8 DESIGN OF STRUCTURAL MEMBERS

8.1 GENERAL

This section gives recommendations for the design of simple members and of members that form part of a frame. Refer to section 7 of the Code for classification of sections and section 3 for material design strength. Member resistance and section capacity should be not less than design load effects. Design requirements should be satisfied for local plate buckling; web buckling and crushing; and deflection and vibration.

The same design principles should be applied when steel materials from countries other than those given in Annex A1.1 are used. The steel material and its manufacturer shall comply with the requirements for Class 2 steel given in clause 3.1.1 of the Code. Material properties provided by the manufacturers may be used together with appropriate values of imperfection constant and material factors given in Appendices 8.1 to 8.4.

When Class 3 uncertified steel is used, the buckling curves for the steel material should be obtained from a reliable source and the material buckling strength so determined should be limited to the material strength given in clause 3.1.4.

The buckling design of asymmetric and unequal-flanged sections is not covered in this section, except for angle and tee sections.

Engineers are recommended to check the structure for torsion and warping from the first principle of stress analysis.

8.2 RESTRAINED BEAMS

Restrained beams refer to beams provided with full lateral restraint to their top flanges and with nominal torsional restraint at their ends. In such a case, lateral-torsional buckling should not occur before plastic moment capacity.

Full lateral restraint can be assumed when the compression flange of a beam is connected positively to a floor or to similar structural elements capable of providing a lateral restraining force. This lateral restraining force shall be 2.5% of the maximum force in the compression flange of the member, or more simply 2.5% of the squash load of the compression flange. This lateral force should be provided uniformly along the flange with the self weight and imposed loading from the floor forming the dominant mode of loading on the member.

8.2.1 Shear capacity

For loads parallel to the webs, the shear capacity V_c given below should not be less than the design shear force V.

$$V_c = \frac{p_y\ A_v}{\sqrt{3}} \geq V \qquad (8.1)$$

in which

A_v is the shear area given by,

Rolled I, H and channels sections	tD
Welded I-sections	td
Rolled and welded rectangular hollow section	$2td$
Rolled and welded T-sections	$t(D-T)$
Circular hollow sections	$0.6A$
Solid rectangular sections	$0.9A$
Others	$0.9A_O$

where

A is the cross-sectional area;

A_O is the area of the rectilinear element in the cross-section with largest dimension parallel to the design shear force direction;

B is the overall breadth;

D is the overall depth;

d is the depth of the web;

T is the flange thickness;

t is the web thickness.

Shear buckling resistance should be checked in accordance with clause 8.4.6.

8.2.2 Moment capacity

The moment capacity of a fully restrained beam is given below and should not be less than the design bending moment.

8.2.2.1 Low shear condition

The following equations should be used when the design shear force V is not larger than 0.6 of the shear capacity V_c .

For Class 1 plastic and Class 2 compact sections:

$$M_c = p_y S \le 1.2 p_y Z \qquad (8.2)$$

For Class 3 semi-compact sections:

$$M_c = p_y Z \qquad (8.3)$$

or $M_c = p_y S_{eff}$ (8.4)

For Class 4 slender sections:

$$M_c = p_y Z_{eff} \qquad (8.5)$$

or $M_c = p_{yr} Z$ (8.6)

where

S is the plastic modulus;

Z is the elastic modulus;

S_{eff} is the effective plastic modulus;

Z_{eff} is the effective elastic modulus;

p_{yr} is the design strength reduced for slender sections.

When high strength steel is used, the use of a Class 4 slender section is not permitted. When ultra high strength steel is used, the use of a plastic modulus is not permitted.

8.2.2.2 High shear condition

The following equations should be used when the design shear force V is larger than 0.6 of the shear capacity V_c .

For Class 1 plastic and Class 2 compact sections:

$$M_c = p_y (S - \rho S_v) \le 1.2 p_y (Z - \rho S_v / 1.5) \qquad (8.7)$$

For Class 3 semi-compact sections:

$$M_c = p_y (Z - \rho S_v / 1.5) \qquad (8.8)$$

or $M_c = p_y (S_{eff} - \rho S_v / 1.5)$ (8.9)

For Class 4 slender sections:

$$M_c = p_y (Z_{eff} - \rho S_v / 1.5) \qquad (8.10)$$

where

S_V is the plastic modulus of shear area A_V in clause 8.2.1.

ρ is given by $\left(\frac{2V}{V_c}-1\right)^2$;

V_c is the shear capacity;

V is the design shear.

When the web slenderness d/t is larger than 70ε for hot-rolled sections, or 62ε for welded sections, the moment capacity should allow for shear buckling as given in clause 8.4.6.

