| CHAPTER 4 |

Steel Column Design

Learning Objectives

- Understanding the concept of effective length and classification of sections.
- Production of valid designs for steel columns under axial force only.
- Production of valid design for simple construction and continuous construction steel columns under combined axial force and moments.

1. Compression Members (Clause 8.7)

1.1 Effective Length

The resistance of a member to buckling depends on (***slenderness***) the effective length to radius of gyration ratio ***($\lambda = L_E/r$).*** The effective length is a function of the actual length between restraints and depends on the type of restraint provided (i.e. rotational and positional restraint). In the majority of cases the effective length will be determined from ***Table 8.6***[1]. Effective length ***L_E*** is equal to the product of ***K*** and the actual length ***L***.

Table 8.6 - Effective length of idealized columns

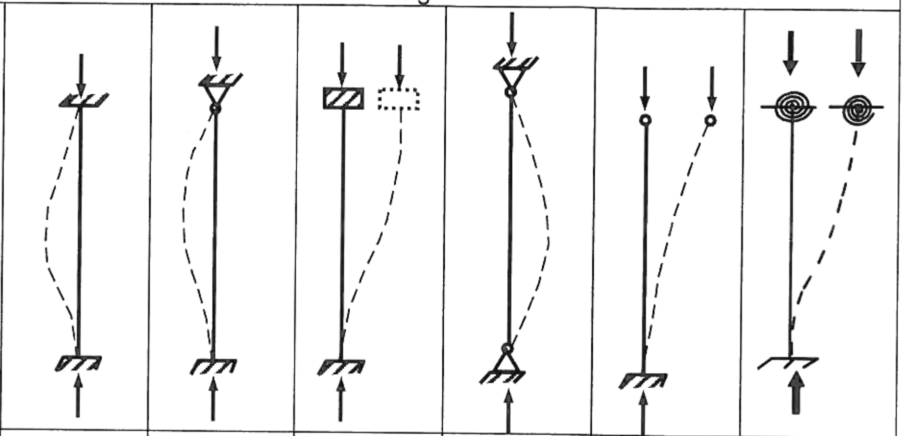
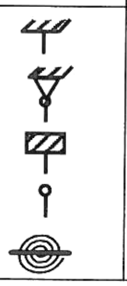
	Flexural Buckling					
Buckled shape of column shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	/
Recommended K value when ideal conditions are approximated	0.70	0.85	1.20	1.00	2.10	1.5
End condition code		Rotation fixed. Transition fixed. Rotation free. Transition fixed. Rotation fixed. Transition free. Rotation free. Transition free. Rotation partially restrained. Transition free.				

Table 1 – Extract of Table 8.6 of the HK Code[1]

Having determined the slenderness (λ), the designer will select a compressive strength (p_c) from the relevant strut table and hence calculate the compression resistance (P_c), i.e.

$$P_c = A_g\, p_c$$

where A_g is the sum of gross cross-sectional area.

Note that this equation is applicable to plastic, compact and semi-compact section

1.2 Multiple strut tables

The compressive strength p_c depends on the types of section, design strength, slenderness and a suitable buckling curve that should be selected from Table 8.7.

Table 8.7 - Designation of buckling curves for different section types

Type of section	Maximum thickness (see note1)	Axis of buckling x-x	Axis of buckling y-y
Hot-finished structural hollow sections with steel grade > S460 or hot-finished seamless structural hollow sections		a_0)	a_0)
Hot-finished structural hollow section < grade S460		a)	a)
Cold-formed structural hollow section of longitudinal seam weld or spiral weld		c)	c)
Rolled I-section	≤ 40 mm > 40 mm	a) b)	b) c)
Rolled H-section	≤ 40 mm > 40 mm	b) c)	c) d)
Welded I- or H-section (see note 2)	≤ 40 mm > 40 mm	b) b)	c) d)
Rolled I-section with welded flange cover plates with 0.25 < U/B < 0.80 as shown in Figure 8.4)	≤ 40 mm > 40 mm	a) b)	b) c)
Rolled H-section with welded flange cover plates with 0.25 < U/B < 0.80 as shown in Figure 8.4)	≤ 40 mm > 40 mm	b) c)	c) d)
Rolled I or H-section with welded flange cover plates with U/B ≥ 0.80 as shown in Figure 8.4)	≤ 40 mm > 40 mm	b) c)	a) b)
Rolled I or H-section with welded flange cover plates with U/B ≤ 0.25 as shown in Figure 8.4)	≤ 40 mm > 40 mm	b) b)	c) d)
Welded box section (see note 3)	≤ 40 mm > 40 mm	b) c)	b) c)
Round, square or flat bar	≤ 40 mm > 40 mm	b) c)	b) c)
Rolled angle, channel or T-section Two rolled sections laced, battened or back-to-back Compound rolled sections		Any axis: c)	

NOTE:
1. For thickness between 40mm and 50mm the value of p_c may be taken as the average of the values for thicknesses up to 40mm and over 40mm for the relevant value of p_y.
2. For welded I or H-sections with their flanges thermally cut by machine without subsequent edge grinding or machining, for buckling about the y-y axis, strut curve b) may be used for flanges up to 40mm thick and strut curve c) for flanges over 40mm thick.
3. The category "welded box section" includes any box section fabricated from plates or rolled sections, provided that all of the longitudinal welds are near the corners of the cross-section. Box sections with longitudinal stiffeners are NOT included in this category.
4. Use of buckling curves based on other recognized design codes allowing for variation between load and material factors and calibrated against Tables 8.8(a_0), (a) to (h) is acceptable. See also footnote under Table 8.8.

Table 2 – Extract of Table 8.7 of the HK Code[1]

Table 8.8(a_0) - Design strength p_c of compression members

Values of p_c in N/mm^2 for strut curve a_0											
λ	Steel grade and design strength (N/mm2) S460 with $\lambda < 110$					λ	Steel grade and design strength (N/mm2) S460 with $\lambda \geq 110$				
	400	410	430	440	460		400	410	430	440	460
15	399	409	429	439	458	110	150	150	151	152	153
20	396	405	425	434	454	112	145	146	147	147	148
25	391	401	420	430	449	114	141	141	142	142	143
30	387	396	415	425	443	116	136	137	137	138	138
35	381	391	409	418	437	118	132	132	133	133	134
40	375	384	402	411	429	120	128	128	129	129	130
42	372	381	399	408	425	122	124	124	125	125	126
44	369	378	395	404	421	124	120	121	121	122	122
46	366	375	391	400	417	126	117	117	118	118	118
48	362	371	387	395	412	128	113	114	114	115	115
50	359	367	383	391	406	130	110	111	111	111	112
52	354	362	378	385	400	135	103	103	103	104	104
54	350	357	372	379	393	140	96	96	96	97	97
56	344	352	366	373	386	145	90	90	90	90	91
58	339	346	359	365	378	150	84	84	85	85	85
60	333	339	352	358	369	155	79	79	79	79	80
62	326	332	344	349	360	160	74	74	75	75	75
64	319	325	335	340	350	165	70	70	70	70	71
66	312	317	327	331	340	170	66	66	66	66	67
68	304	308	317	321	329	175	63	63	63	63	63
70	296	300	308	311	318	180	59	59	59	60	60
72	287	291	298	301	307	185	56	56	56	56	57
74	278	282	288	291	296	190	53	53	54	54	54
76	270	273	278	281	286	195	51	51	51	51	51
78	261	264	269	271	275	200	48	48	48	48	49
80	252	255	259	261	265	210	44	44	44	44	44
82	244	246	250	251	255	220	40	40	40	40	40
84	235	237	241	242	245	230	37	37	37	37	37
86	227	229	232	233	236	240	34	34	34	34	34
88	219	221	223	225	227	250	31	31	31	31	31
90	212	213	215	216	219	260	29	29	29	29	29
92	204	205	208	209	210	270	27	27	27	27	27
94	197	198	200	201	203	280	25	25	25	25	25
96	190	191	193	194	195	290	23	23	23	23	23
98	184	185	186	187	188	300	22	22	22	22	22
100	177	178	180	180	182	310	20	20	20	21	21
102	171	172	174	174	175	320	19	19	19	19	19
104	166	166	168	168	169	330	18	18	18	18	18
106	160	161	162	163	163	340	17	17	17	17	17
108	155	156	157	157	158	350	16	16	16	16	16

Table 3a – Extract of Table 8.8a_o of the HK Code[1]

Table 8.8(a) - Design strength p_c of compression members

λ	1) Values of p_c in N/mm^2 with λ < 110 for strut curve a														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
15	235	245	255	265	275	315	325	335	345	355	399	409	429	439	458
20	234	244	254	264	273	312	322	332	342	351	395	405	424	434	453
25	232	241	251	261	270	309	318	328	338	347	390	400	419	429	448
30	229	239	248	258	267	305	315	324	333	343	385	395	414	423	442
35	226	236	245	254	264	301	310	320	329	338	380	389	407	416	434
40	223	233	242	251	260	296	305	315	324	333	373	382	399	408	426
42	222	231	240	249	258	294	303	312	321	330	370	378	396	404	422
44	221	230	239	248	257	292	301	310	319	327	366	375	392	400	417
46	219	228	237	246	255	280	299	307	316	325	363	371	388	396	413
48	218	227	236	244	253	288	296	305	313	322	359	367	383	391	407
50	216	225	234	242	251	285	293	302	310	318	355	363	378	386	401
52	215	223	232	241	249	282	291	299	307	315	350	358	373	380	395
54	213	222	230	238	247	279	287	295	303	311	345	353	367	374	388
56	211	220	228	236	244	276	284	292	300	307	340	347	361	368	381
58	210	218	226	234	242	273	281	288	295	303	334	341	354	360	372
60	208	216	224	232	239	269	277	284	291	298	328	334	346	352	364
62	206	214	221	229	236	266	273	280	286	293	321	327	338	344	354
64	204	211	219	226	234	262	268	275	281	288	314	320	330	335	344
66	201	209	216	223	230	257	264	270	276	282	307	312	321	326	334
68	199	206	213	220	227	253	259	265	270	276	299	303	312	316	324
70	196	203	210	217	224	248	254	259	265	270	291	295	303	306	313
72	194	201	207	214	220	243	248	253	258	263	282	286	293	296	302
74	191	198	204	210	216	238	243	247	252	256	274	277	283	286	292
76	188	194	200	206	212	232	237	241	245	249	265	268	274	276	281
78	185	191	197	202	208	227	231	235	239	242	257	259	264	267	271
80	182	188	193	198	203	221	225	229	232	235	248	251	255	257	261
82	179	184	189	194	199	215	219	222	225	228	240	242	246	248	251
84	176	181	185	190	194	209	213	216	219	221	232	234	237	239	242
86	172	177	181	186	190	204	207	209	212	214	224	225	229	230	233
88	169	173	177	181	185	198	200	203	205	208	216	218	220	222	224
90	165	169	173	177	180	192	195	197	199	201	209	210	213	214	216
92	162	166	169	173	176	186	189	191	193	194	201	203	205	206	208
94	158	162	165	168	171	181	183	185	187	188	194	196	198	199	200
96	154	158	161	164	166	175	177	179	181	182	188	189	191	192	193
98	151	154	157	159	162	170	172	173	175	176	181	182	184	185	186
100	147	150	153	155	157	165	167	168	169	171	175	176	178	178	180
102	144	146	149	151	153	160	161	163	164	165	169	170	172	172	174
104	140	142	145	147	149	155	156	158	159	160	164	165	166	166	168
106	136	139	141	143	145	150	152	153	154	155	158	159	160	161	162
108	133	135	137	139	141	146	147	148	149	150	153	154	155	156	157

