Equaljoints Plus

Valorisation of knowledge for European pre-QUALified steel JOINTS

European Commission

Research Fund for Coal & Steel

Worked example

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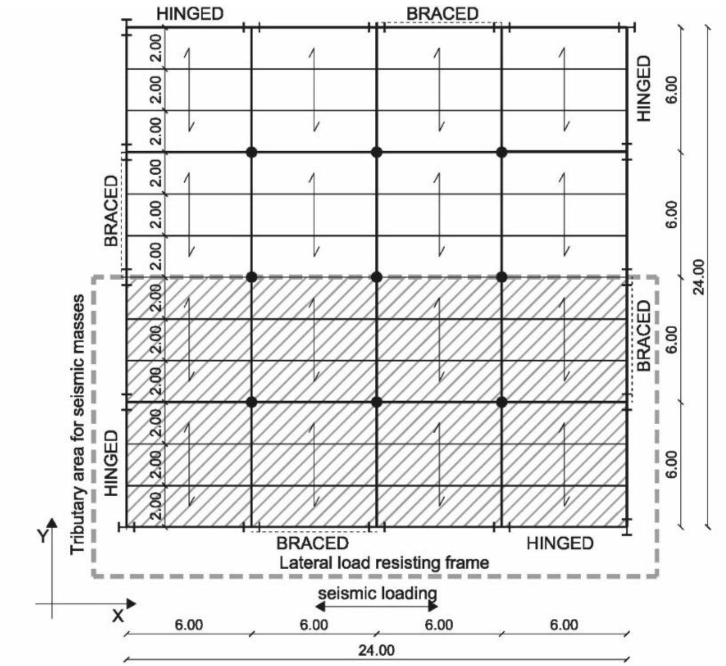
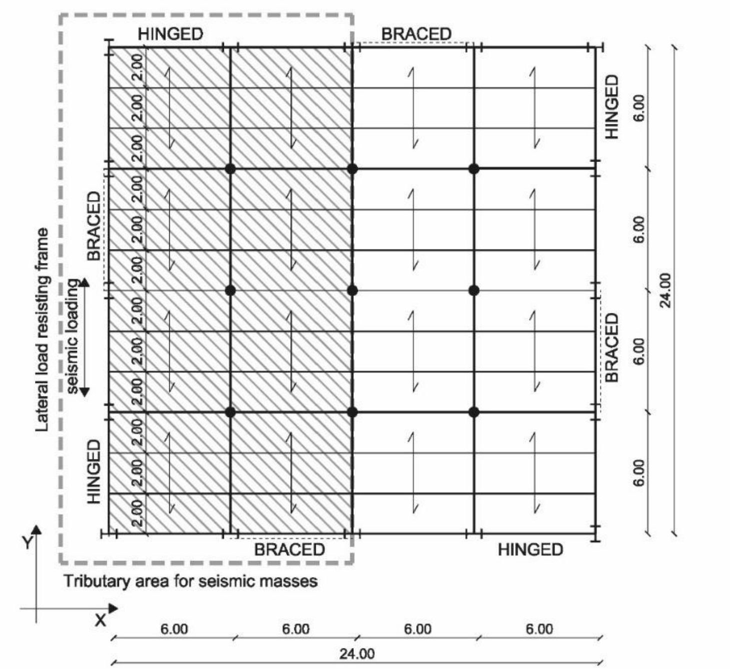
Contents

Structural configuration

- **Perimetral seismic resistant system**
- **Moment resisting frame (MRF)**



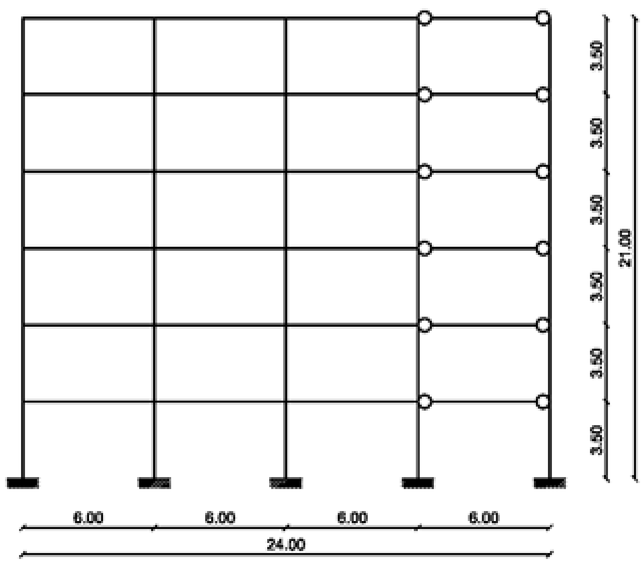
Plan configuration of the building with identification of the lateral load resisting system for X direction