8.2.2.3 Notched ends

When parts of the flanges are curtailed for notched end connections of I, H or channel sections, the moment capacity should be taken as follows:

8.2.2.3.1 Low shear condition at notched ends

When the design shear force V is not larger than 0.75 of the shear capacity V_c of the notched area,

$$M_c = p_y Z_r \tag{8.11}$$

8.2.2.3.2 High shear condition at notched ends

When the design shear force V is larger than 0.75 of the shear capacity V_c of the notch area,

$$M_c = 1.5 p_y Z_r \sqrt{1-\left(\frac{V}{V_c}\right)^2} \tag{8.12}$$

where,

Z_r is the elastic modulus of the section after deduction for the notched material.

8.2.3 Beams with web openings

Where openings are present in beams a reduction in their strength and stiffness should be considered in design. When the strength of such beams is inadequate, reinforcement should be provided. The minimum net section properties should be used for the lateral-torsional buckling check of beams with openings, refer to clause 8.3.

8.2.3.1 Isolated circular openings

8.2.3.1.1 Unreinforced openings

Net section properties are not required in design when the following conditions are met:

a) The member is a plastic Class 1 or compact Class 2 section.

b) The cross-section is symmetrical about the plane of bending.

c) The opening is located within the middle third of the depth of the section and the middle half of the span of the member.

d) The spacing of openings along the member longitudinal axis is not less than 2.5 times the opening diameter or the larger of the opening diameter when they are of different size.

e) The distance between the centre of each opening to the nearest point load is not less than the depth of the member.

f) The load is generally uniformly distributed.

g) The shear due to a point load is not larger than 10% of the shear capacity of the cross section and the maximum shear for the whole beam is not larger than 50% of the shear capacity.

8.2.3.1.2 Reinforced openings

When the requirements in clause 8.2.3.1.1 are not satisfied, web reinforcement may be provided adjacent to the opening to compensate for the removed material. Reinforcement should be carried past the opening for a distance such that the local shear stress caused by force transfer between the reinforcement and the web is not larger than $p_y / \sqrt{3}$.

8.2.3.2 *Members with reinforced isolated openings*

8.2.3.2.1 Local buckling

Compression elements should be checked against local plate buckling as given in clauses 7.5 or 7.7.

8.2.3.2.2 Shear

Secondary Vierendeel moments due to shear forces at openings should be considered. The shear stress at an opening should not exceed $p_y/\sqrt{3}$.

8.2.3.2.3 Moment capacity

The moment capacity of the cross-section should be determined from the net sectional properties and allowing for the Vierendeel moments due to shear at opening.

8.2.3.2.4 Point loads

Load bearing stiffeners should be provided when a point load is closer to an opening than the depth of the member. When the point load is within the opening, reference should be made to specialist literature or a detailed analysis should be carried out using the finite element method.

8.2.3.2.5 Deflection

Additional deflections caused by the removal of material at the opening should be considered.

8.2.3.3 *Members with reinforced multiple openings*

8.2.3.3.1 Local buckling

Compression elements should be checked against local plate buckling as given in clause 7.5 or 7.7.

8.2.3.3.2 Shear

Secondary Vierendeel moments due to shear forces at openings should be considered. Shear stresses at openings should not exceed $p_y/\sqrt{3}$. The shear stress across a web post between two openings, based on the shear area of the web post at its narrowest point, should not be larger than $0.7p_y$.

8.2.3.3.3 Moment capacity

The moment capacity of the cross-section should be determined from the net sectional properties allowing for the Vierendeel moments caused by shear at openings.

8.2.3.3.4 Point loads

The load capacity and buckling resistance of the web should be checked in accordance with clauses 8.2.3.2.4 and 8.4 for the provision of stiffeners. When the point load is within the opening, reference should be made to specialist literature or a detailed analysis should be carried out using the finite element method.

8.2.3.3.5 Deflection

Additional deflections caused by the removal of material at the opening should be considered.

8.2.3.3.6 Web posts

Stability of web posts between openings and at ends of members should be checked and stiffeners should be provided when necessary.

8.2.4 Castellated beams

Typical castellated beams as shown in Figure 8.1 are fabricated from rolled I- or H-sections or from channels. The web posts are assumed to be stable when the ratio d/t is not larger than 70ε. This does not apply to castellated beams with other types of openings nor to opening of other shapes.

