DESIGN PROCEDURES

Rita Peres, Rita Bento, Miguel Castro

Manuscript prepared April, 2011

Table of contents

INTRODUCTION

In the scope of the research project POCI/ECM/59306/2004, a group of steel structures is being selected and designed in order to be assessed, in terms of seismic performance, by the non linear static procedures.

The aim of the research is to extend the use of these procedures to 3D irregular structures, therefore the evaluation of the torsional phenomena, and its effects in seismic response are crucial for a complete understanding of the study results. For this reason the following groups of 1 storey structures, with different torsional characteristics and different hysteretic behaviors, was selected:

GROUP 1- Steel structures with stable hysteretic behavior (the seismic lateral resisting systems selected were Moment Resisting Frames)

1. Laterally unrestrained regular structures (CM=CR=CV);
2. Laterally unrestrained irregular structures (different locations for CM,CR and CV);
3. Laterally restrained regular structures (CM=CR=CV);
4. Laterally restrained irregular structures (different locations for CM, CR and CV).

GROUP 2- Steel structures with unstable hysteretic behavior (the seismic lateral resisting systems selected were Concentric Braces Frames)

5. Laterally unrestrained regular structures (CM=CR=CV);
6. Laterally unrestrained irregular structures (different locations for CM,CR and CV);
7. Laterally restrained regular structures (CM=CR=CV);
8. Laterally restrained irregular structures (different locations for CM, CR and CV).

In the present report, a description of the structures configuration and the design procedures is presented. The static design procedure followed the EC3 [CEN, 2005] recommendations and the seismic design followed the IFBD procedure [Villani, 2009] which consists of a more rational sequence of the design steps prescribed by EC8 [CEN, 2004] and a more realist selection of the structure's behaviour factor.

DESIGN OF THE STUDIED STRUCTURES

1 STRUCTURES CONFIGURATION (1 storey structures)

1.1 GROUP 1 - Structures with stable hysteretic behavior (MRF)

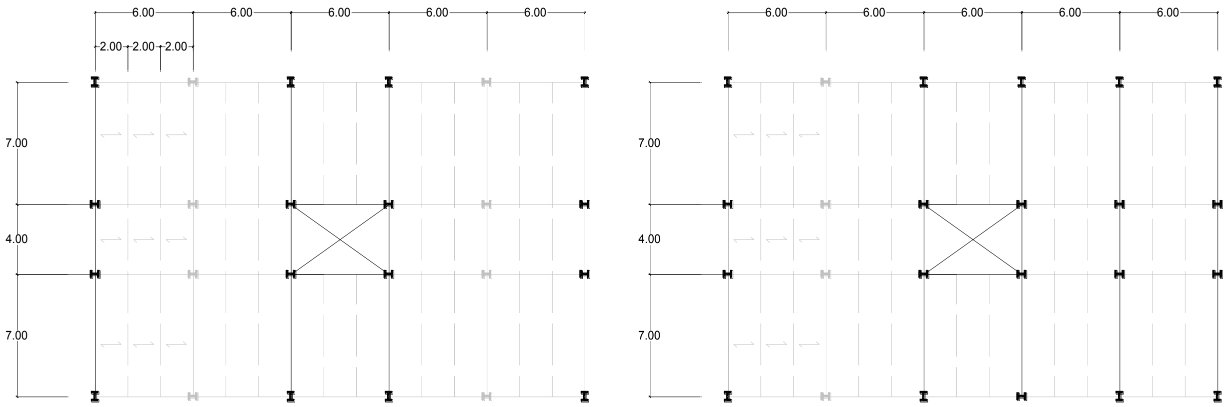

Figure 1- Laterally unrestrained regular& irregular structures (structures 1 & 2)

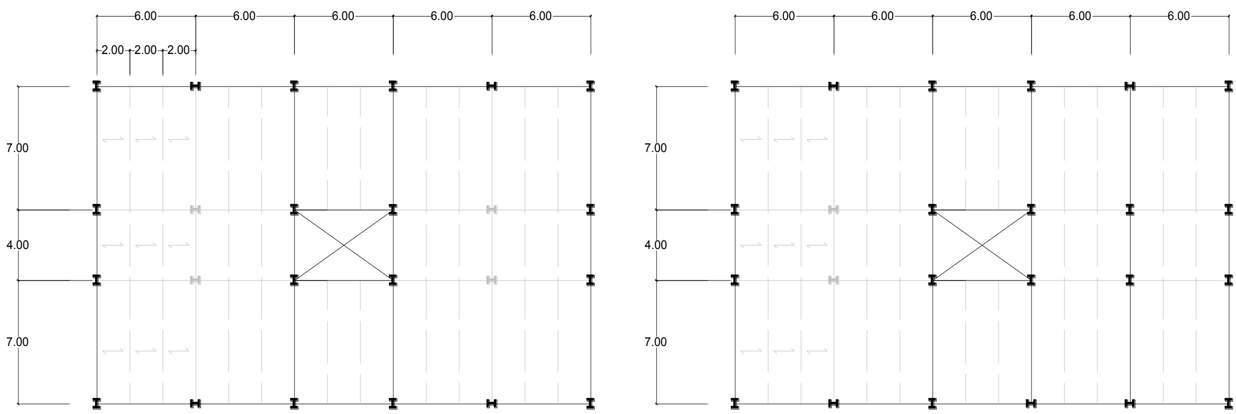

Figure 2- Laterally restrained regular & irregular structures (structures 3&4)

1.2 GROUP 2 - Structures with unstable hysteretic behavior (CBF)

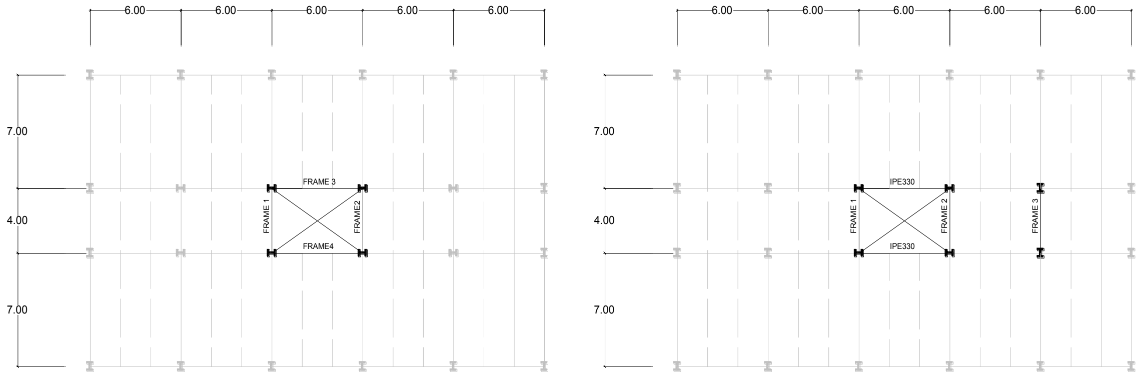

Figure 3- Laterally unrestrained regular & irregular structures (structure 5 & 6)

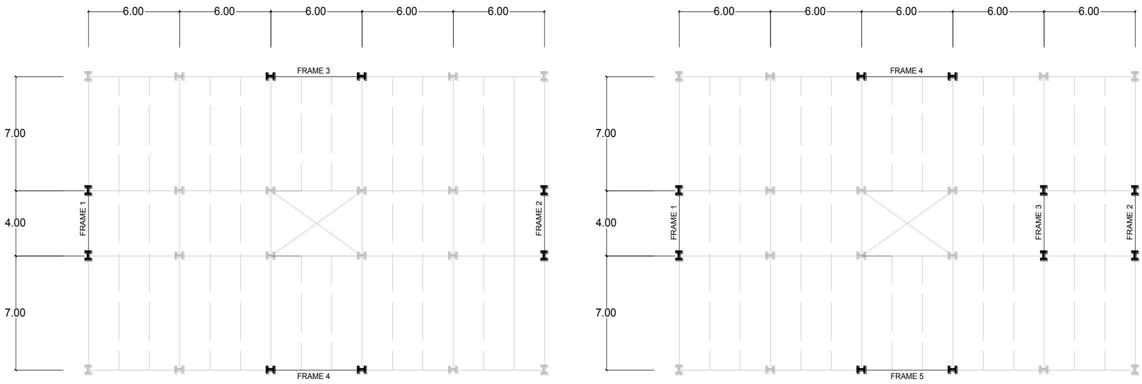

Figure.4- Laterally restrained regular & irregular structure (structure 7&8)

