_{ck} = 25MPa

    $$V_{jhd} \le \eta * f_{cd} * \sqrt{1 - \frac{\nu_d}{\eta}} * b_j * h_{jd}$$

  - So, $\leftrightarrow 847785 \le 0.54 * 16.7 * \sqrt{1 - \frac{0.319}{0.54}} * 400 * 444$ → the condition is met.

    $\leftrightarrow 847785 \le 1024595$

VII.4 Confinement Mechanism

- According to 5.5.3.3 (3) – EC8 [3], adequate confinement (both horizontal and vertical) of the joint should be provided, to limit the maximum diagonal tensile stress of concrete max σ_{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:

  $$\frac{A_{sh} * f_{ywd}}{b_j * h_{jw}} \ge \frac{\left(\frac{V_{jhd}}{b_j * h_{jc}}\right)^2}{f_{ctd} + \nu_d * f_{cd}} - f_{ctd}$$

  Where:

  A_{sh} is the total area of the horizontal hoops.

  A_{sh} = 4 legs (Φ8mm)*50.24mm^2*(500/100)=1004.8mm^2.

f_{ywd} = 434.8MPa; b_j = 400mm, h_{jw} = 500-40-7-10=443mm;h_{jc} =444mm;
f_{ctd} = 2.2MPa; υ_d = 0.319

$$\rightarrow \frac{1004.8*434.8}{400*443} \geq \frac{\left(\frac{847785}{400*444}\right)^2}{2.2+0.319*16.7} - 2.2$$. So the condition is satisfied.

$$2.467 \geq 0.827$$

- According to 5.5.3.3 (6) – EC8 [3], Adequate vertical reinforcement of the column passing through the joint should be provided, so that:

  $$A_{sv,i} \geq \frac{2}{3} * A_{sh} * \frac{h_{jc}}{h_{jw}}$$

  Where:
  - $A_{sv,i}$ denotes the total area of the intermediate bars placed in the relevant column faces between corner bars of the column (including bars contributing to the longitudinal reinforcement of columns). So $A_{sv,i}$ = 12ϕ16 = 2412mm^2.
  - A_{sh} = 1004.8mm^2.
  - h_{jc} = 444; h_{jw} = 443

  So, we have: $A_{sv,i} \geq \frac{2}{3} * A_{sh} * \frac{h_{jc}}{h_{jw}} \leftrightarrow 2412 \geq \frac{2}{3} * 1004.8 * \frac{444}{443} = 671 \rightarrow$ the condition is met.

Alternatively, Eurocode 8 indicates another design check about the transverse reinforcements in the beam-column node: $A_{sh}f_{ywd} \geq \gamma_{Rd}(A_{s1}+A_{s2})f_{yd}(1-0.8\,\nu_d)$

A_{sh} = 1004.8mm^2 A_{s1} = 1190 mm^2 A_{s2} = 620 mm^2 γ_{Rd} = 1.2 ν_d=0.319
1005 x 434 >? 1,2 (1190 + 620)434(1-0.8 x 0.319)
436 kN >? 702 kN => NO =>change diameter of rebars.
Diameter 10 provides 680 ≈ 702 kN Diameter 12 provides 979 kN>702 kN

VII.5 Specific Measures

In accordance with 5.5.3.3 – EC8 [3], the specific measures for detailing local ductility will be checked as follows:

- The diameter d_{bw} of the hoops is not less than 6mm. The real diameter of the hoops is minimum 8mm, so this condition is satisfied.
- If framing beams are present on all four faces of the column, the spacing s of hoops may be increased to: s = min (h_c/2 ; 150mm). The spacing s of the hoops is of 100mm, so this condition is met.
- According to 5.4.3.3 (2) – EC8 [3], If beams frame into all four sides of the joint and their width is at least three-quarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice, but may not exceed 150 mm. So this condition is satisfactory.
- According to 5.4.3.3 (2) – EC8 [3], at least one intermediate (between column corner bars) vertical bar shall be provided at each side of a joint of primary seismic beams and columns.

→ All the conditions are met.

PART 2: PUSHOVER ANALYSIS OF THE DESIGNED BUILDING.

The behavior of the building under un-given seismic actions in the design example which is presented in the first part can be checked by non linear analysis methods. One is the Nonlinear Static Analysis or Pushover method. Nonlinear static Analysis (Pushover) is presented in 4.3.3.4.2 of EC8[3] Part 1. According to EC8 [3], pushover analysis may be used to assess the structural behavior of existing or of newly designed buildings. Pushover is based on analyzing the structure under constant gravity loads and monotonically increasing horizontal loads. The purposes of Pushover Analysis are:

- To verify or revise the over-strength ratio values $\alpha u/ \alpha_1$;
- To estimate the expected plastic mechanisms and the distribution of damage;
- To assess the structural performance of existing or retrofitted buildings for the purposes of EN 1998-3;
- As an alternative to the design based on linear-elastic analysis which uses the behaviour factor *q*.

Under constant gravity loads and monotonically increasing horizontal loads presenting for seismic excitations (the base shear forces at the bottom of the frames), the frames could be considered as working under an un-given earthquake. After obtaining behavior of the frames from the relationship between displacements of the control node which is usually the roof displacement and the base shear forces which are calculated by monotonically increasing horizontal loads, one could estimate the largest magnitude of the earthquake that the designed frames can suffer. The horizontal load patterns used in Pushover are Modal Load Pattern and Uniform Load Pattern. Two types of these patterns are presented in 4.3.3.4.2 of EC8[3] Part 1.

In order to perform Pushover analysis, there are some properties of the frames will be determined such as: Plastic hinge properties of each element in the frames including plastic moments of critical sections taking into account the strain of concrete and reinforcements and rotation capacity of the hinges. To determine such properties, it is necessary to use experimental data (for example the rotation angles of the hinges corresponding to reinforcement ratio and stirrup distance) which are presented in some public documents such as FEMA356…

The computer program which is used to perform Pushover Analysis is SAP2000, version 9.0.3.

The data needed to carry out Pushover are:

- Plastic moments of critical sections in the frames
- Load Patterns
- Plastic rotation properties of each hinge.

I Plastic moment Determination for User Defined Hinges in SAP 2000

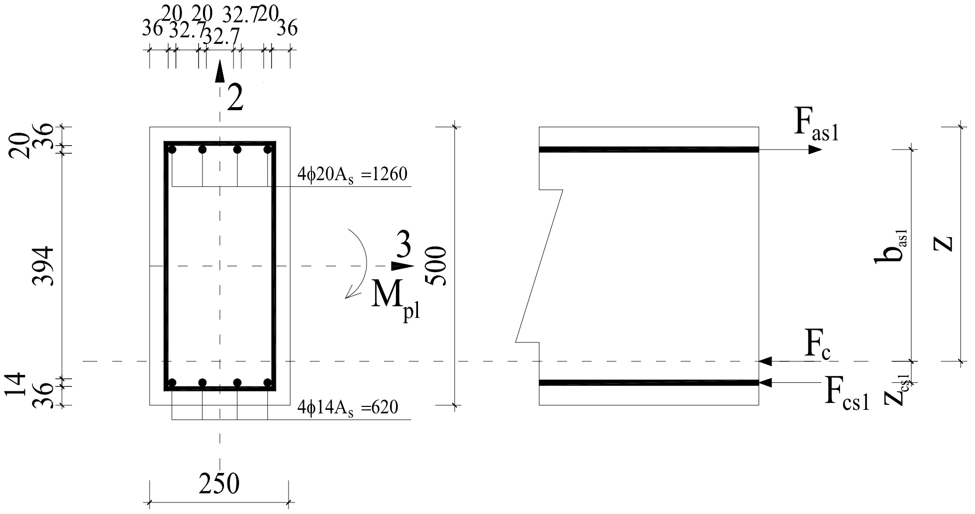
I.1 Transverse Frames

I.1.1 Beams

I.1.1.1 Beams of the first and the second floors ((part 1 – III and IV):

- Section at the joint between external columns and beams
  - Negative Moments:

PLASTIC MOMENT CALCULATION



$F_{as1} = A_s * f_{yd} = 1260 * 434.8 = 547826.1N$; $F_{cs1} = A_s^{'} * f_{yd} = 620 * 434.8 = 269565.2$

$$F_c = \frac{\alpha_c * f_{ck}}{\gamma_c} * b * (h - z) = 4175 * (500 - z)$$

$$\rightarrow F_{as1} = F_{cs1} + F_c \leftrightarrow 547826 = 269565.2 + 4175(500 - z) \rightarrow 500 - z = 66.78 \rightarrow z = 433.22m$$

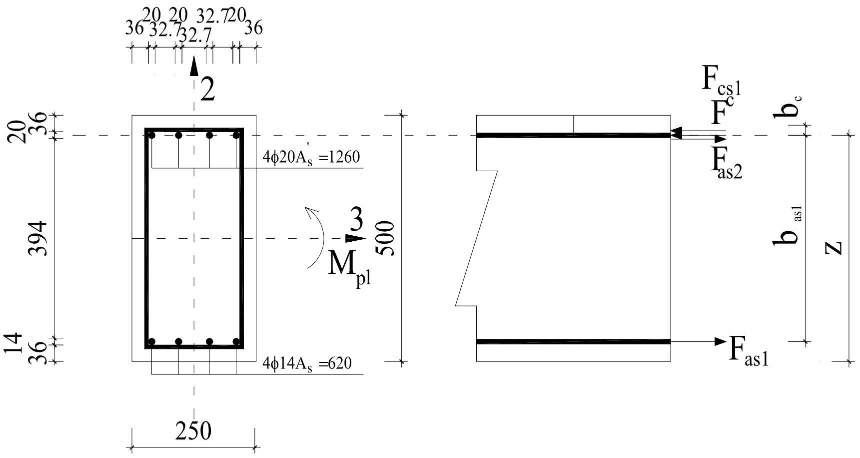
The condition of z is that: (36+14) < 500-z→ 50<105. So this condition is met.
Plastic moment will be calculated hereafter:

$$M_{pl} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c = 547826 * 387.22 + 269565 * 23.78 + 4175 * 66.78 * 33.39$$
$$= 233010586Nmm = 233KNm$$

- Positive Moments:

$$F_{as1} = A_s * f_{yd} = 620 * 434.8 = 269576N \; ; F_{as2} = A_s^{'} * f_{yd} * \frac{z - 444}{20} = 547848 * \frac{z - 444}{20}$$

PLASTIC MOMENT CALCULATION



$$F_{cs1} = A_s^{'} * f_{yd} * \left(1 - \frac{z - 444}{20}\right) = 547848 * (1 - \frac{z - 444}{20}); \; F_c = \frac{\alpha_c * f_{ck}}{\gamma_c} * b * (h - z) = 4175 * (500 - z)$$

$$\rightarrow F_{as1} + F_{as2} = F_{cs1} + F_c \leftrightarrow 269576 + 547848 * \frac{z - 444}{20} = 547848 * \left(1 - \frac{z - 444}{20}\right) + 4175(500 - z)$$

$$\rightarrow z = 452.7mm$$

The condition of z is: 444 < z < 444+20=464 →This condition is met.
Plastic moment will be calculated hereafter:

$$b_{as1} = z - a_{s1} = 452.7 - (30 + 6 + 7) = 409.7mm; b_{as2} = z - a_{s2} = 452.7 - 444 = 8.7mm$$

$$b_{cs1} = 20 - b_{as2} = 11.3mm; b_c = \frac{500 - z}{2} = 23.7mm$$

$$M_{pl} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c =$$
$$= 269576 * 409.7 + 547848 * \frac{452.7 - 444}{20} * 8.7 + 547848 * \left(1 - \frac{452.7 - 444}{20}\right) * 11.3 + 4175 * 47.4 * 23.7$$
$$= 120706465Nmm = 121KNm$$

- Section at the joint between internal columns and beams: In order to calculate the plastic moment of the sections conveniently, we create the excel sheets or small calculation tables using MathCad program. So, we can obtain plastic moment of the other sections as following:

Beams of Floor	Position of column	Sections of the beams	Top reinforcement area (mm^2)	Bottom reinforcement area(mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
1-2	Internal	End	1190	620	223	120.4
1-2	Internal	Middle	628	620	118	118

I.1.1.2 Beams of the other floors (part 1 – III and IV):

Plastic moment at any section of all remaining beams of the frame will be determined similarly as above and they are summarized briefly as following table:

- Negative Plastic Moment of the section at the external joint of the beam on the third and the fourth floors:

Section at the external joint - Negative Plastic Moment Calculation - Unit Nmm

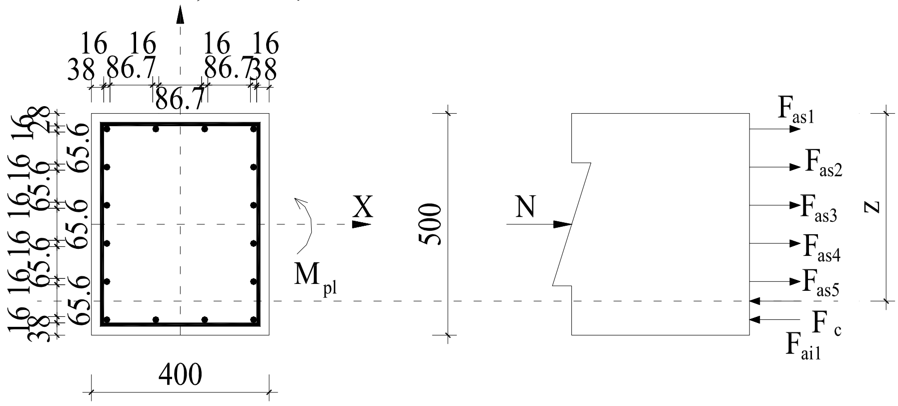
Beam width - b_w	Beam height-h_w	Column width - b_c	Column height- h_c	ϕ_{link} - hoop diameter	A_{s1}	A_{s2}	A_{s3}	A'_{s1}	A'_{s2}	d – effective depth
250	500	400	500	6	762	402		620		438.768
			f_{cd}	**f_{yd}**	ϕ_{As1}	ϕ_{As2}	ϕ_{As3}	$\phi_{A's1}$	$\phi_{A's2}$	
			16.666667	434.7826	18	16		14		
$c_{min,b}$	$c_{min,dur}$	c_{min}	Δc_{dev}	c_{nom}	a_{s1}	a_{s2}	a_{s3}	a'_{s1}	a'_{s2}	
20	15	20	10	30	45	92		43		
					F_{as1}	F_{as2}	F_{as3}	F_{cs1}	F_{cs2}	Fc
					331304.3	174782.6	0	269565.2	0	236521.7
→	z			b'_c	b_{as1}	b_{as2}	b_{as3}	b_{cs1}	b_{cs2}	
	443.2348			28.38261	398.2348	351.2348		13.76522		
		→	M_{pl}							
			203750374							

- Plastic Moment Table of all sections of the remaining beams

Beams of Floor	Position of column	Sections of the beams	Top reinforcement area (mm^2)	Bottom reinforcement area(mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
3-4	Internal	End	1071.5	620	189.6	124.4
3-4	Internal	Middle	508	620	96.7	117.3
5	External	End	804	462	149.6	89.8
5	Internal	End	804	462	149.6	89.8
5	Internal	Middle	402	462	77.4	88.4
6	External	End	462	462	88	88
6	Internal	End	462	462	88	88
6	Internal	Middle	462	462	88	88

I.1.2 Columns

Properties taken from Part 1:$A_{s,tot}$ = 16ϕ16 = 3216mm^2; N = 1295000N



PLASTIC MOMENT CALCULATION

- We assume that 12ϕ16 are in the tensile area, 4ϕ16 are in the compression area and z is the distance taking from compressed fiber to the neutral axis

$$F_{as} = \sum F_{asi} = \sum A_{si} * f_{yd} = 12*201*434.8 = 1048737.6$$

$$F_{ai} = \sum F_{aii} = \sum A_{si} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_c = f_{cd} * b * (h - z) = 16.7*400*(500 - z) = 6680*(500 - z)$$

$$\rightarrow F_{as} + N = F_{ai} + F_c; \leftrightarrow 1048737.6 + 1295000 = 349579.2 + 6680(500 - z)$$

$$\rightarrow 500 - z = 298.5 \rightarrow z = 201.5mm;$$

- The condition of z is that: (38+16)<500-z<(38+16+69.6)→ 54<298.5<123, so the value of z is not correct.
- We assume that the neutral axis will run through the right – third layer reinforcement; so there are over 6ϕ16 in compression zone and over 8 ϕ16 in tension zone.

$$F_{as1} = A_{s1} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_{as2} = F_{as3} = A_{s2} * f_{yd} = 2*201*434.8 = 219789.6$$

$$F_{as4} = A_{s4} * f_{yd} * \frac{z - 282.8}{16}$$

$$F_{ai1} = A_{cs1} * f_{yd} = 4*201*434.8 = 349579.2$$

$$F_{ai2} = A_{cs2} * f_{yd} = 2*201*434.8 = 219789.6$$

$$F_{ai3} = A_{s4} * f_{yd} * (1 - \frac{z - 282.8}{16})$$

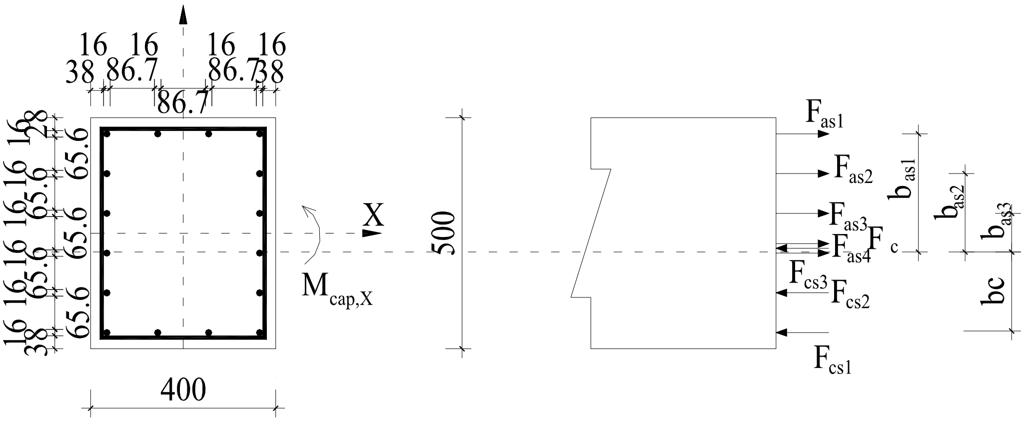
$$F_c = f_{cd} * b * (h - z) = 16.7*400*(500 - z) = 6680*(500 - z)$$

$$\rightarrow \sum F_{si} + N = \sum F_{ai} + F_c$$

$$\rightarrow z = 287.4mm$$

The condition of z is that 282.8 <=z<=282.8+16=298.8, so this condition is met

- Plastic moment will be calculated as following:



PLASTIC MOMENT CACULATION

$b_{as1} = z - a_{s1} = 287.4 - (38 + 8) = 241.4mm; b_{as2} = z - a_{s2} = 287.4 - (38 + 16 + 65.6 + 8) = 159.8mm$

$b_{as3} = z - a_{s3} = 287.4 - (38 + 16 * 2 + 65.6 * 2 + 8) = 78.2mm; b_{as4} = 287.4 - 282.8 = 4.6mm$

$b_{cs1} = h - z - a_{cs1} = 500 - 287.4 - (38 + 8) = 166.6mm$

$b_{cs2} = h - z - a_{cs2} = 500 - 287.4 - (38 + 16 + 65.6 + 8) = 85mm; b_{cs3} = 16 - b_{as4} = 16 - 4.6 = 11.4mm;$

$b_c = \frac{500 - z}{2} = 106.3mm;$

$F_c = 16.7 * (500 - 287.4) * 400 = 1420168N$

$M_{x,cap} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c =$

$= 349579.2 * (241.4 + 166.6) + 219789.6 * (159.8 + 78.2 + 85) + 219789.6 * \frac{4.6}{16} * 4.6$

$+ 219789.6 * \frac{11.4}{16} * 11.4 + 1420168 * 106.3 = 366660126Nmm = 367KNm$

Similarly, we can obtain the plastic moment of other section of the remaining column as following table:

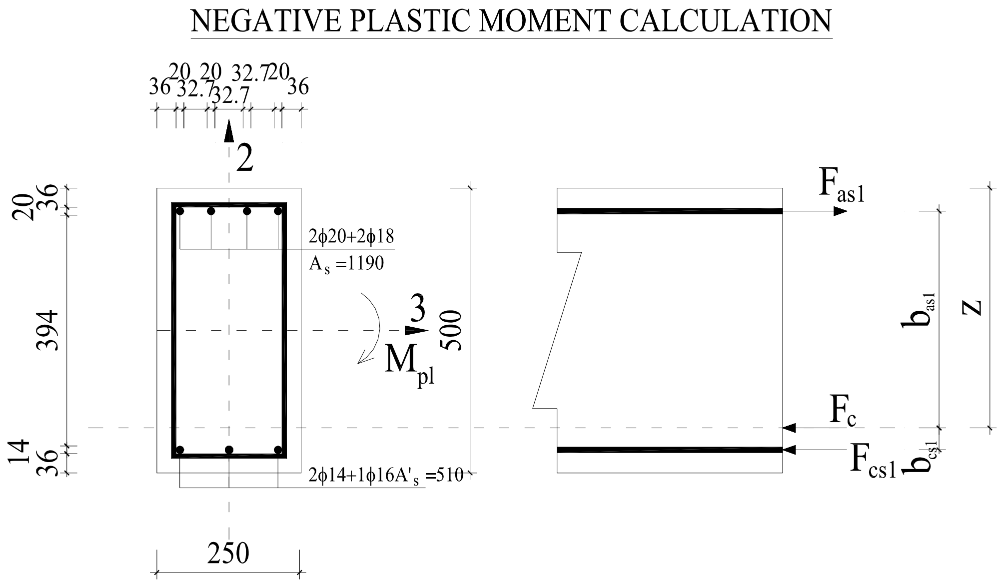
Floor	Width of column (mm)	Height of column (mm)	Reinforcement area (mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
3-4	400	500	$16\phi14 - 2464$	265	265
5-6	400	500	$16\phi14 - 2464$	265	265

I.2 Longitudinal Frames

I.2.1 Beams

I.2.1.1 Beams of the first and the second floor

- Section at the joint between external columns and beams
  - Negative Moments:



$F_{as1} = A_s * f_{yd} = 1190 * 434.8 = 517393.3N;$

$F_{cs1} = A_s^{'} * f_{yd} = 510 * 434.8 = 221739.1N$

$F_c = \dfrac{\alpha_c * f_{ck}}{\gamma_c} * b * (h - z) = 4175 * (500 - z)$

$\rightarrow F_{as1} = F_{cs1} + F_c \leftrightarrow 517393.3 = 221739.1 + 4175(500 - z)$

$\rightarrow z = 429.04m$

The condition of z is that: (36+14) < 500-z → 50<71. So this condition is met.
Plastic moment will be calculated hereafter:

$M_{pl} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c = 215KNm$

- Positive Moments:

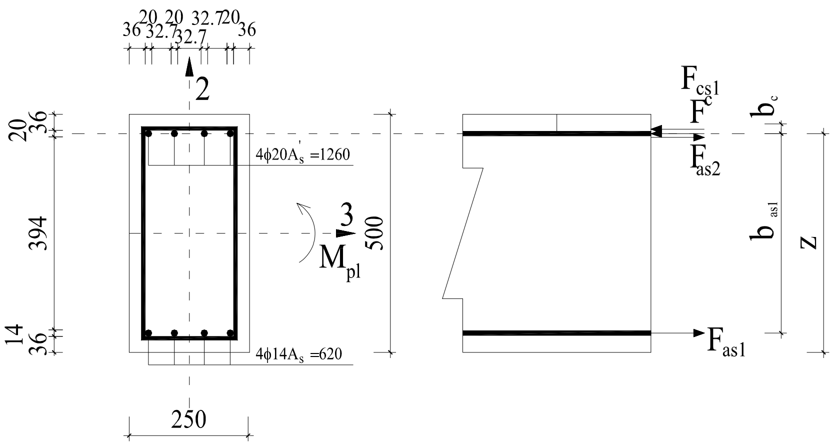
$F_{as1} = A_s * f_{yd} = 510 * 434.8 = 221739.1N$

$F_{as2} = A_s^{'} * f_{yd} * \dfrac{z - 444}{20} = 517393.3 * \dfrac{z - 444}{20}$

$F_{cs1} = A_s^{'} * f_{yd} * \left(1 - \dfrac{z - 444}{20}\right) = 517393.3 * (1 - \dfrac{z - 444}{20})$

$F_c = \dfrac{\alpha_c * f_{ck}}{\gamma_c} * b * (h - z) = 4175 * (500 - z)$

PLASTIC MOMENT CALCULATION



$$\rightarrow F_{as1} + F_{as2} = F_{cs1} + F_c \leftrightarrow 221739.1 + 517393.3 * \frac{z-444}{20} = 517393.3 * \left(1 - \frac{z-444}{20}\right) + 4175(500-z)$$

$$\rightarrow z = 453.5mm$$

The condition of z is: 444 < z < 444+20=464 → This condition is met.

