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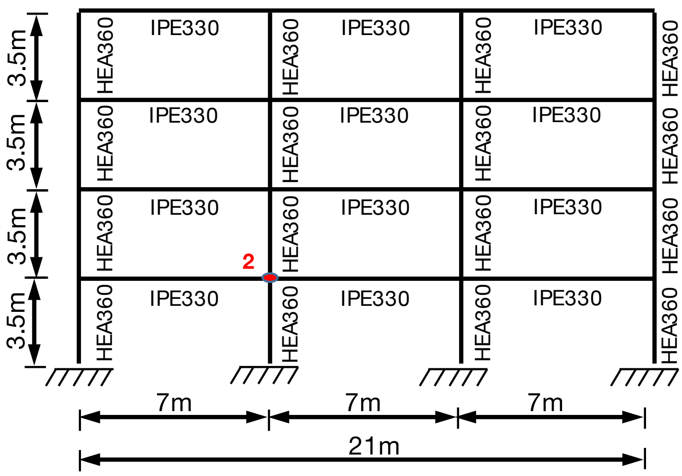
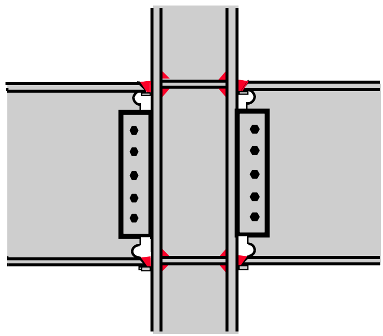
Exercise #6 – Seismic design of steel moment resisting frames

Problem #1: Seismic design of steel beams

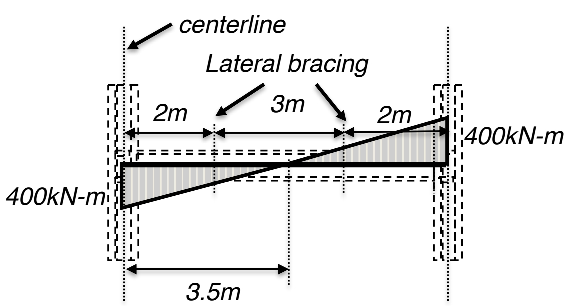
The steel moment-resisting frame (MRF) shown in Figure 1-1a has been designed in a high seismicity zone for gravity and seismic loading. The steel beam-to-column connections can be assumed as rigid as shown in Figure 1-1b. A behaviour factor of $q = 5$ has been adopted. The cross-sectional profiles shown in Figure 1a represent the final design of the steel MRF. Steel beams and columns have been designed with a S355J2 profile ($f_y = 355MPa$, $E = 210GPa$, $v = 0.3$). Steel beams are braced laterally at two locations as shown in Figure 1-1c. The moment distribution due to earthquake and gravity loading is shown in the same figure.



a. Moment resisting frame design



b. Rigid connection



c. Moment distribution in interior IPE330 steel beam and lateral support locations

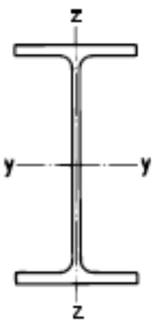
Figure 1-1. Steel moment-resisting frame design and typical beam-to-column connection detail

The following questions should be answered:

1. Check if the first-floor interior beam IPE330 is adequate in terms of cross section classification as discussed in SIA 263 or the Eurocode provisions (calculations should be explicitly done).
2. Check if the first-floor interior beam IPE330 satisfies the lateral bracing requirements for lateral torsional buckling. If not, compute the reduced flexural strength of the first-floor interior steel beam because of lateral torsional buckling. List your assumptions clearly. In any case, the flexural resistance of the beam, reduced by the effects of lateral torsional buckling, cannot be less than 210kNm.

IPE, PEA

IPE- und IPEA-Träger — ***Profilés IPE et IPEA***



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

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

$S_y = ½\, W_{ply}$

$S_z = ½\, W_{plz}$

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

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

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

Maximale Lagerlängen / *Longueurs maximales en stock:*
$h \le 180$ 18 m
$h \ge 200$ 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.*