2 MATERIALS

Table.1– Materials properties

Materials properties	
Modulus of elasticity, E	210 GPa
Yield strength, f_y	275 MPa
Ultimate strength, f_u	430 MPa
Poisson coefficient, υ	0,3

3 LOADS

3.1 Static Loads

Table.1– Static unit loads

	Dead Loads- G $[kN/m^2]$	*Live Loads- Q* $[kN/m^2]$
Slab self weight	2,93	2,00
Finishings	1,00	

Note: The wind load is not considered in the design

3.2 Seismic Loads

As prescribed in Part1 of Eurocode 8 “the ground motion at given point on the surface is represented by an elastic ground acceleration response spectrum”,

It was assumed a Type 1 response spectra and soil type B: S =1,2; T_B =0,15; T_C =0,50 and T_D =2,0.

The design response spectrum, $S_d(T)$, is obtained from the elastic response spectrum by dividing it by the behaviour factor (q).

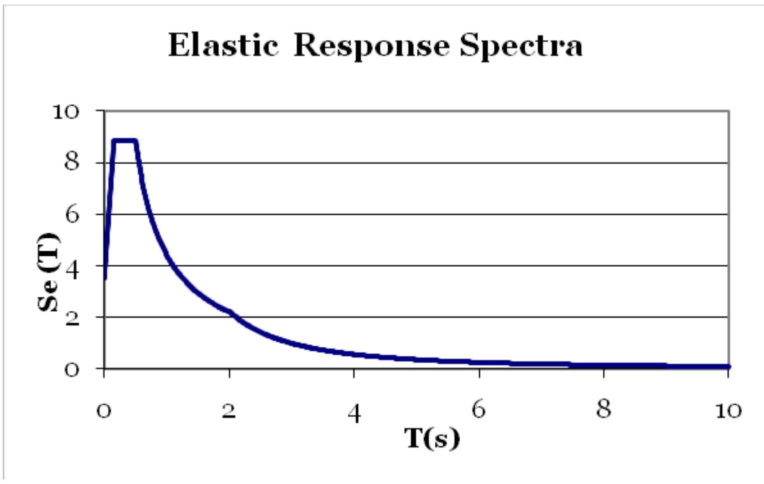

Figure.1- Elastic Response Spectra (Type 1; Soil type B; PGA=0,30g)

4 LOAD COMBINATIONS

The design load combinations considered are related with the ultimate and serviceability limit states, as recommended by ECO [CEN, 2002]:

Ultimate limit state combinations (ULS)

- Persistent and transient design situation

$$F = \gamma_G \cdot G_k + \gamma_Q \cdot Q_k \tag{4.1}$$

with γ_G=1,35 and γ_Q=1,5.

- Seismic combination

$$F_d = G_k + \gamma_E E_{Ek} + \psi_2 \cdot Q_k \tag{4.2}$$

with γ_I =1,00 and ψ_2=1,0 or ψ_2=0,0 (if the last floor is the roof).

Serviceability limit state combination (SLS)

$$F_d = G_k + Q_k \tag{4.3}$$

5 STATIC DESIGN

The static design procedure followed the next sequence of steps:

1. **Selection of initial columns and beams sections**

   The required columns and beams sections are selected based on a preliminary analysis of the vertical loading.

2. **Structural analysis**

   The second order effects, due to lateral displacement as result of the vertical loading, do not need to be incorporated in the structural analysis if $\alpha_{cr} \geq 10$. However, if $\alpha_{cr} \geq 3.0$ a first order analysis can also be performed through the amplification of the horizontal loading.

   The amplification factor is that given by:

$$\frac{1}{1-\dfrac{1}{\alpha_{cr}}} \tag{5.1}$$

   The parameter α_{cr} can be computed through the expression:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right) \cdot \left(\frac{h}{\delta_{H,Ed}}\right) \tag{5.2}$$

where

H_{Ed} is the reaction in the base of the storey to the horizontal loads applied to the structure;

V_{Ed} is the total vertical load applied to the structure;

$\delta_{H,Ed}$ is the horizontal displacement due to the horizontal loads applied to the structure;

h is the storey height.

The equivalent horizontal loads due to the effect of imperfections, H_{Ed}, are given as the product of the vertical loads, V_{Ed} applied to the structure and the equivalent geometric imperfection, ϕ:

$$H_{Ed} = \phi \cdot V_{Ed} \, . \tag{5.3}$$

The equivalent geometric imperfection considered in the global analysis, that leads to lateral displacements and consequently to second order effects, was calculated using the following expression, given in Eurocode 3:

$$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m \tag{5.4}$$

where

$\theta_0 = 1/200$

$\alpha_h = 2/\sqrt{h}$ is the reduction coefficient associated with the storey height $\left(2/3 \leq \alpha_h \leq 1.0\right)$

$\alpha_h = \sqrt{0.50 \cdot \left(1 + \frac{1}{m}\right)}$ is reduction coefficient associated to the number of columns in each storey. The parameter m represents the number of columns in each storey that are subjected to an axial force equal or higher than 50% of mean value for column in the vertical plan considered.

The member imperfections are accounted for in the individual stability member checks, as prescribed in Section 6.3 of EC3.

3. ULS checks

After computing the real actions applied to the structure, the selected member sections have to satisfy the ULS strength requirements in terms of cross section and member stability (Sections 6.2 and 6.3 of EC3, respectively).

<u>Strength requirements</u>

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \tag{5.5}$$

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \tag{5.6}$$

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \tag{5.7}$$

$$M_{Ed} \leq M_{N,Rd} \tag{5.8}$$

where N_{Ed} , M_{Ed} and V_{Ed} are the design axial force, bending moment and shear, respectively;

$N_{c,Rd}$, $M_{c,Rd}$, $V_{c,Rd}$ and $M_{N,Rd}$ are the design resistances computed as following (Sections 6.2.4, 6.2.5,6.2.5 and 6.2.8, respectively):

$N_{c,Rd} = \dfrac{A \cdot f_y}{\gamma_{M0}}$ for class 1,2 and 3 cross-sections

$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v \cdot f_y / \sqrt{3}}{\gamma_{M0}}$$

$M_{c,Rd} = M_{pl,Rd} = \dfrac{w_{pl} \cdot f_y}{\gamma_{M0}}$ for class 1 and 2 sections

$M_{N,y,Rd} = M_{pl,y,Rd} \cdot (1-n)/(1-0.5 \cdot a)$; $M_{N,y,Rd} \leq M_{pl,y,Rd}$ ·

$n \leq a : M_{N,z,Rd} = M_{pl,z,Rd}$

$n \geq a : M_{N,z,Rd} = M_{pl,z,Rd}$

With $n = N_{Ed} / N_{pl,Rd}$; $a = \left(A - 2 \cdot b \cdot t_f\right) / A$ and $a \leq 0.50$;

γ_{M0}=1,0

<u>Stability requirements</u>

Elements subjected to compression should satisfy the following expression:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{5.9}$$

where

N_{Ed} and $N_{b,Rd}$ are the design axial load and design bucking resistance, respectively, given by

$N_{b,Rd} = \dfrac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$ (for class 1,2 and 3 cross-sections)

$\chi = \dfrac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}}, (\chi \leq 1.0)$ $\Phi = 0.5\left[1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right]$

$\overline{\lambda} = \lambda / \lambda_1$

$\lambda = l_e / i$

$\lambda_1 = \pi \cdot \sqrt{\dfrac{E}{f_y}}$

γ_{M1}=1,0

Elements subjected to bending moment :

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \qquad (5.10)$$

where M_{Ed} and $M_{b,Rd}$ are the design bending moment and the bucking bending moment resistance, respectively. The buckling resistance for cross-sections class 1 and 2 can be computed through the expression $M_{b,Rd} = \dfrac{\chi_{LT} \cdot W_{pl} \cdot f_y}{\gamma_{M1}}$.