Table 3b – Extract of Table 8.8a of the HK Code[1]

Table 8.8(a) - Design strength p_c of compression members (cont'd)

λ	2) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve a														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	130	132	133	135	137	142	143	144	144	145	148	149	150	150	151
112	126	128	130	131	133	137	138	139	140	141	144	144	145	146	146
114	123	125	126	128	129	133	134	135	136	136	139	140	141	141	142
116	120	121	123	124	125	129	130	131	132	132	135	135	136	137	137
118	117	118	120	121	122	126	126	127	128	128	131	131	132	132	133
120	114	115	116	118	119	122	123	123	123	125	127	127	128	128	129
122	111	112	113	114	115	119	119	120	120	121	123	123	124	124	125
124	108	109	110	111	112	115	116	116	117	117	119	120	120	121	121
126	105	106	107	108	109	112	113	113	114	114	116	116	117	117	118
128	103	104	105	105	106	109	109	110	110	111	112	113	113	114	114
130	100	101	102	103	103	106	106	107	107	108	109	110	110	110	111
135	94	95	95	96	97	99	99	100	100	101	102	102	103	103	103
140	88	89	90	90	91	93	93	93	94	94	95	95	96	96	96
145	83	84	84	85	85	87	87	87	88	88	89	89	90	90	90
150	78	79	79	80	80	82	82	82	82	83	83	84	84	84	84
155	74	74	75	75	75	77	77	77	77	78	78	79	79	79	79
160	70	70	70	71	71	72	72	73	73	73	74	74	74	74	75
165	66	66	67	67	67	68	68	69	69	69	70	70	70	70	70
170	62	63	63	63	64	64	65	65	65	65	66	66	66	66	66
175	59	59	60	60	60	61	61	61	61	62	62	62	62	63	63
180	56	56	57	57	57	58	58	58	58	58	59	59	59	59	59
185	53	54	54	54	54	55	55	55	55	55	56	56	56	56	56
190	51	51	51	51	52	52	52	52	53	53	53	53	53	53	53
195	48	49	49	49	49	50	50	50	50	50	50	51	51	51	51
200	46	46	46	47	47	47	47	47	48	48	48	48	48	48	48
210	42	42	42	43	43	43	43	43	43	43	44	44	44	44	44
220	39	39	39	39	39	39	39	40	40	40	40	40	40	40	40
230	35	36	36	36	36	36	36	36	36	36	37	37	37	37	37
240	33	33	33	33	33	33	33	33	33	33	34	34	34	34	34
250	30	30	30	30	30	31	31	31	31	31	31	31	31	31	31
260	28	28	28	28	28	28	29	29	29	29	29	29	29	29	29
270	26	26	26	26	26	26	27	27	27	27	27	27	27	27	27
280	24	24	24	24	24	25	25	25	25	25	25	25	25	25	25
290	23	23	23	23	23	23	23	23	23	23	23	23	23	23	23
300	21	21	21	21	21	22	22	22	22	22	22	22	22	22	22
310	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
320	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19
330	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18
340	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17
350	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16

Table 3c – Extract of Table 8.8a[1]

Table 8.8(b) - Design strength p_c of compression members

λ	3) Values of p_c in N/mm^2 with $\lambda < 110$ for strut curve b														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
15	235	245	255	265	275	315	325	335	345	355	399	409	428	438	457
20	234	243	253	263	272	310	320	330	339	349	391	401	420	429	448
25	229	239	248	258	267	304	314	323	332	342	384	393	411	421	439
30	225	234	243	253	262	298	307	316	325	335	375	384	402	411	429
35	220	229	238	247	256	291	300	309	318	327	366	374	392	400	417
40	216	224	233	241	250	284	293	301	310	318	355	364	380	388	404
42	213	222	231	239	248	281	289	298	306	314	351	359	375	383	399
44	211	220	228	237	245	278	286	294	302	310	346	354	369	377	392
46	209	218	226	234	242	275	283	291	298	306	341	349	364	371	386
48	207	215	223	231	239	271	279	287	294	302	336	343	358	365	379
50	205	213	221	229	237	267	275	283	290	298	330	337	351	358	372
52	203	210	218	226	234	264	271	278	286	293	324	331	344	351	364
54	200	208	215	223	230	260	267	274	281	288	318	325	337	344	356
56	198	205	213	220	227	256	263	269	276	283	312	318	330	336	347
58	195	202	210	217	224	252	258	265	271	278	305	311	322	328	339
60	193	200	207	214	221	247	254	260	266	272	298	304	314	320	330
62	190	197	204	210	217	243	249	255	261	266	291	296	306	311	320
64	187	194	200	207	213	238	244	249	255	261	284	289	298	302	311
66	184	191	197	203	210	233	239	244	249	255	276	281	289	294	301
68	181	188	194	200	206	228	233	239	244	249	269	273	281	285	292
70	178	185	190	196	202	223	228	233	238	242	261	265	272	276	282
72	175	181	187	193	198	218	223	227	232	236	254	257	264	267	273
74	172	178	183	189	194	213	217	222	226	230	246	249	255	258	264
76	169	175	180	185	190	208	212	216	220	223	238	241	247	250	255
78	166	171	176	181	186	203	206	210	214	217	231	234	239	241	246
80	163	168	172	177	181	197	201	204	208	211	224	226	231	233	237
82	160	164	169	173	177	192	196	199	202	205	217	219	223	225	229
84	156	161	165	169	173	187	190	193	196	199	210	212	216	218	221
86	153	157	161	165	169	182	185	188	190	193	203	205	208	210	213
88	150	154	158	161	165	177	180	182	185	187	196	198	201	203	206
90	146	150	154	157	161	172	175	177	179	181	190	192	195	196	199
92	143	147	150	153	156	167	170	172	174	176	184	185	188	189	192
94	140	143	147	150	152	162	165	167	169	171	178	179	182	183	185
96	137	140	143	146	148	158	160	162	164	165	172	173	176	177	179
98	134	137	139	142	145	153	155	157	159	160	167	168	170	171	173
100	130	133	136	138	141	149	151	152	154	155	161	162	164	165	167
102	127	130	132	135	137	145	146	148	149	151	156	157	159	160	162
104	124	127	129	131	133	141	142	144	145	146	151	152	154	155	156
106	121	124	126	128	130	137	138	139	141	142	147	148	149	150	151
108	118	121	123	125	126	133	134	135	137	138	142	143	144	145	147

Table 3d – Extract of Table 8.8b of the HK Code[1]

Table 8.8(b) - Design strength p_c of compression members (cont'd)

λ	4) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve b														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	115	118	120	121	123	129	130	131	133	134	138	139	140	141	142
112	113	115	117	118	120	125	127	128	129	130	134	134	136	136	138
114	110	112	114	115	117	122	123	124	125	126	130	130	132	132	133
116	107	109	111	112	114	119	120	121	122	122	126	126	128	128	129
118	105	106	108	109	111	115	116	117	118	119	122	123	124	124	125
120	102	104	105	107	108	112	113	114	115	116	119	119	120	121	122
122	100	101	103	104	105	109	110	111	112	112	115	116	117	117	118
124	97	99	100	101	102	106	107	108	109	109	112	112	113	114	115
126	95	96	98	99	100	103	104	105	106	106	109	109	110	111	111
128	93	94	95	96	97	101	101	102	103	103	106	106	107	107	108
130	90	92	93	94	95	98	99	99	100	101	103	103	104	105	105
135	85	86	87	88	89	92	93	93	94	94	96	97	97	98	98
140	80	81	82	83	84	86	87	87	88	88	90	90	91	91	92
145	76	77	78	78	79	81	82	82	83	83	84	85	85	86	86
150	72	72	73	74	74	76	77	77	78	78	79	80	80	80	81
155	68	69	69	70	70	72	72	73	73	73	75	75	75	76	76
160	64	65	65	66	66	68	68	69	69	69	70	71	71	71	72
165	61	62	62	62	63	64	65	65	65	65	66	67	67	67	68
170	58	58	59	59	60	61	61	61	62	62	63	63	63	64	64
175	55	55	56	56	57	58	58	58	59	59	60	60	60	60	60
180	52	53	53	53	54	55	55	55	56	56	56	57	57	57	57
185	50	50	51	51	51	52	52	53	53	53	54	54	54	54	54
190	48	48	48	48	49	50	50	50	50	50	51	51	51	51	52
195	45	46	46	46	46	47	47	48	48	48	49	49	49	49	49
200	43	44	44	44	44	45	45	45	46	46	46	46	47	47	47
210	40	40	40	40	41	41	41	41	42	42	42	42	42	43	43
220	36	37	37	37	37	38	38	38	38	38	39	39	39	39	39
230	34	34	34	34	34	35	35	35	35	35	35	36	36	36	36
240	31	31	31	31	32	32	32	32	32	32	33	33	33	33	33
250	29	29	29	29	29	30	30	30	30	30	30	30	30	30	30
260	27	27	27	27	27	27	28	28	28	28	28	28	28	28	28
270	25	25	25	25	25	26	26	26	26	26	26	26	26	26	26
280	23	23	23	23	24	24	24	24	24	24	24	24	24	24	24
290	22	22	22	22	22	22	22	22	22	22	23	23	23	23	23
300	20	20	21	21	21	21	21	21	21	21	21	21	21	21	21
310	19	19	19	19	19	20	20	20	20	20	20	20	20	20	20
320	18	18	18	18	18	18	18	19	19	19	19	19	19	19	19
330	17	17	17	17	17	17	17	17	17	18	18	18	18	18	18
340	16	16	16	16	16	16	16	16	17	17	17	17	17	17	17
350	15	15	15	15	15	16	16	16	16	16	16	16	16	16	16