		Statische Werte / *Valeurs statiques*												
IPE	m kg/m	A mm^2	A_v mm^2	A_w mm^2	I_y mm^4	W_{ely} mm^3	$\overline{W}_y$ mm^3	W_{ply} mm^3	i_y mm	I_z mm^4	W_{elz} mm^3	W_{plz} mm^3	i_z mm	$K = I_x$ mm^4
					x 10^6	x 10^3	x 10^3	x 10^3		x 10^6	x 10^3	x 10^3		x 10^6
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
PEA														
120	8,7	1100	542	428	2,57	43,8	45,8	49,9	48,3	0,224	7,00	11,0	14,2	0,0101
140	10,5	1340	620	501	4,35	63,3	66,0	71,6	57,0	0,364	9,98	15,5	16,5	0,0133
160	12,7	1620	780	604	6,89	87,8	91,2	99,1	65,3	0,544	13,3	20,7	18,3	0,0191
180	15,4	1960	920	733	10,6	120	124	135	73,7	0,819	18,0	28,0	20,5	0,0265
200	18,4	2350	1147	855	15,9	162	167	182	82,3	1,17	23,4	36,5	22,3	0,0402
220	22,2	2830	1355	1050	23,2	214	222	240	90,5	1,71	31,2	48,5	24,6	0,0559
240	26,2	3330	1631	1190	32,9	278	288	312	99,4	2,40	40,0	62,4	26,8	0,0820
270	30,7	3920	1875	1420	49,2	368	381	412	112	3,58	53,0	82,3	30,2	0,101
300	36,5	4650	2225	1760	71,7	483	498	542	124	5,19	69,2	107	33,4	0,131
330	43,0	5470	2699	2060	102	626	645	702	137	6,85	85,6	133	35,4	0,190
360	50,2	6400	2972	2280	145	812	839	907	151	9,44	111	172	38,4	0,269
400	57,4	7310	3578	2700	203	1020	1050	1140	167	11,7	130	202	40,0	0,350
450	67,2	8560	4226	3300	298	1330	1370	1490	186	15,0	158	246	41,9	0,462
500	79,4	10100	5047	4050	429	1730	1780	1950	206	19,4	194	302	43,8	0,636
550	92,1	11700	6030	4780	600	2190	2260	2480	226	24,3	232	362	45,5	0,879
600	108	13700	7014	5680	829	2780	2860	3140	246	31,2	283	442	47,7	1,21

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.

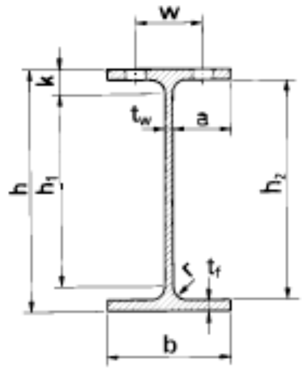
Les profilés PER, IPEo et IPEv figurent dans le programme de laminage de quelques aciéries. Les PEA 80 et PEA 100, également normalisés, sont peu économiques.

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

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

Walztoleranzen siehe Seite 116

Tolérances de laminage voir p. 116



IPE	m kg/m	Profilmasse *Dimensions de la section* h mm	b mm	t_w mm	t_f mm	r mm	Konstruktionsmasse *Dimensions de construction* h_1 mm	k mm	a mm	h_2 mm	w mm	$Ø_{max}$	Oberfläche *Surface* U_m m^2/m	U_t m^2/t	IPE
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	**750x 137**
750 x 147		753	265	13,2	17,0	17	685	34	126	719	120	M27	2,51	17,1	**750x 147**
750 x 173		762	267	14,4	21,6	17	685	39	126	719	120	M27	2,53	14,6	**750x 173**
750 x 196		770	268	15,6	25,4	17	685	42	126	719	120	M27	2,55	13,0	**750x 196**
PEA															**PEA**
120	**8,7**	118	64	3,8	5,1	7	93	12	30	107	36	M10	0,472	54,5	**120**
140	**10,5**	137	73	3,8	5,6	7	111	13	34	126	38	M10	0,547	52,1	**140**
160	**12,7**	157	82	4,0	5,9	9	127	15	39	145	44	M12	0,619	48,7	**160**
180	**15,4**	177	91	4,3	6,5	9	145	16	43	164	50	M12	0,694	45,1	**180**
200	**18,4**	197	100	4,5	7,0	12	159	19	47	183	56	M12	0,764	41,5	**200**
220	**22,2**	217	110	5,0	7,7	12	177	20	52	202	60	M16	0,843	38,0	**220**
240	**26,2**	237	120	5,2	8,3	15	189	24	57	220	68	M16	0,918	35,0	**240**
270	**30,7**	267	135	5,5	8,7	15	219	24	64	250	72	M20	1,04	33,9	**270**
300	**36,5**	297	150	6,1	9,2	15	247	25	71	279	80	M20	1,16	31,8	**300**
330	**43,0**	327	160	6,5	10,0	18	271	28	76	307	86	M24	1,25	29,1	**330**
360	**50,2**	357	170	6,6	11,5	18	297	30	81	334	90	M24	1,35	26,9	**360**
400	**57,4**	397	180	7,0	12,0	21	331	33	86	373	96	M27	1,46	25,4	**400**
450	**67,2**	447	190	7,6	13,1	21	377	35	91	421	106	M27	1,60	23,8	**450**
500	**79,4**	497	200	8,4	14,7	21	425	36	95	468	110	M27	1,74	21,9	**500**
550	**92,1**	547	210	9,0	15,7	24	467	40	100	516	120	M27	1,88	20,4	**550**
600	**108**	597	220	9,8	17,5	24	513	42	105	562	120	M27	2,01	18,6	**600**

Problem #2: Seismic design of welded beam-to-column connections

The steel moment-resisting frame (MRF) shown in Figure 2-1 has been designed in a high seismicity zone for gravity and earthquake loading. The cross sections shown in the figure represent the final design of the steel MRF in the East-West (EW) loading direction. Steel beams and columns have been designed with S355J2 ($f_y = 355MPa$) profiles. The total floor weight due to gravity loading is $G = 5kN/m^2$ (all included). A behaviour factor of $q = 4$ has been adopted as part of the design process. Welded beam-to-column connections are realised in the steel MRF.