Plastic moment will be calculated hereafter:

$$b_{as1} = z - a_{s1} = 453.5 - (30+6+7) = 410.5mm$$

$$b_{as2} = z - a_{s2} = 453.5 - 444 = 9.5mm$$

$$b_{cs1} = 20 - b_{as2} = 10.5mm$$

$$b_c = \frac{500-z}{2} = 23.25mm$$

$$M_{pl} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c =$$

$$= 221739.1 * 410.5 + 517393.3 * \frac{9.5}{20} * 9.5 + 547848 * \left(\frac{10.5}{20}\right) * 10.5 + 4175 * 47.4 * 23.25$$

$$= 100716426.2 Nmm = 101 KNm$$

- Section at the joint between internal columns and beams:

In order to calculate the plastic moment of the sections conveniently, we create the excel sheets or small calculation tables using MathCad program. So, we can obtain plastic moment of the other sections as following:

Beams of Floor	Position of column	Sections of the beams	Top reinforcement area (mm^2)	Bottom reinforcement area(mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
1-2	Internal	End	1020	462	186.1	91
1-2	Internal	Middle	508	308	95.8	61.4

I.2.1.2 Beams of the other floors:

Plastic moment at any section of all remaining beams of the frame will be determined similarly as above and they are summarized briefly as following table:

- Negative Plastic Moment of the section at the external joint of the beam on the third and the fourth floors:

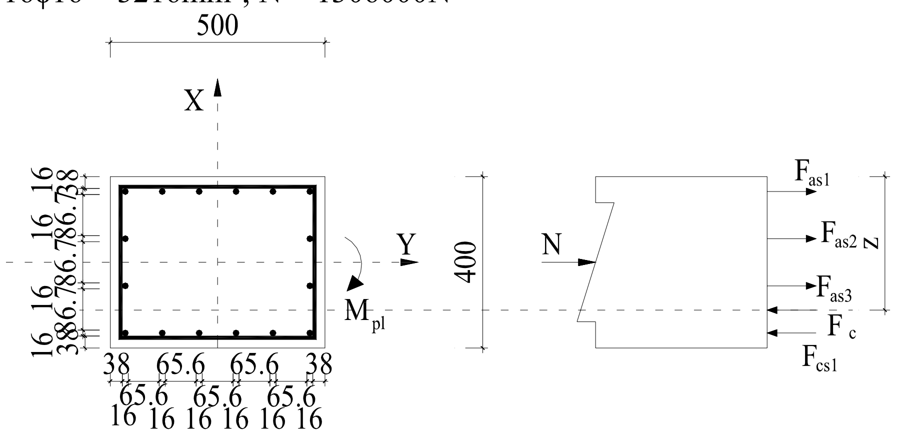
Beam width - b_w	Beam height- h_w	Column width - b_c	Column height- h_c	ϕ_{link} - hoop diameter	A_{s1}	A_{s2}	A_{s3}	A'_{s1}	A'_{s2}	d - effective depth
250	500	500	400	6	1020	0		462	0	455
			f_{cd}	**f_{yd}**	ϕ_{As1}	ϕ_{As2}	ϕ_{As3}	$\phi_{A's1}$	$\phi_{A's2}$	
			16.666667	434.7826	18	0		14	0	
$c_{min,b}$	$c_{min,dur}$	c_{min}	Δc_{dev}	c_{nom}	a_{s1}	a_{s2}	a_{s3}	a'_{s1}	a'_{s2}	
20	15	20	10	30	45	84		43	80	
→	z	→	M_{pl}		F_{as1}	F_{as2}	F_{as3}	F_{cs1}	F_{cs2}	Fc
	441.7739		**186082140**		443478.3	0	0	200869.6	0	242608.7
	Z IS TRUE			b'_c	b_{as1}	b_{as2}	b_{as3}	b_{cs1}	b_{cs2}	
				29.11304	396.7739	357.7739		15.22609		

- Plastic Moment Table of all sections of the remaining beams

Beams of Floor	Position of column	Sections of the beams	Top reinforcement area (mm^2)	Bottom reinforcement area(mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
3-4	External	End	1020	462	186.1	91.0
3-4	Internal	End	816	462	151.4	90.2
3-4	Internal	Middle	508	462	96.3	89.0
5	External	End	620	308	116.4	61.2
5	Internal	End	620	308	116.4	61.2
5	Internal	Middle	620	308	116.4	61.2
6	External	End	462	308	88	61
6	Internal	End	462	308	88	61
6	Internal	Middle	462	308	88	61

I.2.2 Columns

- $A_{s,tot} = 16\phi16 = 3216mm^2$; N = 1306000N



PLASTIC MOMENT CACULATION

- We assume that 10ϕ16 are in the tensile area, 6ϕ16 are in the compression area and z is the distance taking from compressed fiber to the neutral axis

$$F_{as} = \sum F_{asi} = \sum A_{si} * f_{yd} = 10*201*434.8 = 873948.0; F_{ai} = \sum F_{aii} = \sum A_{si} * f_{yd} = 6*201*434.8 = 524368.8$$

$$F_c = f_{cd} * b * (h - z) = 16.7*500*(400 - z) = 8350*(400 - z);$$

$$\rightarrow F_{as} + N = F_{ai} + F_c \leftrightarrow 873948.0 + 1306000 = 524368.8 + 8350(400 - z); \rightarrow 400 - z = 198.3 \rightarrow z = 201.7mm$$

- The condition of z is that: (38+16)<400-z<(38+16+86.7)→ 54<198.3<140, so the value of z is not correct.
- We assume that the neutral axis will run through the first layer reinforcement; so there are less than 8ϕ16 in compression zone and over 8 ϕ16 in tension zone.

$$F_{as1} = A_{s1} * f_{yd} = 6*201*434.8 = 524368.8$$

$$F_{as2} = A_{s2} * f_{yd} = 2*201*434.8 = 174789.6$$

$$F_{as3} = A_{s3} * f_{yd} * \frac{z - 243.4}{16} = 2*201*434.8*\frac{z - 243.4}{16} = 174789.6*\frac{z - 243.4}{16}$$

$$F_{cs1} = 6*201*434.8 = 524368.8N$$

$$F_{cs2} = A_{as3} * f_{yd} * (1 - \frac{z - 243.4}{16}) = 174789.6*(1 - \frac{z - 243.4}{16})$$

$$F_c = f_{cd} * b * (h - z) = 16.7*500*(400 - z) = 8350*(400 - z)$$

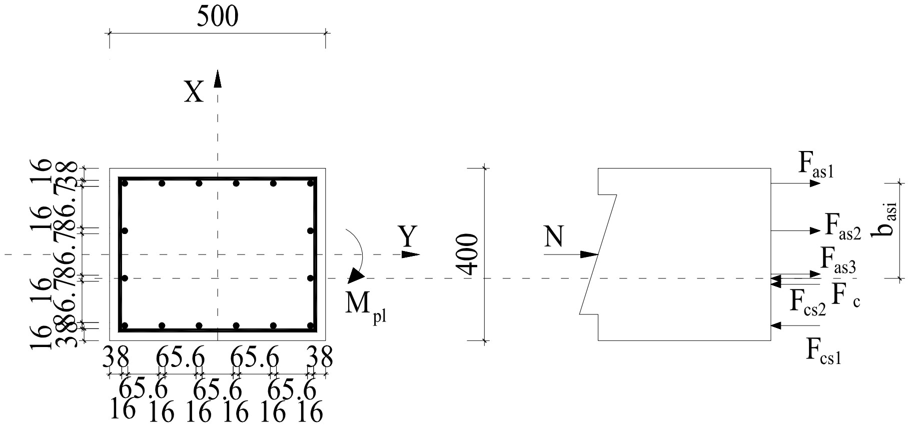
$$\rightarrow \sum F_{si} + N = \sum F_{ai} + F_c$$

$$\leftrightarrow 174789.6 + 174789.6*\frac{z - 243.4}{16} + 1306000 = 174789.6*(1 - \frac{z - 243.4}{16}) + 8350*(400 - z)$$

$$\rightarrow z = 243.5mm$$

The condition of z is that 243.4 <z<=243.4+16, so this condition is met

- Plastic moment will be calculated as following:



PLASTIC MOMENT CACULATION

$$b_{as1} = z - a_{s1} = 243.5 - (38 + 8) = 197.5mm; b_{as2} = z - a_{s2} = 243.5 - (38 + 16 + 86.7 + 8) = 94.8mm$$

$$b_{as3} = z - a_{s3} = 243.5 - 243.4 = 0.1mm; b_{cs1} = h - z - a_{cs1} = 400 - 243.5 - (38 + 8) = 110.5mm$$

$$b_{cs2} = 16 - b_{as3} = 16 - 0.1 = 15.9mm; b_c = \frac{400 - z}{2} = 78.25mm$$

$$F_c = 16.7*(400 - 243.5)*500 = 1306775N$$

$$M_{x,cap} = \sum F_{asi} * b_{asi} + \sum F_{csi} * b_{csi} + F_c * b_c = 524368.8*(197.5 + 110.5) + 174789.6*94.8 +$$

$$174789.6*\frac{0.1}{16}*0.1 + 174789.6*\frac{15.9}{16}*15.9 + 1306775*78.25 = 283092682Nmm = 283KNm$$

Similarly, we can obtain the plastic moment of other section of the remaining column as following table:

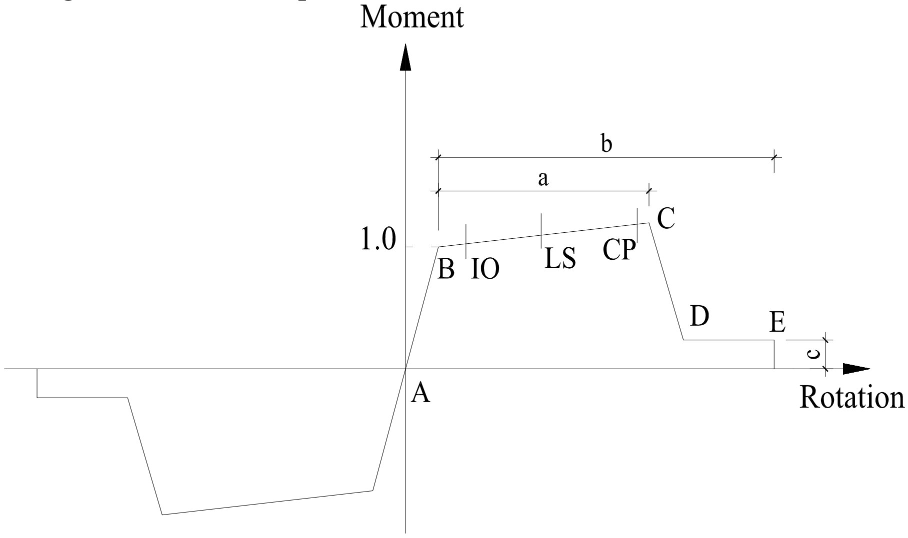
Floor	Width of column (mm)	Height of column (mm)	Reinforcement area (mm^2)	Negative Plastic Moment (kNm)	Positive Plastic Moment (kNm)
4	500	400	16ϕ14 – 2464	217.2	217.2
5-6	500	400	16ϕ14 – 2464	217.2	217.2