Table 3e – Extract of Table 8.8b of the HK Code[1]

Table 8.8(c) - Design strength p_c of compression members

5) Values of p_c in N/mm^2 with λ < 110 for strut curve c															
λ	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
15	235	245	255	265	275	315	325	335	345	355	398	408	427	436	455
20	233	242	252	261	271	308	317	326	336	345	387	396	414	424	442
25	226	235	245	254	263	299	308	317	326	335	375	384	402	410	428
30	220	228	237	246	255	289	298	307	315	324	363	371	388	396	413
35	213	221	230	238	247	280	288	296	305	313	349	357	374	382	397
40	206	214	222	230	238	270	278	285	293	301	335	343	358	365	380
42	203	211	219	227	235	266	273	281	288	296	329	337	351	358	373
44	200	208	216	224	231	261	269	276	284	291	323	330	344	351	365
46	197	205	213	220	228	257	264	271	279	286	317	324	337	344	357
48	195	202	209	217	224	253	260	267	274	280	311	317	330	337	349
50	192	199	206	213	220	248	255	262	268	275	304	310	323	329	341
52	189	196	203	210	217	244	250	257	263	270	297	303	315	321	333
54	186	193	199	206	213	239	245	252	258	264	291	296	308	313	324
56	183	189	196	202	209	234	240	246	252	258	284	289	300	305	315
58	179	186	192	199	205	229	235	241	247	252	277	282	292	297	306
60	176	183	189	195	201	225	230	236	241	247	270	274	284	289	298
62	173	179	185	191	197	220	225	230	236	241	262	267	276	280	289
64	170	176	182	188	193	215	220	225	230	235	255	260	268	272	280
66	167	173	178	184	189	210	215	220	224	229	248	252	260	264	271
68	164	169	175	180	185	205	210	214	219	223	241	245	252	256	262
70	161	166	171	176	181	200	204	209	213	217	234	238	244	248	254
72	157	163	168	172	177	195	199	203	207	211	227	231	237	240	246
74	154	159	164	169	173	190	194	198	202	205	220	223	229	232	238
76	151	156	160	165	169	185	189	193	196	200	214	217	222	225	230
78	148	152	157	161	165	180	184	187	191	194	207	210	215	217	222
80	145	149	153	157	161	176	179	182	185	188	201	203	208	210	215
82	142	146	150	154	157	171	174	177	180	183	195	197	201	203	207
84	139	142	146	150	154	167	169	172	175	178	189	191	195	197	201
86	135	139	143	146	150	162	165	168	170	173	183	185	189	190	194
88	132	136	139	143	146	158	160	163	165	168	177	179	183	184	187
90	129	133	136	139	142	153	156	158	161	163	172	173	177	178	181
92	126	130	133	136	139	149	152	154	156	158	166	168	171	173	175
94	124	127	130	133	135	145	147	149	151	153	161	163	166	167	170
96	121	124	127	129	132	141	143	145	147	149	156	158	160	162	164
98	118	121	123	126	129	137	139	141	143	145	151	153	155	157	159
100	115	118	120	123	125	134	135	137	139	140	147	148	151	152	154
102	113	115	118	120	122	130	132	133	135	136	143	144	146	147	149
104	110	112	115	117	119	126	128	130	131	133	138	139	142	142	144
106	107	110	112	114	116	123	125	126	127	129	134	135	137	138	140
108	105	107	109	111	113	120	121	123	124	125	130	131	133	134	136

Table 3f – Extract of Table 8.8c of the HK Code[1]

Table 8.8(c) - Design strength p_c of compression members (cont'd)

λ	6) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve c														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	102	104	106	108	110	116	118	119	120	122	126	127	129	130	132
112	100	102	104	106	107	113	115	116	117	118	123	124	125	126	128
114	98	100	101	103	105	110	112	113	114	115	119	120	122	123	124
116	95	97	99	101	102	108	109	110	111	112	116	117	118	119	120
118	93	95	97	98	100	105	106	107	108	109	113	114	115	116	117
120	91	93	94	96	97	102	103	104	105	106	110	110	112	112	113
122	89	90	92	93	95	99	100	101	102	103	107	107	109	109	110
124	87	88	90	91	92	97	98	99	100	100	104	104	106	106	107
126	85	86	88	89	90	94	95	96	97	98	101	102	103	103	104
128	83	84	86	87	88	92	93	94	95	95	98	99	100	100	101
130	81	82	84	85	86	90	91	91	92	93	96	96	97	98	99
135	77	78	79	80	81	84	85	86	87	87	90	90	91	92	92
140	72	74	75	76	76	79	80	81	81	82	84	85	85	86	87
145	69	70	71	71	72	75	76	76	77	77	79	80	80	81	81
150	65	66	67	68	68	71	71	72	72	73	75	75	76	76	76
155	62	63	63	64	65	67	67	68	68	69	70	71	71	72	72
160	59	59	60	61	61	63	64	64	65	65	66	67	67	67	68
165	56	56	57	58	58	60	60	61	61	61	63	63	64	64	64
170	53	54	54	55	55	57	57	58	58	58	60	60	60	60	61
175	51	51	52	52	53	54	54	55	55	55	56	57	57	57	58
180	48	49	49	50	50	51	52	52	52	53	54	54	54	54	55
185	46	46	47	47	48	49	49	50	50	50	51	51	52	52	52
190	44	44	45	45	45	47	47	47	47	48	49	49	49	49	49
195	42	42	43	43	43	45	45	45	45	45	46	46	47	47	47
200	40	41	41	41	42	43	43	43	43	43	44	44	45	45	45
210	37	37	38	38	38	39	39	39	40	40	40	40	41	41	41
220	34	34	35	35	35	36	36	36	36	36	37	37	37	37	38
230	31	32	32	32	32	33	33	33	33	34	34	34	34	34	35
240	29	29	30	30	30	30	31	31	31	31	31	31	32	32	32
250	27	27	27	28	28	28	28	28	29	29	29	29	29	29	29
260	25	25	26	26	26	26	26	26	27	27	27	27	27	27	27
270	23	24	24	24	24	24	25	25	25	25	25	25	25	25	25
280	22	22	22	22	22	23	23	23	23	23	23	24	24	24	24
290	21	21	21	21	21	21	21	22	22	22	22	22	22	22	22
300	19	19	20	20	20	20	20	20	20	20	21	21	21	21	21
310	18	18	18	19	19	19	19	19	19	19	19	19	19	19	20
320	17	17	17	17	18	18	18	18	18	18	18	18	18	18	18
330	16	16	16	16	17	17	17	17	17	17	17	17	17	17	17
340	15	15	15	16	16	16	16	16	16	16	16	16	16	16	16
350	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15

Table 3g – Extract of Table 8.8c of the HK Code[1]

Table 8.8(d) - Design strength p_c of compression members

7) Values of p_c in N/mm^2 with $\lambda < 110$ for strut curve d															
λ	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
15	235	245	255	265	275	315	325	335	345	355	397	407	425	435	453
20	232	241	250	259	269	305	314	323	332	341	381	390	408	417	434
25	223	231	240	249	257	292	301	309	318	326	365	373	390	398	415
30	213	222	230	238	247	279	287	296	304	312	348	356	372	380	396
35	204	212	220	228	236	267	274	282	290	297	331	339	353	361	375
40	195	203	210	218	225	254	261	268	275	283	314	321	334	341	355
42	192	199	206	214	221	249	256	263	270	277	307	314	327	333	346
44	188	195	202	209	216	244	251	257	264	271	300	306	319	325	337
46	185	192	199	205	212	239	245	252	258	265	293	299	311	317	329
48	181	188	195	201	208	234	240	246	252	259	286	291	303	309	320
50	178	184	191	197	204	228	235	241	247	253	278	284	295	301	311
52	174	181	187	193	199	223	229	235	241	246	271	277	287	292	303
54	171	177	183	189	195	218	224	229	235	240	264	269	279	284	294
56	167	173	179	185	191	213	219	224	229	234	257	262	271	276	285
58	164	170	175	181	187	208	213	218	224	229	250	255	264	268	277
60	161	166	172	177	182	203	208	213	218	223	243	247	256	260	268
62	157	163	168	173	178	198	203	208	212	217	236	240	248	252	260
64	154	159	164	169	174	193	198	202	207	211	229	233	241	245	252
66	150	156	160	165	170	188	193	197	201	205	223	226	234	237	244
68	147	152	157	162	166	184	188	192	196	200	216	220	226	230	236
70	144	149	153	158	162	179	183	187	190	194	210	213	219	222	228
72	141	145	150	154	158	174	178	182	185	189	203	207	213	215	221
74	138	142	146	150	154	170	173	177	180	183	197	200	206	209	214
76	135	139	143	147	151	165	169	172	175	178	191	194	199	202	207
78	132	136	139	143	147	161	164	167	170	173	186	188	193	195	200
80	129	132	136	140	143	156	160	163	165	168	180	182	187	189	194
82	126	129	133	136	140	152	155	158	161	163	175	177	181	183	187
84	123	126	130	133	136	148	151	154	156	159	169	171	176	177	181
86	120	123	127	130	133	144	147	149	152	154	164	166	170	172	175
88	117	120	123	127	129	140	143	145	148	150	159	161	165	167	170
90	114	118	121	123	126	137	139	141	144	146	154	156	160	161	164
92	112	115	118	120	123	133	135	137	139	142	150	152	155	156	159
94	109	112	115	117	120	129	132	134	136	138	145	147	150	152	154
96	107	109	112	115	117	126	128	130	132	134	141	143	146	147	150
98	104	107	109	112	114	123	125	126	128	130	137	138	141	143	145
100	102	104	107	109	111	119	121	123	125	126	133	134	137	138	141
102	99	102	104	106	108	116	118	120	121	123	129	131	133	134	136
104	97	99	102	104	106	113	115	116	118	120	126	127	129	130	132
106	95	97	99	101	103	110	112	113	115	116	122	123	125	126	128
108	93	95	97	99	101	107	109	110	112	113	119	120	122	123	125

