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Exercise #11: Seismic Design of Steel Frames with Eccentric Bracings

The steel office building shown in Figure 1 is comprised of steel frames with eccentric bracings (EBFs) in the East-West (EW) loading direction and steel moment-resisting frames (MRFs) in the North-South (NS) loading direction. Both frames have been designed for gravity and seismic loading. The cross sections shown in the figure represent the final design of the steel EBF. The behaviour factors are $q = 3$ and $q = 5$ for the EBF and MRF systems, respectively. The steel members (beams, columns and braces) have been designed with S355J2 steel profile (i.e., $E = 200GPa$, $f_y = 355MPa$). The stability coefficient θ is less than 0.10 in all stories. All the EBF links are designed to be 500mm long.

The following questions should be answered:

1. Check if the EBF link in the first storey of the frame is shear, intermediate or flexure-critical link.
2. Check if the maximum overstrength Ω from all the EBF links does not differ from the minimum one by more than 25%. In case that the difference between Ω_{min} and Ω_{max} exceeds 25% assume that $\Omega = \Omega_{min}$.
3. Design and draw the stiffeners for the EBF link in the first storey by assuming that $\gamma_p = 0.02\, rads$.
4. Check if the non-dissipative beam segment outside the EBF link (between the steel column and the bracing member) is adequate to resist the combined axial load, bending and shear demands due to gravity and seismic actions. The axial load due to gravity in the steel beam is equal to zero.

The force diagrams of the steel frame with eccentric bracings for the gravity and the seismic action are shown in Figure 2 to Figure 6.