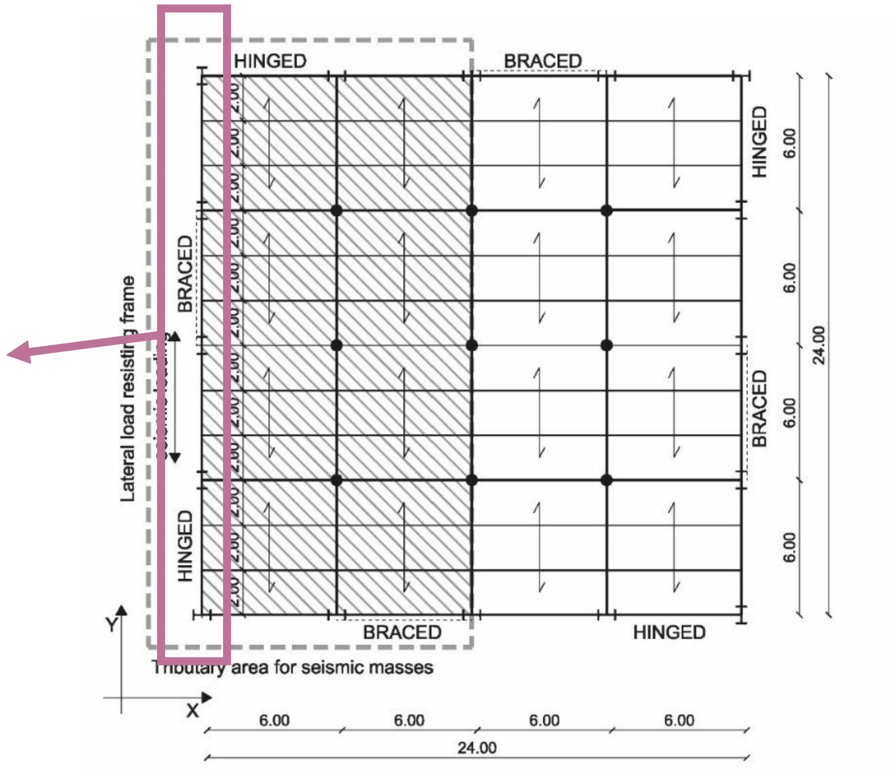
Plan configuration of the building with identification of the lateral load resisting system for Y direction

Structural configuration

- **Perimetral seismic resistant system**
- **Moment resisting frame (MRF)**



Elevation



Gravity loads

- **Live loads for office buildings : q_k=3.00 kN/m²**
- **Structural permanent loads:**

The floor slab is a **composite steel-concrete slab** with HI-BOND A 75 / P760 corrugated steel sheet and C20/25 grade concrete cast. The total thickness of the slab is equal to 125 mm. The corrugated sheet is made of S280GD steel, having a thickness equal to 1.2 m.

Weight of concrete cast is 1.60 kN/m² +

Weight of corrugated steel sheet is 0.15 kN/m² ->

-> the total structural permanent load is **g_{k1}=1.75 kN/m²**

Gravity loads

- **Non-structural permanent loads:**

Soundproof insulation **-** The acoustic insulation of t = 10 mm and with a weight per unit volume of $\gamma = 0.30\ kN/m^3$

Floor screed The floor screed is made by lightweight aggregates with t= 50 mm and $\gamma = 7.2\ kN/m^3$.

Floor of ceramic tiles with a $\gamma = 10\ kN/m^3$ and t = 20 mm.

Thermal insulation made of fiberglass of t = 100 mm and $\gamma = 0.10\ kN/m^3$

Ceiling made of plasterboards t = 20 mm with $\gamma = 0.177\ kN/m^2$.

Internal partition walls have a unit weight less than 1 kN/m, hence, according to EC1, it is possible to model their weight as a uniform load equal to $0.50\ kN/m^2$.

	Weight per unit volume (kN/m^3)	Thickness (m)	Loads (kN/m^2)
Soundproof insulation	0.30	0.010	0.003
Floor screed	7.20	0.050	0.360
Floor	10.00	0.020	0.200
Thermal insulation	0.10	0.100	0.010
Ceiling			0.177
Internal partition walls			0.5
Total value of non-structural permanent loads		g_{k2}=	**1.25 kN/m^2**

STRUCTURAL CONFIGURATION
ACTIONS
DESIGN FOR GRAVITY LOADING
SEISMIC DESIGN
MODELLING ASSUMPTIONS
STRUCTURAL ANALYSIS
COMPARATIVE ASSESSMENTS

Gravity loads

- Structural permanent loads: $g_{k1} = 1.75\ kN/m^2$
- Non-structural permanent loads: $g_{k2}=1.25\ kN/m^2$
- Live loads: $q_k=3.00\ kN/m^2$

Ultimate limit state combination for gravity loads:

$$q_d = \gamma_g(g_{k1} + g_{k2}) + \gamma_q q_k$$
$$= 1.35(1.75 + 1.25) + 1.5 \times 3.0 = 8.55\ kN/m^2$$

Eq (6.10), Table A1.1 and A1.2 – EN 1990:2004

Design of beams of gravity load resisting system

- The beams are designed to withstand a load combination of $q_d = 8.55\ \mathrm{kN/m^2}$.
- The reactions corresponding to the supports is:

$$R_i = 1.10\, q_d l = 1.10 \times 8.55 \times 2 = 18.81\ kN/m$$

$$R_e = 0.40\, q_d l = 0.40 \times 8.55 \times 2 = 6.84\ kN/m$$

- The maximum moment in the midspan of secondary beams is:

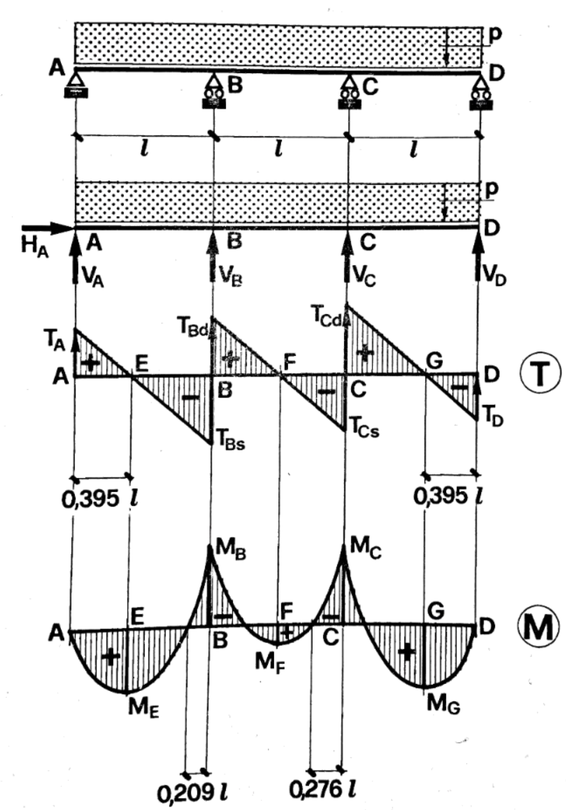
$$M_{max} = R_i \frac{L^2}{8} = 18.48 \times \frac{6^2}{8} = 84.6\ kN\ m$$

$$\rightarrow \quad W_{pl} = \frac{M_{max}}{f_y} = \frac{84.6 \times 1000}{355} = 238.3\ cm^3$$

-> IPE220

$$M_{Rd} = \frac{285.4 \times 10^3 \times 355}{1.00} \cong 101.32\ kNm$$

- The secondary beams have been also checked against serviceability requirements.