Table 3h – Extract of Table 8.8d of the HK Code[1]

Table 8.8(d) - Design strength p_c of compression members (cont'd)

λ	8) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve d														
	Steel grade and design strength p_y (N/mm^2)														
	S275					S355					S460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	91	93	95	96	98	105	106	108	109	110	115	116	118	119	121
112	88	90	92	94	96	102	103	105	106	107	112	113	115	116	118
114	86	88	90	92	94	99	101	102	103	104	109	110	112	113	114
116	85	86	88	90	91	97	98	99	101	102	106	107	109	110	111
118	83	84	86	88	89	95	96	97	98	99	103	104	106	107	108
120	81	82	84	86	87	92	93	94	95	96	101	101	103	104	105
122	79	81	82	84	85	90	91	92	93	94	98	99	100	101	102
124	77	79	80	82	83	88	89	90	91	92	95	96	98	98	99
126	76	77	78	80	81	86	87	88	89	89	93	94	95	96	97
128	74	75	77	78	79	84	85	85	86	87	91	91	93	93	94
130	72	74	75	76	77	82	83	83	84	85	88	89	90	91	92
135	68	70	71	72	73	77	78	79	79	80	83	84	85	85	86
140	65	66	67	68	69	73	73	74	75	75	78	79	80	80	81
145	62	63	64	65	65	69	69	70	71	71	74	74	75	75	76
150	59	60	60	61	62	65	66	66	67	67	69	70	71	71	72
155	56	57	57	58	59	62	62	63	63	64	66	66	67	67	68
160	53	54	55	55	56	58	59	59	60	60	62	62	63	63	64
165	50	51	52	53	53	55	56	56	57	57	59	59	60	60	61
170	48	49	49	50	51	53	53	54	54	54	56	56	57	57	57
175	46	47	47	48	48	50	51	51	51	52	53	53	54	54	55
180	44	45	45	46	46	48	48	49	49	49	50	51	51	51	52
185	42	43	43	44	44	46	46	46	47	47	48	48	49	49	49
190	40	41	41	42	42	44	44	44	44	45	46	46	46	47	47
195	38	39	39	40	40	42	42	42	42	43	44	44	44	45	45
200	37	37	38	38	39	40	40	40	41	41	42	42	42	43	43
210	34	34	35	35	35	37	37	37	37	37	38	38	39	39	39
220	31	32	32	32	33	34	34	34	34	34	35	35	36	36	36
230	29	29	30	30	30	31	31	31	32	32	32	33	33	33	33
240	27	27	28	28	28	29	29	29	29	29	30	30	30	30	31
250	25	25	26	26	26	27	27	27	27	27	28	28	28	28	28
260	24	24	24	24	24	25	25	25	25	25	26	26	26	26	26
270	22	22	22	23	23	23	23	23	24	24	24	24	24	24	25
280	21	21	21	21	21	22	22	22	22	22	23	23	23	23	23
290	19	20	20	20	20	20	21	21	21	21	21	21	21	21	21
300	18	18	19	19	19	19	19	19	19	20	20	20	20	20	20
310	17	17	17	18	18	18	18	18	18	18	19	19	19	19	19
320	16	16	16	17	17	17	17	17	17	17	18	18	18	18	18
330	15	15	16	16	16	16	16	16	16	16	17	17	17	17	17
340	15	15	15	15	15	15	15	15	15	15	16	16	16	16	16
350	14	14	14	14	14	14	14	15	15	15	15	15	15	15	15

Table 3i – Extract of Table 8.8d of the HK Code[1]

2. Compression Members under Combined Axial Force and Moments (Clause 8.9)

Compression members should be checked for ***cross sectional capacity*** and ***member buckling resistance***. Special requirements are given for columns in simple construction (see Clause 8.7.8). A more detailed explanation of this method is dealt with later.

2.1 Cross section capacity check (Clause 8.9.1)

Except for ***Class 4 slender*** cross-section, the cross section capacity can be checked as:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

F_c is the design axial compression at critical location;
A_g is the gross cross-sectional area;
p_y is the design strength;
M_x is the design moment about the major axis at critical location;
M_y is the design moment about the minor axis at critical location;
M_{cx} is the moment capacity about the major axis;
M_{cy} is the moment capacity about the minor axis.

2.2 Member buckling resistance (Clause 8.9.2)

The resistance of the member of a ***sway frame*** can be checked using,

$$\frac{F_c}{P_c} + \frac{m_x \overline{M}_x}{M_{cx}} + \frac{m_y \overline{M}_y}{M_{cy}} \le 1 \qquad \text{(Eqn 8.79)}$$

$$\frac{F_c}{\overline{P}_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \le 1 \qquad \text{(Eqn. 8.80)}$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} \le 1 \qquad \text{(Eqn. 8.81)}$$

where the notations are as given in clause 8.9.2 of the Code as follows:

- F_c is the design axial compression at critical location;
- P_{cx} is compression resistance under ***sway mode*** and about x-axis;
- P_{cy} is compression resistance under ***sway mode*** and about y-axis;
- P_c is the smaller of P_{cx} and P_{cy};
- $\overline{P}_c$ is the smaller of the axial force resistance of the column about x- and y- axis under ***non-sway mode*** and determined from a second-order analysis or taking member length as the effective length.
- M_b is the buckling resistance moment in clause 8.3.5.2
- M_x is the maximum design moment amplified for the P-△-δ effect about the major x-axis;
- $\overline{M}_x$ is the maximum ***first order linear design moment*** about the major x-axis;
- M_y is the maximum design moment amplified for the P-△-δ effect about the minor y-axis;
- $\overline{M}_y$ is the maximum ***first order linear design moment*** about the minor y-axis;
- M_{LT} is the maximum design moment amplified for the P-△-δ effect about the major x-axis governing M_b;
- M_{cx} is the elastic moment capacity p_y*Z_x about the major axis;
- M_{cy} is the elastic moment capacity p_y*Z_y about the minor axis.
- m_x is the equivalent uniform moment factor for flexural buckling about the major axis in Table 8.9;
- m_y is the equivalent uniform moment factor for flexural buckling about the minor axis in Table 8.9;
- m_{LT} is the equivalent moment factor for lateral-flexural buckling in Table 8.4.

Equation 8.79 is for checking against sway due to P-Δ moment where effective length due to member initial curvature is in control;
Equation 8.80 is for checking against the case where sway moment due to lateral force is in control;
Equation 8.81 is for combined axial force and lateral-torsional beam buckling check.
Equation 8.79 is more critical than Equation 8.80 when lateral force is small and vice versa.

2.3 Frame Classification (Cl. 6.3)

2.3.1 Non-sway frames

Except for advanced analysis, a frame is classified as non-sway and the P-Δ effect can be ignored when $\lambda_{cr} \geq 10$

For advanced analysis, a frame is classified as non-sway and the P-Δ effect can be ignored when $\lambda_{cr} \geq 15$

2.3.2 Sway frames

Except for advanced analysis, a frame is classified as sway when $10 > \lambda_{cr} \geq 5$

For advanced analysis, a frame is classified as sway when $15 > \lambda_{cr} \geq 5$

2.3.3 Sway ultra-sensitive frames

A frame is classified as sway ultra-sensitive when $\lambda_{cr} < 5$. ***Only second order P-Δ-δ or advanced analysis can be used for sway ultra-sensitive frames.***

2.4 Moment Amplification (Cl. 8.9)

The effect of moment amplification are automatically considered in a second-order P-△-δ analysis such as by using ***NIDA*** (a computer program developed by CSE of PolyU). Alternatively, $\boldsymbol{\lambda_{cr}}$ (***elastic critical load factor***) can be found by an elastic buckling analysis or by Equation 6.1,

$$\lambda_{cr} = \frac{F_N}{F_V} \cdot \frac{h}{\delta_N} \qquad \text{(Eqn 6.1)}$$

where **F_V** is the factored Dead plus Live loads on and above the floor considered;
F_N is the notional horizontal force taken typically as 0.5% of **F_V** for the building frames;
h is the storey height;
δ_N is the notional horizontal deflection of the upper storey relative to the lower storey due to the notional horizontal force **F_N**.

and used to multiply the maximum moment by the following amplification factor.

$$\frac{\lambda_{cr}}{\lambda_{cr}-1} = \text{larger of } \frac{1}{1-\dfrac{F_v \delta_N}{F_N h}} \text{ and } \frac{1}{1-\dfrac{F_c L_E^2}{\pi^2 EI}}$$

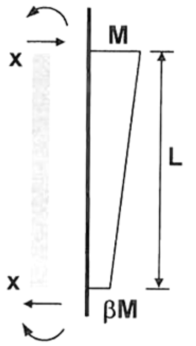
For ***non-sway frame***, only ***Equation 8.80*** and ***Equation 8.81*** are considered. For more exact analysis, $\overline{P}_c$ in equation 8.80 can be replaced by **P_c** using computed effective length. The ***P-δ*** amplification factor $\dfrac{1}{1-\dfrac{F_c L_E^2}{\pi^2 EI}}$ should be used for non-sway frames.

*** ***In order to simplify our study, only column in a non-sway frame is considered. For the design of column in a sway frame, it will not be covered in this course.***

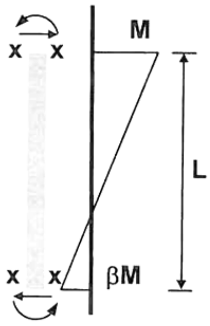
Table 8.9 - Moment equivalent factor m for flexural buckling

Segment with end moment only (value of m from the formula of general case)

β positive



β negative

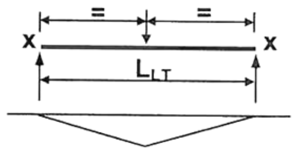


X - lateral restraint

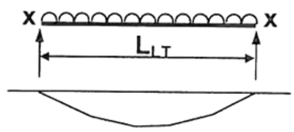
β	m
1.0	1.00
0.9	0.96
0.8	0.92
0.7	0.88
0.6	0.84
0.5	0.80
0.4	0.76
0.3	0.72
0.2	0.68
0.1	0.64
0.0	0.60
-0.1	0.58
-0.2	0.56
-0.3	0.54
-0.4	0.52
-0.5	0.50
-0.6	0.48
-0.7	0.46
-0.8	0.44
-0.9	0.42
-1.0	0.40

Segment between intermediate lateral restraints

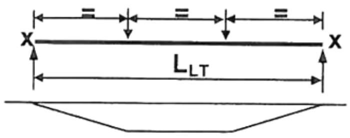
Specific cases



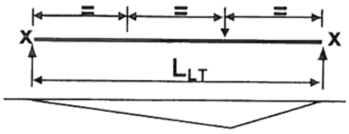
m = 0.90



m = 0.95

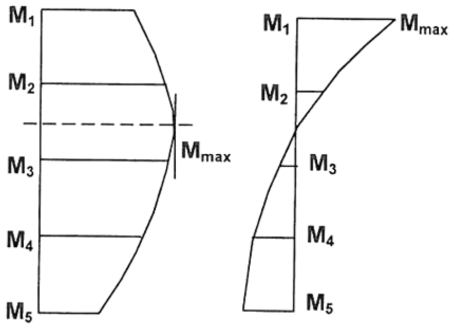


m = 0.95



m = 0.80

General case



$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{max}} \quad \text{but } m \geq \frac{0.8M_{24}}{M_{max}}$$

The moment M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length.