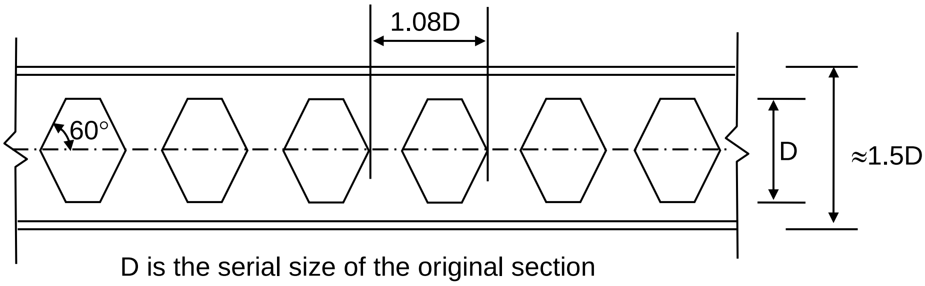


Figure 8.1 - Standard castellated beams

8.3 LATERAL-TORSIONAL BUCKLING OF BEAMS

When the condition for full lateral restraint given in clause 8.2 cannot be satisfied, the resistance of beams to lateral-torsional buckling should be checked. The designer should refer to specialist literature or to carry out a finite element buckling analysis for the buckling of beams with web openings.

8.3.1 Intermediate and end lateral restraints

Restraints, such as those shown in Figure 8.2, may be provided to reduce the effective length of a beam and to increase its buckling resistance moment. These restraints must have adequate stiffness and strength to prevent lateral movement of the compression flange of the beam. The restraints should be as close as practical to the shear centre of the compression flange of the beam. However, if torsional restraint is also provided at the same cross section, the intermediate lateral restraint may be connected at other levels of the section.

The strength requirement for a lateral restraint is 2.5% of the maximum force in the compression flange, or conservatively, 2.5% of its squash load. When more than one lateral restraint is provided at different locations along a beam, the sum of forces provided by these restraints should not be less than 2.5% of the compression flange force and each restraint should provide not less than 1% of the maximum force in the compression flange.

When a bracing system provides lateral restraint to a number of beams, the system should have adequate strength to resist the sum of the 2.5% restraining forces for the maximum forces in compression flanges of these beams, reduced by the factor k_r below:

$$k_r = \sqrt{0.2 + \frac{1}{N_r}} \leq 1 \tag{8.13}$$

in which N_r is the total number of parallel members restrained.

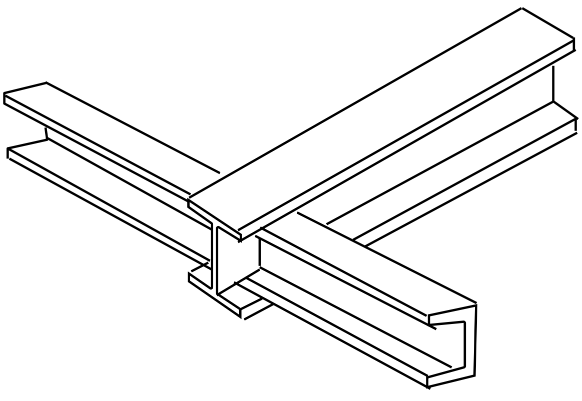


Figure 8.2 - Restraint at end of a cantilever to reduce its effective length

8.3.2 Torsional restraint

Torsional restraint refers to restraint against rotation about the member longitudinal z-axis. It may be provided at any section along a beam or at supports by restraining the lateral movement of both flanges using a pair of lateral restraints as in clause 8.3.1. The strength requirement should satisfy clause 8.3.1 for lateral restraints.

8.3.3 Normal and destabilizing loads

A destabilizing loading condition should be assumed when considering effective length of beams with the dominant loads applied to the top flange and with both the loads and the flange free to deflect and rotate relative to the shear centre of the cross-section. Special consideration and a further reduction should be taken for cases where the loads are applied at a significant distance above the top flange of the beam. Other cases not subject to these conditions should be assumed to be a normal loading condition.

8.3.4 Effective length for lateral-torsional buckling

8.3.4.1 Simple beams without intermediate lateral restraints

(a) For a beam under normal loading conditions with its compression flange restrained against lateral movement at the end supports, but free to rotate on plan and with ends under nominal torsional restraint about the longitudinal axis of the beam at the end supports, the effective length is 1.0 of the span of the beam, i.e.

$$L_E = L_{LT} \tag{8.14}$$

(b) For a beam under normal loading conditions with the compression flange fully restrained against rotation on plan at its end supports, the effective length can be taken as:

$$L_E = 0.8L_{LT} \tag{8.15}$$

(c) For a beam under normal loading conditions with compression flanges unrestrained against lateral movement at end supports and with both flanges free to rotate on plan, the effective length is the sum of 1.2 times the span of the beam and 2 times the beam depth, i.e.

$$L_E = 1.2L_{LT} + 2D \tag{8.16}$$

(d) For beams under destabilizing loads, the effective length should be multiplied by a factor of 1.2.

in which L_{LT} is the segment length between lateral restraints under consideration.

