

Téléphone : +41 21 693 24 27  
Fax : +41 21 693 28 68  
E-mail : [dimitrios.lignos@epfl.ch](mailto:dimitrios.lignos@epfl.ch)  
Site web : <http://resslab.epfl.ch>  
Address: EPFL ENAC IIC RESSLAB  
GC B3 485, Station 18,  
CH-1015, Lausanne

### Exercise #10: Lateral Drift Analysis of Steel Frames with Eccentric Bracings

The steel office building shown in Figure 1 has been designed in Sion, Switzerland with steel frames with eccentric bracings (EBFs) in the East-West loading direction and steel moment-resisting frames (MRFs) in the North-South loading direction. Both frames have been designed for gravity and seismic loading. The cross sections shown in Figure 1 represent the final design of the EBF system. A behavior factor of  $q = 3$  has been considered for the EBF system. The steel components (beams, columns and braces) have been designed with S355 steel profile (i.e.,  $E = 200GPa$ ,  $f_y = 355MPa$ ). The stability coefficient  $\theta$  is less than 0.10 in all stories. All the EBF links are designed to be 500 mm long.

The design location in Sion, CH is site category D with a reference peak ground acceleration,  $a_{gR} = 0.20g$ . The gravity load at each floor of the building is  $G = 5kPa$ . Assume that this only contributes to the seismic mass.

The following questions should be answered:

1. Compute the design base shear,  $V_{d,EW}$ , in the East-West loading direction for one of the two steel frames with eccentric bracings.
2. Use the approximate drift analysis method to calculate the expected lateral storey drift demands at each storey for the seismic design action.
3. Compute the expected EBF link rotation angle,  $\gamma_p$ . Assuming that we have shear-critical links do the corresponding link rotation angle,  $\gamma_p$  satisfy the required limits?

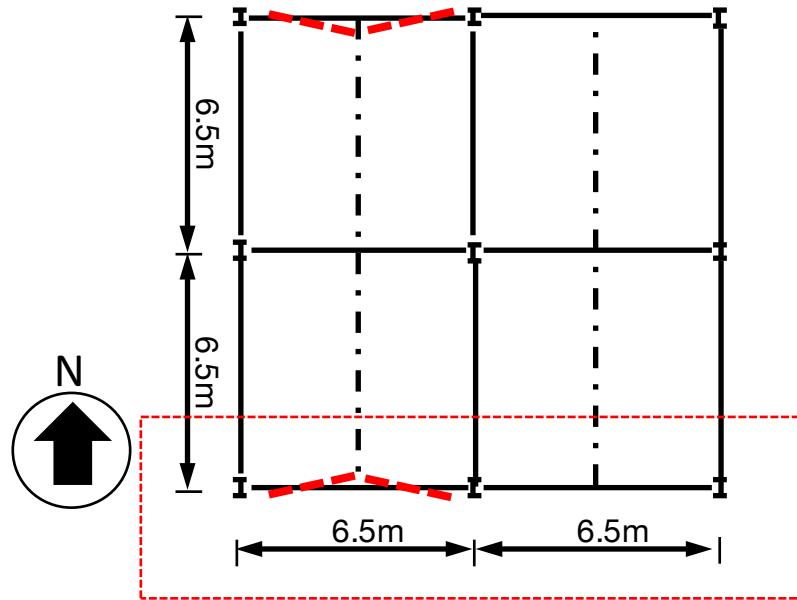
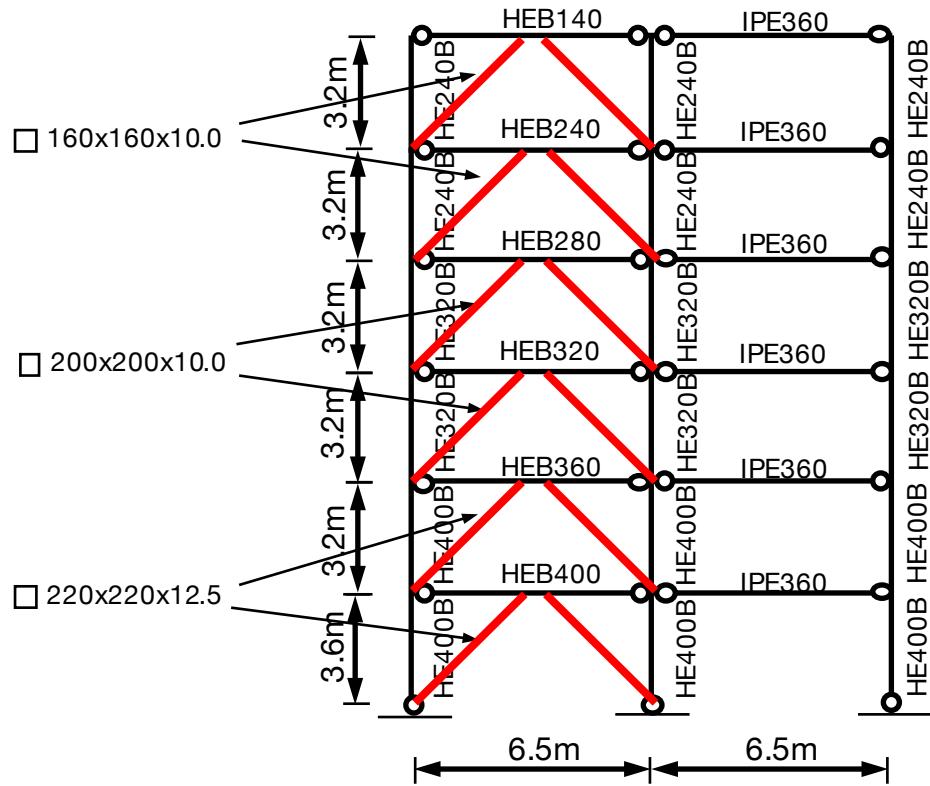
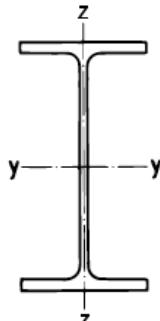