The following questions should be addressed:

3. Do the steel beams satisfy the design requirements for the ductility class the MRFs are designed for?
4. Compute the flexural design resistance, $M_{pl,Rd,f}$ at the column face of the first-floor beam to be used for the seismic design calculations. Assume that the beam satisfies the lateral bracing requirements and can develop its full-plastic bending resistance (i.e., no need to check for lateral torsional buckling).
5. What are the requirements for the complete joint penetration welds between the beam flanges and the column face in terms of fracture toughness?
6. Compute the shear demand, V_{Ed}, of the first-floor interior steel beam.
7. Compute the flexural design resistance at the centre of the connection at joints 1 and 2.
8. Check if at nodes 1 and 2 the column flexural resistance, $M_{N,pl,Rd}$, is larger than that of the respective steel beams intersecting the joints.

Consider the following assumptions:

- Ignore the composite action due to the presence of the concrete slab.
- The axial and shear force diagrams of the columns are shown Figure 2-2.
- The flexural resistance of the columns may be approximated as follows (linear interaction):

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right)$$

N_{Ed} is the design axial load due to earthquake and gravity as shown in Figure 2-2. $N_{pl,Rd}$ is the axial design resistance of the steel column.

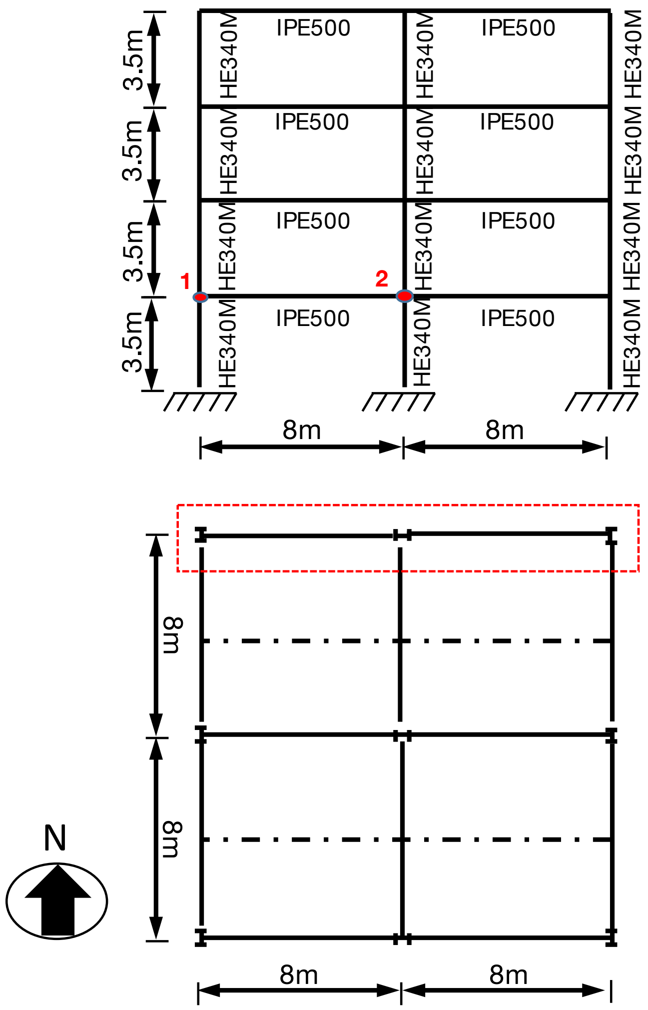
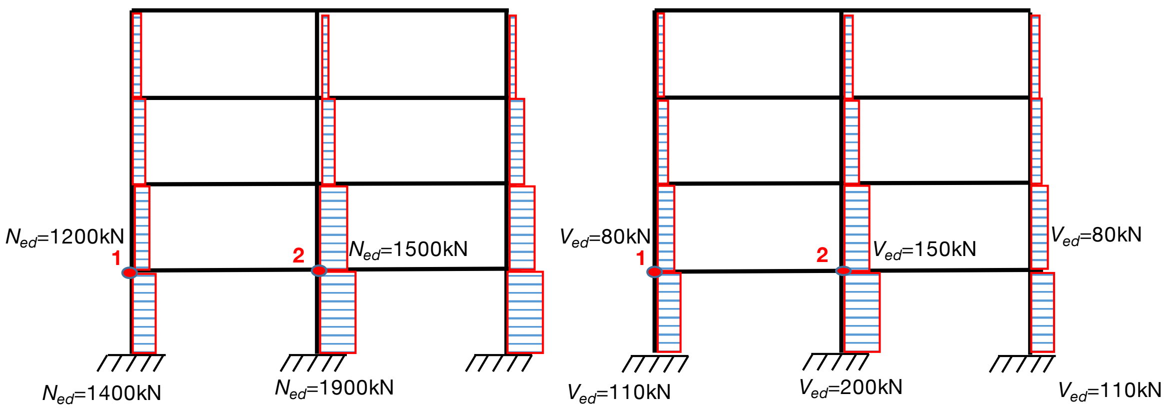


Figure 2-1. Final design of steel MRF



(a) axial force diagram (b) shear force diagram

Figure 2-2. Shear and axial force diagram for steel MRF columns

IPE, PEA

IPE- und IPEA-Träger — *Profilés IPE et IPEA*

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

$A_w = (h - t_f) \cdot t_w$ $\quad W_{ely} = \dfrac{I_y}{h/2}$

$S_y = ½\, W_{ply}$

$S_z = ½\, W_{plz}$ $\quad \overline{W}_y = \dfrac{I_y}{(h-t_f)/2}$

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

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

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○ *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.*