8.3.4.2 Beams with intermediate lateral restraints

For simple beams with adequate intermediate lateral restraints satisfying clause 8.3.1, the effective length L_E is normally taken as $1.0L_{LT}$ for a normal load or $1.2L_{LT}$ for a destabilising load. For beams with one end supported and the other end restrained laterally, the effective length should be taken as the mean value of the effective length in clause 8.3.4.1 and this clause.

8.3.4.3 Cantilevers

8.3.4.3.1 Cantilevers without intermediate restraints

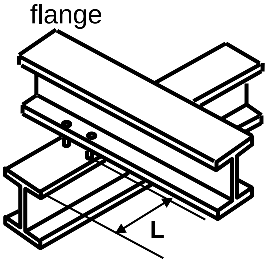
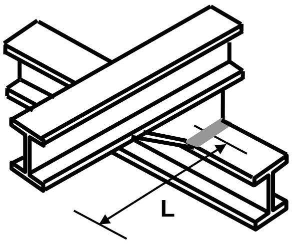
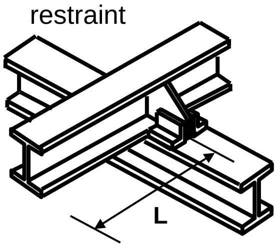
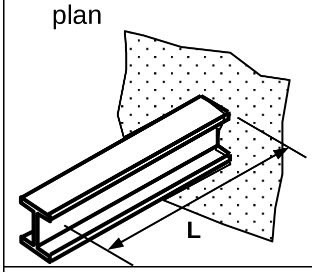
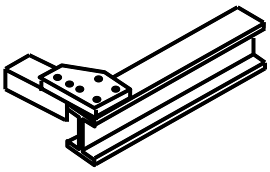
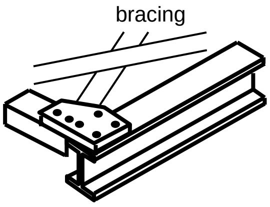
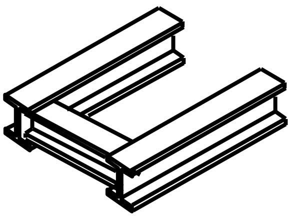
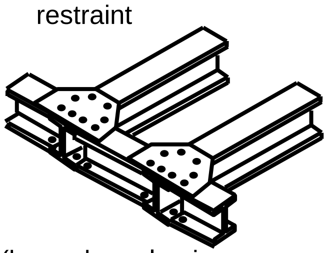
The effective length of a cantilever without intermediate restraint can be taken from Table 8.1 with the following additional considerations:

When a dominant moment exists at the cantilever tip the effective length is increased by the greater of 30% of its original effective length or an addition of $0.3L$ to the effective length where L is the length of the cantilever.

8.3.4.3.2 Cantilevers with intermediate lateral restraint

The effective length is taken as the segment length between lateral restraints when conditions in either case c) 4) or d) 4) of Table 8.1 are met and the cantilever is under normal load. For other conditions, refer to Table 8.1 with the length between segments being taken as the length of the cantilever.

Table 8.1 - Effective length of cantilever without intermediate restraints

Restraint condition		Loading condition	
At support	At tip	Normal	Destabilizing
a) continuous, with lateral restraint to top flange  	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) continuous, with partial torsional restraint  	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) continuous, with lateral and torsional restraint  	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) restrained laterally and torsionally and against rotation on plan  	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L
Tip restraint conditions			
1) Free  (not braced on plan)	2) Lateral restraint to top flange  (braced on plan in one or more bay)	3) Torsional restraint  (not braced on plan)	4) Lateral and torsional restraint  (braced on plan in one or more bay)

8.3.5 Moment resistance to lateral-torsional buckling

8.3.5.1 Limiting slenderness

Lateral-torsional buckling need not be checked in the following cases:

Bending about the minor axis;

CHS, SHS or circular or square solid sections;

RHS with L_E/r_y for relevant value of *D/B* ratio not exceeding the values in Table 8.2;

I -, H -, channel or box sections with λ_{LT} not exceeding values in the bottom row of Table 8.3.

Table 8.2 - limiting value of L_E/r_y for RHS

Ratio *D/B*	Limiting value of L_E/r_y		Ratio *D/B*	Limiting value of L_E/r_y		Ratio *D/B*	Limiting value of L_E/r_y
1.25	770 ε^2		1.50	515 ε^2		2.0	340 ε^2
1.33	670 ε^2		1.67	435 ε^2		2.5	275 ε^2
1.40	580 ε^2		1.75	410 ε^2		3.0	225 ε^2
1.44	550 ε^2		1.80	395 ε^2		4.0	170 ε^2

Key:
B is the width of the section;
D is the depth of the section;
L_E is the effective length for lateral-torsional buckling from clause 8.3.4;
p_y is the design strength in section 3;
r_y is the radius of gyration of the section about its minor axis;
$\varepsilon = \sqrt{275/p_y}$