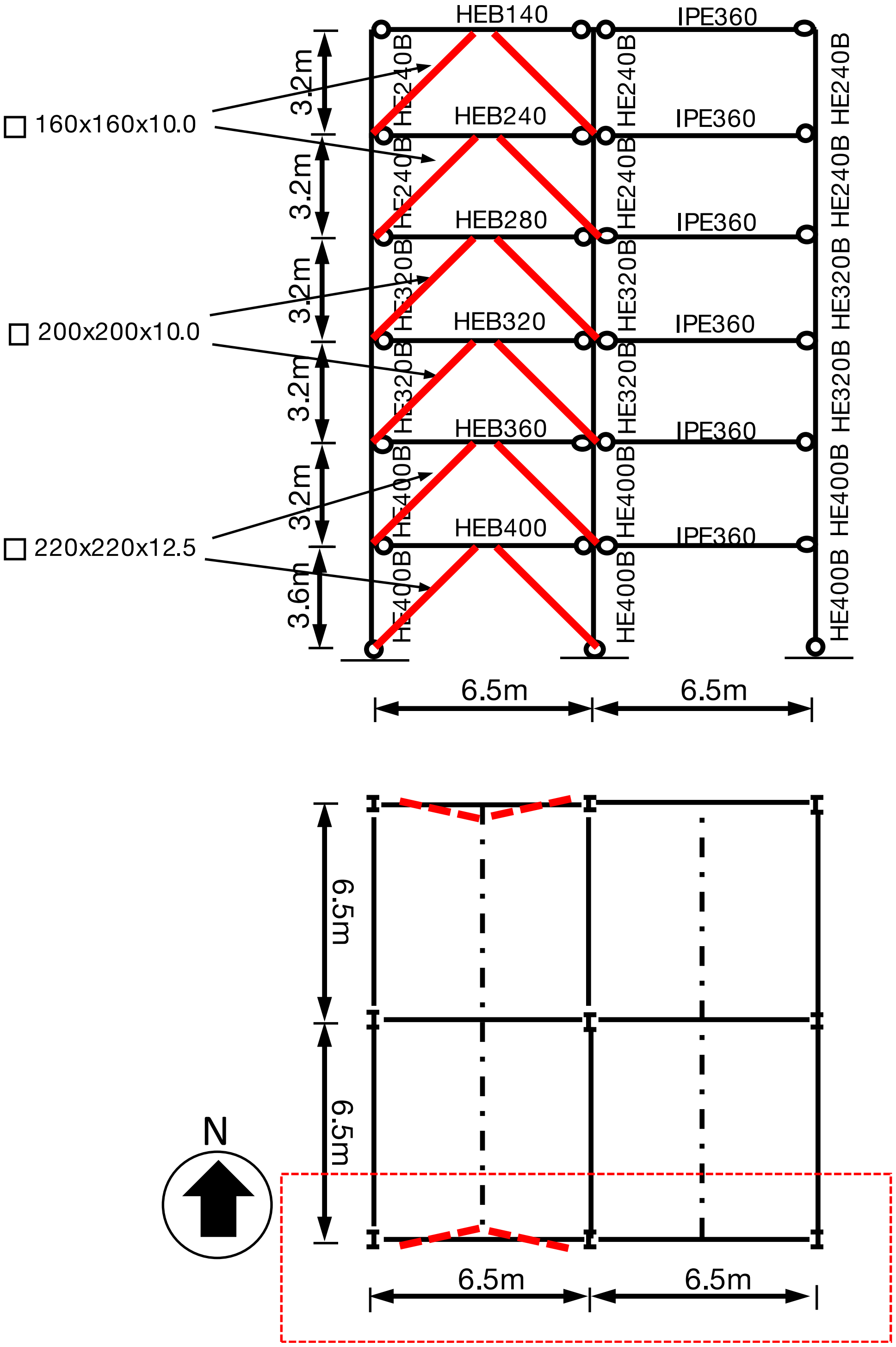


Figure 1. Final design of steel EBF

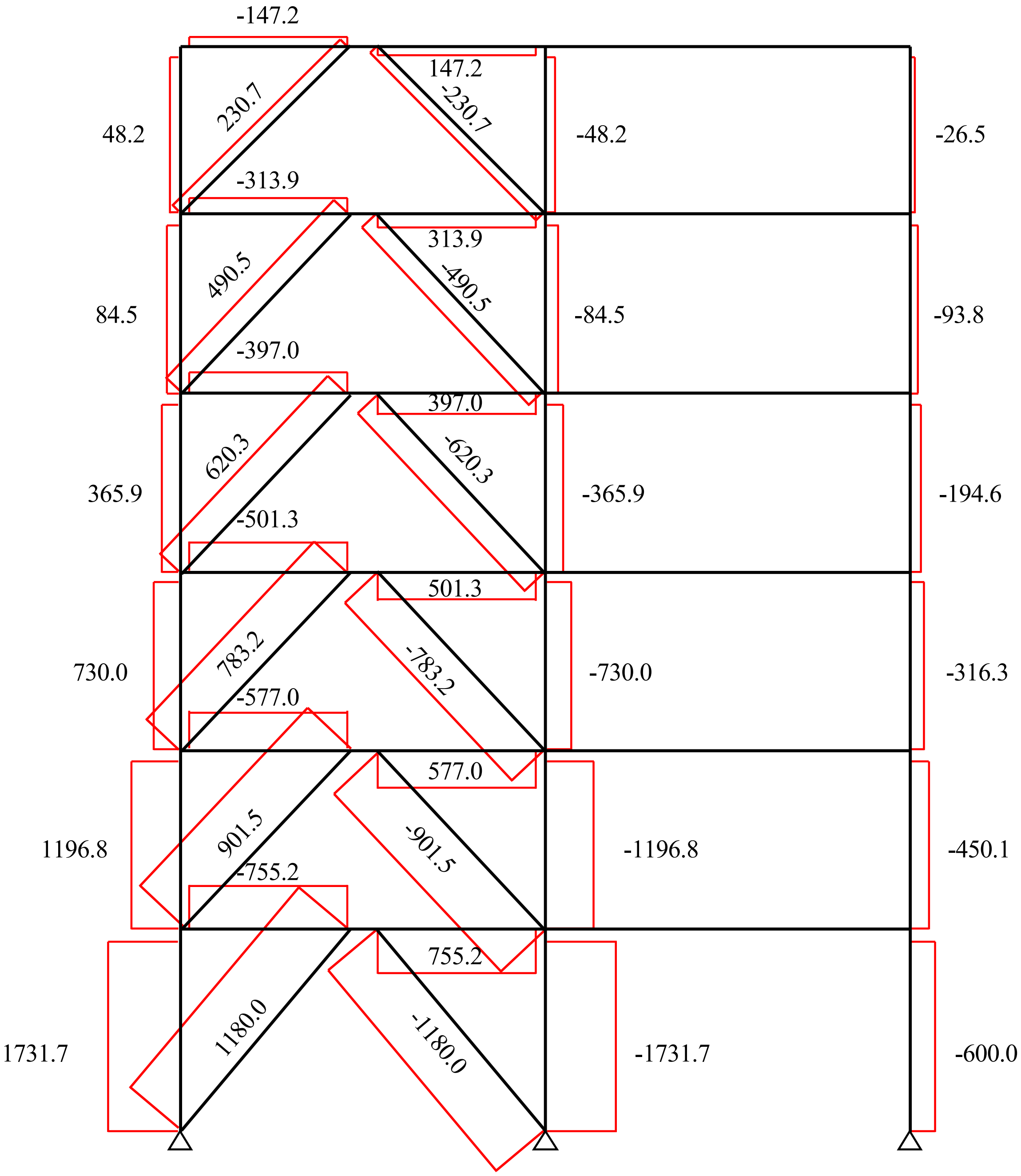


Figure 2. Axial force diagram for seismic loading (units in kN)

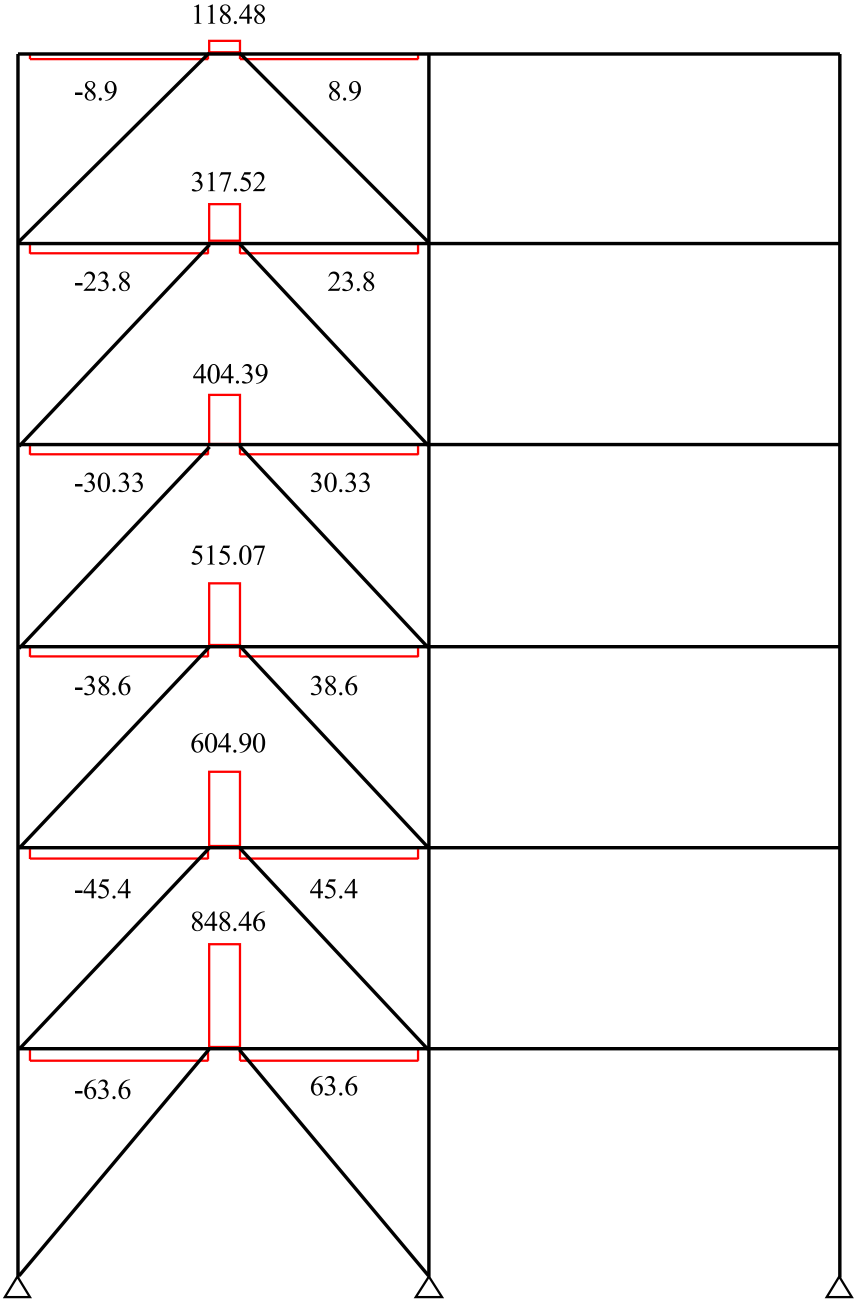


Figure 3. Shear force diagram for seismic loading (units in kN)