II Modeling Parameters and Acceptance Criteria for Nonlinear Static Procedures (for User Defined Hinges in SAP 2000)

II.1 Transverse Frames

II.1.1 Beams

The modeling parameters of frame elements used in Nonlinear Static analysis (Pushover) are all properties of the plastic hinges or plastic zones. These parameters can be determined from FEMA 356[7]. According to FEMA 356[7] part 6, the properties of sections which can be transferred to plastic hinges depend on the relationship between Force-Displacement or Moment – Rotation. The general relationship can be defined as follows:



IO - Immediate Occupancy; LS - Life Safety; CP - Collapse Prevention

Table 2.1 : Properties of the plastic hinge section

Beams of Floor	Span	Position of plastic hinges	ρ of Top reinforc area	ρ of bottom reinforce area	Balanced normalised reinforcement ρ_{bal}	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	$\frac{\rho_{bot}-\rho_{top}}{\rho_{bal}}$
1-2	1	Left	0.011	0.005	0.02107	0.285	<0
1-2	1	Middle	0.005	0.005	0.02107	0	0
1-2	1	Right	0.01	0.005	0.02107	0.237	<0
1-2	2	Left	0.01	0.005	0.02107	0.237	<0
1-2	2	Middle	0.005	0.005	0.02107	0	0
1-2	2	Right	0.01	0.005	0.02107	0.237	<0
1-2	3	Left	0.01	0.005	0.02107	0.237	<0
1-2	3	Middle	0.005	0.005	0.02107	0	0
1-2	3	Right	0.011	0.005	0.02107	0.285	<0
3-4	1	Left	0.008	0.005	0.02107	0.142	<0

3-4	1	Middle	0.0045	0.005	0.02107	<0	0.024
3-4	1	Right	0.007	0.005	0.02107	0.095	<0
3-4	2	Left	0.007	0.005	0.02107	0.095	<0
3-4	2	Middle	0.0045	0.005	0.02107	<0	0.024
3-4	2	Right	0.007	0.005	0.02107	0.095	<0
3-4	3	Left	0.007	0.005	0.02107	0.095	<0
3-4	3	Middle	0.0045	0.005	0.02107	<0	0.024
3-4	3	Right	0.008	0.005	0.02107	0.142	<0
5	1-2-3	Left	0.007	0.004	0.02107	0.142	<0
5	1-2-3	Middle	0.004	0.004	0.02107	0	0
5	1-2-3	Right	0.007	0.004	0.02107	0.142	<0
6	1-2-3	Left	0.004	0.004	0.02107	0	0
6	1-2-3	Middle	0.004	0.004	0.02107	0	0
6	1-2-3	Right	0.004	0.004	0.02107	0	0

Table 2.2 : Modeling Parameters for Negative Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	Transverse Reinforce.	$\frac{V}{b_w * d * \sqrt{f_c^{'}}}$	Modelling Parameters			
						Plastic Rotation Angle		Residual strength ratio	Normalised ratio
						a	b	c	b/a
1-2	1	Left	0.285	C.	<3	0.022	0.039	0.2	1.772727
1-2	1	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	1	Right	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	2	Left	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	2	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	2	Right	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	3	Left	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	3	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	3	Right	0.285	C.	<3	0.022	0.039	0.2	1.772727
3-4	1	Left	0.142	C.	<3	0.024	0.044	0.2	1.833333
3-4	1	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Right	0.095	C.	<3	0.024	0.046	0.2	1.916667
3-4	2	Left	0.095	C.	<3	0.024	0.046	0.2	1.916667
3-4	2	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Right	0.095	C.	<3	0.024	0.046	0.2	1.916667
3-4	3	Left	0.095	C.	<3	0.024	0.046	0.2	1.916667
3-4	3	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Right	0.142	C.	<3	0.024	0.044	0.2	1.833333
5	1-2-3	Left	0.142	C.	<3	0.024	0.044	0.2	1.833333
5	1-2-3	Middle	0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Right	0.142	C.	<3	0.024	0.044	0.2	1.833333
6	1-2-3	Left	0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Middle	0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Right	0	C.	<3	0.025	0.05	0.2	2

Table 2.3: Modeling Parameters for Positive Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{bot}-\rho_{top}}{\rho_{bal}}$	Transverse Reinforce.	$\frac{V}{b_w * d * \sqrt{f_c^{'}}}$	Modelling Parameters			
						Plastic Rotation Angle		Residual strength ratio	Normalised ratio
						a	b	c	b/a
1-2	1	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	1	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	1	Right	<0	C.	<3	0.025	0.05	0.2	2
1-2	2	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	2	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	2	Right	<0	C.	<3	0.025	0.05	0.2	2
1-2	3	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	3	Middle	0	C.	<3	0.025	0.05	0.2	2
1-2	3	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	1	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	2	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	3	Right	<0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Left	<0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Middle	0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Right	<0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Left	0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Middle	0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Right	0	C.	<3	0.025	0.05	0.2	2

Table 2.4: Acceptance Criteria for Negative Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	Tran. Rein.	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Acceptance Criteria					
						Plastic Rotation Angle					
						Performance Level					
						IO	Component Type		Normalised ratio		
							Primary		IO/a	LS/a	CP/a
							LS	CP			
1-2	1	Left	0.285	C.	<3	0.01	0.014	0.022	0.45	0.64	1
1-2	1	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	1	Right	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	2	Left	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	2	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Right	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	3	Left	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Right	0.285	C.	<3	0.01	0.014	0.022	0.45	0.64	1
3-4	1	Left	0.142	C.	<3	0.01	0.017	0.024	0.42	0.71	1

3-4	1	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Right	0.095	C.	<3	0.01	0.018	0.024	0.42	0.75	1
3-4	2	Left	0.095	C.	<3	0.01	0.018	0.024	0.42	0.75	1
3-4	2	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Right	0.095	C.	<3	0.01	0.018	0.024	0.42	0.75	1
3-4	3	Left	0.095	C.	<3	0.01	0.018	0.024	0.42	0.75	1
3-4	3	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Right	0.142	C.	<3	0.01	0.017	0.024	0.42	0.71	1
5	1-2-3	Left	0.142	C.	<3	0.01	0.017	0.024	0.42	0.71	1
5	1-2-3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Right	0.142	C.	<3	0.01	0.017	0.024	0.42	0.71	1
6	1-2-3	Left	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Right	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1

Table 2.5: Acceptance Criteria for Positive Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{bot}-\rho_{top}}{\rho_{bal}}$	Tran. Rein.	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Acceptance Criteria					
						Plastic Rotation Angle					
						Performance Level					
						IO	Component Type		Normalised ratio		
							Primary		IO/a	LS/a	CP/a
							LS	CP			
1-2	1	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	1	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	1	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Left	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Middle	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Right	0	C.	<3	0.01	0.02	0.025	0.4	0.8	1

II.1.2 Column

Table 2.6: Properties of the plastic hinge section

Column of floor	Position of plastic hinges	N – Axial force (N)	V – Shear Force (N)	Gross Area of Column A_g(mm2)	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$
1-2-3	Bottom	1295000	88000	200000	0.259	<3
1-2-3	Top	1276000	88000	200000	0.2552	<3
4-5-6	Bottom	624000	74000	200000	0.1248	<3
4-5-6	Top	616000	74000	200000	0.1232	<3

Table 2.7 : Modeling Parameters for Plastic Moment (According to FEMA 356 – Table 6.8 – Columns controlled by Flexure)

Column of floor	Position of plastic hinges	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Transverse Reinforce.	Modelling Parameters			
					Plastic Rotation Angle		Residual strength ratio	Normalised ratio
					a	b	c	b/a
1-2-3	Bottom	0.259	<3	C	0.017	0.027	0.2	1.59
1-2-3	Top	0.2552	<3	C	0.017	0.027	0.2	1.59
4-5-6	Bottom	0.1248	<3	C	0.02	0.03	0.2	1.5
4-5-6	Top	0.1232	<3	C	0.02	0.03	0.2	1.5

Table 2.7a: Acceptance Criteria for Plastic Moment (According to FEMA 356 – Table 6.8 – Columns controlled by Flexure)

Column of floor	Position of plastic hinges	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Tran. Rein.	Acceptance Criteria					
					Plastic Rotation Angle					
					Performance Level					
					IO	Component Type		Normalised ratio		
						Primary		IO/a	LS/a	CP/a
						LS	CP			
1-2 -3	Bottom	0.259	<3	C.	0.004	0.013	0.017	0.24	0.76	1
1-2-3	Top	0.2552	<3	C.	0.004	0.013	0.017	0.24	0.76	1
4-5-6	Bottom	0.1248	<3	C.	0.005	0.015	0.020	0.25	0.75	1
4-5-6	Top	0.1232	<3	C.	0.005	0.015	0.020	0.25	0.75	1

II.2

II.3 Longitudinal Frames

II.3.1 Beams

Table 2.8 : Properties of the plastic hinge section of longitudinal frame

Beams of Floor	Span	Position of plastic hinges	ρ of top reinforcement area	ρ of bottom reinforcement area	Balanced normalised reinforcement ρbal	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	$\frac{\rho_{bot}-\rho_{top}}{\rho_{bal}}$
1-2	1	Left	0.01	0.0045	0.02107	0.261	-0.261
1-2	1	Middle	0.0045	0.0027	0.02107	0.085	-0.085
1-2	1	Right	0.009	0.004	0.02107	0.237	-0.237
1-2	2	Left	0.009	0.004	0.02107	0.237	-0.237
1-2	2	Middle	0.0045	0.0027	0.02107	0.085	-0.085
1-2	2	Right	0.009	0.004	0.02107	0.237	-0.237
1-2	3	Left	0.009	0.004	0.02107	0.237	-0.237
1-2	3	Middle	0.0045	0.0027	0.02107	0.085	-0.085
1-2	3	Right	0.01	0.0045	0.02107	0.261	-0.261
3-4	1	Left	0.009	0.004	0.02107	0.237	-0.237
3-4	1	Middle	0.0045	0.004	0.02107	0.024	-0.024
3-4	1	Right	0.0072	0.004	0.02107	0.152	-0.152
3-4	2	Left	0.0072	0.004	0.02107	0.152	-0.152
3-4	2	Middle	0.0045	0.004	0.02107	0.024	-0.024
3-4	2	Right	0.0072	0.004	0.02107	0.152	-0.152
3-4	3	Left	0.0072	0.004	0.02107	0.152	-0.152
3-4	3	Middle	0.0045	0.004	0.02107	0.024	-0.024
3-4	3	Right	0.009	0.004	0.02107	0.237	-0.237
5	1-2-3	Left	0.005	0.0027	0.02107	0.109	-0.109
5	1-2-3	Middle	0.005	0.0027	0.02107	0.109	-0.109
5	1-2-3	Right	0.005	0.0027	0.02107	0.109	-0.109
6	1-2-3	Left	0.004	0.0027	0.02107	0.062	-0.062
6	1-2-3	Middle	0.004	0.0027	0.02107	0.062	-0.062
6	1-2-3	Right	0.004	0.0027	0.02107	0.062	-0.062

Table 2.9 : Modeling Parameters for Negative Plastic Moment – longitudinal frame (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	Transverse Reinforce.	$\frac{V}{b_w * d * \sqrt{f_c'}}$	Modelling Parameters			
						Plastic Rotation Angle		Residual strength ratio	Normalised ratio
						a	b	c	b/a
1-2	1	Left	0.261	C.	<3	0.022	0.039	0.2	1.772727
1-2	1	Middle	0.085	C.	<3	0.025	0.05	0.2	2
1-2	1	Right	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	2	Left	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	2	Middle	0.085	C.	<3	0.025	0.05	0.2	2
1-2	2	Right	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	3	Left	0.237	C.	<3	0.023	0.045	0.2	1.956522
1-2	3	Middle	0.085	C.	<3	0.025	0.05	0.2	2
1-2	3	Right	0.261	C.	<3	0.022	0.039	0.2	1.772727
3-4	1	Left	0.237	C.	<3	0.023	0.045	0.2	1.956522

