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Exercise #11: Seismic intervention of existing structures with friction dampers

The steel frame building shown in Figure 1 was designed in 1970s with concentrically braced frames (CBFs) in the North-South (y-y) loading direction and steel moment resisting frames (MRFs) in the East-West (x-x) loading direction without capacity design considerations in Valais (Zone Z3b, Soil Type E). This is an administrative building that must remain functional after an earthquake (building category: COIII according to SIA 261).

An engineering office did a seismic evaluation of the existing building and found that the X-bracing configuration with conventional bracing members (see Figure 2a) is likely to experience bracing connection fractures during the 475-year earthquake of SIA 261. In the MRF direction, the overstrength $\Omega = 2.0$. A seismic intervention (retrofitting) is planned in the y-y loading direction with the use of friction dampers. A single diagonal friction damper may be used to resist the seismic force demands (see Figure 2b). The behaviour of the friction damper may be assumed to be the typical Coulomb type as shown in Figure 3. However, the existing structure should be carefully evaluated to prevent lateral drift demands in the y-y direction as well as overloading of the non-dissipative members after the friction damper installation.

The owner has specified that the lateral storey drift ratios of the building should not exceed 1% of each storey height for a seismic action corresponding to two times the 475-year earthquake for the design location according to SIA 261.

The steel members (beams, columns) of the existing steel frame building have been designed with S355J2 profile (i.e., $E = 210GPa$, $f_y = 355MPa$). The stability coefficient θ is less than 0.10 in all stories in both loading directions. The weight of each floor due to gravity (for all three floors) equals to $G = 7kN/m^2$.

The following questions should be answered:

1. Calculate the corresponding size of the bracing element of the friction damper (made of RRK profile with S355J2 steel, i.e., $E = 210GPa$, $f_y = 355MPa$) to satisfy the retrofitting objectives by assuming that the structure (including the friction dampers) will remain elastic (as if $q = 1$). Because we are using a supplemental damping device, we can assume in this case that the equivalent damping ratio is $\xi = 10\%$ (this is typically verified by doing a qualification test of the damper).
2. What could be the required slip load, N_s, of the friction damper? Explain your assumptions.

3. How many prestressed structural bolts of 10.9 grade should be used for the slip load N_s that you assumed? Assume that the friction damper to be used has a friction coefficient, $\mu = 0.20$.
4. Check the stability of the first story column for the interaction of axial load and biaxial bending. The steel column is pinned in the $y-y$ loading direction. The column is fixed at the base in the $x-x$ loading direction. The force diagrams for the end column of the interior steel MRF are shown in Figure 4. You may assume that the buckling length of the column in the (sway permitted) MRF direction is $1.5L$ (i.e., L is the column length). Does the existing steel column satisfy the stability checks?