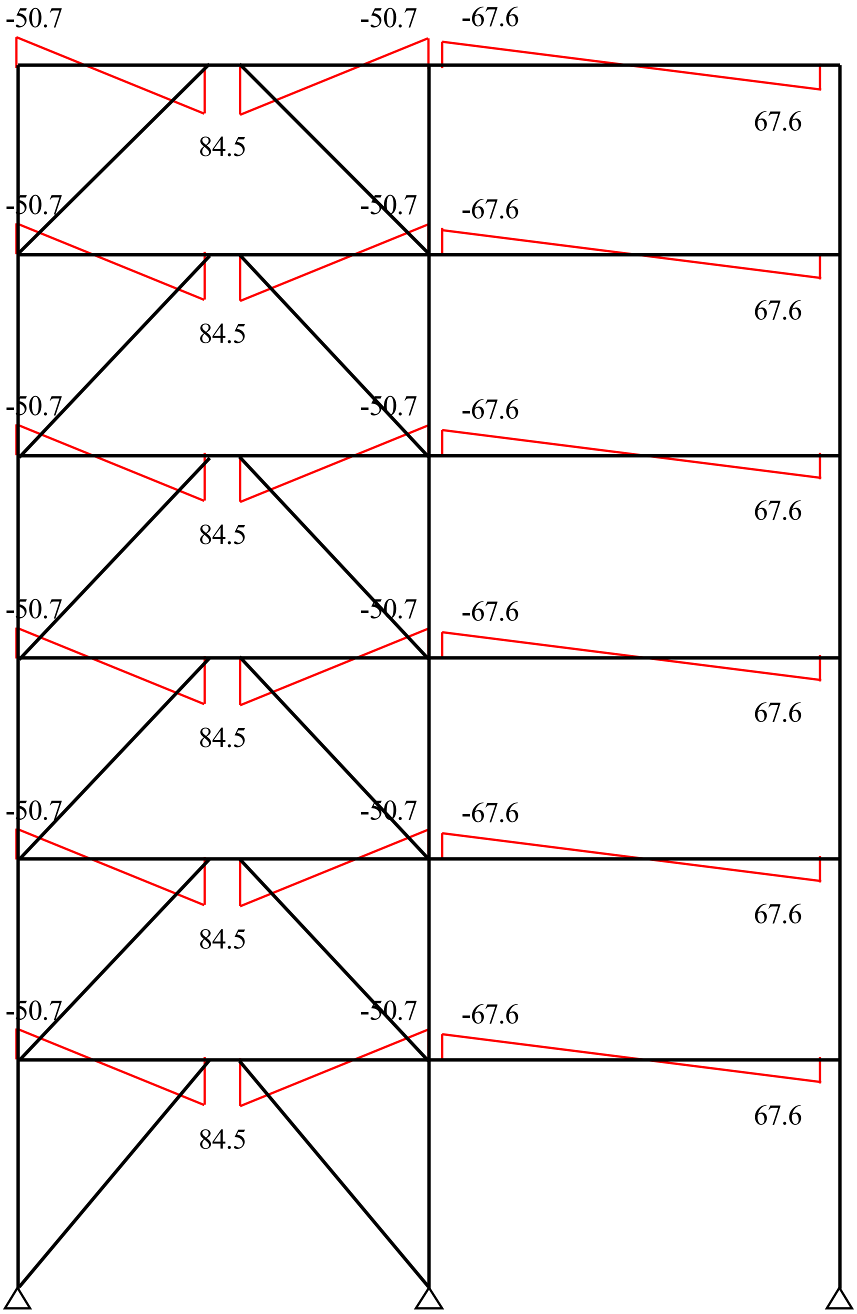


Figure 4. Shear force diagram for gravity loading (units in kN)

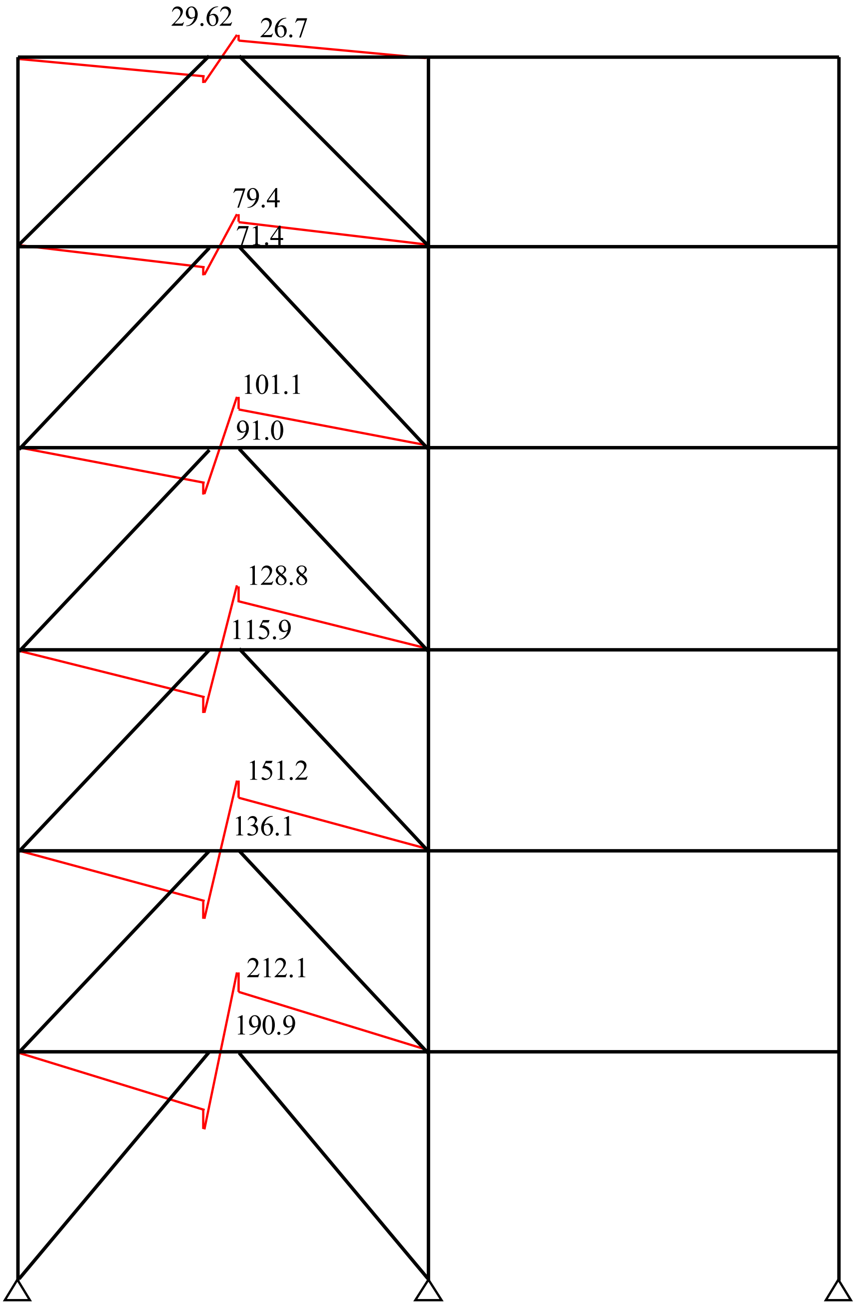


Figure 5. Bending force diagram for seismic loading (units in kN-m)