It was assumed that appropriated measures were adopted in order to avoid lateral torsional buckling of structural elements, therefore both beams and columns were considered laterally restrained $(\chi_{LT} = 1.0)$.

Members of class 1 and 2 cross-sections subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1.0 \qquad ((5.11)$$

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1.0 \qquad (5.12)$$

where

N_{Ed}, $M_{y,Ed}$, and $M_{z,Ed}$ are the design axial load and bending moments;

N_{Rk}, $M_{y,Rk}$, and $M_{z,Rk}$ are the design axial load and bending moments resistences;

χ_y, χ_z are the reduction factors due to buckling in compression;

χ_{LT} is the reduction factor due to lateral buckling;

k_{yy}, k_{yz}, k_{zz}, k_{zy} are the interaction factors, computed as indicated in Annex B.

4. **SLS checks**

The SLS's vertical deformation consists of the following check as prescribed in Section 3.4 of EC:

$$\delta_{\max} < \frac{L}{250} \tag{5.13}$$

Where δ_{mac} is total vertical deflection and L is the beam span.

6 SEISMIC DESIGN

The seismic design procedure followed the IFBD procedure (Improved Forced Based Design) proposed by Villani [2009].

The sequence of design steps of the IFBD procedure is the following:

1. **Selection of the lateral resisting system and static design for gravity loads;**

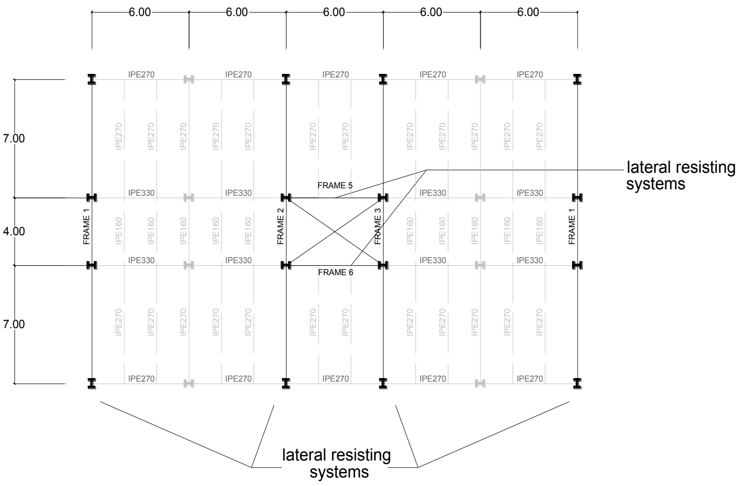

Figure 1- Plan view of the torsionally unrestrained and regular structure

2. **Determination of the seismic elastic forces based on the structure's fundamental period:**

$$V_e = \lambda \cdot m \cdot S_e(T_1) \tag{6.1}$$

where λ is the correction factor (0,85), m is the mass of the system, T_1 is the fundamental period of vibration of the system and $S_e(T_1)$ is the ordinate of the elastic response spectrum;

3. **Serviceability Limit States (SLS) interstorey drift checks and eventual increase of the structural stiffness.** For buildings having ductile structural elements:

$$d_r \cdot \upsilon \leq 0{,}01h \tag{6.2}$$

Where d_r is the inter-storey drift, h is the storey drift and υ is the reduction factor that takes into account the lower return period of the seismic action associated with the damage limitation requirement (it is recommended 0,5 for buildings of classes I and II).

4. **Evaluation of the behaviour factor, 'q'.**

The behaviour factor can be calculated if the the design force (V_d) is intentionally set to be equal to the first yield base shear (V_{1y}):

$$q = \frac{V_{el}}{V_d} = \frac{V_{el}}{V_{1y}} \cdot \frac{V_{1y}}{V_d} = \frac{V_{el}}{V_{1y}} \tag{6.3}$$

where

V_{el} is the elastic seismic force obtained from the elastic response spectrum;

V_y is lateral capacity of the structure;

V_{1y} is lateral force reached at the formation of the first plastic hinge in the structure;

V_d is lateral force considered in the design process;

The first plastic hinge occurs when the bending moment due to seismic action, plus the bending moment due to the gravity loads become equal to the plastic moment of the element under consideration:

$$M_{pl,Rd} = \lambda \cdot M(E_{Ek}) + M(G + \psi_2 \cdot Q_k) \tag{6.4}$$

where :

$M_{pl,Rd}$ is the plastic moment;

$M(E_{Ek})$ is the moment due to a unit seismic action;

λ is the load factor;

$M(G+\psi_2 \cdot Q_k)$ is the moment due to the gravity loads.

5. **Determination of the seismic design forces followed by elastic structural analysis** (in order to get moment, internal forces and displacements):

$$V_d = \lambda \cdot m \cdot S_d(T_1) \tag{6.5}$$

where λ is the correction factor (0,85), m is the mass of the system, T_1 is the fundamental period of vibration of the system and $S_d(T_1)$ is the ordinate of the design response spectrum;

6. **P-Δ checks and possible amplification of the seismic design base shear**:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0.10 \tag{6.6}$$

If $0.10 \leq \theta \leq 0.20$ the second order effects may be approximately taken into account by multiplying the relevant seismic actions effects by the factor $1/(1-\theta)$.

7. **Ultimate Limit State (ULS) checks for the final set of seismic forces.**

Moment Resisting Frames (Group 1): to check ULS, all members should satisfy equations (5.5) to (5.12) .

Concentric Braced Frames (Group 2)

Braces

As recommend by EC8 , braces should satisfy the following of resistance and have their non-dimensional slenderness limited:

$$N_{Ed} \leq N_{pl,Rd} \tag{6.7}$$

$$1.3 \leq \bar{\lambda} \leq 2.0 \tag{6.8}$$

where

N_{Ed} is the design axial force;

$N_{pl,Rd}$ is the design resistance computed as indicated by EC3;~

$\bar{\lambda}$ is the non-dimensional slenderness.

Beams & Columns

The beams and columns should be checked to remain elastic in order to ensure that dissipative behavior is located in the braces. According to Section 6.7.4 of EC8 the design forces are obtained using the following combination:

$$Ed = E_{d,Gk+0,3Qk} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot E_{d,E} \qquad (6.9)$$

where

γ_{ov} is the overstrength factor which is equal to 1.25;

Ω is given by $\Omega = \min\left(\dfrac{N_{pl,rd,i}}{N_{Ed,i}}\right)$.

After computing the beams and columns design forces, as indicated above, equations (5.5) to (5.12) have to be satisfied.

REFERENCES

CEN [2002] *EN1990, Eurocode 0: Basis of structural design,* European Committee for Standardization, Brussels, Belgium.

CEN [2004] *EN1998-1-3, Eurocode 8: Design of structures for earthquake resistance- Part 1: general rules, seismic actions and rules for buildings*, European Committee for Standardization, Brussels, Belgium.

CEN [2005] *EN1998-1-1, Eurocode 3: Design of steel structures - Part 1: general rules, seismic actions and rules for buildings*, European Committee for Standardization, Brussels, Belgium.