The following design procedure should be followed for the design of prismatic and I-, H- and channel sections with equal flanges and of slenderness ratio larger than the limiting slenderness ratio in the bottom rows of Tables 8.3a to 8.3c. For other sections with unequal flanges, varying sections along their length, substantial opening at various locations and unsymmetrical cross-sections, reference should be made to other recognized literature. Alternatively, a buckling analysis may be carried out allowing various factors such as load height, boundary conditions and material properties can be used for computation of critical bending moment M_{cr} and equivalent slenderness as,

$$\lambda_{LT} = \sqrt{\frac{M_p \pi^2 E}{M_{cr} p_y}} \qquad (8.17)$$

where M_p is the plastic moment of the section

8.3.5.2 *Buckling resistance moment*

In each segment of a beam, the buckling resistance moment M_b should satisfy the following.

$$m_{LT} M_x \le M_b \text{ and} \tag{8.18}$$

$$M_x \le M_{cx} \tag{8.19}$$

where

m_{LT} is the equivalent uniform moment factor for lateral-torsional buckling of simple beams from Table 8.4. Conservatively it can be taken as unity. For cantilevers, m_{LT} is equal to 1.

M_x = Maximum bending moment along the beam

For Class 1 plastic and Class 2 compact sections:

$$M_b = p_b S_x \tag{8.20}$$

For Class 3 semi-compact sections:

$$M_b = p_b Z_x \text{ or} \tag{8.21}$$

$$M_b = p_b S_{eff} \tag{8.22}$$

For Class 4 slender sections:

$$M_b = p_b Z_{x,eff} \text{ or} \tag{8.23}$$

$$M_b = p_b \frac{p_{yr}}{p_y} Z_x \tag{8.24}$$

S_{eff} is the effective plastic modulus of the section using the effective width method;

$Z_{y,eff}$ is the effective elastic modulus of the section using the effective width method and

p_{yr} is the design strength allowing for local plate buckling in the effective stress method.

p_b is the buckling strength of the beam, determined from Table 8.3a for hot-rolled sections and Table 8.3b for welded sections using a suitable equivalent slenderness λ_{LT} in clause 8.3.5.3 and relevant design strength p_y.

When high strength steel is used, the use of a Class 4 slender section is not permitted.

Alternatively, formulae in Appendix 8.1 may be used to compute p_b.

Table 8.3a - Bending strength p_b (N/mm^2) for rolled sections

λ_{LT}	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
25	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
30	235	245	255	265	275	315	325	335	345	355	395	403	421	429	446
35	235	245	255	265	273	307	316	324	332	341	378	386	402	410	426
40	229	238	246	254	262	294	302	309	317	325	359	367	382	389	404
45	219	227	235	242	250	280	287	294	302	309	340	347	361	367	381
50	210	217	224	231	238	265	272	279	285	292	320	326	338	344	356
55	199	206	213	219	226	251	257	263	268	274	299	305	315	320	330
60	189	195	201	207	213	236	241	246	251	257	278	283	292	296	304
65	179	185	190	196	201	221	225	230	234	239	257	261	269	272	279
70	169	174	179	184	188	206	210	214	218	222	237	241	247	250	256
75	159	164	168	172	176	192	195	199	202	205	219	221	226	229	234
80	150	154	158	161	165	178	181	184	187	190	201	203	208	210	214
85	140	144	147	151	154	165	168	170	173	175	185	187	190	192	195
90	132	135	138	141	144	153	156	158	160	162	170	172	175	176	179
95	124	126	129	131	134	143	144	146	148	150	157	158	161	162	164
100	116	118	121	123	125	132	134	136	137	139	145	146	148	149	151
105	109	111	113	115	117	123	125	126	128	129	134	135	137	138	140
110	102	104	106	107	109	115	116	117	119	120	124	125	127	128	129
115	96	97	99	101	102	107	108	109	110	111	115	116	118	118	120
120	90	91	93	94	96	100	101	102	103	104	107	108	109	110	111
125	85	86	87	89	90	94	95	96	96	97	100	101	102	103	104
130	80	81	82	83	84	88	89	90	90	91	94	94	95	96	97
135	75	76	77	78	79	83	83	84	85	85	88	88	89	90	90
140	71	72	73	74	75	78	78	79	80	80	82	83	84	84	85
145	67	68	69	70	71	73	74	74	75	75	77	78	79	79	80
150	64	64	65	66	67	69	70	70	71	71	73	73	74	74	75
155	60	61	62	62	63	65	66	66	67	67	69	69	70	70	71
160	57	58	59	59	60	62	62	63	63	63	65	65	66	66	67
165	54	55	56	56	57	59	59	59	60	60	61	62	62	62	63
170	52	52	53	53	54	56	56	56	57	57	58	58	59	59	60
175	49	50	50	51	51	53	53	53	54	54	55	55	56	56	56
180	47	47	48	48	49	50	51	51	51	51	52	53	53	53	54
185	45	45	46	46	46	48	48	48	49	49	50	50	50	51	51
190	43	43	44	44	44	46	46	46	46	47	48	48	48	48	48
195	41	41	42	42	42	43	44	44	44	44	45	45	46	46	46
200	39	39	40	40	40	42	42	42	42	42	43	43	44	44	44
210	36	36	37	37	37	38	38	38	39	39	39	40	40	40	40
220	33	33	34	34	34	35	35	35	35	36	36	36	37	37	37
230	31	31	31	31	31	32	32	33	33	33	33	33	34	34	34
240	28	29	29	29	29	30	30	30	30	30	31	31	31	31	31
250	26	27	27	27	27	28	28	28	28	28	29	29	29	29	29
λ_{L0}	37.1	36.3	35.6	35.0	34.3	32.1	31.6	31.1	30.6	30.2	28.4	28.1	27.4	27.1	26.5