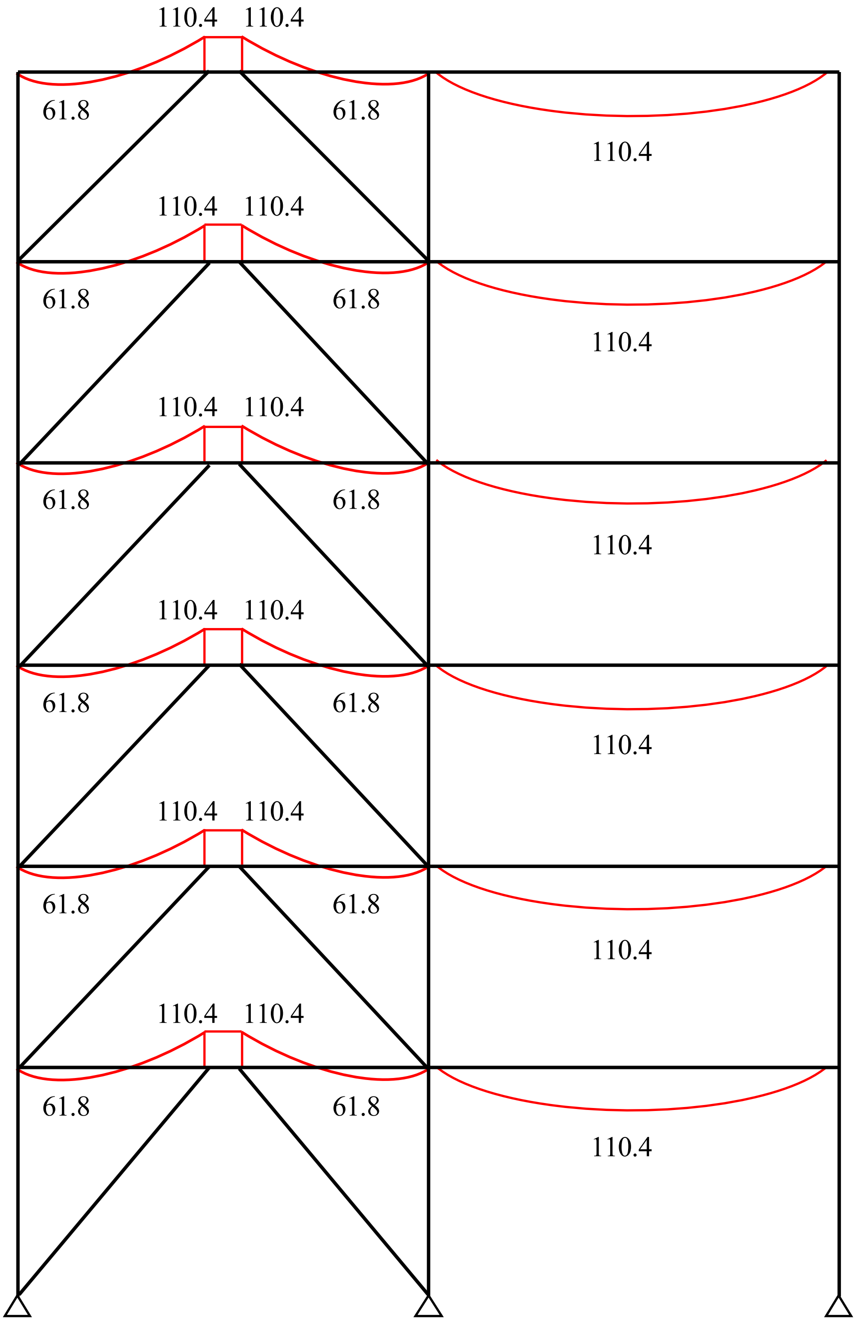
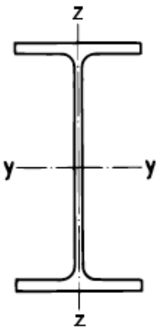


Figure 6. Bending force diagram for gravity loading (units in kN-m)

IPE, PEA

IPE- und IPEA-Träger

Profilés IPE et IPEA



$A_v = A - 2bt_f + (t_w + 2r)\, t_f$

$A_w = (h - t_f) \cdot t_w$

$W_{ely} = \dfrac{I_y}{h/2}$

$S_y = \tfrac{1}{2} W_{ply}$

$S_z = \tfrac{1}{2} W_{plz}$

$\overline{W}_y = \dfrac{I_y}{(h - t_f)/2}$

$W_{elz} = \dfrac{I_z}{b/2}$

Maximale Lagerlängen /
Longueurs maximales en stock:
h ≤ 180 18 m
h ≥ 200 24 m
EURONORM 19 – 57,
DIN 1025/5, ASTM A 6,
Werksnorm/*Norme d'usine*

○ Das Verfahren PP nach SIA 263 ist für dieses Profil aus S355 bei reiner Biegung (n = 0) nicht anwendbar!

* Auch in S355J0 oder S355J2 ab Schweizer Lager erhältlich.

○ *La méthode PP selon SIA 263 n'est pas applicable pour ce profilé en acier S355 en flexion simple (n = 0)!*