APPENDIX- SUMMARY OF THE DESIGN PROCEDURES

STRUCTURE 1- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

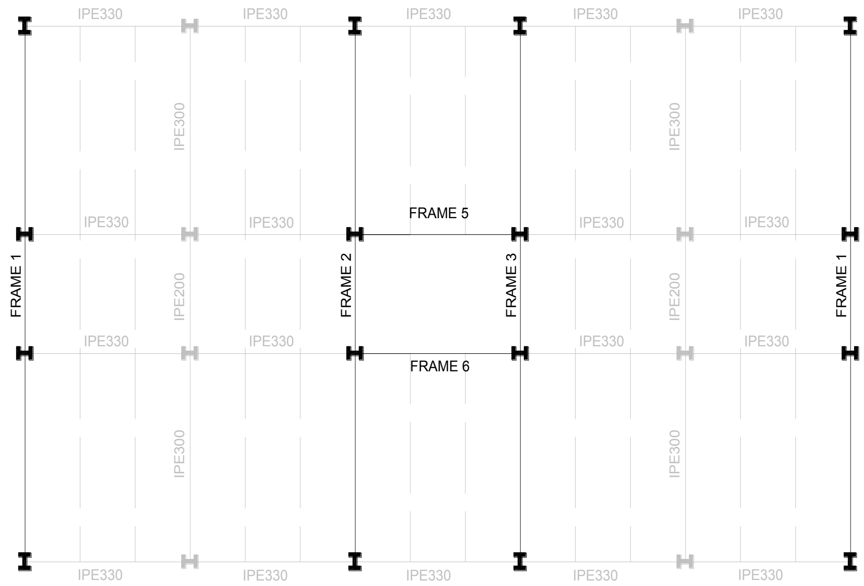

Figure1.1- Torsionally restrained regular structure (structure 1)

Table 1.1– Sections obtained from Static Design

	COLUMNS		BEAMS	
	EXTERNAL	INTERNAL	EXTERNAL	INTERNAL
FRAME 1&4	HEB140	HEB140	IPE 220	IPE 160
FRAME 2&3	HEB180	HEB180	IPE 270	IPE 160
FRAME 5&6	HEB180		IPE270	

Table 1.2– Adopted Sections (Seismic Design)

	COLUMNS		BEAMS	
	EXTERNAL	INTERNAL	EXTERNAL	INTERNAL
FRAME 1&4	HEB180	HEB320	IPE 270	IPE 270
FRAME 2&3	HEB180	HEB320	IPE 270	IPE 270
FRAME 5&6	HEB320		IPE330	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	($\pm$) 1.5	e_{ay} (m)	($\pm$)0.9

Table 2.2–Center of Mass (CM), Center of Stiffness (CR) and Accidental Eccentricities (e_a)

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 3 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	0.00	**-1.50**	-1.50	1.50	1.50
y_i	0.00	0.00	**-0.9**	0.9	-0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.54	0.68	0.55	0.68
$S_e(m/s^2)$	8.175	6.492	8.026	6.492
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1688.18	1340.61	1657.49	1340.61

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS chekcs	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.076	0.038	0.045
Direction xx	4.5	0.082	0.041	0.045

5 Evaluation of the behaviour factor. 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behavior factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6
T(s)	0.55				0.68	
$S_e(m/s^2)$	8.026				6.492	
V^e(kN)	1657.49				1340.61	
V_{ei}(kN)	619.93	293.65	277.34	466.58	662.83	677.79
V_y(kN)	839.48				531.41	
q	1.97				2.52	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6
T(s)	0.55				0.68	
q	2.50				3.00	
$S_d(T)$	4.013				2.102	
V_d(kN)	278.9	127.44	115.92	174.12	226.88	231.04

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	696.38	2025.82	4.50	0.030	0.08	0.05
Direction xx	696.38	2025.82	4.50	0.027	0.07	0.04

7 Ultimate Limit State checks

Equations (5.5) to (5.12) have to be satisfied.

STRUCTURE 2- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

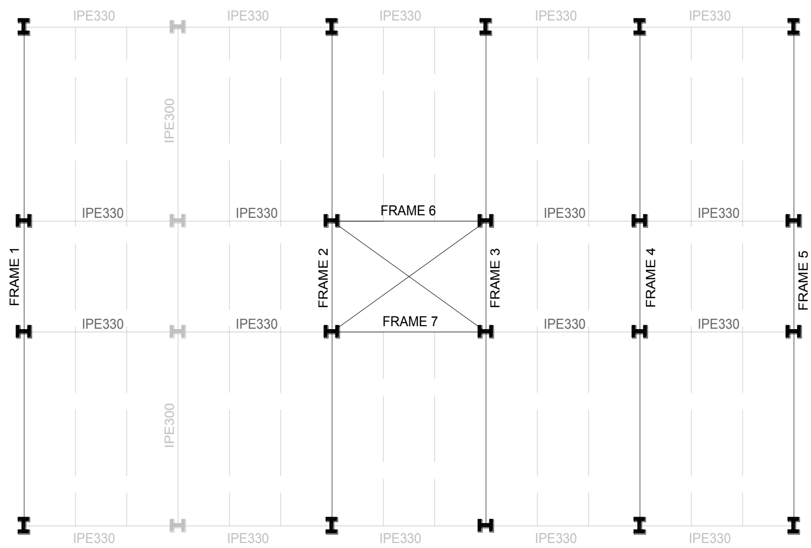

Figure1.1- Torsionally restrained regular structure (structure 2)

Table 1.1– Sections obtained from Static Design

	COLUMNS		BEAMS	
	EXTERNAL	INTERNAL	EXTERNAL	INTERNAL
FRAME 1&5	HEB140	HEB180	IPE 220	IPE 160
FRAME 2&3	HEB140	HEB180	IPE 270	IPE 160
FRAME 4	HEB140	HEB240	IPE 270	IPE 160
FRAME 5&6	HEB180		IPE270	

Table 1.2– Adopted Sections (Seismic Design)

	COLUMNS		BEAMS	
	EXTERNAL	INTERNAL	EXTERNAL	INTERNAL
FRAME 1	HEB240	HEB280	IPE 270	IPE 160
FRAMES 2&3	HEB140	HEB320	IPE 270	IPE 270
FRAME 4	HEB140	HEB320	IPE 270	IPE 270
FRAME 5	HEB140	HEB280	IPE 270	IPE 160
FRAME 5&6	HEB220		IPE300	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	($\pm$) 1.5	e_{ay} (m)	($\pm$)0.9

Table 2.2–Center of Mass (CM). Center of Stiffness (CR) and Accidental Eccentricities (e_a)

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 3 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	0.12	**-1.62**	-1.62	1.62	1.62
y_i	0.00	0.00	**-0.9**	0.9	-0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.63	0.58	0.65	0.58
$S_e(m/s^2)$	7.01	7.61	6.79	7.61
M(ton)	206.51	206.51	206.51	206.51
Ve(kN)	1447.01	1571.75	1402.49	1571.75

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

	h (m)	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.1000	**0.050**	0.045
Direction xx	4.5	0.0792	0.038	0.045

5 Evaluation of the behaviour factor, ‘q’. followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behaviour factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7
T(s)	0.65					0.58	
$S_e(m/s^2)$	6.79					7.61	
V_e(kN)	1402.49					1571.75	
V_{ei} (kN)	413	328	276	248	138	742	766
V_y (kN)	937.87					528.90	
q	1.50					2.97	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7
T(s)	0.65					0.58	
q	2.00					3.50	
$S^e(m/s^2)$	3.40					2.17	
V_d(kN)	260.24	158.32	131.72	117.38	71.80	220.06	241.62

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	739.46	2025.82	4.50	0.050	0.099	0.06
Direction xx	461.68	2025.82	4.50	0.025	0.050	0.05

7 Ultimate Limit State checks

Equations (5.5) to (5.12) have to be satisfied.