λ_{L0} is the maximum slenderness ratio of the member having negligible buckling effect.

Table 8.3b - Bending strength p_b (N/mm^2) for welded sections

(i) S275 ~ S460 steel

λ_{LT}	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
25	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
30	235	245	255	265	275	315	325	335	345	355	390	397	412	419	434
35	235	245	255	265	272	300	307	314	321	328	358	365	378	385	398
40	224	231	237	244	250	276	282	288	295	301	328	334	346	352	364
45	206	212	218	224	230	253	259	265	270	276	300	306	316	321	332
50	190	196	201	207	212	233	238	243	248	253	275	279	288	293	302
55	175	180	185	190	195	214	219	223	227	232	251	255	263	269	281
60	162	167	171	176	180	197	201	205	209	212	237	242	253	258	269
65	150	154	158	162	166	183	188	194	199	204	227	232	242	247	256
70	139	142	146	150	155	177	182	187	192	196	217	222	230	234	242
75	130	135	140	145	151	170	175	179	184	188	207	210	218	221	228
80	126	131	136	141	146	163	168	172	176	179	196	199	205	208	214
85	122	127	131	136	140	156	160	164	167	171	185	187	190	192	195
90	118	123	127	131	135	149	152	156	159	162	170	172	175	176	179
95	114	118	122	125	129	142	144	146	148	150	157	158	161	162	164
100	110	113	117	120	123	132	134	136	137	139	145	146	148	149	151
105	106	109	112	115	117	123	125	126	128	129	134	135	137	138	140
110	101	104	106	107	109	115	116	117	119	120	124	125	127	128	129
115	96	97	99	101	102	107	108	109	110	111	115	116	118	118	120
120	90	91	93	94	96	100	101	102	103	104	107	108	109	110	111
125	85	86	87	89	90	94	95	96	96	97	100	101	102	103	104
130	80	81	82	83	84	88	89	90	90	91	94	94	95	96	97
135	75	76	77	78	79	83	83	84	85	85	88	88	89	90	90
140	71	72	73	74	75	78	78	79	80	80	82	83	84	84	85
145	67	68	69	70	71	73	74	74	75	75	77	78	79	79	80
150	64	64	65	66	67	69	70	70	71	71	73	73	74	74	75
155	60	61	62	62	63	65	66	66	67	67	69	69	70	70	71
160	57	58	59	59	60	62	62	63	63	63	65	65	66	66	67
165	54	55	56	56	57	59	59	59	60	60	61	62	62	62	63
170	52	52	53	53	54	56	56	56	57	57	58	58	59	59	60
175	49	50	50	51	51	53	53	53	54	54	55	55	56	56	56
180	47	47	48	48	49	50	51	51	51	51	52	53	53	53	54
185	45	45	46	46	46	48	48	48	49	49	50	50	50	51	51
190	43	43	44	44	44	46	46	46	46	47	48	48	48	48	48
195	41	41	42	42	42	43	44	44	44	44	45	45	46	46	46
200	39	39	40	40	40	42	42	42	42	42	43	43	44	44	44
210	36	36	37	37	37	38	38	38	39	39	39	40	40	40	40
220	33	33	34	34	34	35	35	35	35	36	36	36	37	37	37
230	31	31	31	31	31	32	32	33	33	33	33	33	34	34	34
240	28	29	29	29	29	30	30	30	30	30	31	31	31	31	31
250	26	27	27	27	27	28	28	28	28	28	29	29	29	29	29
λ_{L0}	37.1	36.3	35.6	35.0	34.3	32.1	31.6	31.1	30.6	30.2	28.4	28.1	27.4	27.1	26.5

λ_{L0} is the maximum slenderness ratio of the member having negligible buckling effect.