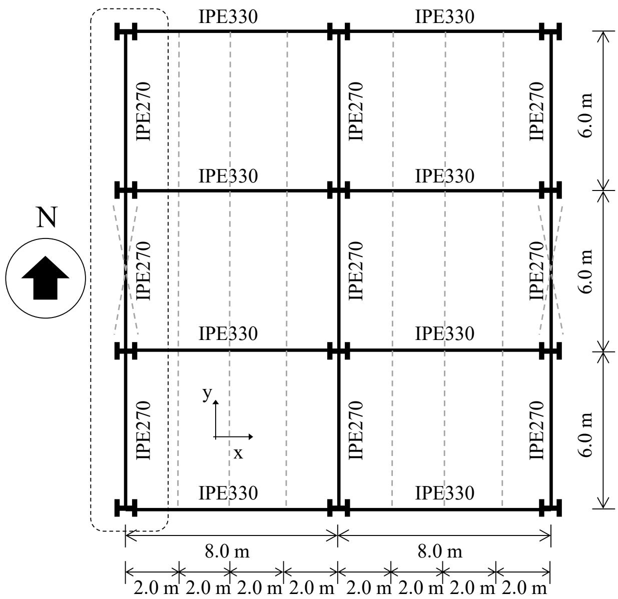


Figure 1. Plan view of the building

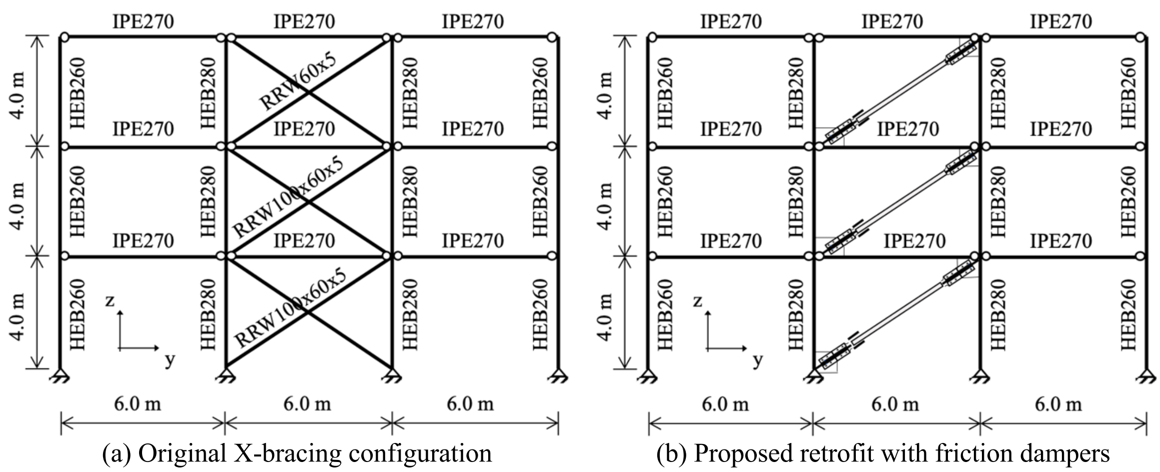


Figure 2. Steel frame with bracings

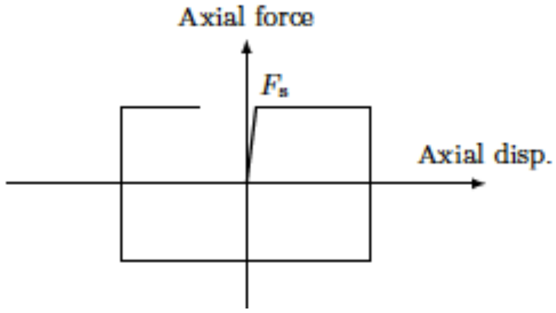


Figure 3. Friction damper hysteretic behaviour (Coulomb type)

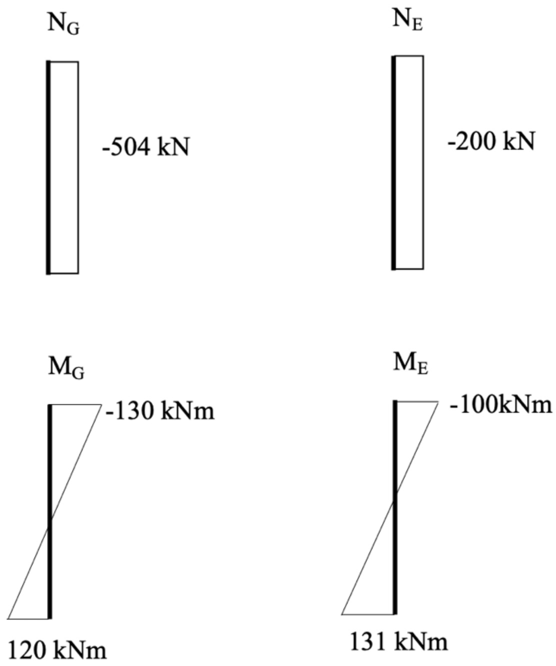
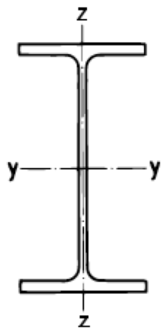


Figure 4. Force diagrams for end first-storey steel column of the interior MRF in the x-direction due to gravity (G) and earthquake (E) loading

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:
$h \leq 180$ 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 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.

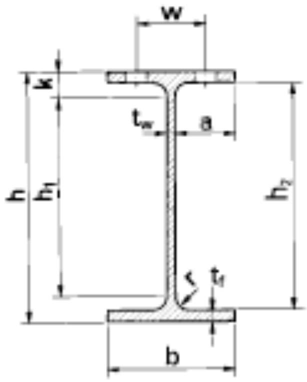
Les profilés PER, IPEo et IPEv figurent dans le programme de laminage de quelques aciéries. Les PEA 80 et PEA 100, également normalisés, sont peu économiques.

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

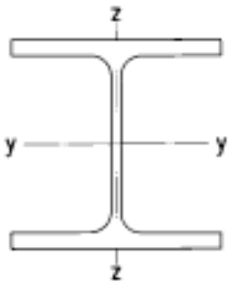
Tolérances de laminage voir p. 116



IPE	m	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*



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

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

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

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

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

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

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

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

EURONORM 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 x 10⁶	W_{ely} mm³ x 10³	$\overline{W}_y$ mm³ x 10³	W_{ply} mm³ x 10³	i_y mm	I_z mm⁴ x 10⁶	W_{elz} mm³ x 10³	W_{plz} mm³ x 10³	i_z mm	K – I_x mm⁴ x 10⁶
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.

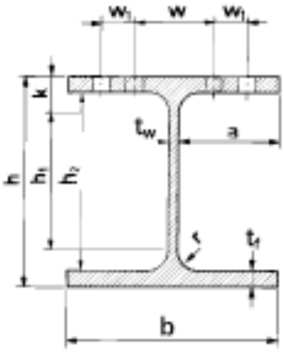
w_1 mit $Ø_{max}$ nur für versetzte Schrauben.