3-4	1	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	1	Right	0.152	C.	<3	0.024	0.046	0.2	1.916667
3-4	2	Left	0.152	C.	<3	0.024	0.046	0.2	1.916667
3-4	2	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	2	Right	0.152	C.	<3	0.024	0.046	0.2	1.916667
3-4	3	Left	0.152	C.	<3	0.024	0.046	0.2	1.916667
3-4	3	Middle	0.024	C.	<3	0.025	0.05	0.2	2
3-4	3	Right	0.237	C.	<3	0.023	0.044	0.2	1.833333
5	1-2-3	Left	0.109	C.	<3	0.024	0.044	0.2	1.833333
5	1-2-3	Middle	0.109	C.	<3	0.024	0.044	0.2	1.833333
5	1-2-3	Right	0.109	C.	<3	0.024	0.044	0.2	1.833333
6	1-2-3	Left	0.062	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Middle	0.062	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Right	0.062	C.	<3	0.025	0.05	0.2	2

Table 2.10 : Modeling Parameters for Positive Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{bot} - \rho_{top}}{\rho_{bal}}$	Transverse Reinforce.	$\frac{V}{b_w * d * \sqrt{f_c^{'}}}$	Modelling Parameters			
						Plastic Rotation Angle		Residual strength ratio	Normalised ratio
						a	b	c	b/a
1-2	1	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	1	Middle	<0	C.	<3	0.025	0.05	0.2	2
1-2	1	Right	<0	C.	<3	0.025	0.05	0.2	2
1-2	2	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	2	Middle	<0	C.	<3	0.025	0.05	0.2	2
1-2	2	Right	<0	C.	<3	0.025	0.05	0.2	2
1-2	3	Left	<0	C.	<3	0.025	0.05	0.2	2
1-2	3	Middle	<0	C.	<3	0.025	0.05	0.2	2
1-2	3	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	1	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	2	Right	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Left	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Middle	<0	C.	<3	0.025	0.05	0.2	2
3-4	3	Right	<0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Left	<0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Middle	<0	C.	<3	0.025	0.05	0.2	2
5	1-2-3	Right	<0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Left	<0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Middle	<0	C.	<3	0.025	0.05	0.2	2
6	1-2-3	Right	<0	C.	<3	0.025	0.05	0.2	2

Table 2.11: Acceptance Criteria for Negative Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{top}-\rho_{bot}}{\rho_{bal}}$	Tran. Rein.	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Acceptance Criteria					
						Plastic Rotation Angle					
						Performance Level					
						IO	Component Type		Normalised ratio		
							Primary		IO/a	LS/a	CP/a
							LS	CP			
1-2	1	Left	0.261	C.	<3	0.01	0.014	0.022	0.45	0.64	1
1-2	1	Middle	0.085	C.	<3	0.01	0.018	0.025	0.42	0.75	1.04
1-2	1	Right	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	2	Left	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	2	Middle	0.085	C.	<3	0.01	0.018	0.025	0.4	0.72	1
1-2	2	Right	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	3	Left	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
1-2	3	Middle	0.085	C.	<3	0.01	0.018	0.025	0.4	0.72	1
1-2	3	Right	0.261	C.	<3	0.01	0.014	0.022	0.45	0.64	1
3-4	1	Left	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
3-4	1	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Right	0.152	C.	<3	0.01	0.017	0.024	0.42	0.71	1
3-4	2	Left	0.152	C.	<3	0.01	0.017	0.024	0.42	0.71	1
3-4	2	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Right	0.152	C.	<3	0.01	0.017	0.024	0.42	0.71	1
3-4	3	Left	0.152	C.	<3	0.01	0.017	0.024	0.42	0.71	1
3-4	3	Middle	0.024	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Right	0.237	C.	<3	0.01	0.015	0.023	0.43	0.65	1
5	1-2-3	Left	0.109	C.	<3	0.01	0.018	0.024	0.42	0.75	1
5	1-2-3	Middle	0.109	C.	<3	0.01	0.018	0.024	0.42	0.75	1
5	1-2-3	Right	0.109	C.	<3	0.01	0.018	0.024	0.42	0.75	1
6	1-2-3	Left	0.062	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Middle	0.062	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Right	0.062	C.	<3	0.01	0.02	0.025	0.4	0.8	1

Table 2.12 : Acceptance Criteria for Positive Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam of Floor	Span	Position of plastic hinges	$\frac{\rho_{bot}-\rho_{top}}{\rho_{bal}}$	Tran. Rein.	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Acceptance Criteria					
						Plastic Rotation Angle					
						Performance Level					
						IO	Component Type		Normalised ratio		
							Primary		IO/a	LS/a	CP/a
							LS	CP			
1-2	1	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	1	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	1	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	2	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
1-2	3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	1	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	2	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
3-4	3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
5	1-2-3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Left	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Middle	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1
6	1-2-3	Right	<0	C.	<3	0.01	0.02	0.025	0.4	0.8	1

II.3.2 <u>Column</u>

Table 2.13 : Properties of the plastic hinge section

Column of floor	Position of plastic hinges	N – Axial force (N)	V – Shear Force (N)	Gross Area of Column A_g(mm2)	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$
1-2-3	Bottom	1306600	79530	200000	0.261	<3
1-2-3	Top	1289200	79530	200000	0.258	<3
4-5-6	Bottom	628800	63902	200000	0.126	<3
4-5-6	Top	614000	23902	200000	0.123	<3

Table 2.14: Modeling Parameters for Plastic Moment (According to FEMA 356 – Table 6.8 – Columns controlled by Flexure)

Column of floor	Position of plastic hinges	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Transverse Reinforce.	Modelling Parameters			
					Plastic Rotation Angle		Residual strength ratio	Normalised ratio
					a	b	c	b/a
1-2-3	Bottom	0.261	<3	C	0.017	0.027	0.2	1.59
1-2-3	Top	0.258	<3	C	0.017	0.027	0.2	1.59
4-5-6	Bottom	0.126	<3	C	0.02	0.03	0.2	1.5
4-5-6	Top	0.123	<3	C	0.02	0.03	0.2	1.5

Table 2.15: Acceptance Criteria for Plastic Moment (According to FEMA 356 – Table 6.8 – Columns controlled by Flexure)

Column of floor	Position of plastic hinges	$\frac{N}{A_g * f_{ck}}$	$\frac{V}{b_w * d * \sqrt{f_{ck}}}$	Tran. Rein.	Acceptance Criteria					
					Plastic Rotation Angle					
					Performance Level					
					IO	Component Type		Normalised ratio		
						Primary		IO/a	LS/a	CP/a
						LS	CP			
1-2 -3	Bottom	0.259	<3	C.	0.004	0.013	0.017	0.24	0.76	1
1-2-3	Top	0.2552	<3	C.	0.004	0.013	0.017	0.24	0.76	1
4-5-6	Bottom	0.1248	<3	C.	0.005	0.015	0.020	0.25	0.75	1
4-5-6	Top	0.1232	<3	C.	0.005	0.015	0.020	0.25	0.75	1

III Lateral Loads

Pushover analysis in EC8 [3] follows the N2 method developed by Fajfar (1999). The method consists of applying two lateral distributions to the frame:

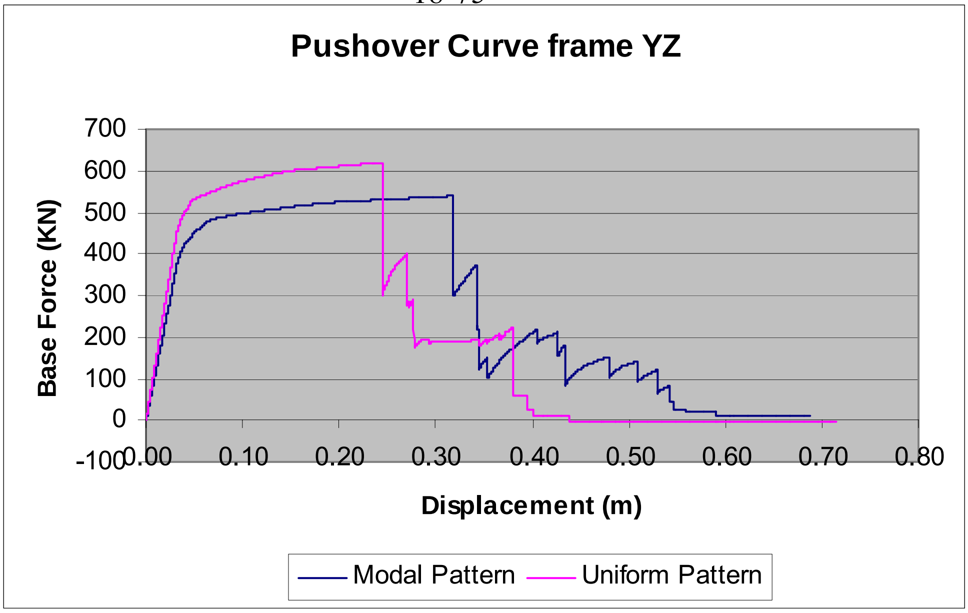
- A "modal" pattern, that is a load shape proportional to the mass matrix multiplied by the first elastic mode shape $P^1 = M\varphi_1$.
- A "uniform" pattern, that is a mass proportional to the load shape $P^2 = MR$

Table 2.16: Lateral Loads

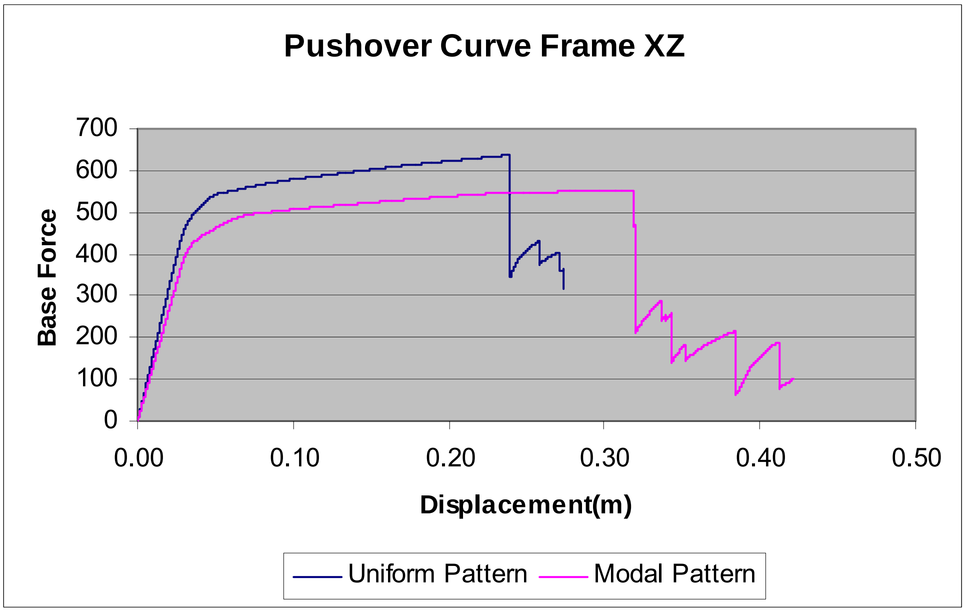
Frame	Floor	Modal Pattern	Uniform Pattern	Frame	Floor	Modal Pattern	Uniform Pattern
YZ	6	1	1	XZ	6	1	1
	5	0.931	1		5	0.939	1
	4	0.809	1		4	0.824	1
	3	0.634	1		3	0.657	1
	2	0.421	1		2	0.45	1
	1	0.191	1		1	0.218	1

IV Pushover Curve (Capacity Curve)

By using Non-linear static Analysis in SAP 2000 version 9.0.3[8], the Pushover Curve of two vertical distributions of the lateral forces which correspond to uniform pattern loads and to modal pattern loads will be plotted as following pictures:



Picture 2.1 : Pushover Curve of frame YZ



Picture 2.2: Pushover Curve of frame XZ

Plastic Mechanisms corresponding to two modal load patterns are presented in following pictures.