Figure 1. Final design of steel frame with eccentric bracings – plan view and elevation



$$\begin{aligned}
 A_v &= A - 2bt_f + (t_w + 2r)t_f \\
 A_w &= (h - t_f) \cdot t_w \quad W_{ely} = \frac{I_y}{h/2} \\
 S_y &= \frac{1}{2} W_{ply} \\
 S_z &= \frac{1}{2} W_{plz} \quad \bar{W}_y = \frac{I_y}{(h - t_f)/2} \\
 & \quad W_{elz} = \frac{I_z}{b/2}
 \end{aligned}$$

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!
- \* Auch in S355J0 oder S355J2 ab Schweizer Lager erhältlich.

- La méthode PP selon SIA 263 n'est pas applicable pour ce profil en acier S355 en flexion simple ( $n = 0$ )!
- \* Livrable en S355J0 ou S355J2 du stock suisse.

IPE	m kg/m	Statische Werte / Valeurs statiques												
		A mm <sup>2</sup>	A <sub>v</sub> mm <sup>2</sup>	A <sub>w</sub> mm <sup>2</sup>	I <sub>y</sub> mm <sup>4</sup>	W <sub>ely</sub> mm <sup>3</sup>	$\bar{W}_y$ mm <sup>3</sup>	W <sub>ply</sub> mm <sup>3</sup>	i <sub>y</sub> mm	I <sub>z</sub> mm <sup>4</sup>	W <sub>elz</sub> mm <sup>3</sup>	W <sub>plz</sub> mm <sup>3</sup>	i <sub>z</sub> mm	K = I <sub>x</sub> mm <sup>4</sup>
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

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

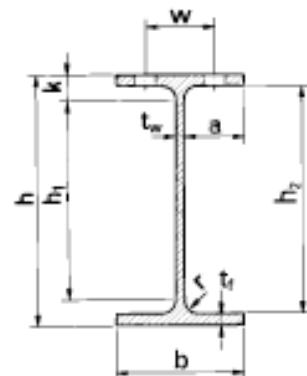
Les profilés PER, IPEo et IPEv figurent dans le programme de laminage de quelques acieries. Les PEA 80 et PEA 100, également normalisés, sont peu économiques.