		Statische Werte / *Valeurs statiques*												
IPE	**m** 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
PEA														
120	8,7	1100	542	428	2,57	43,8	45,8	49,9	48,3	0,224	7,00	11,0	14,2	0,0101
140	10,5	1340	620	501	4,35	63,3	66,0	71,6	57,0	0,364	9,98	15,5	16,5	0,0133
160	12,7	1620	780	604	6,89	87,8	91,2	99,1	65,3	0,544	13,3	20,7	18,3	0,0191
180	15,4	1960	920	733	10,6	120	124	135	73,7	0,819	18,0	28,0	20,5	0,0265
200	18,4	2350	1147	855	15,9	162	167	182	82,3	1,17	23,4	36,5	22,3	0,0402
220	22,2	2830	1355	1050	23,2	214	222	240	90,5	1,71	31,2	48,5	24,6	0,0559
240	26,2	3330	1631	1190	32,9	278	288	312	99,4	2,40	40,0	62,4	26,8	0,0820
270	30,7	3920	1875	1420	49,2	368	381	412	112	3,58	53,0	82,3	30,2	0,101
300	36,5	4650	2225	1760	71,7	483	498	542	124	5,19	69,2	107	33,4	0,131
330	43,0	5470	2699	2060	102	626	645	702	137	6,85	85,6	133	35,4	0,190
360	50,2	6400	2972	2280	145	812	839	907	151	9,44	111	172	38,4	0,269
400	57,4	7310	3578	2700	203	1020	1050	1140	167	11,7	130	202	40,0	0,350
450	67,2	8560	4226	3300	298	1330	1370	1490	186	15,0	158	246	41,9	0,462
500	79,4	10100	5047	4050	429	1730	1780	1950	206	19,4	194	302	43,8	0,636
550	92,1	11700	6030	4780	600	2190	2260	2480	226	24,3	232	362	45,5	0,879
600	108	13700	7014	5680	829	2780	2860	3140	246	31,2	283	442	47,7	1,21

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.

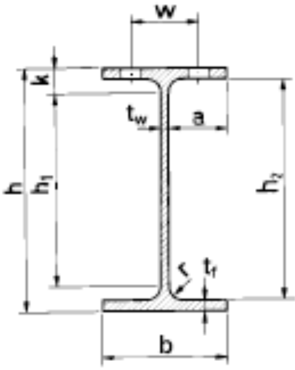
Les profilés PER, IPEo et IPEv figurent dans le programme de laminage de quelques aciéries. Les PEA 80 et PEA 100, également normalisés, sont peu économiques.

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

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

Walztoleranzen siehe Seite 116

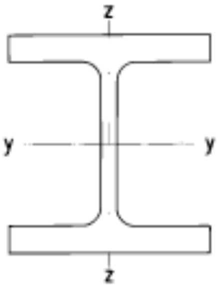
Tolérances de laminage voir p. 116



IPE	m kg/m	Profilmasse *Dimensions de la section* h mm	b mm	t_w mm	t_f mm	r mm	Konstruktionsmasse *Dimensions de construction* h_1 mm	k mm	a mm	h_2 mm	w mm	$Ø_{max}$	Oberfläche *Surface* U_m m²/m	U_t m²/t	IPE
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
PEA															**PEA**
120	8,7	118	64	3,8	5,1	7	93	12	30	107	36	M10	0,472	54,5	120
140	10,5	137	73	3,8	5,6	7	111	13	34	126	38	M10	0,547	52,1	140
160	12,7	157	82	4,0	5,9	9	127	15	39	145	44	M12	0,619	48,7	160
180	15,4	177	91	4,3	6,5	9	145	16	43	164	50	M12	0,694	45,1	180
200	18,4	197	100	4,5	7,0	12	159	19	47	183	56	M12	0,764	41,5	200
220	22,2	217	110	5,0	7,7	12	177	20	52	202	60	M16	0,843	38,0	220
240	26,2	237	120	5,2	8,3	15	189	24	57	220	68	M16	0,918	35,0	240
270	30,7	267	135	5,5	8,7	15	219	24	64	250	72	M20	1,04	33,9	270
300	36,5	297	150	6,1	9,2	15	247	25	71	279	80	M20	1,16	31,8	300
330	43,0	327	160	6,5	10,0	18	271	28	76	307	86	M24	1,25	29,1	330
360	50,2	357	170	6,6	11,5	18	297	30	81	334	90	M24	1,35	26,9	360
400	57,4	397	180	7,0	12,0	21	331	33	86	373	96	M27	1,46	25,4	400
450	67,2	447	190	7,6	13,1	21	377	35	91	421	106	M27	1,60	23,8	450
500	79,4	497	200	8,4	14,7	21	425	36	95	468	110	M27	1,74	21,9	500
550	92,1	547	210	9,0	15,7	24	467	40	100	516	120	M27	1,88	20,4	550
600	108	597	220	9,8	17,5	24	513	42	105	562	120	M27	2,01	18,6	600