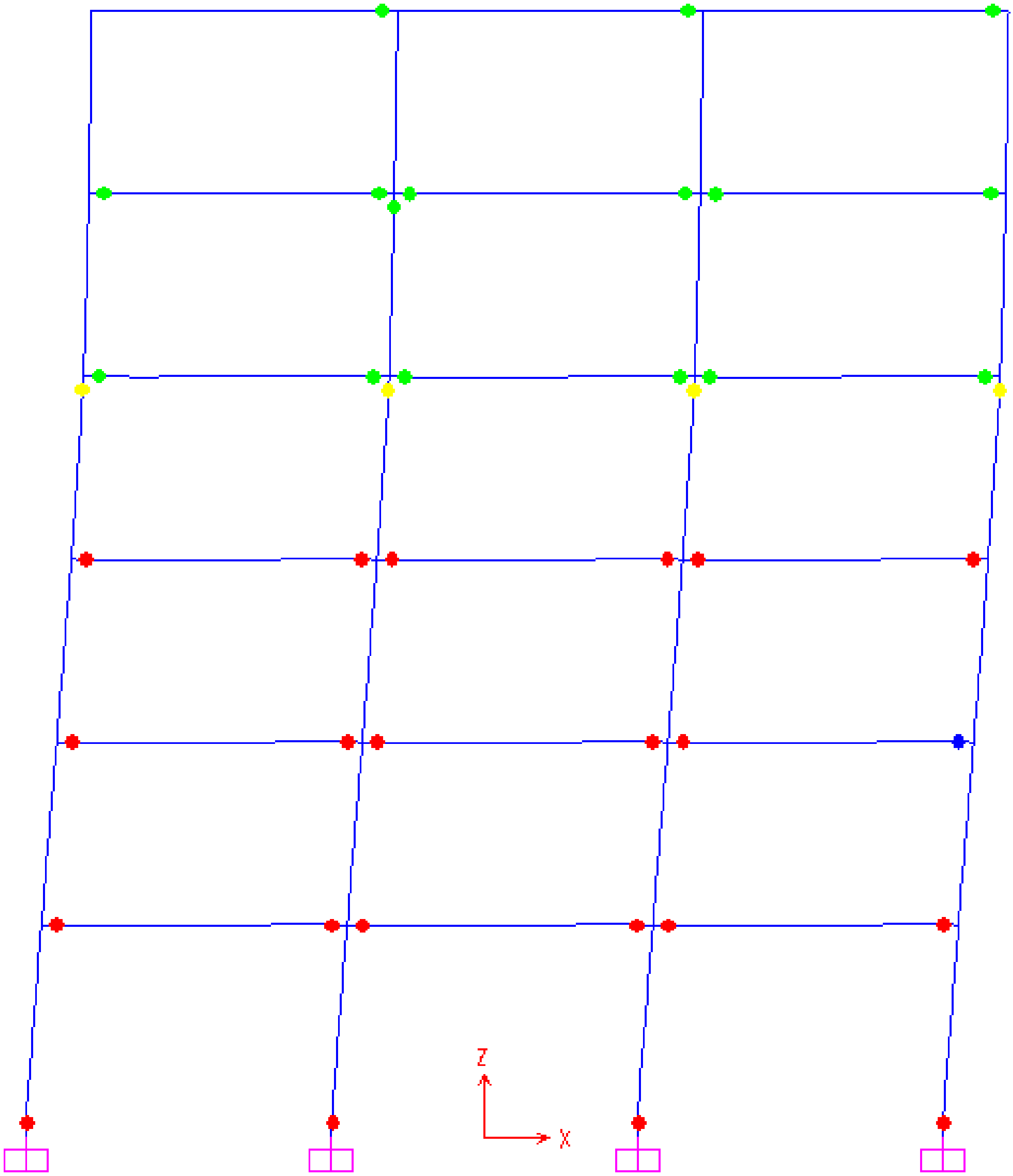


Figure 2.3: Plastic Mechanism - Uniform Load Pattern

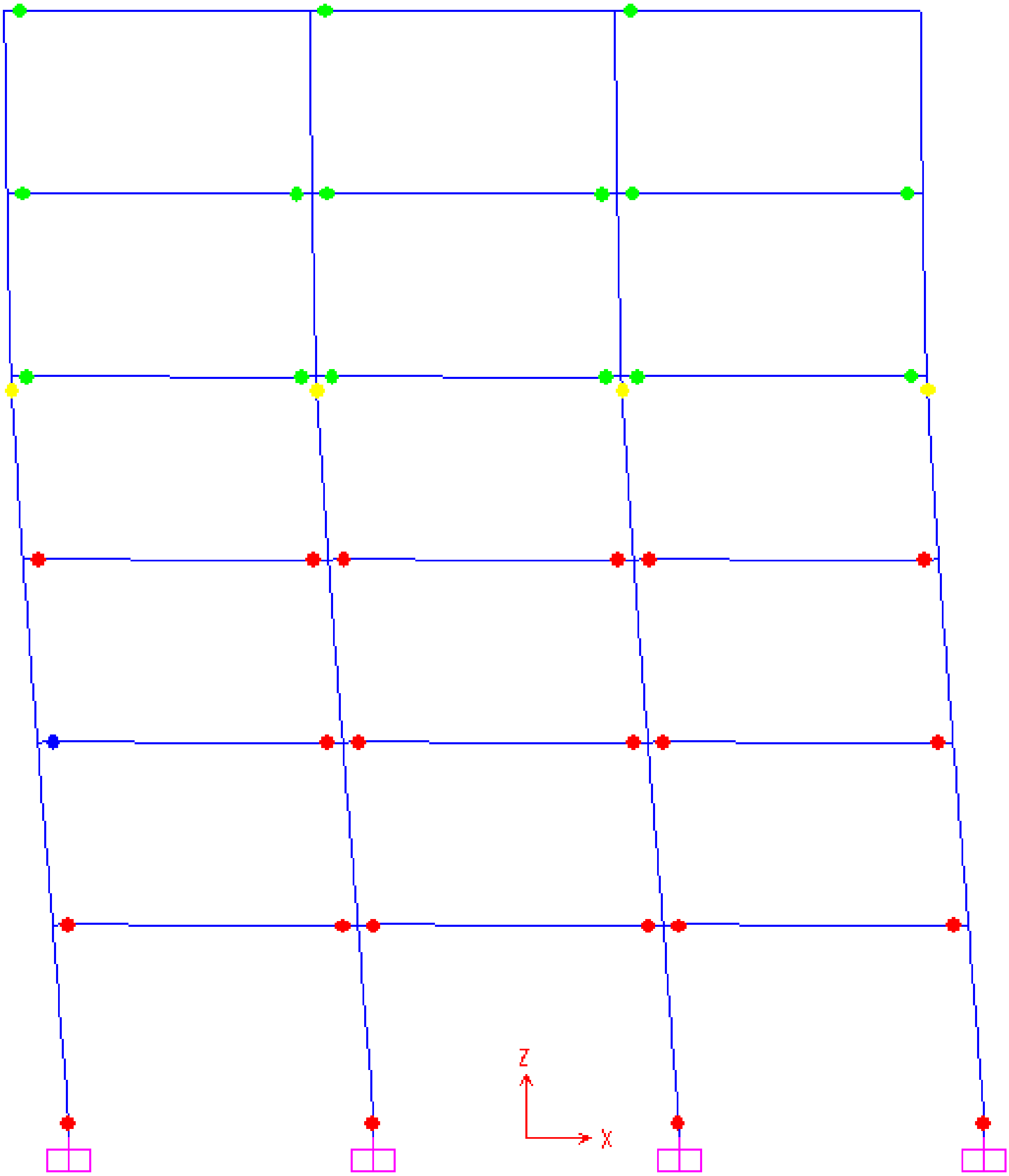


Figure 2.4: Plastic Mechanism Plane YZ - Modal Load Pattern

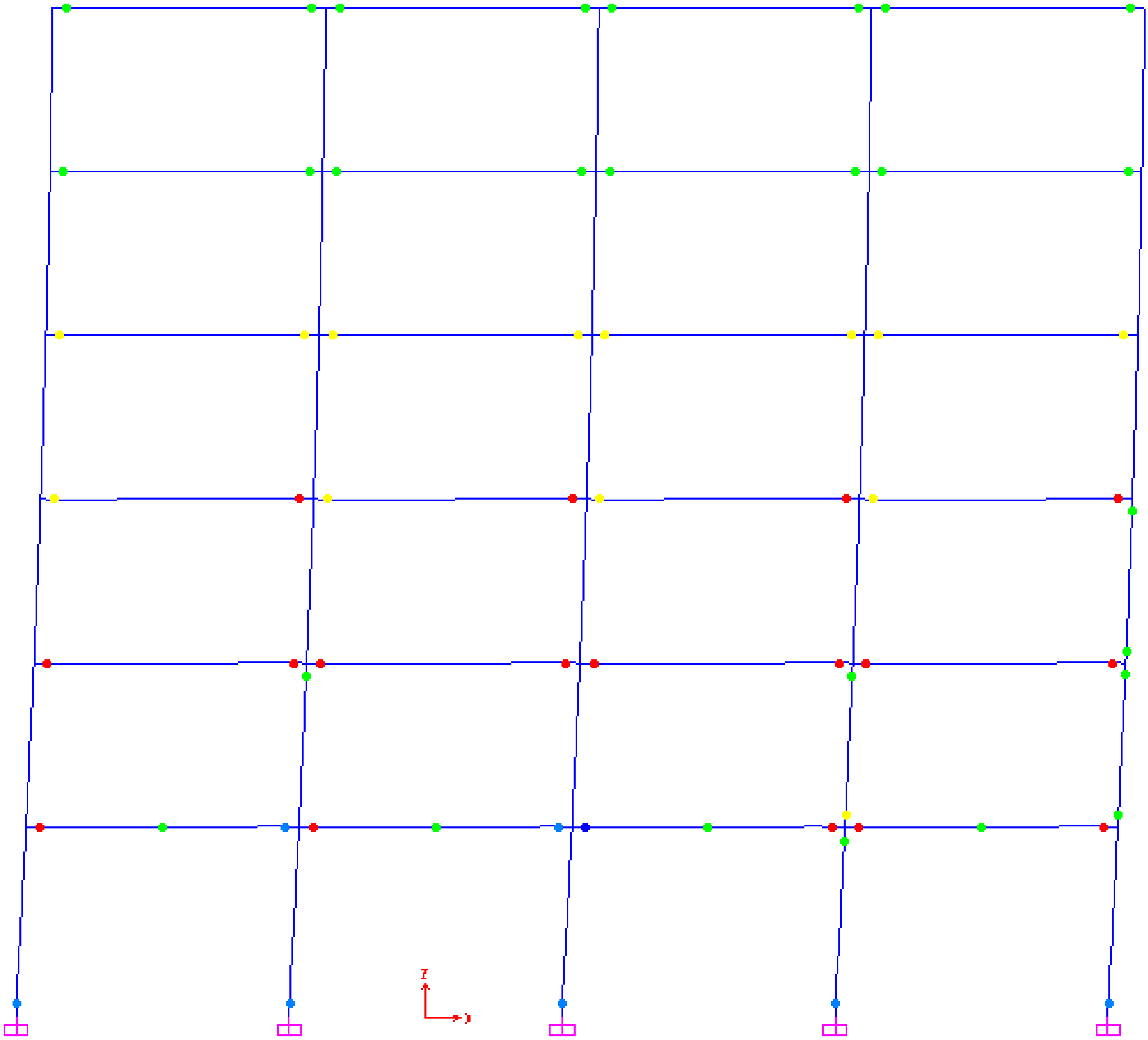


Figure 2.5: Plastic Mechanism Plane XZ - Modal Load Pattern

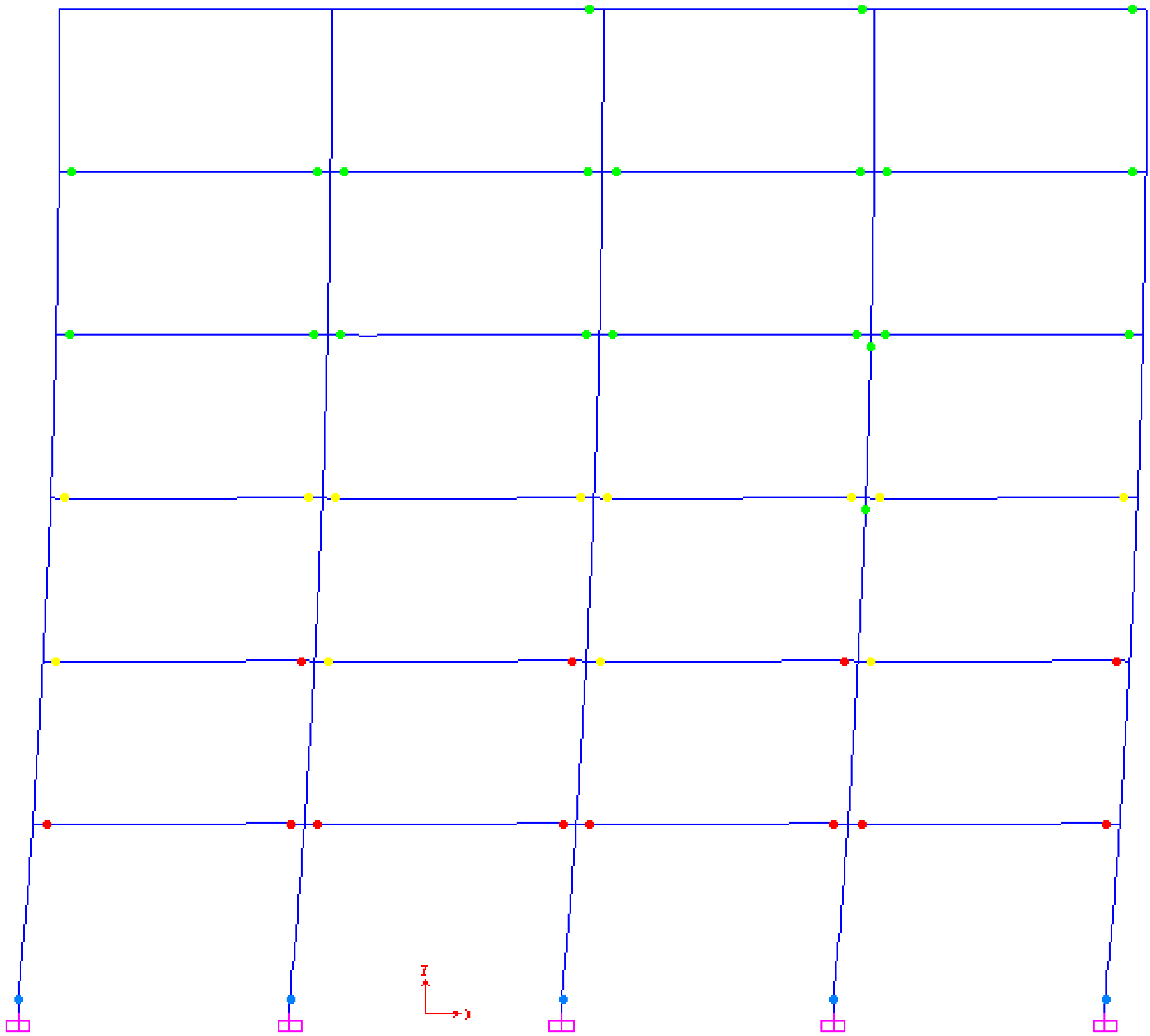


Figure 2.6: Plastic Mechanism Plane XZ - Uniform Load Pattern

V **Target Displacement**

To check performance of structure under earthquake excitations, we will determine the target displacements corresponding to given peak ground accelerations. Three values of PGA will be checked (a_{Rg}=0.15g; 0.3g and 0.6g). The target displacement of the structure will be determined according to ANNEX B – EC8[3]. The structure which is multi degree of freedom will be transformed to Single Degree of Freedom by using the law of energy equilibrium.

Table 2.17 : Frame Properties

Frames	Floors	Masses - m_i (Kg)	Normalized Mode ϕ_i		$m_i\phi_i$		$m_i\phi^2_i$	
			Modal Pattern	Uniform Pattern	Modal Pattern	Uniform Pattern	Modal Pattern	Uniform Pattern
YZ	6	57054.0265	1	1	57054.0265	57054.0265	57054.0265	57054.0265
	5	67568.8073	0.931	1	62906.55963	67568.8073	58566.00702	67568.8073
	4	67568.8073	0.809	1	54663.16514	67568.8073	44222.5006	67568.8073
	3	67568.8073	0.634	1	42838.62385	67568.8073	27159.68752	67568.8073
	2	67568.8073	0.421	1	28446.46789	67568.8073	11975.96298	67568.8073
	1	68588.1753	0.191	1	13100.34149	68588.1753	2502.165224	68588.1753
	Total				Σ=259009.1845	Σ=395917.431	Σ=201480.3498	Σ=395917.431
XZ	6	57054.0265	1	1	57054.0265	57054.0265	57054.0265	57054.0265
	5	67568.8073	0.939	1	63447.11009	67568.8073	59576.83638	67568.8073
	4	67568.8073	0.824	1	55676.69725	67568.8073	45877.59853	67568.8073
	3	67568.8073	0.657	1	44392.70642	67568.8073	29166.00812	67568.8073
	2	67568.8073	0.45	1	30405.9633	67568.8073	13682.68349	67568.8073
	1	68588.1753	0.218	1	14952.22222	68588.1753	3259.584444	68588.1753
	Total				Σ=265928.7258	Σ=395917.431	Σ=208616.7375	Σ=395917.431

Table 2.18 : Target displacement of frame YZ – Modal Load Pattern, a_{Rg} = 0.15g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	Dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4308	1.29	539.5	419.7	0.32	0.25	152	0.07	0.057	1.2	0.6	2.14	0.076	0.076
2	1026		486.8	378.7	0.076	0.059	25.9	0.046	0.036	0.98	0.6	2.58	0.063	0.063
3	850		476.9	371.0	0.063	0.049	19.6	0.044	0.034	0.97	0.6	2.60	0.063	0.063
The target displacement is $d_t = d^*_t * \Gamma$ = 0.08m														

Table 2.19 : Target displacement of frame YZ – Modal Load Pattern, a_{Rg} = 0.3g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4308	1.29	539.5	419.7	0.32	0.25	152	0.07	0.057	1.18	0.6	4.29	0.152	0.152
2	2053		513.6	399.5	0.15	0.12	64.0	0.055	0.043	1.05	0.6	4.85	0.14	0.14
The target displacement is $d_t = d^*_t * \Gamma$ = 0.19m														

Table 2.20 : Target displacement of frame YZ – Modal Load Pattern, a_{Rg} = 0.5g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* S	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4308	1.29	539.5	419.7	0.32	0.25	152	0.07	0.057	1.18	0.6	6.2	0.25	0.25
The target displacement is $d_t = d^*_t * \Gamma$ = 0.32m														

Table 2.21: Target displacement of frame YZ – Uniform Load Pattern, a_{Rg} = 0.15g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* S	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3301	1	620.3	620.3	0.24	0.24	131	0.066	0.066	1.29	0.6	1.97	0.083	0.083
2	1121	1	562.3	562.3	0.083	0.083	34.4	0.043	0.043	1.09	0.6	2.32	0.07	0.07
The target displacement is $d_t = d^*_t * \Gamma$ = 0.07m														