Table 8.3b - Bending strength p_b (N/mm^2) for welded sections (Cont'd)

(ii) S550 ~ S690 steel

λ_{LT}	Strength grade and design strength p_y (N/mm^2)					
	S550			S690		
	490	530	550	630	650	690
25	490	527	543	606	621	652
30	456	484	499	554	568	595
35	417	443	455	505	517	540
40	381	403	414	457	467	487
45	347	366	375	411	422	446
50	315	334	345	391	401	422
55	297	319	329	369	378	396
60	284	303	312	345	353	368
65	269	285	293	321	327	339
70	254	267	274	294	297	303
75	238	248	252	264	267	272
80	219	226	228	239	241	245
85	200	205	208	216	218	221
90	183	187	189	196	198	201
95	168	171	173	179	180	183
100	154	157	159	164	165	167
105	142	145	146	150	151	153
110	131	134	135	138	139	141
115	121	124	125	128	129	130
120	113	115	115	118	119	120
125	105	107	107	110	110	111
130	98	99	100	102	103	104
135	92	93	93	95	96	97
140	86	87	87	89	90	90
145	80	82	82	84	84	85
150	76	77	77	79	79	79
155	71	72	73	74	74	75
160	67	68	68	70	70	70
165	64	64	65	66	66	66
170	60	61	61	62	62	63
175	57	58	58	59	59	59
180	54	55	55	56	56	56
185	51	52	52	53	53	53
190	49	49	50	50	51	51
195	47	47	47	48	48	48
200	44	45	45	46	46	46
210	41	41	41	42	42	42
220	37	37	38	38	38	38
230	34	34	35	35	35	35
240	32	32	32	32	32	32
250	29	29	29	30	30	30
λ_{L0}	25.7	24.7	24.3	22.7	22.3	21.7

Table 8.3c - Bending strength p_b (N/mm^2) for other steel source

(i) Bending strength for rolled sections

λ_{LT}	Steel grade and design strength (N/mm²)							
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690	
	215	305	345	375	410	520	615	630
25	215	305	345	375	410	520	604	618
30	215	305	345	373	403	497	577	589
35	215	299	332	357	386	473	546	557
40	212	286	317	340	367	447	512	522
45	204	272	302	323	347	419	475	484
50	195	259	285	305	326	389	436	443
55	186	245	268	286	305	358	396	402
60	177	230	251	266	283	327	358	363
65	168	216	234	247	261	298	323	326
70	159	202	218	229	241	271	291	294
75	150	188	202	211	221	246	262	264
80	141	175	187	195	203	224	237	239
85	133	163	173	180	187	204	215	216
90	125	151	160	166	172	186	195	196
95	118	141	148	153	158	171	178	179
100	111	131	137	142	146	157	163	164
105	104	122	128	131	135	144	150	150
110	98	114	119	122	125	133	138	138
115	92	106	110	113	116	123	127	128
120	87	99	103	106	108	114	118	118
125	82	93	96	99	101	106	109	110
130	77	87	90	92	94	99	102	102
135	73	82	85	86	88	93	95	95
140	69	77	80	81	83	87	89	89
145	65	73	75	76	78	81	83	84
150	62	69	71	72	73	76	78	79
155	59	65	67	68	69	72	74	74
160	56	61	63	64	65	68	69	70
165	53	58	60	61	62	64	66	66
170	50	55	57	58	58	61	62	62
175	48	52	54	55	55	57	59	59
180	46	50	51	52	53	55	56	56
185	44	47	49	49	50	52	53	53
190	42	45	46	47	48	49	50	50
195	38	43	44	45	45	47	48	48
200	38	41	42	43	43	45	46	46
210	35	38	39	39	40	41	42	42
220	32	35	35	36	36	37	38	38
230	30	32	33	33	33	34	35	35
240	28	30	30	31	31	32	32	32
250	26	27	28	28	29	29	30	30
λ_{L0}	38.8	32.3	30.6	29.4	28.1	25.0	22.9	22.7

λ_{L0} is the maximum slenderness ratio of the member having negligible buckling effect.