* *Livrable en S355J0 ou S355J2 du stock suisse.*

IPE	m	Statische Werte / *Valeurs statiques*													
	kg/m	A mm²	A_v mm²	A_w mm²	I_y mm⁴	W_{ely} mm³	$\overline{W}_y$ mm³	W_{ply} mm³	i_y mm	I_z mm⁴	W_{elz} mm³	W_{plz} mm³	i_z mm	$K = I_x$ mm⁴
					x 10⁶	x 10³	x 10³	x 10³		x 10⁶	x 10³	x 10³		x 10⁶
80*	6,0	764	358	284	0,801	20,0	21,4	23,2	32,4	0,085	3,69	5,82	10,5	0,0067
100*	8,1	1030	508	387	1,71	34,2	36,3	39,4	40,7	0,159	5,79	9,15	12,4	0,0115
120*	10,4	1320	631	500	3,18	53,0	55,9	60,7	49,0	0,277	8,65	13,6	14,5	0,0169
140*	12,9	1640	764	626	5,41	77,3	81,3	88,3	57,4	0,449	12,3	19,2	16,5	0,0240
160*	15,8	2010	966	763	8,69	109	114	124	65,8	0,683	16,7	26,1	18,4	0,0353
180*	18,8	2390	1125	912	13,2	146	154	166	74,2	1,01	22,2	34,6	20,5	0,0472
200*	22,4	2850	1400	1070	19,4	194	203	221	82,6	1,42	28,5	44,6	22,4	0,0685
220*	26,2	3340	1588	1240	27,7	252	263	285	91,1	2,05	37,3	58,1	24,8	0,0898
240*	30,7	3910	1914	1430	38,9	324	338	367	99,7	2,84	47,3	73,9	26,9	0,127
270*	36,1	4590	2214	1710	57,9	429	446	484	112	4,20	62,2	97,0	30,2	0,157
300*	42,2	5380	2568	2050	83,6	557	578	628	125	6,04	80,5	125	33,5	0,198
330*	49,1	6260	3081	2390	117,7	713	739	804	137	7,88	98,5	154	35,5	0,276
360*	57,1	7270	3514	2780	162,7	904	937	1020	150	10,4	123	191	37,9	0,371
400*	66,3	8450	4269	3320	231,3	1160	1200	1310	165	13,2	146	229	39,5	0,504
450*	77,6	9880	5085	4090	337,4	1500	1550	1700	185	16,8	176	276	41,2	0,661
500*	90,7	11600	5987	4940	482,0	1930	1990	2190	204	21,4	214	336	43,1	0,886
550	106	13400	7234	5910	671,2	2440	2520	2790	223	26,7	254	401	44,5	1,22
600	122	15600	8378	6970	920,8	3070	3170	3510	243	33,9	308	486	46,6	1,65
750 x 137		17500	9290	8460	1599	4250	4340	○ 4860	303	51,7	393	614	54,4	1,36
750 x 147		18700	10540	9720	1661	4410	4510	5110	298	52,9	399	631	53,1	1,57
750 x 173		22100	11640	10700	2058	5400	5560	6220	305	68,7	515	810	55,7	2,71
750 x 196		25100	12730	11600	2403	6240	6450	7170	310	81,8	610	959	57,1	4,06

IPE- und IPEA-Träger *Profilés IPE et IPEA* IPE, PEA

Die Profile PER, IPEo und IPEv sind im Walzprogramm einzelner Werke aufgeführt. PEA 80 und PEA 100 sind ebenfalls normiert, aber kaum wirtschaftlich.

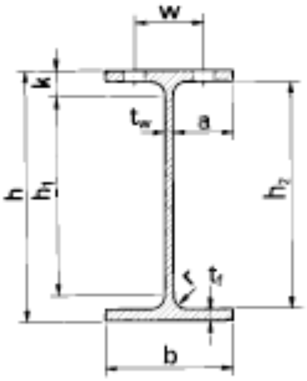
▨ Im allgemeinen nur ab Werk lieferbar. Mindestmengen und Termine beachten.

Walztoleranzen siehe Seite 116

Les profilés PER, IPEo et IPEv figurent dans le programme de laminage de quelques aciéries. Les PEA 80 et PEA 100, également normalisés, sont peu économiques.

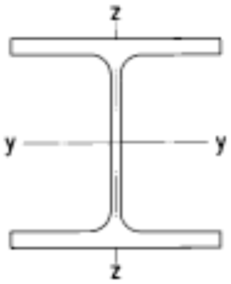
▨ *En général livrable d'usine uniquement. Tenir compte des quantités minimales et des délais.*

Tolérances de laminage voir p. 116



IPE	m	Profilmasse *Dimensions de la section*					Konstruktionsmasse *Dimensions de construction*						Oberfläche *Surface*		IPE
	kg/m	h mm	b mm	t_w mm	t_f mm	r mm	h_1 mm	k mm	a mm	h_2 mm	w mm	$Ø_{max}$	U_m m^2/m	U_t m^2/t	
80	**6,0**	80	46	3,8	5,2	5	60	10	21	70			0,328	54,8	**80**
100	**8,1**	100	55	4,1	5,7	7	74	13	25	89			0,400	49,5	**100**
120	**10,4**	120	64	4,4	6,3	7	92	14	29	107	36	M10	0,475	45,6	**120**
140	**12,9**	140	73	4,7	6,9	7	112	14	34	126	38	M10	0,551	42,6	**140**
160	**15,8**	160	82	5,0	7,4	9	126	17	38	145	44	M12	0,623	39,4	**160**
180	**18,8**	180	91	5,3	8,0	9	146	17	42	164	50	M12	0,698	37,1	**180**
200	**22,4**	200	100	5,6	8,5	12	158	21	47	183	56	M12	0,768	34,3	**200**
220	**26,2**	220	110	5,9	9,2	12	178	21	52	202	60	M16	0,848	32,4	**220**
240	**30,7**	240	120	6,2	9,8	15	190	25	56	220	68	M16	0,922	30,0	**240**
270	**36,1**	270	135	6,6	10,2	15	220	25	64	250	72	M20	1,04	28,8	**270**
300	**42,2**	300	150	7,1	10,7	15	248	26	71	279	80	M20	1,16	27,5	**300**
330	**49,1**	330	160	7,5	11,5	18	270	30	76	307	86	M24	1,25	25,5	**330**
360	**57,1**	360	170	8,0	12,7	18	298	31	81	335	90	M24	1,35	23,6	**360**
400	**66,3**	400	180	8,6	13,5	21	330	35	85	373	96	M27	1,47	22,2	**400**
450	**77,6**	450	190	9,4	14,6	21	378	36	90	421	106	M27	1,61	20,7	**450**
500	**90,7**	500	200	10,2	16,0	21	426	37	94	468	110	M27	1,74	19,2	**500**
550	**106**	550	210	11,1	17,2	24	468	41	99	516	120	M27	1,88	17,7	**550**
600	**122**	600	220	12,0	19,0	24	514	43	104	562	120	M27	2,02	16,6	**600**
750 x 137		753	263	11,5	17,0	17	685	34	126	719	120	M27	2,51	18,3	**750x137**
750 x 147		753	265	13,2	17,0	17	685	34	126	719	120	M27	2,51	17,1	**750x147**
750 x 173		762	267	14,4	21,6	17	685	39	126	719	120	M27	2,53	14,6	**750x173**
750 x 196		770	268	15,6	25,4	17	685	42	126	719	120	M27	2,55	13,0	**750x196**

HEB   Breitflanschträger HEB   *Profilés à larges ailes* HEB



$A_v = A - 2bt_f + (t_w+2r)\,t_f$

$A_w = (h-t_f)\cdot t_w$

$S_y = ½\,W_{ply}$

$S_z = ½\,W_{plz}$

$$W_{ely} = \frac{I_y}{h/2}$$

$$\overline{W}_y = \frac{I_y}{(h-t_f)/2}$$

$$W_{elz} = \frac{I_z}{b/2}$$

Maximale Lagerlängen /
Longueurs maximales en stock:
$h \leq 180$ 18 m
$h \geq 200$ 24 m