8 SUMMARY OF THE DESIGN PROCEDURE (structures 3&4)

Both structures are **torsionally stiff** since the first two modes of vibration are translation in x and y directions. The design is governed by the seismic SLS checks, particularly in the case of the irregular structure, where it was observed that the only way to reduce the storey drift was to increase frame 1 sections in order to reduce the inherent eccentricity introduced by frame 4.

STRUCTURE 3- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

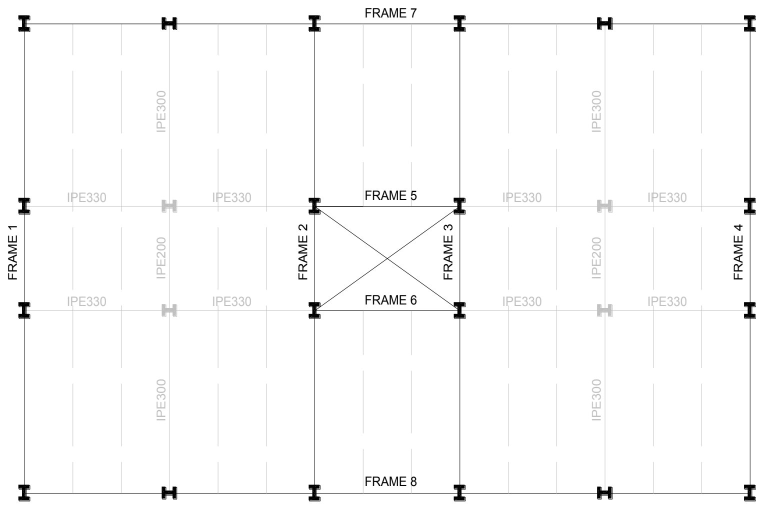

Figure1.1- Torsionally restrained regular structure (structure 3)

Table 1.1– Sections obtained from Static Design

	COLUMNS		BEAMS	
	External	Internal	External	Internal
Frames 1&4	HEB 140	HEB 140	IPE 220	IPE 160
Frames 2&3	HEB 140	HEB 180	IPE 270	IPE 160
Frames 5&6	HEB 180		IPE 300	
Frames7&8	HEB 140	HEB 180	IPE 300	IPE 270

Table 1.2– Adopted Sections (Seismic Design)

	COLUMNS		BEAMS	
	External	Internal	External	Internal
Frames 1&4	**HEB 180**	**HEB 180**	IPE 270	IPE 160
Frames 2&3	**HEB 240**	**HEB 240**	IPE 270	IPE 160
Frames 5&6	**HEB 240**		IPE 300	
Frames7&8	**HEB 180**	**HEB 240**	IPE 300	IPE 270

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	($\pm$) 1.5	e_{ay} (m)	($\pm$) 0.9

Table 2.2–Center of Mass (CM). Center of Stiffness (CR) and Accidental Eccentricities (e_a)

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 3 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	0.00	**-1.50**	-1.50	1.50	1.50
y_i	0.00	0.00	**-0.9**	0.9	-0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.66	0.72	0.67	0.72
$S_e(m/s^2)$	6.689	6.131	6.589	6.131
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1381.24	1266.14	1360.62	1266.14

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS chekcs	**h (m)**	**dr (m)**	**dr.v**	**Limit(m)**
Direction yy	4.5	0.0975	**0.049**	0.045
Direction xx	4.5	0.044	0.022	0.045

5 Evaluation of the behaviour factor, 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behavior factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7	FRAME 8
T(s)	0.66				0.72			
$S_e(m/s^2)$	6.689				6.131			
V_e(kN)	1381.24				1266.14			
V_{ei}(kN)	225.35	465.27	465.27	225.35	138.68	138.68	494.39	494.39
V_y (kN)	881.61				724.30			
q	1.57				1.75			

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7	FRAME 8
T(s)	0.67				0.72			
q	2.00				2.50			
$S_d(m/s^2)$	3.294				2.453			
V_d(kN)	142.1	241.26	214.7	80.96	59.54	61.36	198.68	224.86

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction xx	679.02	2025.82	4.50	0.049	0.097	0.06
Direction yy	544.44	2025.82	4.50	0.036	0.071	0.06

7 Ultimate Limit State checks

Equations (5.5) to (5.12) have to be satisfied.

STRUCTURE 4- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

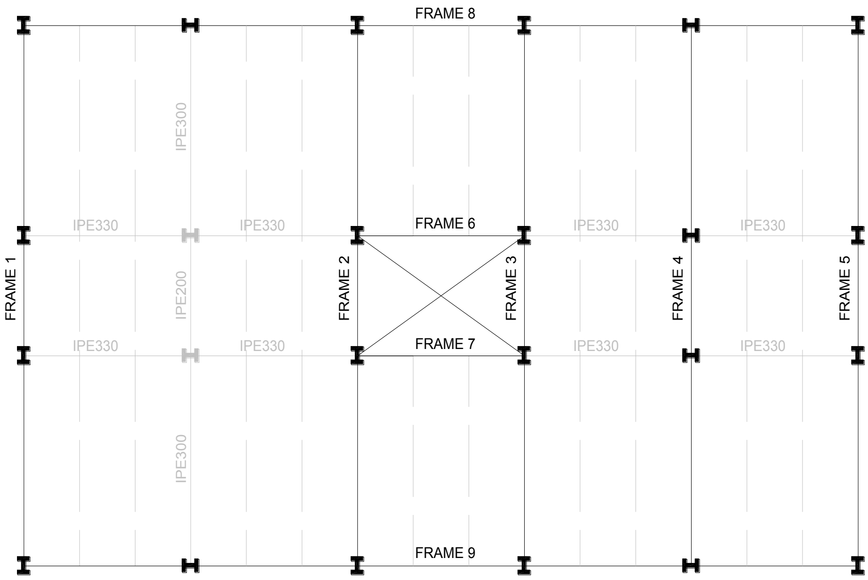

Figure1.1- Torsionally restrained irregular structure (structure 4)

Table 1.1– Sections obtained from Static Design

	COLUMNS		BEAMS	
	External	Internal	External	Internal
Frames 1&5	HEB 140	HEB 140	IPE 220	IPE 160
Frames 2&3	HEB 140	HEB 180	IPE 270	IPE 160
Frame 4	HEB 180	HEB 180	IPE 270	IPE 160
Frames 6&7	HEB 180		IPE 300	
Frames8&9	HEB 140	HEB 180	IPE 300	IPE 270

Table 1.2– Adopted Sections (Seismic Design-1st option)

	COLUMNS		BEAMS	
	External	Internal	External	Internal
Frames 1&5	**HEB 180**	**HEB 180**	IPE 270	IPE 160
Frames 2.3&4	**HEB 240**	**HEB 240**	IPE 270	IPE 160
Frames 6&7	**HEB 240**		IPE 300	
Frames8&9	**HEB 180**	**HEB 240**	IPE 300	IPE 270

Table 1.3– Adopted Sections (Seismic Design- 2nd option)

	COLUMNS		BEAMS	
	External	Internal	External	Internal
Frames 1	HEB 180	**HEB 240**	IPE 270	IPE 160
Frames 2&3	HEB 240	HEB 240	IPE 270	IPE 160
Frames 4	HEB 240	HEB 240	IPE 270	IPE 160
Frames 5	HEB 180	HEB 180		
Frames 6&7	HEB 240		IPE 300	
Frames8&9	HEB 180	HEB 240	IPE 300	IPE 270

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	($\pm$) 1.5	e_{ay} (m)	($\pm$)0.9

Table 2.2– Center of Mass (CM), Center of Stiffness (CR) and Accidental Eccentricities (e_a)