HEM

Breitflanschträger HEM ***Profilés à larges ailes*** **HEM**



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

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

$S_y = ½\, W_{ply}$

$S_z = ½\, W_{plz}$

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

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

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

Maximale Lagerlängen /
Longueurs maximales en stock:
h ≤ 180 18 m
h ≥ 200 24 m

EURONORM 53 – 62, DIN 1025/4

Andere Bezeichnungen / *Autres désignations* } DIR, IPBv

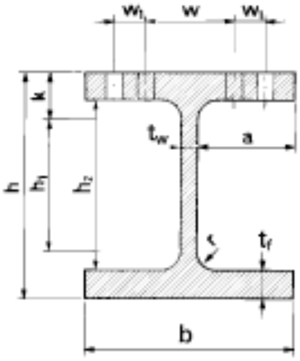
HEM	m	Statische Werte / *Valeurs statiques*												
	kg/m	A mm²	A_v mm²	A_w mm²	I_y mm⁴	W_{ely} mm³	$\overline{W}_y$ mm³	W_{ply} mm³	i_y mm	I_z mm⁴	W_{elz} mm³	W_{plz} mm³	i_z mm	$K = I_x$ mm⁴
					x 10⁶	x 10³	x 10³	x 10³		x 10⁶	x 10³	x 10³		x 10⁶
100	**41,8**	5320	1804	1200	11,4	190	229	236	46,3	3,99	75,3	116	27,4	0,673
120	**52,1**	6640	2115	1490	20,2	288	339	351	55,1	7,03	112	172	32,5	0,905
140	**63,2**	8060	2446	1790	32,9	411	477	494	63,9	11,4	157	240	37,7	1,19
160	**76,2**	9710	3081	2200	51,0	566	649	675	72,5	17,6	212	325	42,6	1,61
180	**88,9**	11300	3465	2550	74,8	748	850	883	81,3	25,8	277	425	47,7	2,01
200	**103**	13100	4103	2920	106,4	967	1090	1140	90,0	36,5	354	543	52,7	2,58
220	**117**	14900	4531	3320	146,0	1220	1360	1420	98,9	50,1	444	679	57,9	3,14
240	**157**	20000	6007	4280	242,9	1800	2040	2120	110	81,5	657	1010	63,9	6,27
260	**172**	22000	6689	4640	313,1	2160	2430	2520	119	104,5	780	1190	69,0	7,22
280	**189**	24000	7203	5120	395,5	2550	2860	2970	128	131,6	914	1400	74,0	8,09
300	**238**	30300	9053	6320	592,0	3480	3930	4080	140	194,0	1250	1910	80,0	14,1
320	**245**	31200	9485	6700	681,3	3800	4270	4440	148	197,1	1280	1950	79,5	15,1
340	**248**	31600	9863	7080	763,7	4050	4530	4720	156	197,1	1280	1950	79,0	15,2
360	**250**	31900	10240	7460	848,7	4300	4780	4990	163	195,2	1270	1940	78,3	15,2
400	**256**	32600	11020	8230	1041	4820	5310	5570	179	193,3	1260	1930	77,0	15,2
450	**263**	33500	11980	9200	1315	5500	6000	6340	198	193,4	1260	1940	75,9	15,4
500	**270**	34400	12950	10200	1619	6180	6690	7090	217	191,5	1250	1930	74,6	15,5
550	**278**	35400	13960	11200	1980	6920	7440	7930	236	191,6	1250	1940	73,5	15,6
600	**285**	36400	14970	12200	2374	7660	8190	8770	256	189,7	1240	1930	72,2	15,7
650	**293**	37400	15970	13200	2817	8430	8970	9660	275	189,8	1240	1940	71,3	15,9
700	**301**	38300	16980	14200	3293	9200	9740	10500	293	188,0	1240	1930	70,1	16,0
800	**317**	40400	19430	16300	4426	10870	11400	12500	331	186,3	1230	1930	67,9	16,6
900	**333**	42400	21440	18300	5704	12540	13100	14400	367	184,5	1220	1930	66,0	16,9
1000	**349**	44400	23500	20300	7223	14330	14900	16600	403	184,6	1220	1940	64,5	17,2

Breitflanschträger HEM — *Profilés à larges ailes* HEM

HEM

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

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



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

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

w_1 mit $Ø_{max}$ nur für versetzte Schrauben.