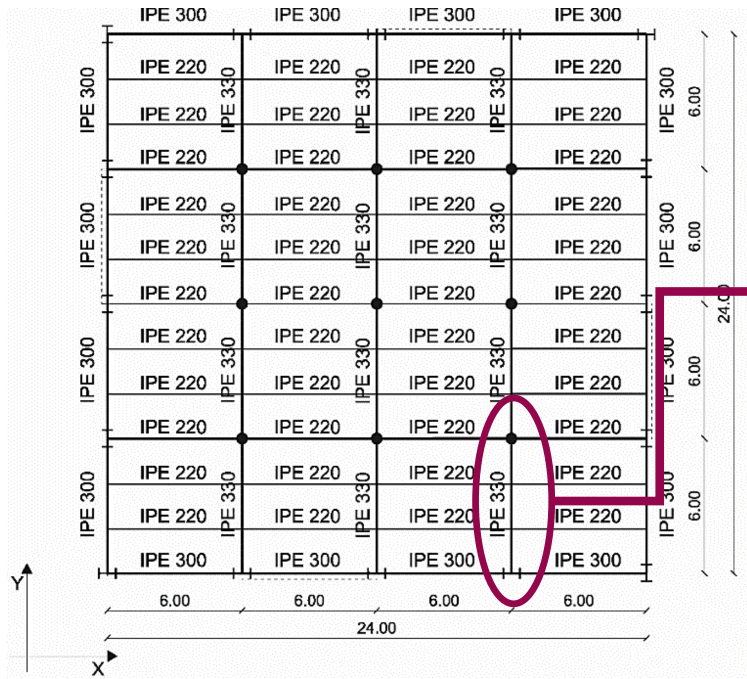


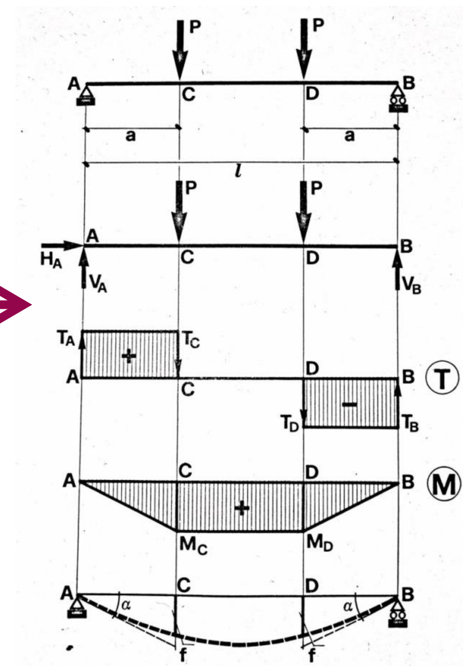
Structural scheme of the composite deck

Design of beams of gravity load resisting system

- The concentrated load due to the adjacent secondary beams is:

$P = (2 \times 18.86 \times 6)/2 = 112.86$ kN





- The maximum moment acting on these beams is equal to:

$$M_{max} = Pa = 112.86 \times 2 = 225.72\ kN\ m$$

$$\rightarrow W_{pl} = \frac{M_{max}}{f_y} = \frac{225.72 \times 1000}{355}$$

$$= 635.8\ cm^3$$ **-> IPE330**

$$M_{Rd} = \frac{804.3 \times 10^3 \times 355}{1.00} \cong 285.526\ kNm$$

Additional permanent loads

- 44 × IPE220 per floor = 67.85 kN
- 12 × IPE 330 per floor = 34.68 kN
- 16 × IPE 300 per floor = 40.48 kN.

○ The total weight of beams is equal to:

$$g_{bk} = 67.85 + 34.68 + 40.48 = 143.00 \text{ kN}/(24\text{m}\times 24\text{m}) = g_{bk} = 0.245 \text{ kN/m}^2$$

- The total permanent loads: $\rightarrow g_{k1} + g_{k2} + g_{bk} = 3.245 \text{ kN/m}^2$
- The weight of external walls of 3.5 m height is 0.16 kN/m² -> 53.76 kN/floor

Total permanent masses for the evaluation of the seismic loads:

- Intermediate storey:

$$3.245 \times (24 \times 24) + 53.76 = 1922.88 \text{ kN} = \mathbf{192.3\ tons}$$

- Roof:

$$3.245 \times (24 \times 24) + 53.76/2 = 1896.00 \text{ kN} = \mathbf{189.6\ tons}$$

Floor mass

Location	Type	Masses for the computation of seismic loads (tons)	Loads (kN/m^2)	$\psi_{2,i}$	$\psi_{E,i}$
Roof	Permanent	G_k= 189.600	3.245		
	Variable	Q_k=172.800	3	0.3	0.24
Intermediate stories	Permanent	G_k=192.288	3.245		
	Variable	Q_k=172.800	3	0.3	0.15

where:

G_k is permanent actions

Q_k is live actions

ψ_2 is coefficient for the quasi-permanent value of the variable actions

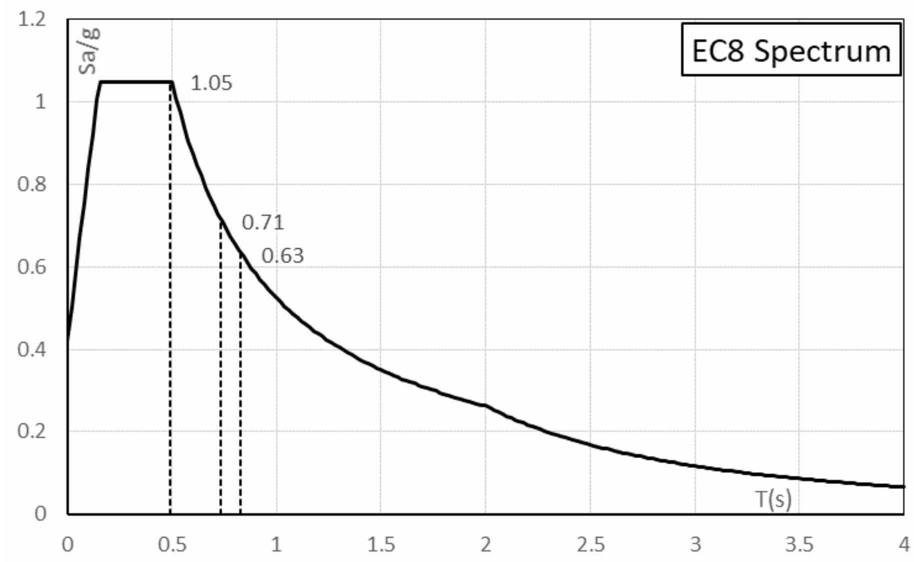
- With reference to the seismic load combination provided by Eurocode 8, masses are evaluated as with:

$$\Sigma G_{k,i} + \Sigma \psi_{E.i} Q_{k,i}$$ *Eq (3.17) – EN1998-1*

Storey	z_i (m)	Floor masses $m_{p,i}$ (tonne)
1	3.5	218.2
2	7.0	218.2
3	10.5	218.2
4	14.0	218.2
5	17.5	218.2
6	21.0	231.1
	m=	1322.1

Seismic forces

- a_g=0.35g;
- Soil Type "B";
- Damping= 5%;
- Type 1 spectrum
- The torsional effects are neglected;
- q=6 (for **MRF**)
- $T_1=C_tH^{3/4}$=0.83sec



a_g	is design ground acceleration
q	is behaviour factor
T_1	is fundamental period
C_t	for moment resistant space steel frames is 0,085 .
H	is a height of the building, in m, from the foundation or top of a rigid basement

Seismic design combination

- **Lateral load resisting frame parallel to the secondary beams**
  - Vertical loads:

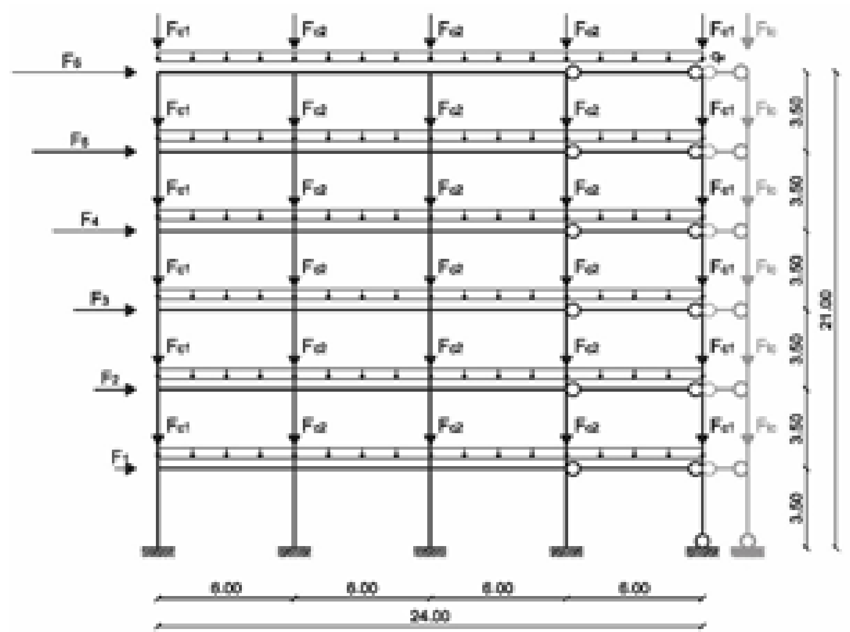
$$\Sigma G_{k,i} + \Sigma \psi_2 Q_{k,i} = 3.245 + 0.3 \times 3 = 4.145\ \mathrm{kN/m^2}$$