EURONORM 53 – 62, DIN 1025/2

Andere Bezeichnungen
Autres désignations   } DIN, IPB

*Auch in S355J0 oder S355J2
ab Schweizer Lager erhältlich

**Livrable en S355J0 ou S355J2*
du stock suisse

HEB	**m** kg/m	Statische Werte / *Valeurs statiques*												
		A mm²	A_v mm²	A_w mm²	I_y mm⁴	W_{ely} mm³	$\overline{W}_y$ mm³	W_{ply} mm³	i_y mm	I_z mm⁴	W_{elz} mm³	W_{plz} mm³	i_z mm	$K = I_x$ mm⁴
					x 10^6	x 10^3	x 10^3	x 10^3		x 10^6	x 10^3	x 10^3		x 10^6
100*	**20,4**	2600	904	540	4,50	89,9	100	104	41,6	1,67	33,5	51,4	25,3	0,0931
120*	**26,7**	3400	1096	708	8,64	144	158	165	50,4	3,18	52,9	81,0	30,6	0,139
140*	**33,7**	4300	1308	896	15,1	216	236	245	59,3	5,50	78,5	120	35,8	0,202
160*	**42,6**	5430	1759	1180	24,9	311	339	354	67,8	8,89	111	170	40,5	0,312
180*	**51,2**	6530	2024	1410	38,3	426	461	481	76,6	13,6	151	231	45,7	0,422
200*	**61,3**	7810	2483	1660	57,0	570	616	643	85,4	20,0	200	306	50,7	0,596
220*	**71,5**	9100	2792	1940	80,9	736	793	827	94,3	28,4	258	394	55,9	0,770
240*	**83,2**	10600	3323	2230	112,6	938	1010	1050	103	39,2	327	498	60,8	1,04
260*	**93,0**	11800	3759	2420	149,2	1150	1230	1280	112	51,3	395	602	65,8	1,26
280*	**103**	13100	4109	2750	192,7	1380	1470	1530	121	65,9	471	718	70,9	1,45
300*	**117**	14900	4743	3090	251,7	1680	1790	1870	130	85,6	571	870	75,8	1,87
320*	**127**	16100	5177	3440	308,2	1930	2060	2150	138	92,4	616	939	75,7	2,29
340*	**134**	17100	5609	3820	366,6	2160	2300	2410	146	96,9	646	986	75,3	2,62
360*	**142**	18100	6060	4220	431,9	2400	2560	2680	155	101	676	1030	74,9	2,98
400*	**155**	19800	6998	5080	576,8	2880	3070	3230	171	108	721	1100	74,0	3,61
450*	**171**	21800	7966	5940	798,9	3550	3770	3980	191	117	781	1200	73,3	4,49
500	**187**	23900	8982	6840	1072	4290	4540	4820	212	126	842	1290	72,7	5,50
550	**199**	25400	10010	7820	1367	4970	5250	5590	232	131	872	1340	71,7	6,12
600	**212**	27000	11080	8840	1710	5700	6000	6420	252	135	902	1390	70,8	6,80
650	**225**	28600	12200	9900	2106	6480	6800	7320	271	140	932	1440	69,9	7,52
700	**241**	30600	13710	11400	2569	7340	7690	8330	290	144	963	1490	68,7	8,42
800	**262**	33400	16180	13400	3591	8980	9360	10230	328	149	994	1550	66,8	9,62
900	291	37100	18880	16000	4941	10980	11400	12580	365	158	1050	1660	65,3	11,5
1000	314	40000	21250	18300	6447	12890	13400	14860	401	163	1090	1720	63,8	12,7

Breitflanschträger HEB *Profilés à larges ailes* HEB HEB

Anstelle des nicht mehr gewalzten Profils HEB 1100 können HL-Profile verwendet werden, siehe Seiten 40/41.

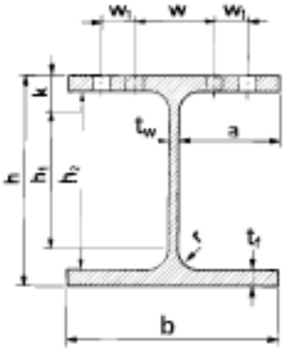
w₁ mit $Ø_{max}$ nur für versetzte Schrauben.

Walztoleranzen siehe Seite 116

Au lieu du profilé HEB 1100 qui n'est plus laminé, on utilisera des profilés HL (voir pages 40/41).

w_1 avec $Ø_{max}$ seulement pour boulons décalés.

Tolérances de laminage voir page 116



HEB	m	Profilmasse *Dimensions de la section*					Konstruktionsmasse *Dimensions de construction*							Oberfläche *Surface*		HEB
	kg/m	h mm	b mm	t_w mm	t_f mm	r mm	h_1 mm	k mm	a mm	h_2 mm	w mm	w_1 mm	$Ø_{max}$	U_m m²/m	U_t m²/t	
100	**20,4**	100	100	6	10	12	56	22	47	80	56		M12	0,567	27,8	**100**
120	**26,7**	120	120	6,5	11	12	74	23	56	98	66		M16	0,686	25,7	**120**
140	**33,7**	140	140	7	12	12	92	24	66	116	76		M20	0,805	23,9	**140**
160	**42,6**	160	160	8	13	15	104	28	76	134	86		M20	0,918	21,5	**160**
180	**51,2**	180	180	8,5	14	15	122	29	85	152	100		M24	1,04	20,3	**180**
200	**61,3**	200	200	9	15	18	134	33	95	170	110		M24	1,15	18,8	**200**
220	**71,5**	220	220	9,5	16	18	152	34	105	188	120		M24	1,27	17,8	**220**
240	**83,2**	240	240	10	17	21	164	38	115	206	96	35	M24	1,38	16,6	**240**
260	**93,0**	260	260	10	17,5	24	176	42	125	225	106	40	M24	1,50	16,1	**260**
280	**103**	280	280	10,5	18	24	196	42	134	244	110	45	M24	1,62	15,7	**280**
300	**117**	300	300	11	19	27	208	46	144	262	120	45	M27	1,73	14,8	**300**
320	**127**	320	300	11,5	20,5	27	224	48	144	279	120	45	M27	1,77	13,9	**320**
340	**134**	340	300	12	21,5	27	242	49	144	297	120	45	M27	1,81	13,5	**340**
360	**142**	360	300	12,5	22,5	27	260	50	143	315	120	45	M27	1,85	13,0	**360**
400	**155**	400	300	13,5	24	27	298	51	143	352	120	45	M27	1,93	12,4	**400**
450	**171**	450	300	14	26	27	344	53	143	398	120	45	M27	2,03	11,9	**450**
500	**187**	500	300	14,5	28	27	390	55	142	444	120	45	M27	2,12	11,3	**500**
550	**199**	550	300	15	29	27	438	56	142	492	120	45	M27	2,22	11,2	**550**
600	**212**	600	300	15,5	30	27	486	57	142	540	120	45	M27	2,32	11,0	**600**
650	**225**	650	300	16	31	27	534	58	142	588	120	45	M27	2,42	10,8	**650**
700	**241**	700	300	17	32	27	582	59	141	636	126	45	M27	2,52	10,5	**700**
800	**262**	800	300	17,5	33	30	674	63	141	734	130	40	M27	2,71	10,4	**800**
900	291	900	300	18,5	35	30	770	65	140	830	130	40	M27	2,91	10,0	900
1000	314	1000	300	19	36	30	868	66	140	928	130	40	M27	3,11	9,9	1000