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

Walztoleranzen siehe Seite 116

Tolérances de laminage voir page 116

HEM	m	Profilmasse *Dimensions de la section*					Konstruktionsmasse *Dimensions de construction*							Oberfläche *Surface*		HEM
	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	41,8	120	106	12	20	12	56	32	47	80	60		M12	0,619	14,8	100
120	52,1	140	126	12,5	21	12	74	33	56	98	68		M16	0,738	14,2	120
140	63,2	160	146	13	22	12	92	34	66	116	76		M20	0,857	13,6	140
160	76,2	180	166	14	23	15	104	38	76	134	86		M20	0,970	12,7	160
180	88,9	200	186	14,5	24	15	122	39	85	152	100		M24	1,09	12,3	180
200	103	220	206	15	25	18	134	43	95	170	110		M24	1,20	11,7	200
220	117	240	226	15,5	26	18	152	44	105	188	120		M24	1,32	11,3	220
240	157	270	248	18	32	21	164	53	115	206	100	35	M24	1,46	9,30	240
260	172	290	268	18	32,5	24	176	57	125	225	110	40	M24	1,57	9,13	260
280	189	310	288	18,5	33	24	196	57	134	244	116	45	M24	1,69	8,94	280
300	238	340	310	21	39	27	208	66	144	262	120	50	M24	1,83	7,70	300
320	245	359	309	21	40	27	225	67	144	279	126	47	M27	1,87	7,63	320
340	248	377	309	21	40	27	243	67	144	297	126	47	M27	1,90	7,67	340
360	250	395	308	21	40	27	261	67	143	315	126	47	M27	1,93	7,77	360
400	256	432	307	21	40	27	298	67	143	352	126	47	M27	2,00	7,81	400
450	263	478	307	21	40	27	344	67	143	398	126	47	M27	2,10	7,97	450
500	270	524	306	21	40	27	390	67	142	444	130	45	M27	2,18	8,07	500
550	278	572	306	21	40	27	438	67	142	492	130	45	M27	2,28	8,20	550
600	285	620	305	21	40	27	486	67	142	540	130	45	M27	2,37	8,32	600
650	293	668	305	21	40	27	534	67	142	588	130	45	M27	2,47	8,42	650
700	301	716	304	21	40	27	582	67	141	636	130	42	M27	2,56	8,50	700
800	317	814	303	21	40	30	674	70	141	734	132	42	M27	2,75	8,66	800
900	333	910	302	21	40	30	770	70	140	830	132	42	M27	2,93	8,80	900
1000	349	1008	302	21	40	30	868	70	140	928	132	42	M27	3,13	8,97	1000

Problem 1 – Solution

1. According to EC8 or SIA-263, for $q > 4$, Class 1 cross section profiles are only permitted since the steel beam ends should be the dissipative zones. Therefore, steel beams should be checked for section classification:

IPE 330:

Flange subject to compression:

$$\frac{b}{t_f} = \frac{(0.5b_f - r - 0.5t_w)}{t_f} = \frac{(0.5 \cdot 160 - 18 - 0.5 \cdot 7.5)}{11.5} = 5.07 < 9 \cdot \sqrt{\frac{235}{355}} = 7.32$$

Therefore, the flange is classified as Class 1

Web subject to pure bending
$b = h - 2 \cdot t_f - 2 \cdot r = 330 - 2 \cdot 11.5 - 2 \cdot 18 = 271mm$ (you could also use h_1 from the table of SZS)

$$\frac{b}{t} = \frac{271}{7.5} = 36.13 < 72 \cdot \sqrt{\frac{235}{355}} = 58.58$$

Both flange and web are classified as Class 1; the IPE330 beams are Class 1 and comply with the seismic requirements for steel MRFs.

2. We should first check the allowable stable length for Class 1 profiles:

$M_{\mathrm{pl,Rd,b}} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = \frac{804 \cdot 10^3 \cdot 0.355}{1.0}$=285.4kNm

Note that $M_{\mathrm{pl,Rd,b}} < 400\mathrm{kNm}$ near the connection; However, we should check the M_{cr} because the question is asking to verify the stability requirements of the steel beam.

Moment at lateral bracing location, $M_{Ed,min} = 400 \cdot 1.5/3.5 = 171.4\mathrm{kNm}$.

$\psi = \frac{M_{Ed,min}}{M_{Ed,max}} = \frac{171.4}{400}$ =0.43 < 1, $\varepsilon = \sqrt{\frac{235}{355}} = 0.81$

Assume:
$L_1 = 2000mm - 350/2mm = 1825mm$ (note that you need to subtract the column depth contribution)
$L_2 = 3000mm$

$L_{stable} = (60 - 40 \cdot 0.43) \cdot \varepsilon \cdot i_z = 43 \cdot 0.813 \cdot 35.5 \sim 1241mm$ < 2000mm and 3000mm lateral bracing spacing. Therefore, the steel beam does not comply with the lateral stability checks. Its flexural resistance should be reduced by the effects of lateral torsional buckling. Because the moment diagram is different in segments L_2 and L_3 the check should be done in both segments.

To compute $M_{cr,b}$,

$$M_{cr,b} = \frac{C_1 \pi^2 E I_z}{k_v k_\varphi L_b^2} \sqrt{\frac{I_\omega}{I_z}\left(\frac{GK\left(k_\varphi L_b\right)^2}{\pi^2 E I_\omega} + 1\right)}$$

The beam is welded to the column face; therefore, fixed end boundary conditions may be assumed. Therefore,

$k_v = 1.0$ (for torsion) and $k_\varphi = 0.5$ (for warping)

For the segment with $L_1 = 1825mm$:

- The moment at the column face, $M_1 = 400 \cdot 3325/3500 = 380$kNm
- The moment at the other end of the beam, $M_2 = 171.4kNm$
- Therefore, κ = -171.4/380 = -0.45
- $C_1 = 1.75 - 1.05 \cdot 0.45 + 0.3 \cdot 0.45^2 = 1.33 < 2.3$
- $E = 210000MPa, v = 0.3, G = \frac{E}{2(1+v)} = 80769.2MPa$
- I_z = 7,88 x 10^6mm^4
- Torsional moment of inertia, K from structural mechanics (Note that the units of K are mm^4)
  $K = \frac{1}{3}\left(2b_f t_f^3 + \left(h - t_f\right)t_w^3\right) = \frac{1}{3}(2 \cdot 160 \cdot 11.5^3 + (330 - 11.5) \cdot 7.5^3) =$ $207015.7mm^4$ (note that value is 24% lower than the one listed in the SZS table in C5 because we ignore the radius cut, *r*)
- Warping constant, I_ω from structural mechanics (Note that the units of I_ω are mm^6)

$$I_\omega = \frac{\left(h - t_f\right)^2 b_f^3 t_f}{24} = \frac{(330 - 11.5)^2 160^3 11.5}{24} = 1{,}99x10^{11}mm^6$$

Therefore,

$$M_{cr,b} = \frac{1.33 \cdot \pi^2 \cdot 210000 \cdot 7{,}88x10^6}{1.0 \cdot 0.5 \cdot 1825^2} \sqrt{\frac{1{,}99x10^{11}}{7{,}88x10^6}\left(\frac{80769 \cdot 207015.7 \cdot (0.5 \cdot 1825)^2}{\pi^2 \cdot 210000 \cdot 1{,}99x10^{11}} + 1\right)} = 2121\ kNm$$

The reduction factor for lateral torsional buckling should be calculated as follows,

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{285.4}{2121}} = 0.367$$

$h/b = 330/160 = 2.06 > 2$ – for a rolled I-beam, EN 1993-1-1 uses the imperfection curve b; therefore, $a_{LT} = 0.34$.

$$\Phi_{LT} = 0.5 \cdot [1 + 0.34 \cdot (0.367 - 0.2) + 0.367^2] = 0.596$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = \frac{1}{0.596 + \sqrt{0.596^2 - 0.367^2}} = 0.938 < 1.0$$

Therefore, $M_{b,Rd} = 0.938 \cdot 285.4/1.05 = 255$ kNm

For the segment with $L_1 = 3000mm$:

- The moment at the column face, $M_1 = 171.4kNm$
- The moment at the other end of the beam, $M_2 = -171.4kNm$
- Therefore, $\kappa = 1$ (member in double curvature)
- $C_1 = 1.75 + 1.05 \cdot 1.00 + 0.3 \cdot 1.00^2 = 3.1 > 2.3 \rightarrow C_1 = 2.3$
- $E = 210000MPa, v = 0.3, G = \frac{E}{2(1+v)} = 80769.2MPa$
- $I_z = 7{,}88 \times 10^6 \text{mm}^4$
- Torsional moment of inertia, K from structural mechanics (Note that the units of K are mm^4)
  $K = \frac{1}{3}\left(2b_f t_f^3 + \left(h - t_f\right)t_w^3\right) = \frac{1}{3}(2 \cdot 160 \cdot 11.5^3 + (330 - 11.5) \cdot 7.5^3) = 207015.7mm^4$ (note that value is 24% lower than the one listed in the SZS table in C5 because we ignore the radius cut)
- Warping constant, I_ω from structural mechanics (Note that the units of I_ω are mm^6)

$$I_\omega = \frac{\left(h - t_f\right)^2 b_f^3 t_f}{24} = \frac{(330 - 11.5)^2 160^3 11.5}{24} = 1{,}99x10^{11}mm^6$$

Therefore,

$$M_{cr,b} = \frac{2.3 \cdot \pi^2 \cdot 210000 \cdot 7{,}88x10^6}{1.0 \cdot 0.5 \cdot 3000^2} \sqrt{\frac{1{,}99x10^{11}}{7{,}88x10^6}\left(\frac{80769 \cdot 207015.7 \cdot (0.5 \cdot 3000)^2}{\pi^2 \cdot 210000 \cdot 1{,}99x10^{11}} + 1\right)} = 1385.8\ kNm$$

The reduction factor for lateral torsional buckling should be calculated as follows,

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{285.4}{1385.8}} = 0.454$$

$h/b = 330/160 = 2.06 > 2$ – for a rolled I-beam EN 1993-1-1 uses an imperfection curve b; therefore, $a_{LT} = 0.34$.

$$\Phi_{LT} = 0.5 \cdot [1 + 0.34 \cdot (0.454 - 0.2) + 0.454^2] = 0.646$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = \frac{1}{0.646 + \sqrt{0.646^2 - 0.454^2}} = 0.905 < 1.0$$

Therefore, $M_{b,Rd} = 0.905 \cdot 285.4/1.05 = 246\ \text{kNm}$

Although we checked both segments, the one with the largest length becomes slightly more critical to reduce the plastic flexural resistance by the effects of lateral torsional buckling. Because the member is in double curvature, the moment diagram helps the member regarding lateral torsional buckling resistance.

Problem 2 – Solution

1. Design requirements for steel beams:

According to EC8, for ductility class medium (i.e., $2 < q \leq 4$) Class 1 & 2 cross sections should be used. Therefore, steel beams should be checked if they comply with the cross-section classifications:

IPE 500:

Flange subject to compression:

$$\frac{c}{t_f} = \frac{(0.5b - r - 0.5t_w)}{t_f} = \frac{(0.5 \cdot 200 - 21 - 0.5 \cdot 10.2)}{16} = 4.62 < 9 \cdot \sqrt{\frac{235}{355}} = 7.32$$

Therefore, the flange of the cross section is classified as Class 1.