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

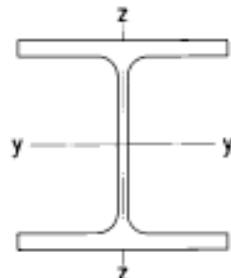


IPE	m kg/m	Profilmasse Dimensions de la section					Konstruktionsmasse Dimensions de construction						Oberfläche Surface	IPE	
		h mm	b mm	t_w mm	t_f mm	r mm	h <sub>1</sub> mm	k mm	a mm	h <sub>2</sub> mm	w mm	Ø <sub>max</sub>			
80	6,0	80	46	3,8	5,2	5	60	10	21	70			0,328	54,8	80
100	8,1	100	55	4,1	6,7	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	750 x 137
750 x 147		753	265	13,2	17,0	17	685	34	126	719	120	M27	2,51	17,1	750 x 147
750 x 173		762	267	14,4	21,6	17	685	39	126	719	120	M27	2,53	14,6	750 x 173
750 x 196		770	268	15,6	25,4	17	685	42	126	719	120	M27	2,55	13,0	750 x 196

# HEB

## Breitflanschträger HEB

## Profils à larges ailes HEB



$$\begin{aligned}
 A_v &= A - 2bt_f + (t_w + 2r)t_f & I_y &= \frac{I_y}{h/2} \\
 A_w &= (h - t_f) \cdot t_w & W_{ely} &= \frac{I_y}{h/2} \\
 S_y &= \frac{1}{2} W_{ply} & S_z &= \frac{1}{2} W_{plz} & W_y &= \frac{I_y}{(h - t_f)/2} \\
 W_y &= \frac{I_y}{(h - t_f)/2} & W_{elz} &= \frac{I_z}{b/2}
 \end{aligned}$$

\* Auch in S355J0 oder S355J2  
ab Schweizer Lager erhältlich

Maximale Lagerlängen /  
Longueurs maximales en stock:

$h \leq 180$  18 m

$h \geq 200$  24 m

EURONORM 53 - 62, DIN 1025/2

Andere Bezeichnungen } DIN, IPB  
Autres désignations } DIN, IPB

\* Livrable en S355J0 ou S355J2  
du stock suisse

HEB	m kg/m	Statische Werte / Valeurs statiques												
		A mm <sup>2</sup>	A <sub>v</sub> mm <sup>2</sup>	A <sub>w</sub> mm <sup>2</sup>	I <sub>y</sub> mm <sup>4</sup>	W <sub>ely</sub> mm <sup>3</sup>	W <sub>y</sub> mm <sup>3</sup>	W <sub>ply</sub> mm <sup>3</sup>	i <sub>y</sub> mm	I <sub>z</sub> mm <sup>4</sup>	W <sub>elz</sub> mm <sup>3</sup>	W <sub>plz</sub> mm <sup>3</sup>	i <sub>z</sub> mm	K = I <sub>x</sub> mm <sup>4</sup>
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	268	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.

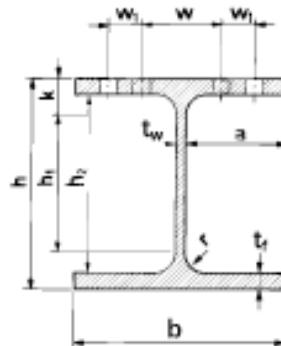
w<sub>1</sub> mit Ø<sub>max</sub> nur für versetzte Schrauben.