If M_2, M_3 and M_4 all lie on the same side of the axis, their values are all take as positive. If they lie both sides of the axis, the side leading to the larger value of m is taken as the positive side.

The values of M_{max} and M_{24} are always taken as positive. M_{max} is the maximum moment in the segment and M_{24} is the maximum moment in the central half of the segment.

Table 4 – Extract of Table 8.9 of the HK Code[1]

3. Design of Columns in Simple Construction

(a) For column design in simple construction, all beams are assumed fully loaded and simply supported on columns. All equivalent moment factors (***m***) in columns should be taken as unity ***1.0.***

(i) The column should satisfy the relationship given in Clause 8.9 for cross section capacity and member buckling resistance check.

(ii) In calculating M_b, the equivalent slenderness of the column between restraints should be taken as

$$\lambda_{LT} = 0.5 \frac{L}{r_y}$$

where L is the length of the column between lateral supports or the storey height (not L_E) and r_y is the radius of gyration about the minor axis.

(b) Clause 8.7.8: The following provisions apply specifically to columns in simple multi-story construction.

(i) Pattern loading need **NOT** be considered.

(ii) The nominal moment on columns due to simply supported beams should be calculated as follows:
For beams resting on the face of steel columns, the reaction position should be taken as the larger of ***100 mm*** from the column face or at the centre of the stiff bearing length.

(iii) In multi-storey buildings where columns are connected rigidly by splices, the net moment applied at any one level should be distributed among members in proportion to the column stiffnesses or to their ***I/L*** ratios.

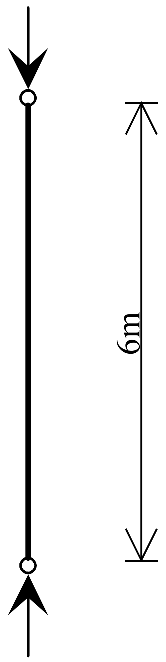
The effective length for columns in bending is not required in the calculation of M_b due to the provisions noted above; i.e. $\lambda_{LT} = 0.5\ L/r_y$.

The effective length for columns to be used when calculating the compressive resistance should be obtained from Table 8.6 depending on the end conditions. It should be noted that where the beam is loaded to more than 90% of its capacity it should be considered as incapable of affording rotational restraint to the column to which it is connected (see Clause 8.7.2).

Summary for Column Design		
Checking	**Columns in Simple Construction (e.g. Connections between beams and columns are pinned)**	**Columns in Continuous Construction (non-sway) (e.g. Columns in rigid frame)**
Cross Section Capacity Check	$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$ For plastic and compact sections, $\boldsymbol{M_{cx} = p_y \cdot S_x \leq 1.2 p_y \cdot Z_x}$ $\boldsymbol{M_{cy} = p_y \cdot S_y \leq 1.2 p_y \cdot Z_y}$ (No need to carry local capacity check as the member buckling resistance check is more critical.)	$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$ For plastic and compact sections, $\boldsymbol{M_{cx} = p_y \cdot S_x \leq 1.2 p_y \cdot Z_x}$ $\boldsymbol{M_{cy} = p_y \cdot S_y \leq 1.2 p_y \cdot Z_y}$
Member Buckling Resistance Check	$\frac{F_c}{P_c} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$ $\frac{F_c}{P_{cy}} + \frac{M_{LT}}{M_b} + \frac{\overline{M}_y}{M_{cy}} \leq 1$ Note that: 1. The equivalent moment factors (***m_x, m_y & m_{LT}***) are set to 1. 2. ***$\lambda_{LT} = 0.5*(L/r_y)$*** and from table 8.3a to determine ***p_b***. Hence ***$M_b = p_b*S_x$***. 3. ***L*** is the column height, but **<u>NOT</u>** the effective column height ***L_E***. 4. $M_{cx} = p_y \cdot Z_x$ $M_{cy} = p_y \cdot Z_y$	$\frac{F_c}{P_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \leq 1$ $\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} \leq 1$ Note that: 1. The equivalent moment factors (***m_x and m_y***) has different values in x-x and y-y directions and to be obtained from **Table 8.9**. 2. The value of ***m_{LT}*** to be obtained from **Table 8.4.** 3. ***$\lambda_{LT} = uv\lambda\sqrt{\beta_w}$*** and from table 8.3a to determine ***p_b***. Hence ***$M_b = p_b*S_x$***. 4. $M_{cx} = p_y \cdot Z_x$ $M_{cy} = p_y \cdot Z_y$

Example 1 (Column with pinned ends)

A grade S355 steel column is subjected to a factored axial force of 2500 kN. The column is 6 m long and is pinned at both ends. Select a suitable column section.



Solution

	Ref.
Factored Load F_c = 2500 kN	
Effective Lengths $L_{ex} = L_{ey} = 6.0$ m	Table 8.6
Assume 305 x 305 x 118 UC (Grade S355)	
From section table,	
T = 18.7 mm, b/T = 8.22, d/t = 20.6, r_x = 13.6 cm, r_y = 7.77 cm,	
A = 150 cm^2	
Design Strength p_y	
T = 18.7 > 16 mm $\Rightarrow p_y$=345 N/mm^2	Table 3.2
Section Class:	
Outstand element of compressive flange	
Semi-compact limit = 13 ε	

where $\varepsilon = \sqrt{\frac{275}{p_y}}$ i.e. $\sqrt{\frac{275}{345}} = 0.89$ Table 7.1

$13\,\varepsilon = 13 \times 0.89 = 11.6$

$\frac{b}{T} = 8.22 < 11.6$

Web, subject to compression throughout, semi-compact limit = 40ε

$40\varepsilon = 40 \times 0.89 = 35.6$

$\frac{d}{t} = 20.6 < 35.6$

$\Rightarrow$ **Section is not slender**

N.B. *Semi-compact criterion has been used to determine whether or not section is slender, since the capacity of slender section has to be reduced for local buckling.*

Slenderness λ $\quad \lambda_x = \frac{L_{EX}}{r_x} = \frac{6.0 \times 10^2}{13.6}$ i.e. 44.1

$\lambda_y = \frac{L_{EY}}{r_y} = \frac{6.0 \times 10^2}{7.77}$ i.e. 77.2

Compressive Strength p_c Strut table selection : -Table 8.7

For $\lambda_x = 44.1$, $p_c = 302\ \text{N/mm}^2$ Table 8.8(b)

$\lambda_y = 77.2$, $p_c = 193\ \text{N/mm}^2$ (governs) Table 8.8(c)

Compressive Resistance $P_c = A_g\, p_c$

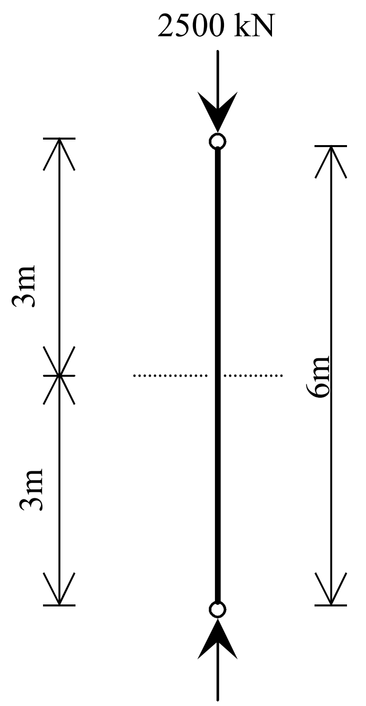
Where A_g =gross cross sectional area = 150 cm^2

$$P_c = \frac{150 \times 10^2 \times 193}{10^3} = 2895\,\text{kN}$$

Since $P_c = 2895 > F_c = 2500$ $\Rightarrow$ Section adequate

Example 2 - (Column with Pinned Ends - Tied in Weaker Axis)

A grade S355 steel column is subjected to a factored axial force of 2500 kN. The column is 6 m long and is pinned at both ends. The column is braced in the weaker axis at the mid-height. Select a suitable column section.



Solution

	Ref.
Factored Load F_c = 2500 kN	
Effective Lengths L_{ex} = 6.0m & L_{ey} = 3.0 m	Table 8.6
<u>Assume 254 x 254 x 73 UC (Grade S355)</u>	
From section table,	
T = 14.2 mm, b/T = 8.96, d/t = 23.3, r_x = 11.1 cm r_y = 6.48 cm,	
A = 93.1 cm^2	
Design Strength p_y	
T = 14.2 < 16 mm $\Rightarrow p_y$ =355 N/mm^2	Table 3.2

Section Class:

Outstand element of compressive flange

Semi-compact limit = 13 ε

where $\varepsilon = \sqrt{\frac{275}{p_y}}$ i.e. $\sqrt{\frac{275}{355}} = 0.88$ Table 7.1

13 ε = 13 x 0.88 = 11.4

$$\frac{b}{T} = 8.96 < 11.4$$

Web, subject to compression throughout, semi-compact limit = 40ε

40ε = 40 x 0.88 = 35.2

$$\frac{d}{t} = 23.3 < 35.2$$

$\Rightarrow$ Section is not slender

N.B. Semi-compact criterion has been used to determine whether or not section is slender, since the capacity of slender section has to be reduced for local buckling.