Web subject to bending and compression.

$c = h - 2 \cdot t_f - 2 \cdot r = 500 - 2 \cdot 16 - 2 \cdot 21 = h_1 = 426$ mm

$$a = \frac{(z_b - t_f - r)}{c} = \frac{500/2 - 16 - 21}{426} = 0.5$$

When $a = 0.5$, it does not matter if you consider the limit for bending and tension or bending and compression because the two limits for c/t_w are the same at $a = 0.5$.

since $a = 0.5$, $c/t_w = 426/10.2 = 41.76 < 36\frac{\varepsilon}{a} = 58.58$

The beam web is Class 1; therefore, the IPE 500 profile is Class 1 and comply with the seismic requirements for steel MRFs for the selected q-factor.

Span-to-depth ratio (Note that one column is oriented with respect to the weak-axis and one with respect to the strong-axis):

$L_o/h = \{[8000 - 377/2 – 21/2)]/2\}/500 = 7.8 > 5$ OK ($q = 4$ has been selected)

2. Flexural resistance at the beam's cross section:

The flexural design resistance of the beam at the column face is as follows:

$$M_{pl,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = 2190 \cdot 10^3 \cdot 0.355 = 777.5 \text{ kNm}$$

To compute the design flexural resistance at the centre of connections 1 and 2, the shear demand, V_{Ed}, shall be computed first.

3. Requirements for complete joint penetration groove welds of the connections

These are demand critical welds; therefore, their CVN values should be as follows:

- 27 Joules at -30°C
- 54 Joules at 21°C

4. Shear demand V_{Ed}

Shear due to gravity load at the beam:

Uniform load on a steel beam
$G = 5 \cdot 4 = 20$ kN/m

$L_b = 8000 – h_c/2 - t_w/2 = 8000 - 377/2 - 21/2 = 7801$ m (Note that one column is oriented with its weak axis)

$V_{Ed,G} = 20 \cdot 7.801/2 = 78.0$ kN

$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov} \cdot (2 \cdot M_{pl,Rd}/L_b) = 78.0 + 1.1 \cdot 1.25 \cdot 2 \cdot 777.5/7.801 = 352.1$ kN

5. Connection flexural resistance at joint centres:

Joint 1 (1-beam intersecting the joint):
$M_{Rd} = 1.1 \cdot \gamma_{ov} \cdot M_{pl,Rd} + V_{Ed} \cdot t_w/2 = 1069 + 352.1 \cdot 21/2/1000 = 1073$ kNm

Joint 2 (2 beams intersecting the joint):
$M_{Rd} = 2 \cdot (1.1 \cdot \gamma_{ov} \cdot M_{pl,Rd} + V_{Ed} \cdot h_c/2) = 2 \cdot 837.6 = 2271$ kNm

6. Flexural resistance ratio comparisons at joints 1 and 2:

Joint 1: Column is oriented with the weak axis with respect to the loading direction of interest:

HE340M

$$M_{pl,Rd} = W_{pl,z} \cdot f_y/\gamma_{M0} = 1950 \cdot 10^3 \cdot 0.355 = 692.3 \text{ kNm}$$

$$N_{pl,Rd} = f_y \cdot A/\gamma_{M0} = 0.355 \cdot 31600 = 11218 \text{ kN}$$

Column above joint 1:

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 692.3 \cdot \left(1 - \frac{1200}{11218}\right) = 618.2 \text{ kNm}$$

Column below joint 1:

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 692.3 \cdot \left(1 - \frac{1400}{11218}\right) = 605.9 \text{ kNm}$$

Therefore,

Summation of column moments in the centre of the joint:

$$\sum M_c = 618.2 \; + \; 605.9 + 80 \cdot \frac{0.50}{2} + 110 \cdot \frac{0.50}{2} = 1271.6 \text{ kNm}$$

Ratio for Joint 1:

$$\frac{\sum M_c}{\sum M_b} = \frac{1271.6}{1073} = 1.19 < 1.30 \rightarrow \text{not OK (columns are not protected)}$$

Joint 2: Column is oriented with the strong axis with respect to the loading direction of interest:

HE340M
$N_{pl,Rd} \; = \; f_y \cdot A/\gamma_{M0} = 11218 \text{ kN}$
$M_{pl,Rd} \; = \; W_{pl,y} \cdot f_y/\gamma_{M0} = 4720 \cdot 10^3 \cdot 0.355 \; = \; 1675.6 \text{ kNm}$

Column above joint 2

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 1675.6 \cdot \left(1 - \frac{1500}{11218}\right) = 1451.5 \text{ kNm}$$

Column below joint 2

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 1675.6 \cdot \left(1 - \frac{1900}{11218}\right) = 1391.8 \text{ kNm}$$

Therefore,

Summation of column moments in the centre of the joint:

$$\sum M_c = 1451.5 + 1391.8 + 150 \cdot \frac{0.500}{2} + 200 \cdot \frac{0.500}{2} = 2930.9 \text{ kNm}$$

Strength ratio for Joint 2:

$$\frac{\sum M_c}{\sum M_b} = \frac{2930.9}{2271} = 1.29 \cong 1.30 \rightarrow \text{marginally OK (columns are protected)}$$