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

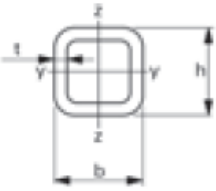


HEB	m	Profilmasse *Dimensions de la section*					Konstruktionsmasse *Dimensions de construction*							Oberfläche *Surface*		HEB
	kg/m	h mm	b mm	t_w mm	t_f mm	r mm	h_1 mm	k mm	a mm	h_2 mm	w mm	w_1 mm	$Ø_{max}$	U_m m²/m	U_t m²/t	
100	**20,4**	100	100	6	10	12	56	22	47	80	56		M12	0,567	27,8	**100**
120	**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

RRK

Quadratische Hohlprofile **kaltgefertigt**

Profils creux carrés ***finis à froid***



Produktbezeichnungen / *Désignations de produits:* **Rautaruukki (RAUTA), VHP, Alessio, Condesa, Tubeurop**

Abmessungen und statische Werte gemäss Norm EN 10 219-2 (Auswahl)

Lagerlängen 12 m (18 m)

- Örtliches Beulen vgl. Seiten 56+57

Erläuterungen S. 20/21, Toleranzen S. 118

Lieferprogramm siehe Seite 56
Bezugsquellen siehe Seite 64

Stahlsorte S355J2H ab Schweizer Lager: siehe Hinweis Seite 59.

Dimensions et valeurs statiques selon norme EN 10 219-2 (sélection)

Longueurs usuelles en stock 12 m (18 m)

- *Voilement local voir pages 56+57*

Explications p. 20/21, tolérances p. 118

Programme de livraison voir page 56
Fournisseurs voir page 64

Nuance d'acier S355J2H du stock suisse: voir indication page 59.

RRK Abm./*Dim.*		Statische Werte / *Valeurs statiques*							Oberfläche *Surface*	
	m	A	I	W_{el}	W_{pl}	i	$K=I_x$	C_t	U_m	U_t
mm	kg/m	mm^2	mm^4	mm^3	mm^3	mm	mm^4	mm^3	m^2/m	m^2/t
h · b · t			x 10^6	x 10^3	x 10^3		x 10^6	x 10^3		
40 · 40 · 3	3,30	421	0,093	4,66	5,72	14,9	0,158	7,07	0,150	45,3
4	4,20	535	0,111	5,54	7,01	14,4	0,194	8,48	0,146	34,8
50 · 50 · 3	4,25	541	0,195	7,79	9,39	19,0	0,321	11,8	0,190	44,7
4	5,45	695	0,237	9,49	11,7	18,5	0,404	14,4	0,186	34,2
5	6,56	836	0,270	10,8	13,7	18,0	0,475	16,6	0,183	27,9
6	7,56	963	0,295	11,8	15,3	17,5	0,532	18,2	0,179	23,7
60 · 60 · 3	5,19	661	0,351	11,7	14,0	23,1	0,571	17,7	0,230	44,3
4	6,71	855	0,436	14,5	17,6	22,6	0,726	22,0	0,226	33,7
6	9,45	1203	0,561	18,7	23,7	21,6	0,984	28,6	0,219	23,2
70 · 70 · 4	7,97	1015	0,721	20,6	24,8	26,7	1,19	31,1	0,266	33,4
5	9,70	1236	0,846	24,2	29,6	26,2	1,42	36,7	0,263	27,1
80 · 80 · 4	9,22	1175	1,11	27,8	33,1	30,7	1,80	41,8	0,306	33,2
5	11,3	1436	1,31	32,9	39,7	30,3	2,18	49,7	0,303	26,9
6	13,2	1683	1,49	37,3	45,8	29,8	2,52	56,6	0,299	22,7
8	16,4	2084	1,68	42,1	53,9	28,4	3,07	66,6	0,286	17,5
90 · 90 · 4	10,5	1335	1,62	36,0	42,6	34,8	2,61	54,2	0,346	33,0
5	12,8	1636	1,93	42,9	51,4	34,3	3,16	64,7	0,343	26,7
8	18,9	2404	2,55	56,6	71,3	32,5	4,56	88,8	0,326	17,3
100 · 100 · 3	8,96	1141	1,77	35,4	41,2	39,4	2,79	53,2	0,390	43,5
4	11,7	1495	2,26	45,3	53,3	38,9	3,62	68,1	0,386	32,9
5	14,4	1836	2,71	54,2	64,6	38,4	4,41	81,7	0,383	26,6
6	17,0	2163	3,11	62,3	75,1	37,9	5,14	94,1	0,379	22,3
8	21,4	2724	3,66	73,2	91,1	36,7	6,45	114	0,366	17,1
10	25,6	3257	4,11	82,2	105,2	35,5	7,50	130	0,357	14,0
110 · 110 · 4	13,0	1655	3,06	55,6	65,2	43,0	4,86	83,6	0,426	32,8
120 · 120 · 3	10,8	• 1381	3,12	• 52,1	• 60,2	47,6	4,88	78,2	0,470	43,3
4	14,2	1815	4,02	67,0	78,3	47,1	6,37	101	0,466	32,7
5	17,5	2236	4,85	80,9	95,4	46,6	7,78	122	0,463	26,4
6	20,7	2643	5,62	93,7	112	46,1	9,13	141	0,459	22,1
8	26,4	3364	6,77	113	138	44,9	11,6	175	0,446	16,9
10	31,8	4057	7,77	129	162	43,8	13,8	203	0,437	13,7
140 · 140 · 5	20,7	2636	7,91	113	132	54,8	12,6	170	0,543	26,2
6	24,5	3123	9,20	131	155	54,3	14,8	198	0,539	22,0
8	31,4	4004	11,3	161	194	53,0	19,0	248	0,526	16,7
10	38,1	4857	13,1	187	230	52,0	22,7	291	0,517	13,6
150 · 150 · 4	18,0	• 2295	8,08	• 108	• 125	59,3	12,6	162	0,586	32,5
5	22,3	2836	9,82	131	153	58,9	15,5	197	0,583	26,2
6	26,4	3363	11,5	153	180	58,4	18,3	230	0,579	21,9
8	33,9	4324	14,1	188	226	57,1	23,6	289	0,566	16,7
10	41,3	5257	16,5	220	269	56,1	28,4	341	0,557	13,5
160 · 160 · 4	19,3	• 2455	9,87	• 123	• 143	63,4	15,4	185	0,626	32,5
5	23,8	3036	12,0	150	175	62,9	19,0	226	0,623	26,1
6	28,3	3603	14,1	176	206	62,5	22,4	264	0,619	21,9
8	36,5	4644	17,4	218	260	61,2	29,0	334	0,606	16,6
10	44,4	5657	20,5	256	311	60,2	34,9	395	0,597	13,4
180 · 180 · 6	32,1	4083	20,4	226	264	70,6	32,2	340	0,699	21,8
8	41,5	5284	25,5	283	336	69,4	41,9	432	0,686	16,5
10	50,7	6457	30,2	335	404	68,4	50,7	515	0,677	13,4
200 · 200 · 5	30,1	• 3836	24,1	• 241	• 279	79,3	37,6	362	0,783	26,0
6	35,8	4563	28,3	283	330	78,8	44,6	426	0,779	21,8
8	46,5	5924	35,7	357	421	77,6	58,2	544	0,766	16,5
10	57,0	7257	42,5	425	508	76,5	70,7	651	0,757	13,3
220 · 220 · 5	33,2	• 4236	32,4	• 294	• 340	87,4	50,4	442	0,863	26,0
6	39,6	5043	38,1	347	• 402	87,0	59,8	521	0,859	21,7
8	51,5	6564	48,3	439	516	85,8	78,1	668	0,846	16,4
250 · 250 · 6	45,2	• 5763	56,7	• 454	• 524	99,2	88,4	681	0,979	21,6
8	59,1	7524	72,3	578	676	98,0	116	878	0,966	16,3
10	72,7	9257	87,1	697	822	97,0	142	1062	0,957	13,2
260 · 260 · 6	47,1	• 6003	64,0	• 493	• 569	103	99,7	739	1,02	21,6
8	61,6	7844	81,8	629	734	102	131	955	1,01	16,3
10	75,8	9657	98,6	759	894	101	160	1156	0,997	13,2
300 · 300 · 10	88,4	11257	155	1035	1211	117	250	1572	1,16	13,1
12,5	108	13704	183	1223	1451	116	306	1892	1,14	10,6