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, avec  $\emptyset_{max}$  seulement pour boulons décalés.*

*Tolérances de laminage voir page 116*



HEB	m kg/m	Profilmasse <i>Dimensions de la section</i>					Konstruktionsmasse <i>Dimensions de construction</i>						Oberfläche <i>Surface</i>	HEB		
		h mm	b mm	t <sub>w</sub> mm	t <sub>f</sub> mm	r mm	h <sub>1</sub> mm	k mm	a mm	h <sub>2</sub> mm	w mm	w <sub>1</sub> mm	Ø <sub>max</sub>			
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

RRW Abm./Dim. mm	m kg/m	Statische Werte / Valeurs statiques							Oberfläche Surface	
		A mm <sup>2</sup>	I mm <sup>4</sup>	W <sub>el</sub> mm <sup>3</sup>	W <sub>pl</sub> mm <sup>3</sup>	i mm	K=I <sub>x</sub> mm <sup>4</sup>	C <sub>t</sub> mm <sup>3</sup>	U <sub>m</sub> m <sup>2</sup> /m	U <sub>t</sub> m <sup>2</sup> /t
h · b · t			x 10 <sup>6</sup>	x 10 <sup>3</sup>	x 10 <sup>3</sup>		x 10 <sup>6</sup>	x 10 <sup>3</sup>		
150 · 150 · 5	22,6	2873	10,0	134	156	59,0	15,5	197	0,587	26,0
5,6	25,1	3201	11,1	147	173	58,8	17,2	217	0,586	23,3
6,3	28,1	3579	12,2	163	192	58,5	19,1	240	0,584	20,8
8	35,1	4475	14,9	199	237	57,7	23,5	291	0,579	16,5
10	43,1	5493	17,7	236	286	56,8	28,3	344	0,574	13,3
12,5	52,7	6707	20,8	277	342	55,7	33,7	402	0,568	10,8
16	65,2	8301	24,3	324	411	54,1	40,3	467	0,559	8,58
160 · 160 · 5	24,1	3073	12,2	153	178	63,1	18,9	226	0,627	26,0
6,3	30,1	3831	15,0	187	220	62,6	23,3	275	0,624	20,7
8	37,6	4795	18,3	229	272	61,8	28,8	335	0,619	16,5
10	46,3	5893	21,9	273	329	60,9	34,8	398	0,614	13,3
12,5	56,6	7207	25,8	322	395	59,8	41,6	467	0,608	10,7
16	70,2	8941	30,3	379	476	58,2	49,9	546	0,599	8,53
180 · 180 · 6,3	34,0	4335	21,7	241	281	70,7	33,6	355	0,704	20,7
8	42,7	5435	26,6	296	349	70,0	41,6	434	0,699	16,4
8,8	46,7	5943	28,8	320	379	69,6	45,2	469	0,697	14,9
10	52,5	6693	31,9	355	424	69,1	50,5	518	0,694	13,2
12,5	64,4	8207	37,9	421	511	68,0	60,7	613	0,688	10,7
16	80,2	10221	45,0	500	621	66,4	73,4	724	0,679	8,46
200 · 200 · 5	30,4	• 3873	24,5	• 245	• 283	79,5	37,6	362	0,787	25,9
6,3	38,0	4839	30,1	301	350	78,9	46,5	444	0,784	20,6
8	47,7	6075	37,1	371	436	78,1	57,8	545	0,779	16,3
10	58,8	7493	44,7	447	531	77,2	70,3	655	0,774	13,2
12,5	72,3	9207	53,4	534	643	76,1	84,9	778	0,768	10,6
16	90,3	11501	63,9	639	785	74,6	103	927	0,759	8,40
220 · 220 · 6,3	41,9	5343	40,5	368	• 427	87,1	62,4	544	0,864	20,6
8	52,7	6715	50,0	455	532	86,3	77,6	669	0,859	16,3
10	65,1	8293	60,5	550	650	85,4	94,7	807	0,854	13,1
12,5	80,1	10207	72,5	659	789	84,3	115	963	0,848	10,6
250 · 250 · 6,3	47,9	• 6099	60,1	• 481	• 556	99,3	92,4	712	0,984	20,5
7,1	53,7	6844	67,0	536	• 622	99,0	103	792	0,982	18,3
8	60,3	7675	74,5	596	694	98,6	115	880	0,979	16,3
10	74,5	9493	90,6	724	851	97,7	141	1065	0,974	13,1
12,5	91,9	11707	109	873	1037	96,6	172	1279	0,968	10,5
16	115	14701	133	1061	1280	95,0	211	1546	0,959	8,31
260 · 260 · 7,1	56,0	7128	75,7	582	• 674	103	116	861	1,02	18,3
8,8	68,8	8759	91,6	705	822	102	142	1039	1,02	14,8
11	85,0	10826	111	855	1006	101	173	1255	1,01	11,9
14,2	108	13745	137	1055	1259	99,9	217	1542	1,00	9,30
300 · 300 · 6,3	57,8	• 7359	105	• 703	• 809	120	161	1043	1,18	20,5
8	72,8	• 9275	131	• 875	• 1013	119	202	1294	1,18	16,2
10	90,2	11493	160	1068	1246	118	248	1575	1,17	13,0
12,5	112	14207	194	1296	1525	117	303	1904	1,17	10,5
16	141	17901	238	1590	1895	115	376	2325	1,16	8,2
350 · 350 · 8	85,4	• 10875	211	• 1207	• 1392	139	324	1789	1,38	16,2
10	106	13493	259	1479	• 1715	139	399	2185	1,37	13,0
12,5	131	16707	315	1802	2107	137	489	2654	1,37	10,4
16	166	21101	389	2225	2630	136	610	3264	1,36	8,20
400 · 400 · 10	122	• 15493	391	• 1956	• 2260	159	601	2895	1,57	12,9
12,5	151	19207	478	2392	2782	158	739	3530	1,57	10,4
16	191	24301	593	2967	3484	156	924	4362	1,56	8,17