Table 2.22: Target displacement of frame YZ – Uniform Load Pattern, a_{Rg} = 0.3g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* S	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3301	1	620.3	620.3	0.24	0.24	131	0.066	0.066	1.29	0.6	3.95	0.17	0.17
2	2296	1	604.8	604.8	0.17	0.17	85.5	0.048	0.048	1.11	0.6	4.56	0.144	0.144
3	1945	1	591.1	591.1	0.144	0.144	69.9	0.052	0.052	1.17	0.6	4.34	0.15	0.15
The target displacement is $d_t = d^*_t * \Gamma$ = 0.15m														

Table 2.23: Target displacement of frame YZ – Uniform Load Pattern, a_{Rg} = 0.5g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* S	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3301	1	620.3	620.3	0.24	0.24	131	0.066	0.066	1.29	0.6	6.58	0.28	0.28
The target displacement is $d_t = d^*_t * \Gamma$ = 0.24m														

Table 2.24: Target displacement of frame XZ – Modal Load Pattern, a_{Rg} = 0.15g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4309	1.28	553.1	433.9	0.319	0.25	157	0.07	0.057	1.15	0.6	2.2	0.074	0.074
2	999		495.9	489.0	0.074	0.058	26.2	0.043	0.034	0.95	0.6	2.7	0.061	0.061
The target displacement is $d_t = d^*_t * \Gamma$ = 0.078m														

Table 2.25: Target displacement of frame XZ – Modal Load Pattern, a_{Rg} = 0.3g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4309	1.28	553.1	433.9	0.319	0.25	157	0.07	0.055	1.154	0.6	4.4	0.149	0.149
2	2012		522.9	410.2	0.149	0.17	64.4	0.051	0.04	1.0	0.6	5.03	0.13	0.13
The target displacement is $d_t = d^*_t * \Gamma$ = 0.17m														

Table 2.26: Target displacement of frame XZ – Modal Load Pattern, a_{Rg} = 0.5g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	4309	1.28	553.1	433.9	0.319	0.25	157	0.07	0.055	1.16	0.6	7.3	0.25	0.25
The target displacement is $d_t = d^*_t * \Gamma$ = 0.32m														

Table 2.27: Target displacement of frame XZ – Uniform Load Pattern, a_{Rg} = 0.15g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3226	1	636.9	636.9	0.24	0.24	130	0.072	0.072	1.33	0.6	1.91	0.085	0.085
2	1147		569.6	569.6	0.085	0.085	36.7	0.042	0.042	1.00	0.6	2.37	0.07	0.07
The target displacement is $d_t = d^*_t * \Gamma$ = 0.07m														

Table 2.28: Target displacement of frame XZ – Uniform Load Pattern, a_{Rg} = 0.3g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* s	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3226	1	636.9	636.9	0.24	0.24	130	0.072	0.072	1.33	0.6	3.83	0.17	0.17
2	2296		611.4	611.4	0.17	0.17	87.0	0.057	0.057	1.2	0.6	4.22	0.16	0.16
The target displacement is $d_t = d^*_t * \Gamma$ = 0.16m														

Table 2.29: Target displacement of frame XZ – Uniform Load Pattern, a_{Rg} = 0.5 g soil C

Ite. Pro.s	Step	Γ	F_b (KN)	F^* (KN)	dm (m)	d^* (m)	E_m (KNm)	d_y (m)	d^*_y (m)	T^* S	T_c s	$S_e(T^*)$	d^*_e (m)	d^*_t (m)
1	3226	1	636.9	636.9	0.24	0.24	130	0.072	0.072	1.33	0.6	6.37	0.28	0.28
2	2296		611.4	611.4	0.17	0.17	87.0	0.057	0.057	1.2	0.6	4.22	0.16	0.16
The target displacement is $d_t = d^*_t * \Gamma$ = 0.24m														

VI Over-strength Factor and behavior factor

- According to 5.2.2.2 – EC8[3], over strength factor is the ratio between the values of α_u, which is the multiplier of the horizontal seismic design action with all other design actions constant, at formation of plastic hinges in a number sections sufficient for the development of overall structural instability and of α_1, which is the multiplier of the horizontal seismic design action at first attainment of member flexural resistance anywhere in the structure. The ratio α_u/α_1 taken into account structural behavior at first yielding and plastic mechanism is equal to the ratio between the values of the base force at plastic mechanism and of the base force at first yielding.