Suggested Solution

Question 1:

EBF check in the first storey:

Shear resistance:

$$M_{p,link} = f_y b t_f (d - t_f) = 355 \cdot 300 \cdot 24 \cdot (400 - 24) \cdot 10^{-6} = 961.1kNm$$

$$V_{p,link} = \frac{f_y}{\sqrt{3}} \cdot t_w \cdot (d - t_f) = \frac{355}{\sqrt{3}} \cdot 13.5 \cdot (400 - 24) \cdot 10^{-3} = 1040.4kN$$

Since the axial load in the EBF is zero, the check $N_{Ed}/N_{pl,Rd} < 0.15$ is satisfied. Threfore,

$$e_s = 1.6 \cdot \frac{M_{p,link}}{V_{p,link}} = 1.6 \cdot \frac{961.1}{1040.4} = 1.48\, m > e = 0.5\, m$$

Therefore, the EBF link is classified as shear-critical.

Question 2:

For shear-critical links, the overstrength Ω_i per link is computed based on the following formula:

$$\Omega_i = 1.5 \cdot \frac{V_{p,link,i}}{V_{Ed,E,i}}$$

$V_{p,link,i}$ is computed as discussed in Question 1.
$V_{Ed,E,i}$ is computed directly from the shear diagram shown in Figure 3 due to seismic loading.

Storey	**Cross Section**	$V_{p,link}$ **[kN]**	$V_{Ed,E,i}$ **[kN]**	Ω	$\frac{\Omega - \Omega_{min}}{\Omega_{min}}$
6	HEB140	183.6	118.5	2.33	26.63%
5	HEB240	457.1	317.5	2.16	17.39%
4	HEB280	563.8	404.4	2.09	13.59%
3	HEB320	705.9	515.1	2.06	11.96%
2	HEB360	864.7	604.9	2.14	16.30%
1	HEB400	1040.4	848.5	1.84	0.00%

Question 3

The maximum allowable γ_p is $0.08 rads$ for shear critical EBF links. For the given link rotation angle, $\gamma_p = 0.02\, rads$.

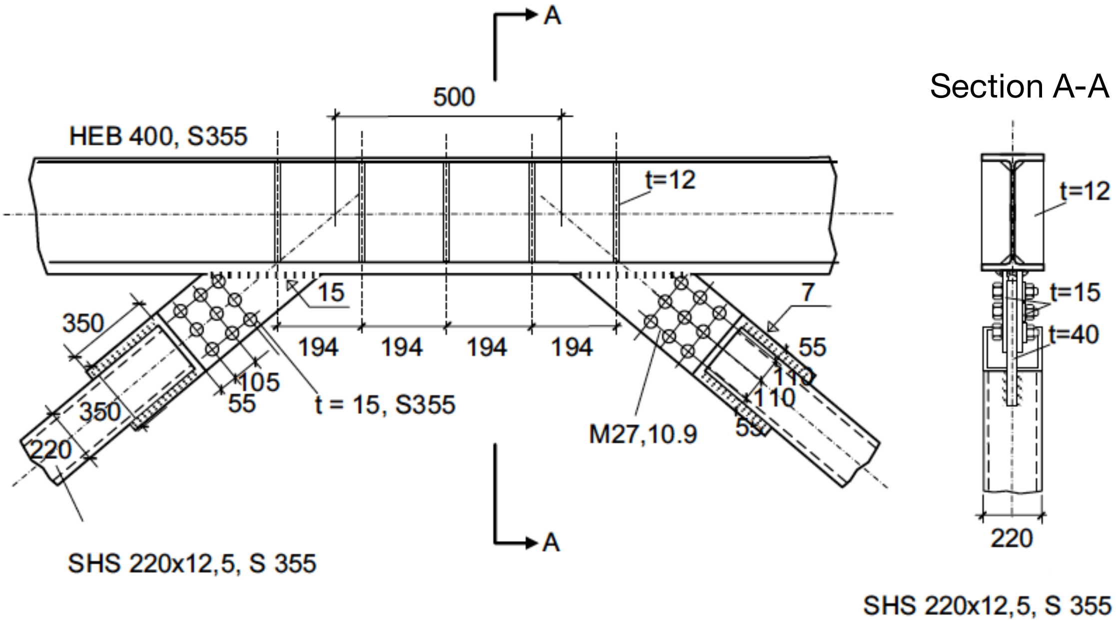
Therefore,

$$s \leq 52t_w - \frac{d}{5} = 52 \cdot 13.5 - \frac{400}{5} = 622\,mm > e = 500\,mm$$

Based on the calculated s distance, we don't have to install stiffeners in this case. However, this is not a good practice and stiffeners will be installed at a minimum distance. A typical drawing is presented that the stiffeners are placed every 194mm < 622mm. Note that in this case the EBF link is able to achieve much larger γ_p than $0.02rads$.

In order to size the stiffeners, the following empirical formula may be used (see stiffener design of panel zones in steel MRF section):

$t > \max(10\,;\,0.75 \cdot t_w) = \max(10\,;\,0.75 \cdot 13.5) = 10.1mm$; therefore, $assume, t = 12mm$.



Question 4

We will consider the beam in the first storey. The beam size is an HEB400:

Cross-section classification:

According to EC8, for $q = 3$, Class 1&2 cross section profiles are permitted. Therefore, the steel beam member outside the EBF link should be first checked for section classification:

Flange subject to compression:

$$\frac{b_f - t_w - 2r}{2 \cdot t_f} = \frac{300 - 13.5 - 2 \cdot 27}{2 \cdot 24} = 4.8 < 9 \cdot \sqrt{\frac{235}{f_y}} = 9 \cdot 0.814 = 7.3$$

Therefore, the flange of the beam is classified as Class 1.

Web subject to bending and compression. Assume the web in compression (conservative assumption):

$$\frac{h_1}{t_w} = \frac{298}{13.5} = 22.1 < \ 33 \cdot \sqrt{\frac{235}{f_y}} = 33 \cdot 0.814 \ = \ 26.9$$

Therefore, both the flange and web are classified as Class 1; hence, the HEB400 beam cross section is Class 1.