=

## Suggested Solution

### Question 1

Given:  $a_{gR} = 0.20g$  and soil type D, the Type 1 design spectrum is defined with the following values:  $S = 1.35$ ,  $T_B = 0.20s$ ,  $T_C = 0.8s$ ,  $T_D = 2s$ .

The design base shear will be obtained using the Type 1 design spectrum. However, we first need to estimate the first mode vibration period,  $T_1$ , of the office building in the East-West loading direction. For this reason, we can use the approximate period formula that is a function of height:

For buildings with heights up to 40 meters,  $T_1 = C_t \cdot H^{3/4}$

For steel frames with eccentric bracings,  $C_t = 0.085$ . The corresponding height of the building in this case is,  $H = 5 \cdot 3.2 + 3.6 = 19.6m$ .

Therefore,  $T_1 = 0.085 \cdot 19.6^{\frac{3}{4}} = 0.79sec$

Therefore,  $T_B < T_1 < T_C$ ; hence,

$$S_d(T) = a_{gR} \cdot S \cdot \frac{2.5}{q}$$

Thus, for each direction, we obtain the following design accelerations:

$$S_d(T_{NS}) = 0.20 \cdot 9.81 \cdot 1.35 \cdot \frac{2.5}{3} = 2.21 \text{ m/s}^2$$

Seismic mass: In this case, we assume that the seismic mass is only attributed due to gravity loading. Therefore,

$$W = 6 \cdot (5kPa \cdot 13m \cdot 13m) = 5070kN$$

Therefore, the design base shear per frame is as follows:

$$V_{d,EW} = S_d \cdot m_{tot} = 2.21 \cdot \frac{5070/2}{9.81} = 570.4kN$$

## Question 2

In order to use the approximate drift analysis method, we should first distribute the design base shear based on the equivalent lateral force method; therefore, per frame with eccentric bracings:

$$F_i = V_{d,EW} \cdot \frac{z_i \cdot m_i}{\sum_{j=1:N} (z_j \cdot m_j)}$$

Floor, $i$	Weight, $W$ [kN]	Mass, $m_i$ $\left[ kN \cdot \frac{s^2}{m} \right]$	$z_i$ [m]	$z_i \cdot m_i$ [m]	$F_i$ [kN]
6	845	86	19.6	1688	161
5	845	86	16.4	1413	134
4	845	86	13.2	1137	108
3	845	86	10.0	861	82
2	845	86	6.8	586	56
1	845	86	3.6	310	30