Slenderness λ $\lambda_x = \frac{L_{EX}}{r_x} = \frac{6.0 \times 10^2}{11.1}$ i.e. 54

$\lambda_y = \frac{L_{EY}}{r_y} = \frac{3.0 \times 10^2}{6.48}$ i.e. 46.3

Compressive Strength p_c Strut table selection : -Table 8.7

For $\lambda_x = 54$, p_c = 288 N/mm² Table 8.8(b)

$\lambda_y = 46.3$, p_c = 285 N/mm² (governs) Table 8.8(c)

Compressive Resistance $P_c = A_g\, p_c$

Where A_g =gross cross sectional area = 93.1 cm²

$$P_c = \frac{93.1 * 10^2 * 285}{10^3} = 2653 \text{ kN}$$

Since $P_c = 2653 > F_c = 2500$ $\Rightarrow$ Section adequate

Example 3 (Column with Axial Load and Moments)

A 203 x 203 x 60 kg/m UC (grade S355) as shown in the figure is subjected to the loadings as tabulated. Check if the column size is adequate. You may make the following assumptions for your design.

<u>Assumptions</u>

(i) The column is effectively continuous and forms part of a structure of simple construction.

(ii) The column is effectively pinned at the base.

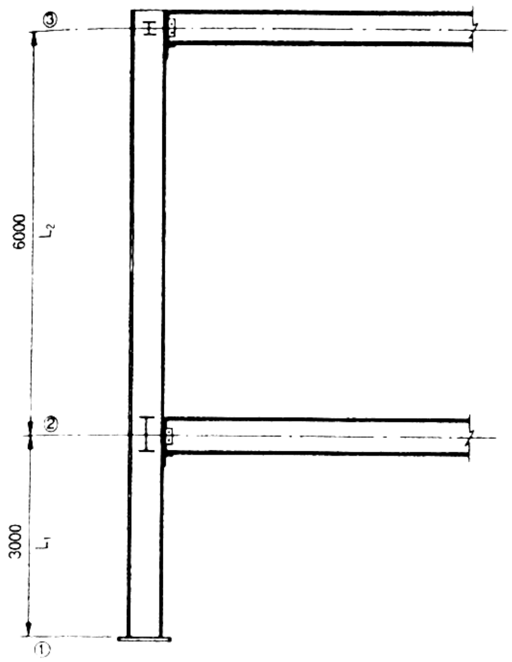
(iii) Main beams are fixed to column flange by web and seating cleats.

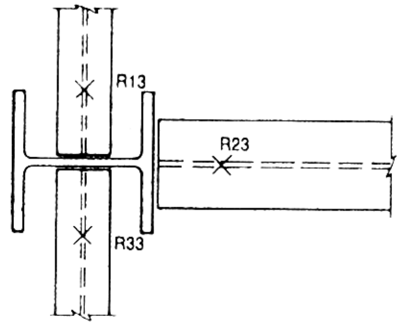
(iv) Secondary beams at level (3) are small tie members.

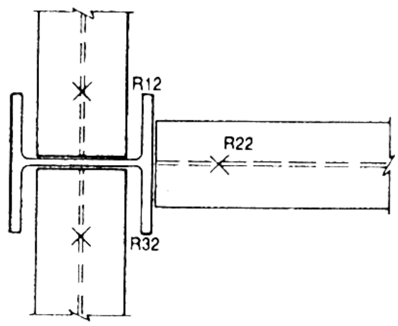
(v) Secondary beams at level (2) are substantial members.

(vi) Moments on the column from the beams are due to nominal eccentricity and not due to partial fixity.

(vii) Moment amplification factor is 1.15 for P-△-δ effect for design moment about x-axis, y-axis and design moment about major axis governing M_b







Load Factors γ_f Dead Load = 1.4
Imposed Load = 1.6

Loading Table				
	Dead Loads (kN)		Imposed Loads (kN)	
Level 3	Unfactored	Factored	Unfactored	Factored
R13	20	28	20	32
R33	20	28	20	32
R23	24	34	80	128
Self wt. (assumed)	20	28		
		118		***192***
Level 2				
R12	30	42	100	160
R32	30	42	100	160
R22	30	42	200	320
Self wt. (assumed)	20	28		
		154		***640***
Total axial load		***272***		***832***

Solution

Consider level (1) to (2)

Check 203 x 203 x 60 UC Grade S355 for column (1)-(2) **Ref**

Section properties are $t = 9.4$ mm, $T = 14.2$ mm, $d = 160.8$ mm $b/T = 7.25$, $d/t = 17.1$, $D = 209.6$ mm, $r_x = 8.96$ cm, $r_y = 5.20$ cm, $S_x = 656$ cm^3, $Z_x = 584$ cm^3 $S_y = 305$ cm^3, $Z_y = 201$ cm^3, $A = 76.4$ cm^2

Section classification

Design Strength $\boldsymbol{p_y}$

$\boldsymbol{T} = 14.2 < 16$ mm, $\Rightarrow$ $\boldsymbol{p_y} = 355$ N/mm^2 Table 3.2

Outstand element of compressive flange

$$\varepsilon = \sqrt{\frac{275}{P_y}} = \sqrt{\frac{275}{355}} = 0.88$$

$9\,\varepsilon = 9 \text{ x } 0.88 = 7.92$

$\frac{b}{T} = 7.25 < 7.92$

$$r_1 = \frac{F_c}{dtp_{yw}} = \frac{1104*10^3}{160.8*9.4*355} = 2.06$$

$$\frac{80\varepsilon}{1 + r_1} = \frac{80 * 0.88}{1 + 1} = 35.2,$$

Use $40\varepsilon = 35.2$,

$d/t = 17.1 < 35.2$

$\Rightarrow$ <u>Section is plastic</u>

<u>Column in simple multi-story construction</u>

For cross section capacity check, the column should satisfy the relationship

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

the equivalent uniform moment factor m is taken as 1.0 for bending about both axes. The equivalent slenderness is taken as $\boldsymbol{\lambda_{LT} = 0.5\, L/r_y}$

Applied axial load, $\boldsymbol{F_c}= 272 + 832 = 1104$ kN

Moment capacity about x-x axis,

$\boldsymbol{M_{cx} = p_y * S_x} = 355 * 656*10^{-3} = \underline{\boldsymbol{232.9}}$ kNm

$\boldsymbol{\leq 1.2p_y * Z_x} = 1.2*355*584*10^{-3} = 248.8$ kNm

Moment capacity about y-y axis,

$\boldsymbol{M_{cy} = p_y * S_y} = 355 * 305*10^{-3} = 108.3$ kNm

$\boldsymbol{\leq 1.2p_y * Z_y} = 1.2*355*201*10^{-3} = \underline{\boldsymbol{85.6}}$ kNm

$\boldsymbol{P_c = A_g\, p_y}$

Where A_g =gross cross sectional area = 76.4 cm^2

$$A_g\, p_y = \frac{76.4 \text{ x } 10^2 \text{ x } 355}{10^3} = 2712 \text{ kN}$$

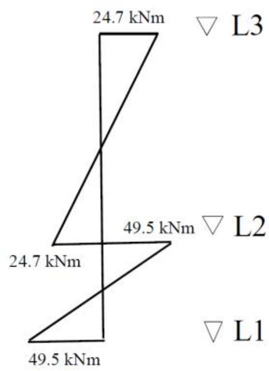
To calculate $\boldsymbol{M_x}$

Total moment at level (2) from main beam $\boldsymbol{M_2 = R_{22} * e_x}$

$\boldsymbol{e_x}$ = D/2 + 100 = 209.6/2 + 100 = 205 mm

$\boldsymbol{M_2}$ = 362 x 205 x 10^{-3} = 74.2 kNm

Assume the same size for columns (1)-(2) and (2)-(3),



$$M_{x\ 2-1} = M_2 \text{ x } \frac{L_2}{L_1 + L_2} = 74.2 \text{ x } \frac{6000}{9000} = 49.5 \text{kNm}$$

$$M_{x\ 2-3} = M_2 \text{ x } \frac{L_1}{L_1 + L_2} = 74.2 \text{ x } \frac{3000}{9000} = 24.7 \text{kNm}$$

$$\overline{M}_x = \boldsymbol{M_{x\ 2\text{-}1}} = 49.5 \text{ kNm}$$

For cross section capacity check,

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

$$\frac{1104}{2712} + \frac{1.15 * 49.5}{232.9} + 0 = 0.41 + 0.24 \ = 0.65 < \ 1$$

Member Buckling Resistance Check:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \leq 1 \qquad \text{(Eqn. 8.80)}$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} \leq 1 \qquad \text{(Eqn. 8.81)}$$

To calculate Compressive Resistance $\overline{P}_c$

Assume at level (2), rotational and translational restraints are provided to the column for buckling about both axes; hence the effective lengths are :

$\boldsymbol{L_{EX}} = \boldsymbol{L_{EY}} = 0.85\ \boldsymbol{L_1}$ $\qquad$ $\boldsymbol{L_{EX}} = \boldsymbol{L_{EY}} = 0.85\text{x } 3000 = 2550$ mm

Slenderness λ $\qquad \lambda_x = \dfrac{L_{EX}}{r_x} = \dfrac{2.55 \text{ x } 10^2}{8.96} = 28.5$

$$\lambda_y = \frac{L_{EY}}{r_y} = \frac{2.55 \text{ x } 10^2}{5.20} = 49.0$$

Strut table selection : - Table 8.7

For $\lambda_x = 28.5$, $p_c = 337\ N/mm^2$ Table 8.8(b)

$\lambda_y = 49.0$, $p_c = 277.5\ N/mm^2$ (governs) Table 8.8(c)

$\overline{P}_c$ **= smaller of P_{cx} and P_{cy}= P_{cy} = $A_g\ p_c$**

where A_g =gross cross sectional area = 76.4 cm^2

$$\overline{P}_c = P_{cy} = A_g\ p_c = \frac{76.4 \times 10^2 \times 277.5}{10^3} = 2120\,kN$$

<u>To calculate</u> M_b

$\lambda_{LT} = 0.5\ L/r_y = 0.5 \times 3000 / 5.20 \times 10 = 28.8$

For $p_y = 355\ N/mm^2$ and $\lambda_{LT} = 28.8$

$\Rightarrow$ $p_b = 355\ N/mm^2$ Table 8.3a

$M_b = S_x\ p_b = 656 \times 355 \times 10^{-3} = 232.9\ kNm$

The amplified moments for P-△-δ effect for use in eqn. 8.80 and 8.81 are

$M_x = 1.15*49.5 = 56.9\ kNm$

$M_{LT} = M_x = 56.9\ kNm$

Also, $m_x = m_y = m_{LT} = 1$ and $\overline{M}_y = 0$

<u>Member Buckling Resistance Check:</u>

$M_{cx} = p_y * Z_x = 355 * 584*10^{-3} = \underline{207.3}\ kNm$

$M_{cy} = p_y * Z_y = 355 * 201*10^{-3} = \underline{71.4}\ kNm$

Check $\dfrac{F_c}{\overline{P}_c} + \dfrac{m_x\ M_x}{M_{cx}} + \dfrac{m_y\ M_y}{M_{cy}} \le 1$

$$\frac{1104}{2120} + \frac{56.4}{207.3} + 0 = 0.52 + 0.27 = 0.79 < 1$$

Check $\dfrac{F_c}{P_{cy}} + \dfrac{m_{LT}\ M_{LT}}{M_b} + \dfrac{m_y\ \overline{M}_y}{M_{cy}} \le 1$

$$\frac{1104}{2120} + \frac{56.9}{232.9} + 0 = 0.52 + 0.24 = 0.76 < 1$$

Hence <u>section is O.K.</u>

<u>Consider Level (2) to (3)</u>

Check **<u>203 x 203 x 60 UC Grade S355</u>** for column (2)-(3) **<u>Ref</u>**

<u>Section classification</u>

Section is the same as for column (1)-(2) and is therefore not slender.