Shear resistance:

$$V_{pl,Rd} = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{6998 \cdot 0.355}{\sqrt{3} \cdot 1.00} = 1434 \, kN$$

Shear demand:

$\Omega = \Omega_{min} = \ 1.84$ (we use the smallest Ω obtained in Question 2 as suggested by the question)

$$V_{Ed} = V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} = 84.5 + 1.1 \cdot 1.25 \cdot 1.84 \cdot 63.6 = 84.5 + 160.9$$
$$= 245.4 \, kN < 1434 \, kN$$

Shear load –flexure interaction:

Shear-bending interaction verification:

$$V_{Ed} = 245.4 \, kN < 717 \, kN = 0.50 V_{pl,Rd}$$

Therefore, no reduction due to shear is required. However, if $V_{Ed} > 0.50 \, V_{pl,Rd}$, the bending resistance shall be reduced due to the effects of shear demand and the reduced flexural resistance shall be used to check the axial-flexure interaction.

Member buckling:

Since the buckling length is the same in the weak and strong axis, the buckling resistance is checked merely according to the weak axis.

Bucking resistance of the steel beam (z-z axis)

$$l_k = \frac{L - t_w - e}{2} = \frac{6500 - 13.5 - 500}{2} = 2993 mm$$

$\frac{h}{b} = \frac{400}{300} = \ 1.33 \ > 1.2$ and $t_f \ = 10.2mm < \ 100mm$; therefore the imperfection curve is *b* and the imperfection factor is $\alpha = \ 0.34$ (z-z axis)

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{l_k^2} = 3.14^2 \cdot 210 \cdot \frac{108 \cdot 10^6}{2993^2} = 24962.5 \, kN$$

$$\bar{\lambda}_z = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}} = \sqrt{\frac{19800 \cdot 0.355}{24962.5}} = 0.53$$

$$\Phi_z = 0.5 \cdot \left(1 + \alpha \cdot (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z{}^2\right) = 0.5 \cdot (1 + 0.34 \cdot (0.53 - 0.2) + 0.53^2) = 0.70$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z{}^2}} = \frac{1}{0.70 + \sqrt{0.70^2 - 0.53^2}} = 0.864$$

$N_{b,z,Rd} = \chi_z \cdot A \cdot \frac{f_y}{\gamma_{M1}} = 0.864 \cdot 19800 \cdot \frac{0.355}{1.05} = 5784\, kN$
$N_{Ed} = N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} = 0 + 1.1 \cdot 1.25 \cdot 1.84 \cdot 755.2 = 1911\, kN < N_{b,z,Rd} = 5784\, kN$

Axial load –flexure interaction:

The beam experiences bending demands only in the strong axis. The bending demand due to gravity and seismic action is as follows:

$M_{Ed} = M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} = 110.4 + 1.1 \cdot 1.25 \cdot 1.84 \cdot 190.9 = 110.4 + 483 = 593\, kN.m$ (brace side)

h/b = 400/300 = 1.33; therefore, the buckling curve to be used is "*a*" (i.e., $a_{LT} = 0.21$), according to EC3.

Plastic bending resistance with respect to y-y axis

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 3230 \cdot 10^3 \cdot \frac{0.355}{1.00} = 1146.7\, kNm$$

Computation of critical moment

$z_g = 0$ (the cross section is doubly symmetric – assume that loads are passing through the cross-section shear center).

$C_1 = 1.75$; the superposition of gravity and seismic moment diagrams may be considered as triangular with a nearly zero moment at one end. This is a conservative assumption given that under gravity moment the moment diagram in the beam changes sign.

The steel beam is connected to the column with a shear connection; therefore, $k_v = k_\varphi = 1.0$.

$L = 2993\, mm$

Shear modulus: $G = \frac{E}{2 \cdot (1+v)} = 80.8 kN/mm^2$

Computation of torsional constant:

$$K = \frac{2 \cdot b_f \cdot t_f^3 + (h - t_f) \cdot t_w^3}{3} = \frac{2 \cdot 300 \cdot 24^3 + (400 - 24) \cdot 13.5^3}{3} = 8.3x10^6 mm^4$$

$$I_w = \frac{t_f \cdot \left(h - t_f\right)^2 \cdot b^3}{24} = \frac{24 \cdot (400 - 24)^2 \cdot 300^3}{24} = 1.435x10^{15} mm^6$$

Therefore, M_{cr} may be computed as follows:

$$M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{k_v k_\varphi (L_D)^2} \cdot \left(\frac{I_w}{I_z} \cdot \left(\frac{\left(k_\varphi \cdot L_D\right)^2 \cdot G \cdot K}{\pi^2 \cdot E \cdot I_\omega} + 1 \right) \right)^{0.5}$$

$$= 1.75 \cdot \frac{\pi^2 \cdot 200 \cdot 108 \cdot 10^6}{1.0 \cdot 1.0 \cdot (2993)^2}$$

$$\cdot \left(\frac{1.435x10^{15}}{108 \cdot 10^6} \cdot \left(\frac{(1.0 \cdot 2993)^2 \cdot 80.8 \cdot 8.3 \cdot 10^6}{\pi^2 \cdot 200 \cdot 1.435x10^{15}} + 1 \right) \right)^{0.5} = 151968\, kNm$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} \cdot f_y}{M_{cr}}} = \sqrt{\frac{3230 \cdot 0.355}{151968}} = 0.089$$

Since $\bar{\lambda}_{LT} < 0.2$, $\chi_{LT} = 1$.

<u>*N-M interaction*</u> (note that $M_{z,Ed} = 0$)

$$\frac{N_{Ed}}{\chi_z \cdot A \cdot \frac{f_y}{\gamma_{M1}}} + \frac{\omega_y}{1 - \frac{N_{Ed}}{N_{y,cr}}} \cdot \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{pl,y,Rd}}{\gamma_{M1}}} \leq 1$$

$\omega_y = 0.6$, since we assume a triangular moment diagram in the beam under examination.

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{l_k^2} = 3.14^2 \cdot 210 \cdot \frac{576.8 \cdot 10^6}{2993^2} = 133318.5\, kN$$

$$\chi_{LT} \frac{M_{pl,y,Rd}}{\gamma_{M1}} = 1.00 \cdot \frac{3230 \cdot\ 0.355}{1.05} = 1092\, kN.m$$

$$\frac{N_{Ed}}{\chi_z \cdot A \cdot \frac{f_y}{\gamma_{M1}}} + \frac{\omega_y}{1 - \frac{N_{Ed}}{N_{y,cr}}} \cdot \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{pl,y,Rd}}{\gamma_{M1}}} = \frac{1911}{5784} + \frac{0.6}{1 - \frac{1911}{133318.5}} \cdot \frac{593}{1092} = 0.33 + 0.33$$

$$= 0.66 < 1$$

Therefore, the beam satisfies all the checks for interaction of axial load and bending, and the seismic design is compliant with the seismic provisions for non-dissipative elements.