The flexural component of deflection of the frame with eccentric bracings may be calculated in the following steps:

**Step 1:** compute the moment of inertia of the column sectional area about their centroid by using the Steiner's theory:

$$I_c \approx 2 \times A_c \left( \frac{L}{2} \right)^2 = \frac{A_c L^2}{2}$$

As an example, for storey 3,

$$I_1 \approx \frac{A_c L^2}{2} = \frac{16100 \times 6500^2}{2} = 3.40 \times 10^{11} \text{ mm}^4$$

**Step 2:** Compute the value of the external moment,  $M$  at each mid-storey level. For example, for storey 3,

$$M = 82 \cdot \frac{3.2}{2} + 108 \cdot \left( \frac{3.2}{2} + 3.2 \right) + 134 \cdot \left( \frac{3.2}{2} + 3.2 + 3.2 \right) + 161 \cdot \left( \frac{3.2}{2} + 3.2 + 3.2 + 3.2 \right) = 3524524 \text{ kN} \cdot \text{mm}$$

**Step 3:** Determine for each storey the value of  $hM/EI_1$ . For example, for storey 3,

$$\frac{hM}{EI_1} = \delta\theta_{3,flexure} = \frac{3200 \times 3524524}{3.4 \times 10^{11} \times E} = \frac{0.0332}{E}$$

**Step 4:** Determine for each storey,  $i$ , the accumulation of value of  $\delta\theta_{i,flexure}$  from storey 1 up to storey 6. For example, the accumulation of  $\delta\theta_{i,flexure}$  up to storey 6 is:

$$\sum_{i=1}^6 h_i \left( \frac{M}{EI} \right)_i = \frac{0.0608 + 0.0395 + 0.0332 + 0.0198 + 0.0141 + 0.0037}{E} = \frac{0.171}{E}$$

**Step 5:** Record the produce of  $h_i$  and  $\theta_{i,flexure}$ . For example, in storey 3 due to flexure:

$$\delta_{3,flexure} = 3200x \frac{(0.0608 + 0.0395 + 0.0332)}{E} = \frac{427}{E} \text{ mm}$$

**Step 6:** At each level where the value of the lateral drift is required, evaluate the accumulation of the storey drifts,  $\delta_{i,flexure}$  from storey 1 up to the considered  $n$ th floor, to give the drift  $\Delta_{flexure}^{(n)}$ . For example, in storey 6 due to flexure:

$$\Delta_{5,flexure} = \frac{219 + 321 + 427 + 490 + 535 + 547}{E} = \frac{2539}{200} = 12.7 \text{ mm}$$

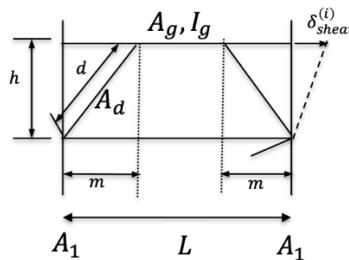
In summary, the flexural deflections along the 6-storey frame are as follows:

Storey	Frame Inertia $I_i$ [mm $^4$ ]	External Moment $M_i$ [kN-mm]	$\delta\theta_i$ [rad/E]	Storey Inclination $\theta_{ii}$ [rad/E]	Storey Drift $\delta_{if}$ [mm/E]	$\Sigma\delta_{if}$ [mm/E]	$\Delta_{m,flexure}$ [mm]
6	2.24E+11	256997	0.0037	0.171	547	2539	12.7
5	2.24E+11	986028	0.0141	0.167	535	1992	10.0
4	3.40E+11	2103176	0.0198	0.153	490	1457	7.3
3	3.40E+11	3524524	0.0332	0.133	427	967	4.8
2	4.18E+11	5166155	0.0395	0.100	321	540	2.7
1	4.18E+11	7058227	0.0607	0.061	219	219	1.1