Applied axial load, $\boldsymbol{F} = 118 + 192 = 310$ kN

<u>To calculate</u> $\boldsymbol{M_x}$

Total moment at level (3) from main beam $\boldsymbol{M_3 = R_{23}} \times \boldsymbol{e_x}$

$\boldsymbol{e_x} = D/2 + 100 = 209.6/2 + 100 = 205$ mm

$\boldsymbol{M_3} = 162 \times 205 \times 10^{-3} = 33.2$ kNm

$\overline{M}_x = \boldsymbol{M_{x\ 3\text{-}2}} = 33.2$ kNm or

$\boldsymbol{M_{x\ 2\text{-}3}} = 24.7$ kNm whichever is greater.

Therefore $\overline{M}_x = 33.2$ kNm

For cross section capacity check,

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$$

$$\frac{310}{2712} + \frac{1.15 * 33.2}{232.9} + 0 = 0.11 + 0.16 = 0.27 < 1$$

<u>To calculate Compressive Resistance</u> $\overline{P}_c$

Assume at level (3) the main beam provides rotational and translational restraints for buckling about the x-x axis but the secondary beam provides translational restraint for buckling about the y-y axis.

Hence the effective lengths are :

$\boldsymbol{L_{EX}} = 0.7\ \boldsymbol{L_2}$ $L_{EX} = 0.7 \times 6000 = 4200$ mm

$\boldsymbol{L_{EY}} = 0.85\ \boldsymbol{L_2}$ $L_{EY} = 0.85 \times 6000 = 5100$ mm Table 8.6

Slenderness λ $\lambda_x = \dfrac{L_{EX}}{r_x} = \dfrac{4.2 \times 10^2}{8.96} = 46.9$

$$\lambda_y = \frac{L_{EY}}{r_y} = \frac{5.1 \times 10^2}{5.20} = 98.1$$

Strut table selection : - Table 8.7

For $\lambda_x = 46.9$, $p_c = 304$ N/mm² Table 8.8(b)

$\lambda_y = 98.1$, $p_c = 145$ N/mm² (governs) Table 8.8(c)

$\overline{P}_c$ **= smaller of P_{cx} and P_{cy}, hence $\overline{P}_c = P_{cy} = A_g\, p_c$**

where A_g = gross cross sectional area = 76.4 cm^2

$$\overline{P}_c = P_{cy} = A_g\, p_c = \frac{76.4 \times 10^2 \times 145}{10^3} = 1108\,\text{kN}$$

<u>To calculate</u> M_b

$\lambda_{LT} = 0.5\, L/r_y$ = 0.5 x 6000 /5.2 x 10 = 57.7

For p_y = 355 N/mm^2 and λ_{LT} = 57.7

$\Rightarrow$ p_b = 265 N/mm^2 Table 8.3a

$M_b = S_x\, p_b$ = 656 x 265 x 10^{-3} = 173.8 kNm

The amplified moments for P-△-δ effect for use in eqn. 8.80 and 8.81 are

M_x = 1.15*33.2 = 38.2 kNm

$M_{LT} = M_x$ = 38.2 kNm

Also, $m_x = m_y = m_{LT} = 1$ and $\overline{M}_y = 0$

<u>Member Buckling Resistance Check:</u>

Check $$\frac{F_c}{\overline{P}_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \le 1$$

$$\frac{310}{1108} + \frac{38.2}{207.3} + 0 = 0.28 + 0.18 = 0.46 < 1$$

Check $$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} \le 1$$

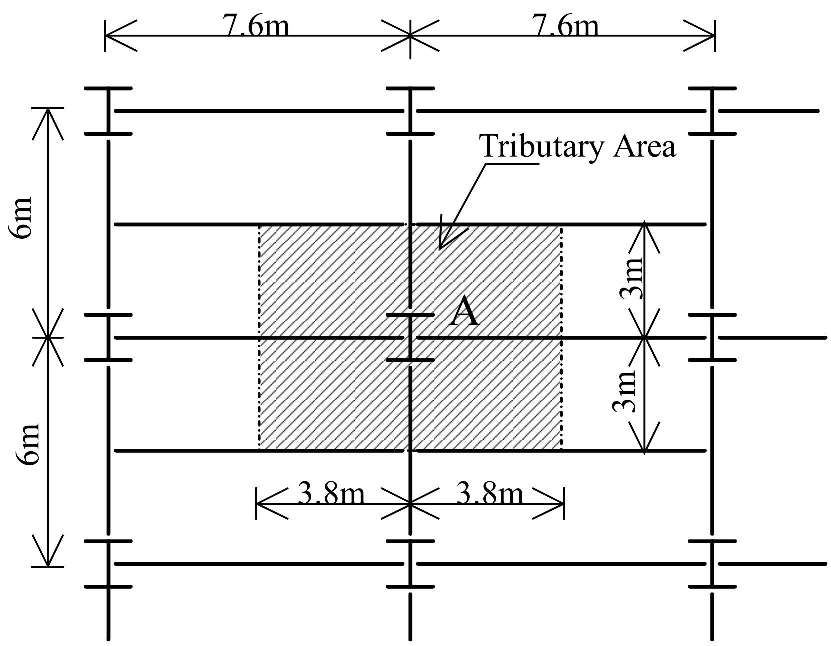
$$\frac{310}{1108} + \frac{38.2}{173.8} + 0 = 0.28 + 0.22 = 0.50 < 1$$

Example 4 (Axially Loaded Column by Tributary Area)

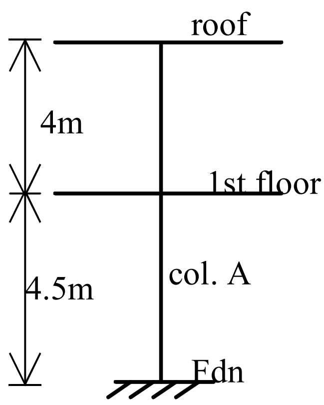
A part plan of an office floor and the elevation of the internal column ***A*** are shown in the figure. The roof and floor loads are as follows

Roof	Dead load (total)	= 5 kN/m^2
	Imposed load	= 1.5 kN/m^2
Floors	Dead load (total)	= 7 kN/m^2
	Imposed load	= 3 kN/m^2

Design column ***A*** (grade S355 steel) for axial load only. All dead loads include the self-weight of the beam and column. The roof and floor steel structures have the same layout. The foundation of the column is assumed to be fixed. Column A is effectively held in position at both ends and unrestrained in rotation by floor beams.



Floor Plan



Elevaton

Solution

When calculating the loads on the column, the imposed loads may be reduced in accordance with clause 3.7.3.1 of the ***Code of Practice for Dead and Imposed Loads 2011***. This is permitted, because it is unlikely that all floors will be fully loaded simultaneously. Values from the table are:

Imposed Load Reduction	
No. of Floors Carried by Member	**Reduction in Imposed Load (%)**
1	0
2	5
3	10
4	15
5	20
6	25
7	30
8	35
Over 8	40 maximum

The roof is regarded as a floor for imposed load reduction purposes.

The slabs for the floors and roof are one-way spanning slabs. The dead and imposed loads are calculated separately.

Loading

Since all beams are simply supported, column load may be calculated from the "***tributary area***" of column, i.e. the area multiplied by the UDL will give the load on the column.

Tributary Area of Column = $7.6 * 6 = 45.6\ m^2$

Location	Dead Load (kN)	Imposed Load (kN)
Below Roof	5 * 45.6 = 228	1.5 * 45.6 = 68.4
Σ	**228**	**68.4**
Below 1st Floor	7 * 45.6 = 319.2	3 * 45.6 = 136.8
Σ	**547.2**	**205.2**

Column Design

Roof to 1st Floor

Total factored load = 228 * 1.4 + 68.4 * 1.6 = ***428.6*** kN

Try 152 x 152 x 30 UC

T = 9.4mm, b/T = 8.13, d/t = 19.0, r_y = 3.83 cm, A = 38.3 cm^2

T = 9.4 mm < 16 mm $\Rightarrow p_y = 355\ N/mm^2$

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

b/T = 8.13 < 13*0.88 = 11.4

d/t = 19.0 < 40*0.88 = 35.2

$\Rightarrow$ the section is NOT slender.

L_E = 1 * 4000 = 4000 mm

Slenderness $\lambda = L_E / r_y$ = 4000 / 38.3 = 104.4

From table 8.8(c), $p_c = 132\ N/mm^2$

$P_c = A_g * p_c = 38.3*10^2*132 = 505.6$ kN > 428.6 kN $\Rightarrow$ O.K.

1st Floor to Foundation (Support 2 floors – reduction in LL = 5%):

Total factored load = 547.2 * 1.4 + (205.2 * 1.6) * 0.95 = ***1078*** kN

Try 203 x 203 x 46 UC

T = 11mm, b/T = 9.25, d/t = 22.3, r_y = 5.13 cm, A = 58.7 cm^2

T = 11 mm < 16 mm $\Rightarrow p_y = 355\ N/mm^2$

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

b/T = 9.25 < 13*0.88 = 11.4
d/t = 22.3 < 40*0.88 = 35.2
⇒ the section is NOT slender.