Suggested Solution

While the final design requires explicit nonlinear dynamic analysis with 7 or 11 site-specific ground motions to verify the retrofitting objectives, the procedure outlined below may be used for preliminary design of the friction dampers for the planned intervention. I assume that you have the information for the SIA 261 elastic spectrum from the seismic engineering course.

Question 1

An estimate of the seismic forces in the y-y loading direction should be computed for the given design location based on SIA-261, Chapter 16.

Design location : Valais, CH, Z3b ($a_{gd} = 1.6m/s^2$),

Soil Type E ($S = 1.40$, $T_B = 0.15sec, T_C = 0.50sec, T_D = 2.0sec$),

Building Class: COIII ($\gamma_f = 1.4$),

Steel structure to be retrofitted; the supplemental damping ratio, $\xi = 0.10$; therefore,

$$n = \sqrt{\frac{1}{0.5 + 10 \cdot \xi}} = \sqrt{\frac{1}{0.5 + 10 \cdot 0.10}} = 0.82 > 0.55$$

The design base shear will be obtained using the elastic design spectrum according to SIA-261 (Clause 16.2.3). We first need to estimate the first mode vibration period, T_1, in the y-y loading direction. For this reason, we can use the approximate period formula that is a function of height according to SIA-261,

For buildings with heights up to 40 meters, the following equation holds true,

$$T_1 = C_t \cdot H^{3/4}$$

For steel frames with concentric bracings, $C_t = 0.05$. The corresponding height of the building in this case is, $H = 3 \cdot 4.0m = 12m$.

Therefore,

$$T_1 = 0.05 \cdot 12^{\frac{3}{4}} = 0.32sec$$

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

$$S_e(T) = 2.5 \cdot a_{gd} \cdot S \cdot n$$

Thus, for the y-y direction of interest, we obtain the following design accelerations:

$$S_e(T_1) = 2.5 \cdot 1.6 \cdot 1.4 \cdot 0.82 = 4.57\ m/s^2$$

Note: when we use the elastic spectrum according to SIA 261 we do not amplify by spectral ordinate by $\gamma_f = 1.4$.