  All of these values will be displayed as following tables:

Table 2.30: Over-strength Factor

Frame	Modal Load Pattern					Uniform Load Pattern				
	Base Force at first yielding $\alpha_1 V_b$	Displace. at first yielding d_y	Base Force at plastic mecha. $\alpha_u V_b$	Displac at plastic mecha. d_y	α_u/α_1	Base Force at first yielding $\alpha_1 V_b$	Displace. at first yielding d_y	Base Force at plastic mecha. $\alpha_u V_b$	Displac at plastic mecha. d_y	α_u/α_1
YZ	370.1KN	0.032m	539.5KN	0.32m	1.46	399.4	0.028m	620.3KN	0.244m	1.55
XZ	346.8	0.0264m	553.1KN	0.32m	1.6	386.1	0.025m	636.9KN	0.239m	1.65

- According to 3.2.2.5 - EC8 [3], the behaviour factor *q* is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The q – factor will be determined as following table:

Table 2.31: Behaviour factors

Frame	Modal Load Pattern				Uniform Load Pattern			
	Base Force at yielding (equi. SDOF)	Maximum Displac . (equi. SDOF d_y^*)	Displace. at first yielding d_y (SDOF)	q	Base Force at yielding (equi. SDOF)	Maximum Displac . (equi. SDOF d_y^*)	Displace. at first yielding d_y (SDOF)	q
YZ	419.7KN	0.25m	0.057m	4.4	620.3	0.24	0.072	3.3
XZ	433.9	0.25	0.057	4.4	636.9	0.24	0.072	3.3

VII Check the performance of the structure

The performance of the structure under given earthquake excitation will be obtained by comparing capacity of the structure (capacity curve) and demand of the earthquake excitation. (the target displacements). The state of the structure at target displacement under a given earthquake represented by elastic response acceleration will be summarized as follows:

Table 2.32: Plastic Hinge Distributions under different earthquake excitations – Modal Pattern

Frame	a_{Rg} – Soil C	dt (m)	Step	Modal Load Pattern (Number of plastic hinges)				
				B-IO	IO-LS	LS-CP	CP-C	C-D
YZ	0.15g	0.08	1080	34	0	0	0	0
	0.3g	0.19	2567	23	19	0	0	0
	0.5g	0.32	4308	18	5	20	0	1
XZ	0.15g	0.078	1053	41	0	0	0	0
	0.3g	0.17	2296	26	21	0	0	0
	0.5g	0.32	4309	16	16	20	0	1

Table: Plastic Hinge Distributions under different earthquake excitations – Uniform Pattern

Frame	a_{Rg} – Soil C	dt (m)	Step	Uniform Load Pattern (Number of plastic hinges)				
				B-IO	IO-LS	LS-CP	CP-C	C-D
YZ	0.15g	0.07	945	24	0	0	0	0
	0.3g	0.15	2026	27	10	0	0	0
	0.5g	0.24	3301	15	10	15	0	1
XZ	0.15g	0.07	945	33	0	0	0	0
	0.3g	0.16	2161	20	21	0	0	0
	0.5g	0.24	4309	19	16	12	0	1

The behaviors of the structure under PGA 0.15g excitation which are observed at target displacements in two planar frames are similar. There is no soft-story mechanism happening to the structure. The states of all plasticized hinges are in the range of yielding and immediate occupancy (B-IO) that means all of these hinges just begin to yield. The performances of two frames under PGA 0.3g excitation in two load patterns are also similar. Under the PGA 0.5g excitation, the plastic mechanisms are observed in two frames. The desired plastic mechanism in which the plastic hinges are concentrated at the end of all beams and at the base of column is only attained in Modal Load Pattern for both frames. The collapse of the frames happens when the rotation of plastic hinge at the column's base section exceeds limit value.

VIII Conclusion

- The reinforced concrete frames in a given seismic region which are correspondingly designed to EC8 [3] are checked by Nonlinear Static Analysis (Pushover). For the acceleration a_{gR}=0,15g equal to the designed acceleration, the behavior of the frames is quite good (the structure can reach to the target displacement with some plastic hinges). There is no soft-story mechanism observed.
- Under greater earthquakes (for a_{gR}=0,2g; a_{gR}=0,3g), the structure still reach to target displacements with more plastic hinges. There is no soft-story mechanism observed with these accelerations.
- Under the earthquake with a_{gR}=0,5g, the structure can not reach to target displacement with because of some local plastic hinge collapses.

Symbols

The symbols which are used in the design example are:

A_{Ed} : design value of seismic action (= $\gamma_I . A_{Ek}$)
A_{Ek} : characteristic value of the seismic action for the reference return period
E_d: design value of action effects
Q: variable action
$S_e(T)$: elastic horizontal ground acceleration response spectrum also called "elastic response spectrum". At T=0, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S.
$S_{ve}(T)$: elastic vertical ground acceleration response spectrum
$S_{De}(T)$: elastic displacement response spectrum
$S_d(T)$: design spectrum (for elastic analysis). At T=0, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S
S: soil factor
T: vibration period of a linear single degree of freedom system
T_s: duration of the stationary part of the seismic motion
a_{gR} : reference peak ground acceleration on type A ground
a_g : design ground acceleration on type A ground
d_g : design ground displacement
g : acceleration of gravity
q: behaviour factor
γ_I : importance factor
η damping correction factor
ξ viscous damping ratio (in percent)
$\psi_{2,i}$: combination coefficient for the quasi-permanent value of a variable action i
$\psi_{E,i}$: combination coefficient for a variable action i, to be used when determining the effects of the design seismic action
E_{Edx}, E_{Edy} : design values of the action effects due to the horizontal components (x and y) of the seismic action
E_{Edz} : design value of the action effects due to the vertical component of the seismic action
F_i : horizontal seismic force at storey i
F_b : base shear force
H : building height from the foundation or from the top of a rigid basement
L_{max}, L_{min} : larger and smaller in plan dimension of the building measured in orthogonal directions
R_d : design value of resistance
S_a : seismic coefficient for non-structural elements
T_1 : fundamental period of vibration of a building
d : displacement
d_r : design interstorey drift
e_a : accidental eccentricity of the mass of one storey from its nominal location
h: interstorey height
m_i : mass of storey i
n : number of storeys above the foundation or the top of a rigid basement
q_d : displacement behaviour factor
s_i : displacement of mass m_i in the fundamental mode shape of a building
z_i : height of mass m_i above the level of application of the seismic action
α :ratio of the design ground acceleration to the acceleration of gravity
θ: interstorey drift sensitivity coefficient

A_c: Area of section of concrete member
A_{sh}: total area of horizontal hoops in a beam-column joint
A_{si} : total area of steel bars in each diagonal direction of a coupling beam
A_{st} : area of one leg of the transverse reinforcement
A_{sv} : total area of the vertical reinforcement in the web of the wall
$A_{sv,i}$: total area of column vertical bars between corner bars in one direction through a joint
ΣM_{Rb} : sum of design values of moments of resistance of the beams framing into a joint in the direction of interest
ΣM_{Rc} : sum of design values of the moments of resistance of the columns framing into a joint in the direction of interest
D_o : diameter of confined core in a circular column
$M_{i,d}$: end moment of a beam or column for the calculation of its capacity design shear
$M_{Rb,i}$: design value of beam moment of resistance at end i
$M_{Rc,i}$: design value of column moment of resistance at end i
N_{Ed} : axial force from the analysis for the seismic design situation
T_1 :fundamental period of the building in the horizontal direction of interest
T_C : corner period at the upper limit of the constant acceleration region of the elastic spectrum
$V_{Ed,max}$: maximum acting shear force at end section of a beam from capacity design calculation
$V_{Ed,min}$: minimum acting shear force at end section of a beam from capacity design calculation
$V_{Rd,c}$: design value of shear resistance for members without shear reinforcement in accordance with EN1992-1-1:2004
$V_{Rd,S}$: design value of shear resistance against sliding
b : width of bottom flange of beam
b_c : cross-sectional dimension of column
b_{eff} : effective flange width of beam in tension at the face of a supporting column
b_i : distance between consecutive bars engaged by a corner of a tie or by a cross-tie in a column
b_o: width of confined core in a column or in the boundary element of a wall (to centreline of hoops)
b_w : thickness of confined parts of a wall section, or width of the web of a beam
d : effective depth of section
d_{bL} : longitudinal bar diameter
d_{bw} : diameter of hoop
f_{cd} : design value of concrete compressive strength
f_{ctm} : mean value of tensile strength of concrete
f_{yd} : design value of yield strength of steel
$f_{yd,h}$: design value of yield strength of the horizontal web reinforcement
$f_{yd,v}$: design value of yield strength of the vertical web reinforcement
f_{yld} : design value of yield strength of the longitudinal reinforcement
f_{ywd} : design value of yield strength of transverse reinforcement
h : cross-sectional depth
h_c : cross-sectional depth of column in the direction of interest
h_f : flange depth
h_{jc}: distance between extreme layers of column reinforcement in a beam-column joint
h_{jw} : distance between beam top and bottom reinforcement
h_o : depth of confined core in a column (to centreline of hoops)
h_s : clear storey height
h_w : height of wall or cross-sectional depth of beam

k_D : factor reflecting the ductility class in the calculation of the required column depth for anchorage of beam bars in a joint, equal to 1 for DCH and to 2/3 for DCM
k_w : factor reflecting the prevailing failure mode in structural systems with walls
l_{cl} :clear length of a beam or a column
l_{cr} : length of critical region
n : total number of longitudinal bars laterally engaged by hoops or cross ties on perimeter of column section
q_o : basic value of the behaviour factor
s: spacing of transverse reinforcement
x_u : neutral axis depth
z: internal lever arm
α :confinement effectiveness factor, angle between diagonal bars and axis of a coupling beam
α_o: prevailing aspect ratio of walls of the structural system
α_1 : multiplier of horizontal design seismic action at formation of first plastic hinge in the system
α_u : multiplier of horizontal seismic design action at formation of global plastic mechanism
γ_c : partial factor for concrete
γ_{Rd} model uncertainty factor on design value of resistances in the estimation of capacity design action effects, accounting for various sources of overstrength
γ_s: partial factor for steel
ε_{cu2} : ultimate strain of unconfined concrete
$\varepsilon_{cu2,c}$: ultimate strain of confined concrete
$\varepsilon_{su,k}$: characteristic value of ultimate elongation of reinforcing steel
$\varepsilon_{sy,d}$: design value of steel strain at yield
η : reduction factor on concrete compressive strength due to tensile strains in transverse direction
ζ ratio, $V_{Ed,min}/V_{Ed,max}$, between the minimum and maximum acting shear forces at the end section of a beam
μ_φ : curvature ductility factor
μ_δ : displacement ductility factor
ν : axial force due in the seismic design situation, normalised to $A_c f_{cd}$
ξ : normalised neutral axis depth
ρ : tension reinforcement ratio
ρ' : compression steel ratio in beams
σ_{cm} : mean value of concrete normal stress
ρ_{max} : maximum allowed tension steel ratio in the critical region of primary seismic beams
ρ_w : shear reinforcement ratio
ω_v : mechanical ratio of vertical web reinforcement
ω_{wd} : mechanical volumetric ratio of confining reinforcement
Γ: transform factor
m^*: the mass of the equivalent SDOF system
F^* : The force F^* and displacement d^* of the equivalent SDOF system
d^* :The displacement d^* of the equivalent SDOF system
d_y^*: the yield displacement of the idealised SDOF system
E_m^*: the actual deformation energy up to the formation of the plastic mechanism.
d_m^*: the displacement of the idealised SDOF system at plastic mechanism of MDOF system.
T^* : the idealized equivalent SDOF system

F_y^*: the yield force of the idealised SDOF system
d_{et}^*: The target displacement of the structure with period T^* and unlimited elastic behaviour
$S_e(T^*)$: the elastic acceleration response spectrum at the period T^*.
d_t^* : target displacement of the idealised SDOF system
d_t : The target displacement of the MDOF system

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