The shear component of deflection of the frame with eccentric bracings may be calculated in the following steps:

**Step 1:** Compute the value of the external shear  $V_i$  acting in each storey  $i$  due to seismic loading. For instance, for storey 1,  $V_1 = 570.4 \text{ kN}$

**Step 2:** Compute for each storey  $i$  the storey drift due to shear,  $\delta_{i,shear}$ , by substituting the value of the storey shear and member properties into the appropriate formula for frames with eccentric bracings.



$$\delta_{shear}^{(i)} = \frac{V}{E} \cdot \left[ \frac{d^3}{2m^2 A_d} + \frac{m}{2A_g} + \frac{h^2(L-2m)^2}{12I_g L} \right]$$

And using this to compute the drift in storey 3 due to shear,

$$\delta_3^s = \frac{485.1}{200} \cdot \left[ \frac{4386.3^3}{2 \cdot 3000^2 \cdot 7493} + \frac{3000}{2 \cdot 16100} + \frac{3200^2(6500 - 2 \cdot 3000)^2}{12 \cdot 308.2 \cdot 10^6 \cdot 6500} \right] = 2.0 \text{ mm}$$

**Step 3:** Sum the storey drifts due to shear up to and including storey 6 to obtain the total shear drift at floor levels 6. For example, the drift due to shear at floor 3:

$$\Delta_{shear}^{(3)} = 2.0 + 1.7 + 2.0 = 5.7 \text{ mm}$$

In summary, the shear deflections along the 6-storey frame are as follows:

Storey	Storey Height $h_i$ [mm]	Shear $V_i$ [kN]	Storey Drift $\delta_{is}$ [mm]	$\Delta_{m,shear}$ [mm]	$\Delta_{total}$ [mm]
6	3200	160.6	2.7	12.0	24.7
5	3200	295.0	1.8	9.3	19.3
4	3200	403.2	1.8	7.5	14.8
3	3200	485.1	2.0	5.7	10.5
2	3200	540.9	1.7	3.7	6.4
1	3600	570.4	2.0	2.0	3.1

In the last column of the above table, the total deflection due to shear and flexure is summarized.

Note herein that in reality, the storey drift of storey  $i$  is given by the integration of the curvature along its height,  $h_i$ . The assumption we did to solve the problem with the approximate drift analysis technique is that we assumed that the bending moment from floor to floor is constant over the height; hence, the curvature is also constant. This would naturally give us larger estimates of displacement demands. However, the expected differences from the real solution is not more than 10% (for shorter buildings) or 20% (for taller ones).

Therefore, the corresponding storey drift ratio,  $SDR_i$  for each storey is,

$$SDR_i = \left( \frac{\Delta_{total}^i - \Delta_{total}^{i-1}}{h_i} \right) \cdot q$$

Note that the above storey drift ratios should be multiplied by the behaviour factor,  $q$ , to estimate the real drifts of the structure (equal displacement rule) as discussed during week 2 of Advanced Steel Design. Therefore,

Storey	Storey Height $h_i$ [mm]	Shear $V_i$ [kN]	$\Delta_{total}$ [mm]	$SDR_i$ [rads]	$\gamma_p$ [rads]
6	3200	160.6	24.7	0.005	0.066
5	3200	295.0	19.3	0.004	0.055
4	3200	403.2	14.8	0.004	0.052
3	3200	485.1	10.5	0.004	0.050
2	3200	540.9	6.4	0.003	0.040
1	3600	570.4	3.1	0.003	0.034

### Question 3

Regarding the allowable EBF rotational angle,

$$\theta_{p,i} = SDR_i$$

Therefore, with  $L = 6500mm$  and  $e = 500mm$ :

$$\gamma_{p_i} = \frac{L}{e} \cdot \theta_{p_i} = \frac{6500}{500} \cdot \theta_{p_i}$$

The link rotation angles per storey have been calculated in the last column of the table above. Assuming that the links are all shear critical (based on Question 3) the upper limit of  $0.08 \text{ rads}$  for shear-critical links is satisfied.