→ distributed loads acting on beams of seismic resistant schemes

$$q_d = 0.40 \times 4.145 \times 2 = 3.316\ \mathrm{kN/m^2}$$

  - Concentrated loads on columns based on the seismic load

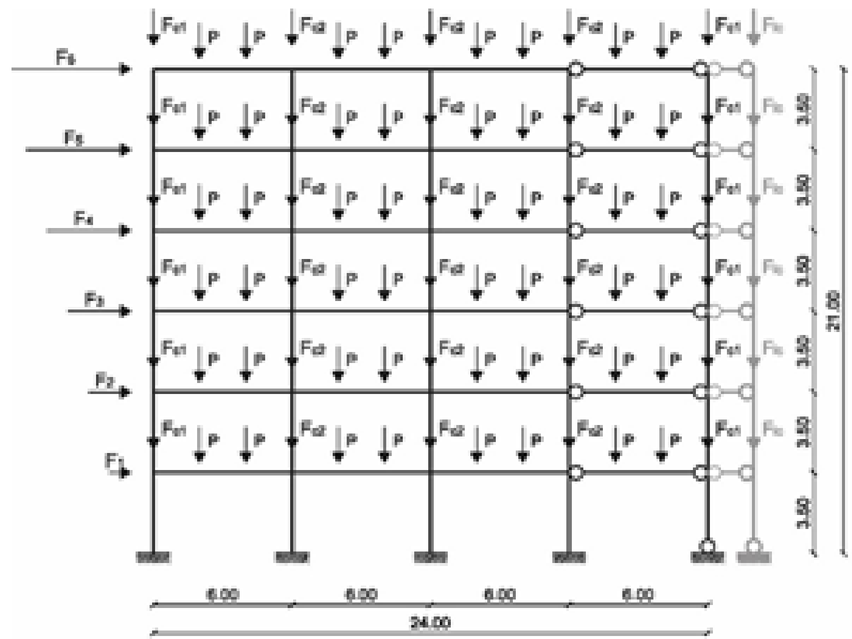
Storey	$\boldsymbol{F_{c1}}$ (kN)	$\boldsymbol{F_{c2}}$ (kN)	$\boldsymbol{F_{lc}}$ (kN)
1	30.72	58.10	905.40
2	30.72	58.10	905.40
3	30.72	58.10	905.40
4	30.72	58.10	905.40
5	30.72	58.10	905.40
6	29.04	56.40	900.36



Seismic design combination

- **Lateral load resisting frame orthogonal to the secondary beams**
  - Concentrated loads on columns based on the seismic load

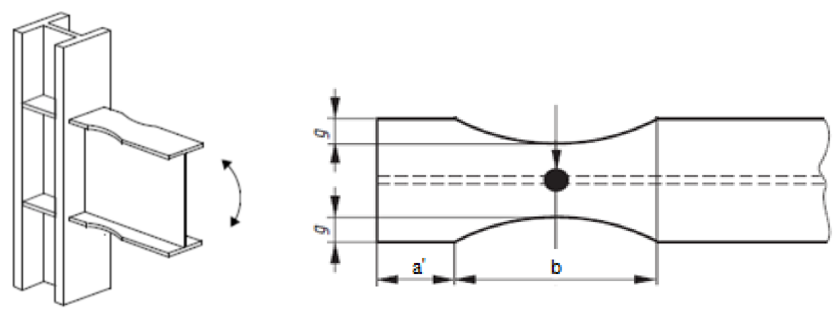
Storey	P (kN)	F_{c1} (kN)	F_{c2} (kN)
1	27.36	13.31	23.26
2	27.36	13.31	23.26
3	27.36	13.31	23.26
4	27.36	13.31	23.26
5	27.36	13.31	23.26
6	27.36	11.63	21.58



  - Lateral load resisting frames arranged orthogonal to secondary beams do not have distributed loads but concentrated loads with a span of 2 m (P).

Modelling assumptions of MRFs

- Reduced beam sections (RBS / Dog-bones)



(d)

Joint Type	Geometry
DB-S: Full-strength with strong panel zone (Type "d")	$a = 0.6b_f$ $b = 0.75d_b$ $s = a + \frac{b}{2}$ $g = 0.2b_f$

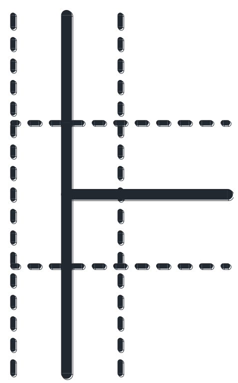
Equaljoints design manual

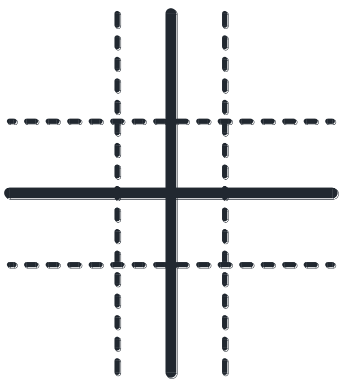
- S355 steel (γ_{ov}=1.25)
- Rigid constraints - at each floor
- "P-Δ column" (leaning column) - modelling second-order effects from "gravity" frames
- Seismic mass - assigned to nodes of lateral load resisting frame

Modelling assumptions

Simplified approach

- Eurocode-based design and preliminary assessment of seismic performance of frames;
- Beams intersect columns in nodes
- Elastic beam elements only





Structural analysis

- **Seismic load combinations**:

  Ultimate Limit State (ULS):

  - for dissipative elements: $\sum G_{k.j} + \sum \psi_{2,i} Q_{k,i} + \alpha(A_{Ed} + I)$
  - for non-dissipative elements: $\sum G_{k.j} + \sum \psi_{2,i} Q_{k,i} + \alpha(\Omega_T A_{Ed} + I)$

- **Second order effects** should be accounted for, by multiplying the seismic action by α if θ > 0.1

where

$$\alpha = 1/(1-\theta)$$

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.1$$

Eq. (4.28) – EN1998-1

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

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

V_{tot} is the total seismic storey shear;

h is the interstorey height;

Ω_T is multiplicative factor on design seismic action, for the design of the non-dissipative members (or structural system overstrength) *(6.6.3 and 6.8.3 – EN1998-1)*

Structural analysis

- **Seismic load combinations**:
  - **Serviciability Limit State (SLS):**
    - $\sum G_{k.j} + \sum \psi_{2,i} Q_{k,i} + \nu q A_{Ed} + I$ *(4.3.4 and 4.4.3.2 - EN 1998-1)*

    q is behaviour factor considered for assessment of displacements induced by the design seismic action resulted from a linear elastic analysis (4.3.4 – EN1998-1)

    v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement and is considered 0.5.