Coordinates (m)	CM	(1st option) CR (w/out e_a)	(2nd option) CR (w/out e_a)	CASE 1 e_a	CASE 3 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	1.83	0.73	**-2.23**	0.77	**-2.23**	0.77
y_i	0.00	0.00	0.00	**-0.9**	-0.9	0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.62	0.71	0.60	0.71
$S_e(m/s^2)$	7.120	6.218	7.358	6.218
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1470.35	1283.97	1519.36	1283.97

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1- 1st option)

SLS chekcs	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.106	**0.053**	0.045
Direction xx	4.5	0.089	0.045	0.045

Table 4.2– SLS checks (CASE 1-2nd option)

SLS chekcs	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.091	**0.046**	0.045
Direction xx	4.5	0.0863	0.043	0.045

5 Evaluation of the behaviour factor, 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behaviour factors in x and y directions (CASE 1-2nd option)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7	FRAME 8	FRAME 9
T(s)	0.60					0.71			
Se(m/s2)	7.358					6.218			
Ve(kN)	1519.36					1283.97			
Vei(kN)	369.08	399.83	369.08	246.05	135.33	178.22	177.96	448.17	479.62
Vy (kN)	971.57					757.12			
q	1.56					1.70			

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1-2nd option)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5	FRAME 6	FRAME 7	FRAME 8	FRAME 9
T(s)	0.60					0.71			
q	2.00					2.50			
Sd(T)	3.679					2.487			
Vd(kN)	208.34	213.66	182.74	116.50	62.40	73.30	74.76	201.04	219.02

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1-2nd option)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	845.04	2025.82	4.50	0.045	0.091	0.05
Direction xx	628.25	2025.82	4.50	0.035	0.069	0.05

7 Ultimate Limit State checks

Equations (5.5) to (5.12) have to be satisfied.

8 SUMMARY OF THE DESIGN PROCEDURE (structures 3&4)

The structures 3 and 4 are also **torsionally stiff** since their first two modes of vibration are translational in both directions.

Conversely to structures 1 and 2, structures 3 and 4 are torsionally restrained. For this reason it was possible to decrease the internal columns sections (HEB 320 to HEB 240).

The design was also governed by the serviceability drift limits imposed by EC8. As before, it was observed that the only way to reduce the storey drift it to reduce the introduced accidental (regular structure) or inherent (irregular structure) eccentricities.

In the limit, it would mean to lose regularity criteria, in the case of the regular structure, and eliminate the irregularities (introduced by considering frame 4), in the case of the irregular structure.

The 1st and 2nd options presented for structure 4 illustrate the above mentioned. The reduction of the inherent eccentricity from 1.83 to 0.73 reduces the story drift from 0.53 to 0.46.

STRUCTURE 5- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

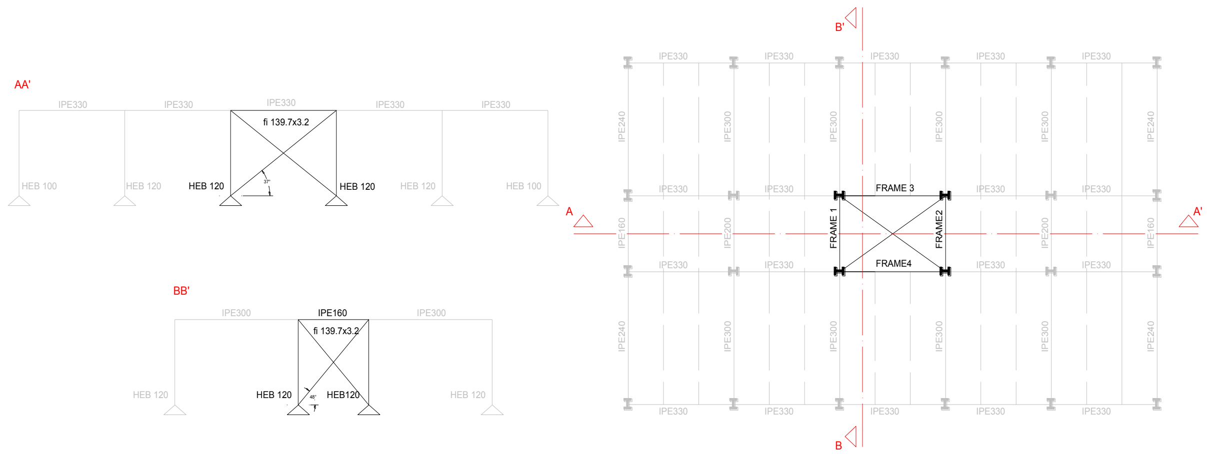

Figure1.1- Torsionally unrestrained regular structure (structure 5)

Table 1.1– Sections obtained from Static Design/ Sections Adopted

	STATIC DESIGN		SEISMIC DESIGN	
	Columns	Beams	Columns	Beams
Frames 1&2	HEB120	IPE 160	**HEB140**	IPE 160
Frames 3&4	HEB120	IPE 330	**HEB140**	IPE 330
Braces	C H S 139.7 x 3.2		C H S 139.7 x 3.2	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	(±) 1.5	e_{ay} (m)	(±)0.9

Table 2.2– Center of Mass (CM) , Center of Stiffness (CR) with and without Accidental Eccentricities

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 2 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	0.00	**-1.50**	-1.50	1.50	1.50
y_i	0.00	0.00	**-0.9**	0.9	-0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.48	0.42	0.48	0.41
$S_e(m/s^2)$	8.829	8.829	8.829	8.829
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1823.23	1823.23	1823.23	1823.23

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS chekcs	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.082	0.041	0.045
Direction xx	4.5	0.050	0.025	0.045

5 Evaluation of the behaviour factor, ‘q’, followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behavior factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4
T(s)	0.48		0.41	
$S_e(m/s^2)$	8.83		8.83	
V_e(kN)	1823.23		1823.23	
V_{ei}(kN)	1207.78	627.07	760.08	1058.58
V_{yi}(kN)	488.80		670.13	
q(i)	3.73		2.72	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4
T(s)	0.48		0.41	
q_{adopt}	4.50		3.00	
$S_d(T)$	2.207		2.943	
V_d(kN)	224.22	257.40	313.57	297.13

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	481.61	2025.82	4.50	0.0223	0.100	0.09
Direction xx	610.70	2025.82	4.50	0.0164	0.049	0.04

7 Ultimate Limit State checks

7.1 Braces design checks

Table 7.1- Brace design checks (Case 1)

Braces	N_{ed} (kN)	$N_{pl.Rd}$(kN)	**Ω**	$\bar{\lambda}$	Ω_{min}
Frame 1	-335.09	486.35	1.45	1.829	**1.24**
Frame 2	-384.67	486.35	1.26	1.829	
Frame 3	-392.63	486.35	1.24	0.000	
Frame 4	-372.05	486.35	1.31	0.000	

7.2 Beams & Columns design Checks

The beams and columns should be checked to remain elastic in order to ensure that dissipative behavior is located in the braces. After computing the beams and columns design forces equations (5.5) to (5.12) have to be satisfied.