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

$$W = 3 \cdot (7kPa \cdot 16m \cdot 18m) = 6048kN$$

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

$$V_e = S_e \cdot m_{tot} = 4.57 \cdot \frac{\frac{6048}{2}}{9.81} = 1409.4kN \; (2 \; lateral \; resisting \; frames \; in \; y-y)$$

According to the retrofitting objectives the friction dampers should be designed for two times the 475-year earthquake; therefore, the base shear for the preliminary design should be as follows, $V = 2V_e = 2818.8kN$.

We should distribute the base shear based on the equivalent lateral force method; therefore, per frame:

$$F_i = V_e \cdot \frac{z_i \cdot m_i}{\sum_{j=1:N}(z_j \cdot m_j)}$$

Table 1. Elastic forces for two times the 475-year earthquake

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 $[kN]$
3	2016	206	12.0	2466	1409
2	2016	206	8.0	1644	940
1	2016	206	4.0	822	470

Based on the lateral force distribution in the y-y loading direction we are going to use the approximate drift analysis method to estimate the lateral drift demands per storey. Because with have steel frames with single diagonal bracings (we assume that the friction dampers are single diagonal bracings), we will have to estimate the flexural and shear contributions to the lateral drift demand in the same way we did in steel frames with eccentric bracings. The main issue is that because the friction damper size is to be found, the shear contribution will be a function of the area of the main diagonal brace, A_d. Because we know the target drift limit imposed by the owner, we will use this one to identify A_d. In this example, we are dealing with a 3-storey building; therefore, we anticipate that shear deformations will be the main contributor to deflections. However, we will also compute the flexural ones.

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 1,

$$I_1 \approx \frac{A_c L^2}{2} = \frac{13100x6000^2}{2} = 2.36x10^{11} mm^4$$

Step 2: Compute the value of the external moment, M at each mid-storey level. For example, for storey 1,

$$M = 470 \cdot \frac{4.0}{2} + 940 \cdot \left(\frac{4.0}{2} + 4.0\right) + 1409 \cdot \left(\frac{4.0}{2} + 4.0 + 4.0\right) = 20672195 kN \cdot mm$$

Step 3: Determine for each storey the value of hM/EI_1. For example, for storey 1,

$$\frac{hM}{EI_1} = \delta\theta_{1,flexure} = \frac{4000x20672195}{2.36x10^{11}xE} = \frac{0.351}{E}$$

Step 4: Determine for each storey, i, the accumulation of value of $\delta\theta_{i,flexure}$ from storey 1 up to storey 3. For example, the accumulation of $\delta\theta_{i,flexure}$ up to storey 3 is:

$$\sum_{i=1}^{3} h_i \left(\frac{M}{EI}\right)_i = \frac{0.351 + 0.175 + 0.0478}{E} = \frac{0.574}{E}$$

Step 5: Record the product of h_i and $\theta_{i,flexure}$. For example, in storey 1 due to flexure:

$$\delta_{1,flexure} = 4000x \frac{(0.351)}{E} = \frac{1403}{E}\ mm$$

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 nth floor, to give the drift $\Delta^{(n)}_{flexure}$ For example, in storey 3 due to flexure:

$$\Delta_{3,flexure} = \frac{1403 + 2104 + 2295}{E} = \frac{5802}{210} = 27.6mm$$

In summary, the flexural deflections along the 3-storey frame are as follows:

Table 2. Summary of flexural deformations along the building height

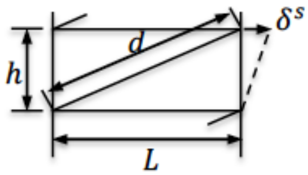
Storey	Frame Inertia I_i [mm^4]	External Moment M_i [kN-mm]	$\delta\theta_i$ [rad/E]	Storey Inclination θ_{if} [rad/E]	Storey Drift δ_{if} [mm/E]	$\Sigma\delta_{if}$ [mm/E]	$\Delta_{m,flexure}$ [mm]
3	2.36E+11	2818936	0.0478	0.574	2295	5802	27.6
2	2.36E+11	10336098	0.1753	0.526	2104	3507	16.7
1	2.36E+11	20672195	0.3507	0.351	1403	1403	6.7

NOTE: As expected, the flexural deformations in this case are fairly small and could be neglected in a preliminary design

The shear component of deflection of the frame 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, $V_1 = 2818.9kN$

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 (see Slide 26 in Frames with Eccentric Bracings).