  - **Limitation of interstorey drift** $d_r \nu = 0.010\ h$ *(4.4.3.2c) - EN 1998-1)*

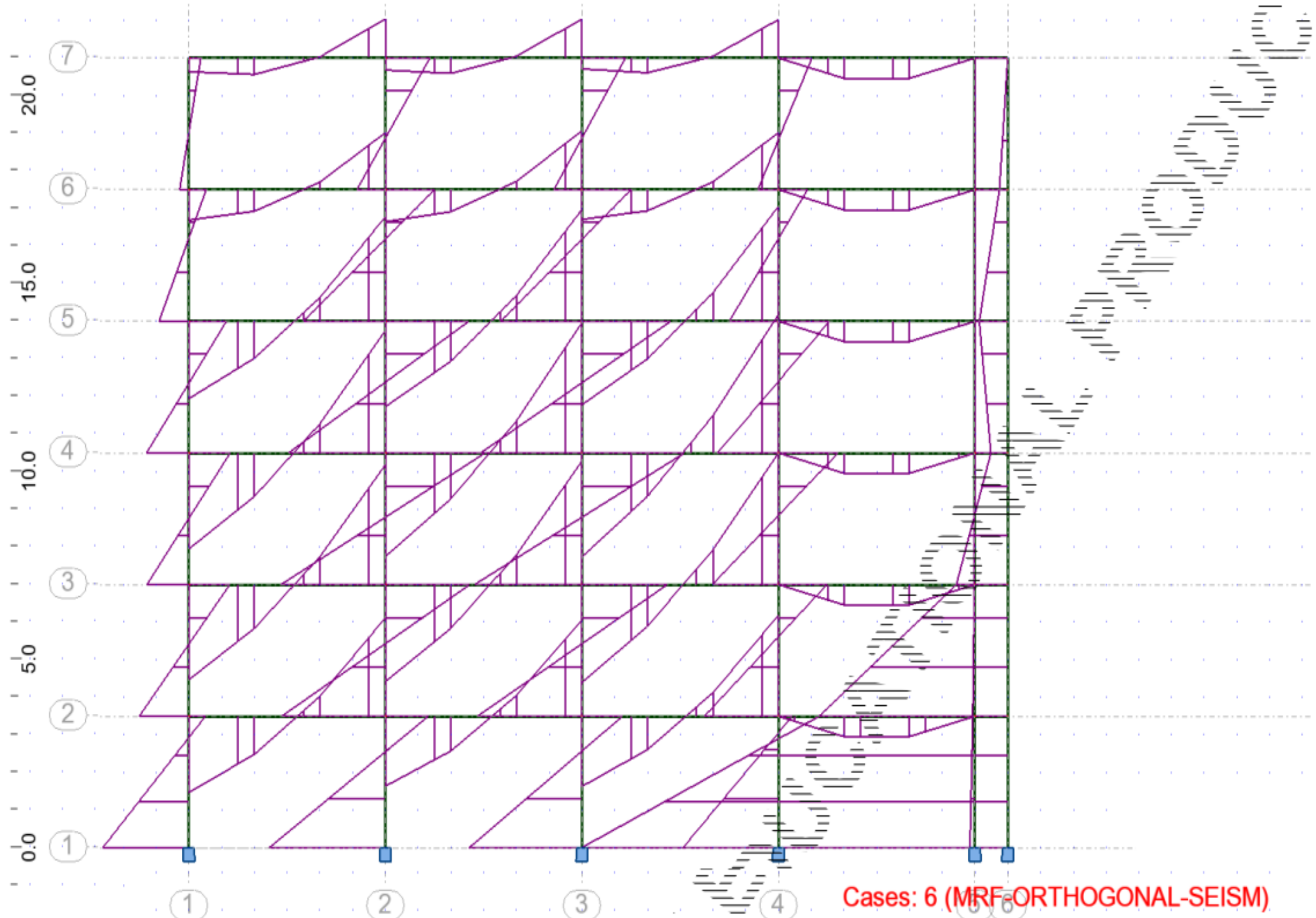
    d_r is the design interstorey drift as defined in *4.4.2.2(2) – EN1998-1*

    h is the storey height;

- **Modal response** spectrum elastic analysis *(4.3.3.3 – EN 1998-1)* – RSA2016
- **Global imperfections** considered by applying Equivalent Horizontal Force (EHF) at each story.

Structural analysis

Simplified approach



Moment diagrams from the seismic load combination

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Structural analysis

○ **ULS**

- Dissipative elements and structural over-strength:
  - Beams
  - Global structural dissipative behaviour - individual values of ratios Ω_i do not exceed the minimum value by more than 25%
- Non-dissipative elements:
  - Columns
- Second order effects was not accounted as θ = 0.1

○ **SLS**

Limitation of interstorey drifts: limit of 0.01h => increase sections

- T_1=0.85sec

→ **MRF Design governed by serviceability limitations (drift limits)**

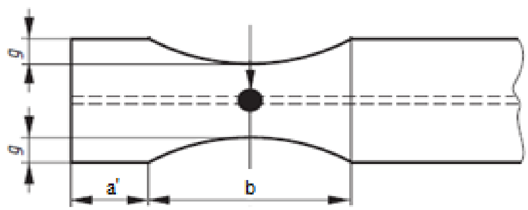
2D	Edge columns			Central columns		
Storey	Section	W_{pl}×10^3 (mm^3)	MRd (kNm)	Section	W_{pl}× 10^3 (mm^3)	MRd (kNm)
1	HEM450	7094	2518	HEM500	7094	2518
2	HEB450	3982	1413	HEM450	6331	2247
3	HEB400	3232	1147	HEM450	6331	2247
4	HEB400	3232	1147	HEM450	6331	2247
5	HEB360	2683	952	HEM450	6331	2247
6	HEB360	2683	952	HEB450	3982	1413