STRUCTURE 6- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

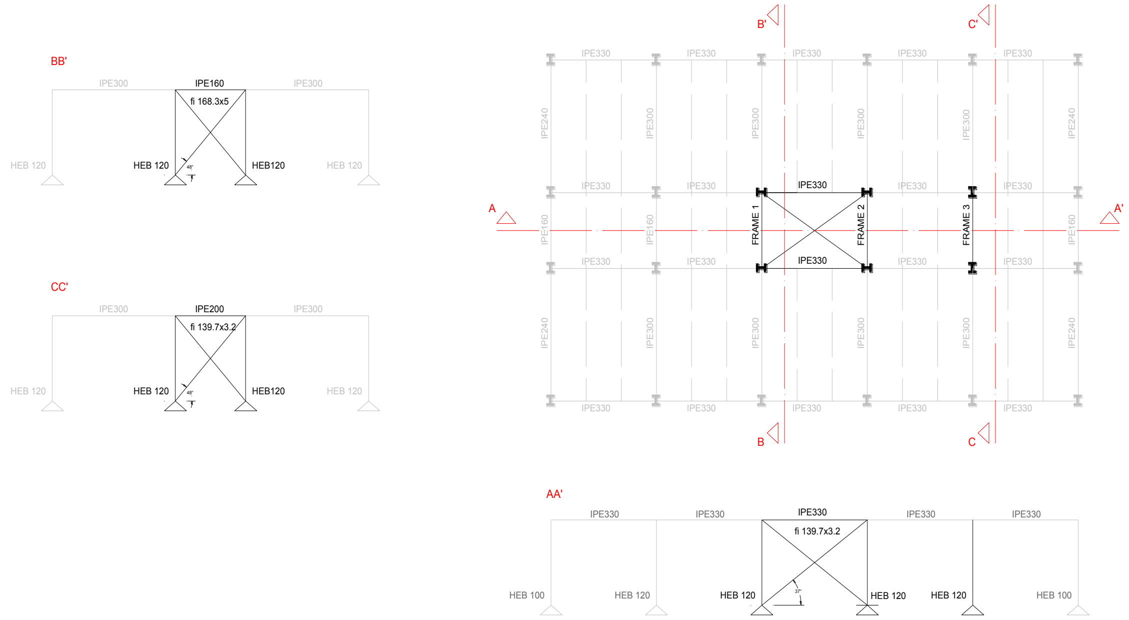

Figure1.1- Torsionally unrestrained irregular structure (structure 6)

Table 1.1– Sections obtained from Static Design/ Sections Adopted

	STATIC DESIGN		SEISMIC DESIGN	
	Columns	Beams	Columns	Beams
Frames 1&2	HEB120	IPE 160	**HEB140**	IPE 160
Frames 3	HEB120	IPE 200	**HEB140**	IPE 200
Frames 4&5	HEB120	IPE 330	**HEB140**	IPE 330
Braces 3. 4&5	C H S 139.7 x 3.2		C H S 139.7 x 3.2	
Braces 1&2	C H S 139.7 x 3.2		C H S 168.3 x 5	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	(±) 1.5	e_{ay} (m)	(±)0.9

Table 2.2– Center of Mass (CM). Center of Stiffness (CR) and Accidental Eccentricities

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 2 e_a	CASE 3 e_a	CASE 4 w/ e_a
x_i	0.00	2.69	**-4.19**	1.19	-4.19	1.19
y_i	0.00	0.00	**-0.9**	-0.9	0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Without Accidental eccentricities		With Accidental eccentricity(Case 1)	
	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.35	0.43	0.28	0.43
$S_e(m/s^2)$	8.83	8.83	8.83	8.83
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1823.23	1823.23	1823.23	1823.23

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS chekcs	**h (m)**	**dr (m)**	**dr.v**	**Limit(m)**
Direction yy	4.5	0.070	0.037	0.045
Direction xx	4.5	0.054	0.027	0.045

5 Evaluation of the behaviour factor, 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behaviour factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 3	FRAME 4
T(s)	0.28			0.43	
$S_e(m/s^2)$	8.829			8.83	
V_e(kN)	1823.23			1823.23	
V_{ei}(kN)	1251.70	586.81	3.23	873.66	946.46
V_y (kN)	883.01			749.51	
q	2.06			2.43	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4	FRAME 5
T(s)	0.28			0.43	
q	2.50			2.70	
$S_d(T)$	3.270			3.270	
V_d(kN)	373.47	256.33	222.83	363.58	364.72

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	852.64	2025.82	4.50	0.0277	0.069	0.04
Direction xx	728.30	2025.82	4.50	0.0199	0.054	0.03

7 Ultimate Limit State checks

7.1 Braces design checks

Table 7.1- Brace design checks (Case 1)

BRACES	Braces	N_{ed} (kN)	$N_{pl.Rd}$(kN)	**Ω**	$\overline{\lambda}$
Frame 1	-558.14	-912.35	1.63	1.227	**1.06**
Frame 2	-383.09	-912.35	2.38	1.227	
Frame 3	-366.04	-486.35	1.33	1.468	
Frame 4	-455.25	-486.35	1.07	1.829	
Frame 5	-456.67	-486.35	1.06	1.829	

7.2 Beams & Columns design Checks

After computing the beams and columns design forces equations (5.5) to (5.12) have to be satisfied.

8 SUMMARY OF THE DESIGN PROCEDURE (structures 5 &6)

Both structures are **torsionally flexible** since the 1^{st} mode of vibration is rotational.

The beams design was governed by the static serviceability limits states; the columns design was ruled by the criterion that beams and columns must remain elastic during the seismic event in order to ensure that the dissipative behavior is located in the brace; and braces design were governed by the resistance of the sections (frames 1 and 2 braces sections had to be increased comparatively to frames 3.4 and 5).

STRUCTURE 7- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

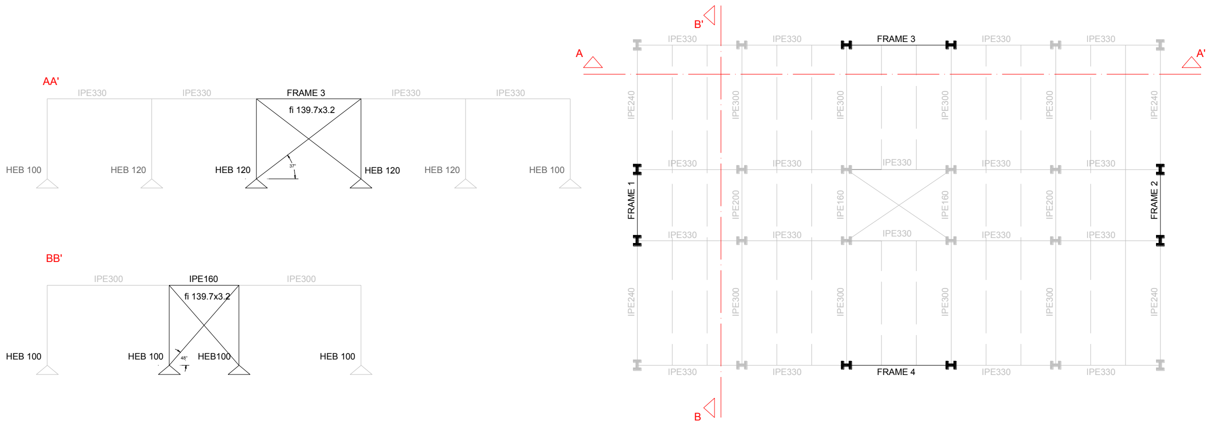

Figure1.1- Torsionally restrained regular structure (structure 7)

Table 1.1– Sections obtained from Static Design/ Sections Adopted

	STATIC DESIGN		SEISMIC DESIGN	
	Columns	Beams	Columns	Beams
Frames 1&2	HEB100	IPE 160	**HEB140**	IPE 160
Frames 3&4	HEB120	IPE 330	HEB120	IPE 330
Braces	C H S 139.7 x 3.2		C H S 139.7 x 3.2	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	($\pm$) 1.5	e_{ay} (m)	($\pm$)0.9

Table 2.2– Center of Mass (CM), Center of Stiffness (CR) and Accidental Eccentricities

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 2 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	0.00	**-1.50**	-1.50	1.50	1.50
y_i	0.00	0.00	**-0.9**	0.9	-0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Direction yy	Direction xx
T(s)	0.47	0.43
$S_e(m/s^2)$	8.829	8.829
M (ton)	206.51	206.51
V_e(kN)	1823.23	1823.23

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS chekcs	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.055	0.027	0.045
Direction xx	4.5	0.043	0.022	0.045

5 Evaluation of the behaviour factor, 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behaviour factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4
T(s)	0.47		0.43	
Se(m/s2)	8.829		8.83	
V_e(kN)	1823.23		1823.23	
V_{ei}(kN)	975.99	853.99	882.40	937.73
V_y (kN)	603.69		756.49	
q	3.02		2.41	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 4
T(s)	0.47		0.43	
q_{adop}	3.50		3.00	
$S_d(m/s^2)$	2.523		2.943	
V_d(kN)	292.58	247.05	299.87	324.12

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	539.63	2025.82	4.50	0.0156	0.055	0.05
Direction xx	623.99	2025.82	4.50	0.0143	0.043	0.03

7 Ultimate Limit State checks

7.1 Braces design checks

Table 7.1- Brace design checks (Case 1)

Braces	N_{ed} (kN)	$N_{pl.Rd}$(kN)	**Ω**	$\bar{\lambda}$	**Ω_{min}**
Frame 1	437.25	486.35	1.11	1.631	**1.11**
Frame 2	369.21	486.35	1.32	1.631	
Frame 3	-375.47	486.35	-1.30	2.032	
Frame 4	-405.84	486.35	-1.20	2.032	

7.2 Beams & Columns design Checks

After computing the beams and columns design forces equations (5.5) to (5.12) have to be satisfied.