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

And using this to compute the drift in storey 1 due to shear,

$$\delta_1^s = \frac{2818.9}{210} \cdot \left[\frac{7211.1^3}{6000^2 \cdot A_d} + \frac{6000}{4590}\right]$$

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

$$\Delta_{shear}^{(3)} = \frac{2818.9}{210} \cdot \left[\frac{7211.1^3}{6000^2 \cdot A_d} + \frac{6000}{4590}\right] + \frac{2349.1}{210} \cdot \left[\frac{7211.1^3}{6000^2 \cdot A_d} + \frac{6000}{4590}\right] + \frac{1409.5}{210} \cdot \left[\frac{7211.1^3}{6000^2 \cdot A_d} + \frac{6000}{4590}\right] = \frac{6577.5}{210} \cdot \left[\frac{7211.1^3}{6000^2 \cdot A_d} + \frac{6000}{4590}\right]$$

The target lateral drift limit imposed by the owner is 1%; therefore, each one of the stories should satisfy this limit; hence,

$$\delta_1^s + \delta_1^{flexure} < 1\% \rightarrow A_d = 9657mm^2 (RRK260x260x10)$$

In summary, the shear deflections along the 3-storey steel frame are as follows for the selected friction damper tube assuming an elastic design (i.e., the damper is not activated):

Table 3. Summary of shear and total deformations along the building height

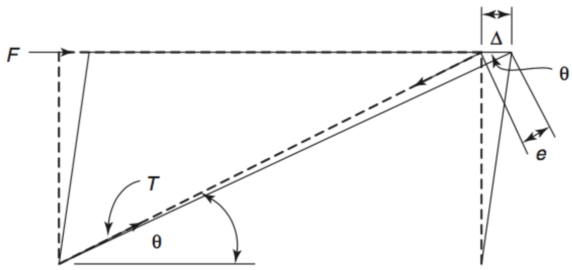
Load F [kN]	Storey	Storey Height h_i [mm]	Shear V_i [kN]	Storey Drift δ_{is} [mm]	$\Delta_{m,shear}$ [mm]	Δ_{total} [mm]	SDR_i [rads]
1409	3	4000	1409.5	16.0	74.7	102.4	0.007
940	2	4000	2349.1	26.7	58.7	75.4	0.009
470	1	4000	2818.9	32.0	32.0	38.7	0.010

In the last column of the above table, the storey drift ratios (SDR) are summarized for the selected RRK profile. Note that the drift limits are below or at most 1%. In principle, we could have optimized the brace size per storey to reach 1%. However, this would imply that the damper connections should be different, and this will cost more than the reduction in the steel weight we would benefit. Therefore, we keep those all the same.

Question 2

For the selected RRK,

$$N_{cr} = \frac{\pi^2 E I_B}{l_B^2} = \frac{\pi^2 210 \cdot 98.6x10^6}{7211.1^2} = 3930kN$$



(b)

Assuming that we do elastic design, this should resist the lateral load at the upper storey and should not buckle. Therefore, $N_{cr} > \frac{F}{cos\theta} = \frac{1409}{\cos(0.59)} = 1694kN$

$$\theta = \arctan\left(\frac{4}{6}\right) = 0.59rads$$

Therefore, member buckling is prevented for the selected RRK260x260x10. assuming that the damper behaves elastically (no slip). However, we would like the damper to be activated at a slip load, F_s, lower than the expected elastic force that we have calculated in Table 1. The selected slip load, F_s, should be such that when it is used in stability verifications of existing members, these would not have to be reinforced, if possible.

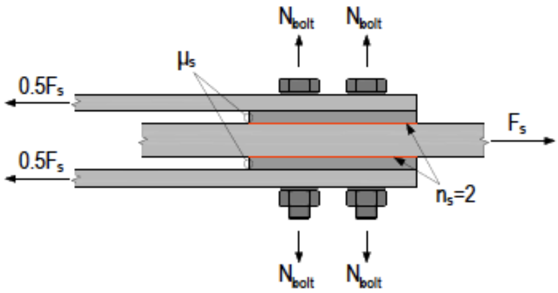
Let us assume that we design the friction damper for a slip load, $F_s = 650kN$ that is approximately one third to one fourth of the expected elastic axial load inside the bracing element, in this case, the expected second order moment in the friction damper would be,

$$M^B = \frac{F_s(u_o + e)}{1 - \frac{F_s}{N_{cr}}} = 650 \cdot \frac{\frac{7211.1}{100} + 0}{1 - \frac{650}{3930}} = 56161kN < 759x10^3 \cdot 0.355 = 269445kNmm$$

NOTE: in the calculation above I assumed that the corresponding imperfection of the member is $L/100$ to properly account for member P-Delta effects (typical manufacturing limits are $L/1000$).

Question 3

The required number of bolts depends on the sliding interfaces and the coefficient of friction. In this case, $\mu_s = 0.20$, the sliding interfaces, $n_s = 2$. Therefore,



$$F_s = n_s \cdot \mu_s \cdot N_{tot} \rightarrow N_{tot} = \frac{650}{0.20 \cdot 2} = 1625kN$$

By using 12 bolts, then each bolt should be preloaded to $N_{bolt} = 135kN$ to achieve the required slip load, $F_s = 650kN$. By using M20 10.9 grade bolts then $A_s = 245mm^2$ and $f_{u,bolt} = 1040MPa$. We need to check if the required preload is less than 65% of the ultimate preload that the M16 bolts could be pretensioned; therefore,

$$N^u_{bolt} = 0.65 \cdot 0.90 \cdot f_{u,bolt} \cdot A_s = 0.65 \cdot 0.90 \cdot 1.04 \cdot 245 = 149kN > 135kN$$

Therefore, 12 M20 10.9 grade bolts suffice for the connection design of the friction damper for a target slip load of 650kN.