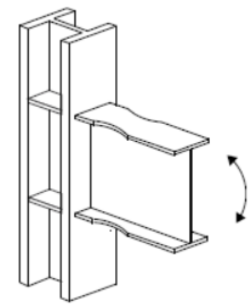
Beams				Verification for non-dis. conn.		
Section	W_{pl}×10^3 (mm^3)	MRd (kNm)	$M_{Ed,E}/M_{Rd}$	Ω_i	Ω_T= 1.1γ_{ov} Ωi.min	M_{Ed}/M_{Rd} (central col)
IPE600	3512	1247	0.23	4.42	3.21	0.50
IPE600	3512	1247	0.27	3.67		0.48
IPE600	3512	1247	0.27	3.73		0.41
IPE550	2787	989	0.31	3.22		0.43
IPE400	1307	464	0.43	2.33		0.42
IPE360	1019	362	0.26	3.81		0.46

Storey	d_s	d_s×q	d_r	d_r×v		0.01h	Δ%
Base	0	0	0	0	<	35	0.00
1	4	24	24	12	<	35	0.34
2	11	66	42	21	<	35	0.60
3	19	114	48	24	<	35	0.69
4	28	168	54	27	<	35	0.77
5	37	222	54	27	<	35	0.77
6	47	282	60	30	<	35	0.86

Structural analysis

- RBS (dog-bone) capacity check





(d)

Storey	Section	$W_{pl,y}$	A_s	A_v	h	b	t_w	t_f	r	$M_{Rd,b}$
		mm^3	mm^2	mm^2	mm	mm	mm	mm	mm	kNm
1	IPE600	3512000	15600	8378	550	210	11.1	17.2	24	1247
2	IPE600	3512000	15600	8378	550	210	11.1	17.2	24	1247
3	IPE600	3512000	15600	8378	550	210	11.1	17.2	24	1247
4	IPE550	2787000	13400	7234	500	200	10.2	16	24	989
5	IPE400	1307000	8450	4270	400	180	8.6	13.5	21	464
6	IPE360	1019000	7270	3514	360	170	8	12.7	21	362

Section	a	b	s	g	b_{RBS}	A	A_v	W_{ply}	$M_{Rd,RBS}$	$M_{Ed}/M_{Rd,RBS}$
	mm	mm	mm	mm	mm	mm^2	mm^2	mm^3	kNm	-
IPE600	126	412.5	332.25	42	126	11760	6941	2404746	854	**0.33**
IPE600	126	412.5	332.25	42	126	11760	6941	2404746	854	**0.40**
IPE550	120	375	307.5	40	120	10058	5893	1892643	672	**0.39**
IPE550	120	375	308	40	120	10058	5893	1893643	672	**0.46**
IPE400	108	300	258	36	108	6124	3330	862635	306	**0.65**
IPE360	102	270	237	34	102	5268	2802	673906	239	**0.40**

Structural analysis

- **Weak beam – strong column check**

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

to prevent formation of a soft storey plastic mechanism in multi-storey buildings, *4.4.2.3(4) - EN 1998-1* at all joints (except last storey)

M_{Rc} is the sum of the design values of the moments of resistance of the columns framing the joint
M_{Rb} is the sum of the design values of the moments of resistance of the beams framing the joint

Edge column							
Storey	ΣM_{Rc}	ΣM_{Rb}	$\Sigma M_{Rb,RBS}$	$1.3\times \Sigma M_{Rb}$	$1.3\times \Sigma MR_{b,RBS}$	$\Sigma M_{Rc}/ 1.3\Sigma M_{Rb}$	$\Sigma M_{Rc}/ 1.3\times \Sigma M_{Rb,RBS}$
1	3932	1247	854	1621	1110	2.43	**3.54**
2	2561	1247	854	1621	1110	1.58	**2.31**
3	2295	1247	854	1621	1110	1.42	**2.07**
4	2100	989	672	1286	874	1.63	**2.40**
5	1905	464	306	603	398	3.16	**4.78**
6	952	362	239	470	311	2.03	**3.06**

Central Column							
Storey	ΣM_{Rc}	ΣM_{Rb}	$\Sigma M_{Rb,RBS}$	$1.3\times \Sigma MRb$	$1.3\times \Sigma M_{Rb,RBS}$	$\Sigma M_{Rc}/ 1.3\Sigma M_{Rb}$	$\Sigma M_{Rc}/ 1.3\Sigma M_{Rb,RBS}$
1	4766	2494	1707	3242	2220	1.47	**2.15**
2	4495	2494	1707	3242	2220	1.39	**2.03**
3	4495	2494	1707	3242	2220	1.39	**2.03**
4	4495	1979	1344	2572	1748	1.75	**2.57**
5	3661	928	612	1206	796	3.03	**4.60**
6	1414	723	478	941	622	1.50	**2.27**