$L_E = 0.85 * 4500 = 3825$ mm
Slenderness $\lambda = L_E / r_y = 3825 / 51.3 = 74.6$
From table 8.8(c), $p_c = 204$ N/mm^2
$P_c = A_g * p_c = 58.7*10^2*204 = 1197$ kN > 1078 kN ⇒ O.K.

Example 5 (Column with Axial Force and Bending Moments)

A braced column in a non-sway frame 4.5m long is subjected to the factored end loads due to rigid frame action as listed below. The column is fixed at one end and pinned at the other end. Check that a 254 x 254 x 89 UC in grade S355 steel is adequate.

Factored axial load	=	1650 kN
Factored bending moment about x-x axis, M_x, at top	=	+80 kNm
Factored bending moment about x-x axis, M_x, at bottom	=	+24 kNm
Amplification factor for P-△-δ effect in x-x axis = 1.08		
Factored bending moment about y-y axis, M_y, at top	=	+30 kNm
Factored bending moment about y-y axis, M_y, at bottom	=	-21 kNm
Amplification factor for P-△-δ effect in y-y axis = 1.15		

Factored bending moment amplified for P-△-δ effect governing M_b

M_{LT} = 80*1.08 = 86.4 kNm (distribution same as M_x)

(*+ve* - clockwise, *-ve* - anti-clockwise)

Solution

From section table, T = 17.3 mm, b/T = 7.41, d/t = 19.4, d = 200.3mm

t = 10.3 mm, r_y = 6.55 cm, Z_x = 1096 cm^3,

Z_y = 379 cm^3, S_x = 1224 cm^3, S_y = 575 cm^3,

u = 0.850, x = 14.5, A = 113 cm^2

T = 17.3 mm > 16 mm, $\Rightarrow$ p_y = 345 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{345}} = 0.89$$

b/T = 7.41 < 9ε = 9*0.89 = 8.01 and

$$r_1 = \frac{F_c}{dtp_{yw}} = \frac{1650*10^3}{200.3*10.3*345} = 2.32$$

$$\frac{80\varepsilon}{1+1r_1} = \frac{80*0.89}{1+1} = 35.6, \qquad \text{Use } 40\varepsilon = 35.6$$

d/t = 19.4 < 35.6

$\Rightarrow$ <u>Section is plastic</u>

Cross section capacity check

Moment capacity about x-x axis,

$M_{cx} = p_y * S_x = 345 * 1224*10^{-3} = \underline{422.3}$ kNm

$\leq 1.2 p_y * Z_x = 1.2*345*1096*10^{-3} = 453.7$ kNm

Moment capacity about y-y axis,

$M_{cy} = p_y * S_y = 345 * 575*10^{-3} = 198.4$ kNm

$\leq 1.2 p_y * Z_y = 1.2*345*379*10^{-3} = \underline{156.9}$ kNm

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

$$\frac{1650x10^3}{113*10^2*345} + \frac{1.08*80}{422.3} + \frac{1.15*30}{156.9} = 0.423 + 0.205 + 0.220 = 0.848 < 1.0$$

The section is satisfactory with respect to cross section capacity.

Member Buckling Resistance Check

The effective length $L_E = 0.85 * 4500 = 3825$ mm

Slenderness $\lambda = L_E / r_y = 3825/65.5 = 58.4$

From table 8.8 (c), $p_c = 246$ N/mm^2

Consider M_x, the ratio of end moments $\beta = -24/80 = -0.3$

From table 8.9, $m_x = 0.54$

Consider M_y, the ratio of end moments $\beta = 21/30 = 0.7$

From table 8.9, $m_y = 0.88$

Moment capacity about x-x axis,

$M_{cx} = p_y * Z_x = 345 * 1096*10^{-3} = \underline{378.1}$ kNm

Moment capacity about y-y axis,

$M_{cy} = p_y * Z_y = 345 * 379*10^{-3} = \underline{130.8}$ kNm

Check $\frac{F_c}{P_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \leq 1$

$$\frac{1650*10^3}{113*10^2*246} + \frac{0.54*1.08*80}{378.1} + \frac{0.88*1.15*30}{130.8} = 0.59 + 0.12 + 0.23$$

$$= 0.94 < 1$$

The equivalent slenderness $\lambda_{LT} = uv\lambda \sqrt{\beta_w}$

$u = 0.850$, $\lambda = 58.4$, $x = 14.5$

$$v = \frac{1}{\left(1 + 0.05(\frac{\lambda}{x})^2\right)^{0.25}} = 0.862$$

For class 1 & 2 section, $\beta_w = 1$

hence $\lambda_{LT} = 0.85*0.862*58.4 = 42.8$

From table 8.3a, $p_b = 308.6$ N/mm^2

Buckling resistance moment

$M_b = p_b*S_x = 308.6*1224*10^{-3} = 377.7$ kNm

Consider M_x, the ratio of end moments $\beta = -24/80 = -0.3$

From table 8.4a, $m_{LT} = 0.48$

Check $$\frac{F_c}{P_{cy}} + \frac{m_{LT}\ M_{LT}}{M_b} + \frac{m_y\ \overline{M}_y}{M_{cy}} \leq 1$$

$$\frac{1650*10^3}{113*10^2*246} + \frac{0.48*86.4}{377.7} + \frac{0.88*30}{130.8} = 0.59 + 0.11 + 0.20$$

$$= \quad 0.90 < 1.0$$

The section is also satisfactory with respect to member resistance buckling.

📖 **Revision**

Read reference 2 on P.144 -184.

📖 **Main Reference**

1. *Code of practice for Structural Use of Steel 2011, Buildings Department, the Government of HKSAR*
2. *Structural Steelwork, Design to Limit State Theory, 3rd edition (2004), Dennis Lam, Thien-Cheong Ang, Sing-Ping Chiew, Elsevier.*
3. *Limit States Design of Structural Steelwork, 3rd edition (2001), D.A. Nethercot, Spon Press.*
4. *The Behaviour and Design of Steel Structures to BS5950, 3rd edition (2001), N.S. Trahair, M.A. Bardford, D.A. Nethercot, Spon Press.*
5. *Steel Designers' Manual, 6th edition (2003), Oxford: Blackwell Science, Steel Construction Institute.*
6. *Structural Steelwork, Design to Limit State Theory, 2nd edition, T.J. MacGinley and T.C. Ang, Butterworths.*

| TUTORIAL 4 |

Q1. Given a column section of 305 x 305 x 118 kg/m UC with an effective length L_E = 4.5 m, find the compressive resistance P_c of the column. Use grade S355 steel.

Q2. Check the column in Q1 for structural adequacy with the following ultimate design force and bending moments, from the 1st order analysis, ***if the column is in a non-sway frame.***
Axial force including self weight of the column F = 1450 kN
Bending Moment M_x = 220 kNm, M_y = 35 kNm, M_{LT} = 200 kNm
Amplification factor for P-△-δ effect in x-x axis = 1.1
Amplification factor for P-△-δ effect in y-y axis = 1.15

Q3. A column between floors of a non-sway multi-story building frame is subjected to biaxial bending at the top and bottom. The column member is of the Grade S355 with 305 x 305 x 198 kg/m UC section. The effective length of the column is 6m. Investigate its adequacy if the design loads data, from 1st order analysis, are as follows:

Ultimate axial compression = 2500 kN
Ultimate moments,
Top – about major axis = +350 kNm
- about minor axis = +90 kNm
Amplification factor for P-△-δ effect in x-x axis = 1.1
Bottom –about major axis = +150 kNm
- about minor axis = -70 kNm
Amplification factor for P-△-δ effect in y-y axis = 1.15
Ultimate moment amplified for P-△-δ effect governing M_b
M_{LT} = 350 kNm
(+ve – clockwise, -ve – anti-clockwise)

Q4. Check the adequacy of a 203 x 203 x 60 kg/m UC (grade S355) column with an effective length $L_E = L$ = 4.5 m, if the total ultimate vertical loads, from 1st order analysis, acting on it at the roof level of a building are as follows, assuming ***simple construction*** in the design:

F_1 = 650 kN including self weight acting at the center of the column.
F_x = 140 kN (along X-axis) with an eccentricity e_y of 105 mm from the y-axis, due to beam reactions.
F_y = 100 kN (along Y-axis) with an eccentricity e_x of 208 mm from the x-axis, due to beam reactions.
Moment Amplification factor for P-△-δ effect for all moments = 1.15

Q5. A column between floors of a ***simple construction*** multi-story building is subjected to the following loadings due to beam reactions and the axial force above. It is assumed that the stiffness of the upper and the lower columns of the floor under consideration is the same. Check the adequacy of a grade S355, 254 x 254 x 89 kg/m UC section if the effective length of the column is 5.1m and the actual column height is 6m. All the loadings shown below are ***factored*** loads from the 1st order analysis.

F = 1200 kN including self weight acting at the center of the column.

F_{x1} = 150 kN with an eccentricity e_y of 220 mm from the y-axis, due to beam reactions.

F_{x2} = 100 kN with an eccentricity e_y of 110 mm from the y-axis, due to beam reactions.

F_{y1} = 180 kN with an eccentricity e_x of 230 mm from the x-axis, due to beam reactions.

F_{y2} = 300 kN with an eccentricity e_x of 270 mm from the x-axis, due to beam reactions.

Moment Amplification factor for P-△-δ effect for all moments = 1.15

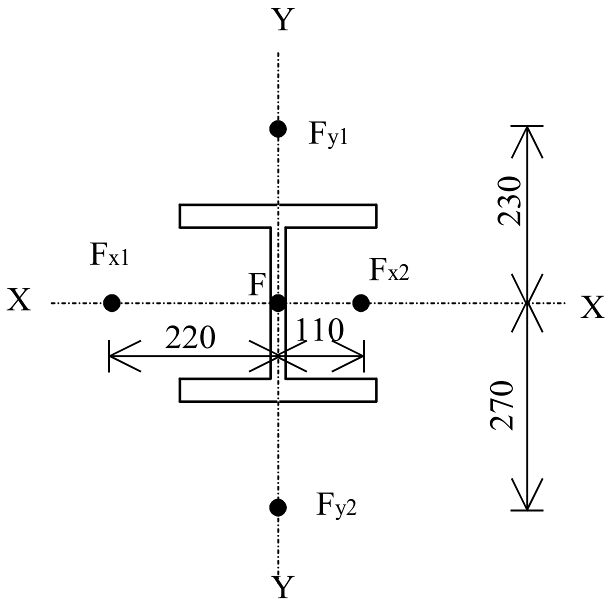


Figure Q5