STRUCTURE 8- SEISMIC DESIGN (3D)

1 Selection of the lateral resisting system

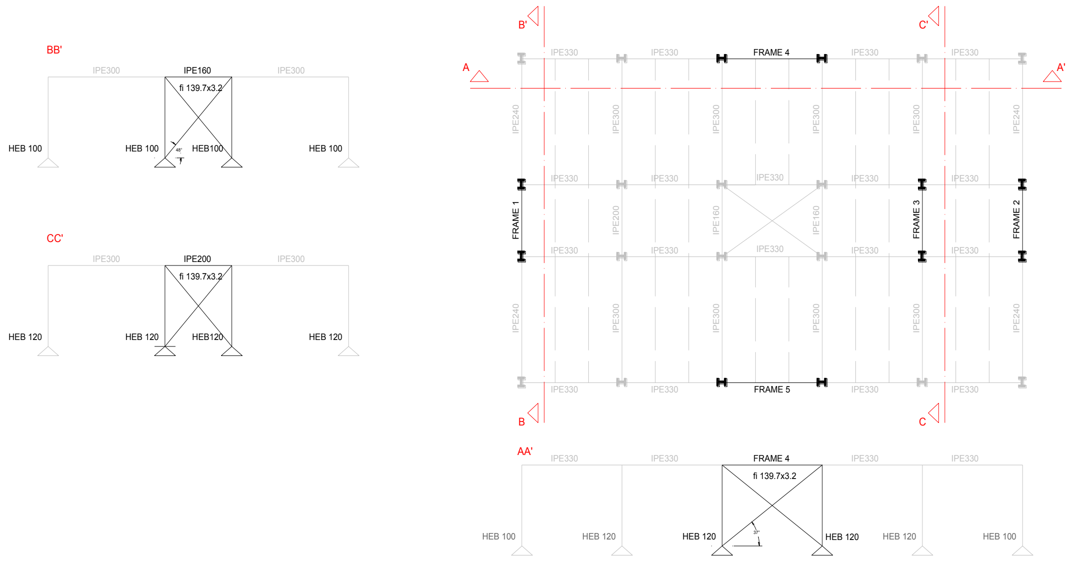

Figure1.1- Torsionally restrained irregular structure (structure 8)

Table 1.1– Sections obtained from Static Design/ Sections Adopted

	STATIC DESIGN		SEISMIC DESIGN	
	Columns	Beams	Columns	Beams
Frames 1&2	HEB100	IPE 160	**HEB140**	IPE 160
Frame 3	HEB120	IPE 200	HEB120	IPE 200
Frames 4&5	HEB120	IPE 330	HEB120	IPE 330
Braces	C H S 139.7 x 3.2		C H S 139.7 x 3.2	

2 Consideration of accidental eccentricities

2.1– Accidental eccentricities

ACCIDENTAL ECCENTRICITIES (e_a)			
Lx(m)	30	Ly(m)	18
e_{ax} (m)	(±) 1.5	e_{ay} (m)	(±)0.9

Table 2.2– Center of Mass (CM), Center of Stiffness (CR) and Accidental Eccentricities

Coordinates(m)	CM	CR (w/out e_a)	CASE 1 e_a	CASE 2 e_a	CASE 3 e_a	CASE 4 e_a
x_i	0.00	3.63	**-5.13**	2.13	-5.13	2.13
y_i	0.00	0.00	**-0.9**	-0.9	0.9	0.9

3 Modal analysis to determine periods of vibration in x and y directions and the elastic seismic forces

Table 3.1– Elastic Seismic Forces (with and without accidental eccentricities)

	Direction yy	Direction xx	Direction yy	Direction xx
T(s)	0.40	0.43	0.41	0.43
$S_e(m/s^2)$	8.829	8.829	8.829	8.829
M (ton)	206.51	206.51	206.51	206.51
V_e(kN)	1823.23	1823.23	1823.23	1823.23

4 Serviceability Limit State checks (drift limits are obtained from the spectral analysis considering q=1.0)

Table 4.1– SLS checks (CASE 1)

SLS checks	**h (m)**	**dr (m)**	**dr.ν**	**Limit(m)**
Direction yy	4.5	0.048	0.024	0.045
Direction xx	4.5	0.044	0.022	0.045

5 Evaluation of the behaviour factor, 'q', followed by spectral analysis (in order to get moment. internal forces and displacements)

Table 5.1– Estimation of the structure behaviour factors in x and y directions (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 3	FRAME 4
T(s)	0.41			0.43	
$S_e(m/s^2)$	8.829			8.83	
V_e(kN)	1823.23			1823.23	
V_{ei}(kN)	827.15	478.23	530.69	887.15	936.09
V_y(kN)	744.39			757.82	
q	2.45			2.41	

Table 5.2– Adopted behavior factors in x and y direction and design forces obtained from the spectral analysis (CASE 1)

	FRAME 1	FRAME 2	FRAME 3	FRAME 3	FRAME 4
T(s)	0.41			0.43	
q_{adop}	3.00			3.00	
$S_d(m/s^2)$	2.94			2.94	
V_d(kN)	301.58	151.14	177.05	309.98	330.20

6 P-Δ checks

Table 6.1- P-Δ Checks (Case 1)

P-Δ checks	H_{ed} (kN)	P_{tot} (kN)	H(m)	d_e (m)	dr=de*q	**θ**
Direction yy	629.76	2025.82	4.50	0.0160	0.048	0.03
Direction xx	640.18	2025.82	4.50	0.0146	0.044	0.03

7 Ultimate Limit State checks

7.1 Braces design checks

Table 7.1- Brace design checks (Case 1)

Braces	N_{ed} (kN)	$N_{pl.Rd}$(kN)	**Ω**	$\bar{\lambda}$ $\bar{\lambda}$	**Ω_{min}**
Frame 1	450.71	486.35	1.08	1.631	
Frame 2	225.87	486.35	2.15	1.631	
Frame 3	264.59	486.35	1.84	1.631	**1.08**
Frame 4	388.13	486.35	1.25	2.032	
Frame 5	413.46	486.35	1.18	2.032	

7.2 Beams & Columns design Checks

After computing the beams and columns design forces equations (5.5) to (5.12) have to be satisfied.

8 SUMMARY OF THE DESIGN PROCEDURE (structures 7 &8)

The structures are **torsionally stiff** since the 1st and 2nd modes of vibration are translation in y and x directions, respectively.

The beams design was governed by the static serviceability limits states; the columns design in y direction was ruled by the criterion that beams and columns must remain elastic during the seismic event in order to ensure that the dissipative behavior is located in the brace, while in x direction the sections size was determined by stability checks; braces design was imposed by the non-dimensional slenderness parameter.