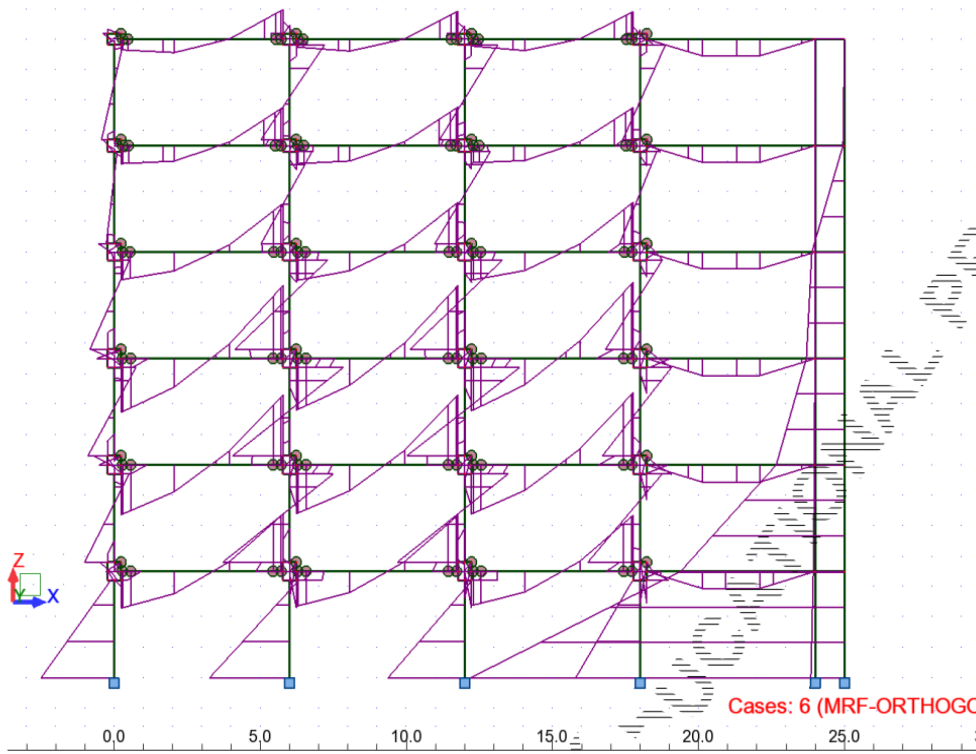
Structural analysis

Refined approach

- The same verifications were made as for the simplified case in terms of dissipative elements and structural over-strength, weak beam strong column, capacity at RBS, et.c
- T_1=1.27sec

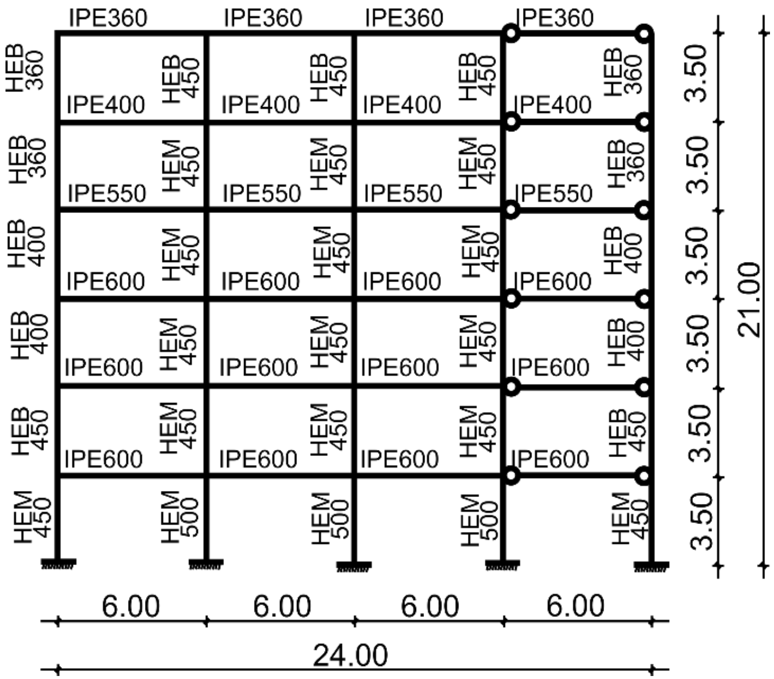
→ **MRF Design governed by serviceability limitations (drift limits)**

Storey	$d_r \times v$
1	0
2	9
3	18
4	21
5	24
6	24
	<35

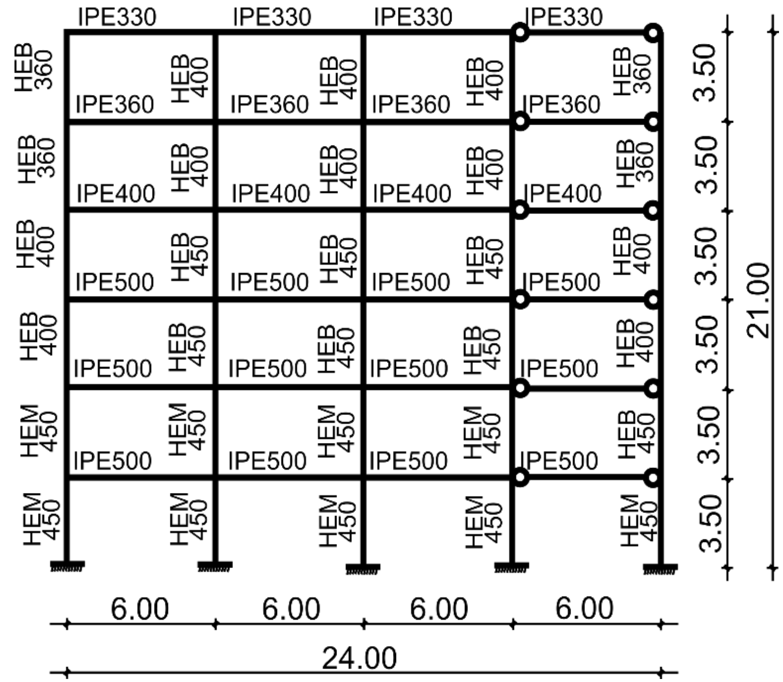


Moment diagrams from the seismic load combination

Comparative assessment



- Simplified



- **Refined approach**

Simplified → Refined approach

- increase in T_1
- reduction of design seismic forces
- reduction in some of the MRF cross-sections
- drifts within codified limits

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Valorisation of knowledge for European pre-QUALified steel JOINTS

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Thank you for your attention!

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20.06.2019

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