Question 4

The column axial load demand comes from three sources:

Gravity load from tributary area (see also Figure 4):

$N_{Ed,G} = 3 \cdot (6 \cdot 4) \cdot 7 = 504kN$

Axial load due to seismic action in the interior MRF (x-x) amplified by the effects of overstrength in the MRF direction:

$N^{MRF}_{Ed,E} = 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N^{MRF}_{Ed,E} = 1.1 \cdot 1.25 \cdot 2.0 \cdot 200 = 550kN$

Axial load due to seismic action in the friction damper (y-y) loading direction, $F_s = 650kN$

The angle between the brace and the column, $\theta_1 = \arctan\left(\frac{6}{4}\right) = 0.982rads$

$N^{Friction\ damper}_{Ed,E} = F_s \cdot \cos(0.982) = 361kN$

Therefore, the total axial load applied to the column is as follows:

$$N_{Ed,1} = N_{Ed,G} + N_{Ed,E,1}^{Friction\ damper} + 0.3N_{Ed,E}^{MRF} = 504 + 361 + 0.3 \cdot 550 = 1030kN$$

NOTE: Because we check the columns in the friction damper (y-y) loading direction, when we combine the seismic effects in the two loading directions, we reduce the demands coming from the perpendicular direction by 30% based on the design code load combinations. This is because when an earthquake strikes a building, there is a principal loading direction (in this case we assume is y-y) and a secondary one (in this case we assume it is x-x). The 30% reduction can be found in the Eurocode design provisions but it is not mandatory according to the Swiss design provisions. However, you may want to be careful with this if a column is part of a MRF and braces intersect it in the perpendicular loading direction as in this example. For seismic interventions of existing structures I would always use the 30% rule in this case.

Buckling resistance of the steel column (same as Exercise #13):

Strong axis (MRF direction)

$l_k = 1.5L = 1.5 \cdot 4000 = 6000mm$

$\frac{h}{b} = 1.0 < 1.2$ and $t_f < 100mm$; buckling coefficient $\alpha = 0.34$

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{l_k^2} = 3.14^2 \cdot 210 \cdot \frac{192.7 \cdot 10^6}{6000^2} = 11083kN$$

Weak axis (friction damper direction)

$l_k = L = 4000mm$ (the column is pinned at the bottom. The beams intersecting at the top are pinned to the column in the weak axis)

$\frac{h}{b} = 1.0 < 1.2$ and $t_f < 100mm$; buckling coefficient $\alpha = 0.49$

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{l_k^2} = 3.14^2 \cdot 210 \cdot \frac{65.9 \cdot 10^6}{4000^2} = 8528kN$$

$$\bar{\lambda}_y = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}} = \sqrt{\frac{13100 \cdot 0.355}{11083}} = 0.65,\ \ \bar{\lambda}_z = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}} = \sqrt{\frac{13100 \cdot 0.355}{8528}} = 0.74$$

$$\Phi_y = 0.5 \cdot \left(1 + \alpha \cdot \left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^{\,2}\right) = 0.5 \cdot (1 + 0.34 \cdot (0.65 - 0.2) + 0.65^2) = 0.79$$

$$\Phi_z = 0.5 \cdot \left(1 + \alpha \cdot \left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^{\,2}\right) = 0.5 \cdot (1 + 0.49 \cdot (0.74 - 0.2) + 0.74^2) = 0.90$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^{\,2}}} = \frac{1}{0.79 + \sqrt{0.79^2 - 0.65^2}} = 0.81$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^{\ 2}}} = \frac{1}{0.90 + \sqrt{0.90^2 - 0.74^2}} = 0.71$$

Therefore,

$$N_{b,y,Rd} = \chi_y \cdot A \cdot \frac{f_y}{\gamma_{M1}} = 0.81 \cdot 13100 \cdot \frac{0.355}{1.05} = 3588kN > 1030kN$$

$$N_{b,z,Rd} = \chi_z \cdot A \cdot \frac{f_y}{\gamma_{M1}} = 0.71 \cdot 13100 \cdot \frac{0.355}{1.05} = 3145kN > 1030kN$$

The friction damper installation does not require any further enhancement to the steel column due to flexural buckling.

Axial load –flexure interaction:

h/b = 280/280 = 1.0 < 2; the buckling curve is "a" (i.e., a_{LT} = 0.21), according to EC3.

Plastic bending resistance with respect to strong axis bending

$$M_{pl,y,Rd} = W_{pl,y} \cdot \frac{f_y}{\gamma_{M0}} = 1530 \cdot 10^3 \cdot 0.355/1.00 \cong 543.2kNm$$

Plastic bending resistance with respect to weak axis bending

$$M_{pl,z,Rd} = W_{pl,z} \cdot \frac{f_y}{\gamma_{M0}} = 718 \cdot 10^3 \cdot 0.355/1.00 \cong 254.9kNm$$

Computation of critical moment:

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

From Figure 4:

$$M_{y,Ed,top} = M_{G,top} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{E,top} = 130 + 1.1 \cdot 1.25 \cdot 2 \cdot 100 = 405kNm$$

$$M_{y,Ed,bot.} = M_{G,bot.} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{E,bot.} = 120 + 1.1 \cdot 1.25 \cdot 2 \cdot 131 = 480.2kNm$$

therefore, $k = \frac{405}{480.2} = 0.84$

The steel column is fixed at the base in the y-y direction; however, conservatively, we assume that the warping constant is $k_v = 1.0$

From, $k_v = 1.0$, $k_\varphi = 1.0$ (conservative assumption), k = 0.84, $C_1 > 2.3, C_1 = 2.3$, $L_D = 4000mm$

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

Computation of torsional and warping constants:

$$K = \frac{2 \cdot b \cdot t_f^3 + \left(h - t_f\right) \cdot t_w^3}{3} = \frac{2 \cdot 280 \cdot 18^3 + (280 - 18) \cdot 10.5^3}{3} = 1.19x10^6 mm^4$$

$$I_{\omega}=\frac{t_f\cdot\left(h-t_f\right)^2\cdot b^3}{24}=\frac{18\cdot(280-18)^2\cdot 280^3}{24}=1.13x10^{12}mm^6$$

Therefore, the computation of M_{cr} is as follows:

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

$$=2.3\cdot\frac{\pi^2\cdot 210\cdot 65.9\cdot 10^6}{1.0\cdot 1.0\cdot(4000)^2}$$

$$\cdot\left(\frac{1.13\cdot 10^{12}}{65.9\cdot 10^6}\cdot\left(\frac{(1.0\cdot 4000)^2\cdot 80.8\cdot 1.19\cdot 10^6}{\pi^2\cdot 210\cdot 1.13\cdot 10^{12}}+1\right)\right)^{0.5}\sim 3309kNm$$

$$\bar{\lambda}_{LT}=\sqrt{\frac{W_{pl,y}\cdot f_y}{M_{cr}}}=\sqrt{\frac{543.2}{3309}}=0.41>0.40$$

Therefore, the column bending resistance should be reduced due to lateral torsional buckling.

$$\Phi_{LT}=0.5\cdot\left(1+\alpha_{LT}\cdot\left(\bar{\lambda}_{\mathrm{LT}}-0.2\right)+\bar{\lambda}_{LT}{}^2\right)=0.5\cdot(1+0.21\cdot(0.41-0.2)+0.41^2)$$

$$=0.60$$

$$\chi_{LT}=\frac{1}{\Phi_{\mathrm{LT}}+\sqrt{\Phi_{\mathrm{LT}}^2-\bar{\lambda}_{LT}{}^2}}=\frac{1}{0.60+\sqrt{0.60^2-0.41^2}}=0.95$$

Strong Axis Interaction: (Note that $M_{z,Ed}=0$ because the column is pinned in the CBF direction):

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

To compute ω_y you should consider the moment sign in this case such that the moment gradient can reduce the interaction due to bending if the member is in double curvature. Therefore,

$$\omega_y=0.6+0.4\cdot\left(-\frac{405}{480.2}\right)=0.26<0.40; therefore,\omega_y=0.40$$

Interaction of bending and axial load and bending:

Option 1:

$$\frac{N_{Ed_1}}{\chi_z\cdot A\cdot\frac{f_y}{\gamma_{M1}}}+\frac{\omega_y}{1-\frac{N_{Ed,1}}{N_{y,cr}}}\cdot\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{pl,y,Rd}}{\gamma_{M1}}}=\frac{1030}{3145}+\frac{0.40}{1-\frac{1030}{11083}}\cdot\frac{480.2}{0.95\cdot 517.3}=0.33+0.43$$

$$=0.76<1.00$$

Therefore, the existing steel column satisfies all the checks for interaction of axial load and bending for both BRB options.

SOME NOTES:

(1) As a general remark in this case, the existing steel columns (and beam) are not overloaded by the installation of the selected friction damper. If this was the case, we could have lowered the activation force, F_s, from $650kN$ to something less. The main problem that may occur in this case is that the lower the activation force the larger slot displacement you need to design your friction damper for. Just to put things into perspective, if an activation force larger than 1800kN is used then the existing steel column will need to be retrofitted because the interaction of axial load and bending will not be satisfied. Moreover, if you were to check the stability of the steel beam due to interaction of axial load and bending (as we did in Exercise #13), with an activation force larger than 1100kN, the beam would have to be reinforced. You can do this for practice, if you wish but this should essentially be the same as the last question in Exercise #13.

(2) The preliminary design we did herein assumes that when the damper is activated, then the expected lateral storey drift ratio demands will still remain below 1% (i.e., this is an application of the commonly used equal displacement rule). This should be verified by nonlinear response history analysis with ground motions that should be selected for the design site. Typically, 7 or 11 ground motions suffice in this case based on the current design standards. A nonlinear building model of the retrofitted structure is needed. If you are interested to see the entire process, you should talk to me for a potential master thesis project in RESSLab.

(3) Friction dampers come with a friction coefficient that is determined by qualification testing similar to that shown in buckling restrained braces. In this case, the friction coefficient is determined with a min and max value. When conducting nonlinear dynamic analysis for verification of the performance objectives of interest, the min and max values (lower and upper bound analysis is typically conducted).