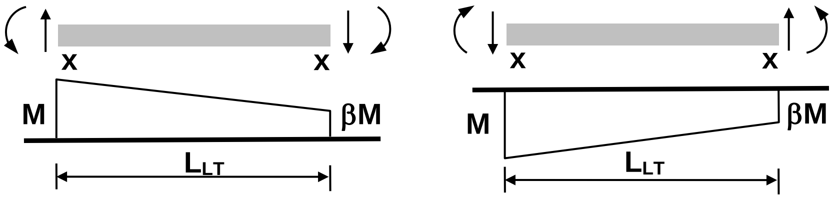
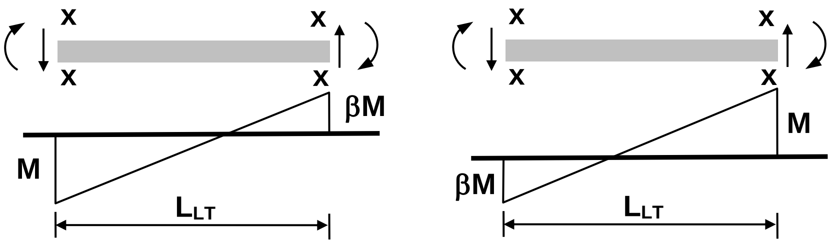
Table 8.3c - Bending strength p_b (N/mm^2) for other steel source

(ii) Bending strength for welded sections

λ_{LT}	Steel grade and design strength (N/mm²)							
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690	
	215	305	345	375	410	520	615	630
25	215	305	345	375	410	520	594	606
30	215	305	345	371	397	477	544	554
35	215	293	321	341	365	437	496	505
40	211	269	295	313	334	398	449	457
45	194	248	270	287	306	361	405	411
50	179	228	248	263	279	328	382	391
55	165	209	227	240	255	314	362	369
60	153	193	209	223	242	298	340	345
65	141	177	199	215	232	281	316	321
70	131	171	192	206	222	264	291	294
75	122	165	184	197	210	246	262	264
80	116	159	176	187	199	224	237	239
85	113	152	167	177	187	204	215	216
90	109	146	159	166	172	186	195	196
95	106	139	148	153	158	171	178	179
100	102	131	137	142	146	157	163	164
105	99	122	128	131	135	144	150	150
110	95	114	119	122	125	133	138	138
115	91	106	110	113	116	123	127	128
120	87	99	103	106	108	114	118	118
125	82	93	96	99	101	106	109	110
130	77	87	90	92	94	99	102	102
135	73	82	85	86	88	93	95	95
140	69	77	80	81	83	87	89	89
145	65	73	75	76	78	81	83	84
150	62	69	71	72	73	76	78	79
155	59	65	67	68	69	72	74	74
160	56	61	63	64	65	68	69	70
165	53	58	60	61	62	64	66	66
170	50	55	57	58	58	61	62	62
175	48	52	54	55	55	57	59	59
180	46	50	51	52	53	55	56	56
185	44	47	49	49	50	52	53	53
190	42	45	46	47	48	49	50	50
195	40	43	44	45	45	47	48	48
200	38	41	42	43	43	45	46	46
210	35	38	39	39	40	41	42	42
220	32	35	35	36	36	37	38	38
230	30	32	33	33	33	34	35	35
240	28	30	30	31	31	32	32	32
250	26	27	28	28	29	29	30	30
λ_{L0}	38.8	32.3	30.6	29.4	28.1	25.0	22.9	22.7

λ_{L0} is the maximum slenderness ratio of the member having negligible buckling effect.

Table 8.4a - Equivalent uniform moment factor m_{LT} for lateral-torsional buckling of beams under end moments and typical loads

Segment with end moment only (values of m_{LT} from the formula for the general case)	β	m_{LT}
	1.0	1.00
β positive	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
X lateral restraint	0.1	0.64
β negative	0.0	0.60
	-0.1	0.56
	-0.2	0.52
	-0.3	0.48
	-0.4	0.46
	-0.5	0.44
	-0.6	0.44
	-0.7	0.44
	-0.8	0.44
	-0.9	0.44
	-1.0	0.44

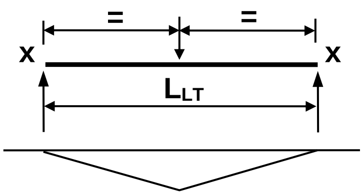
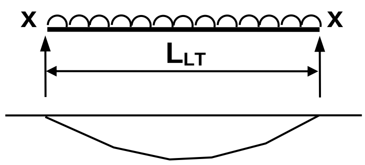
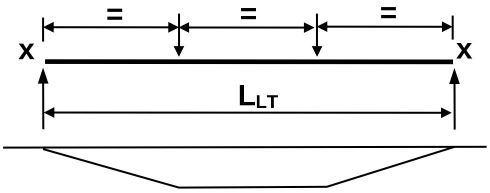
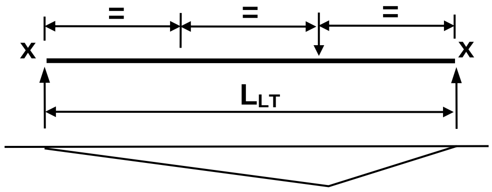
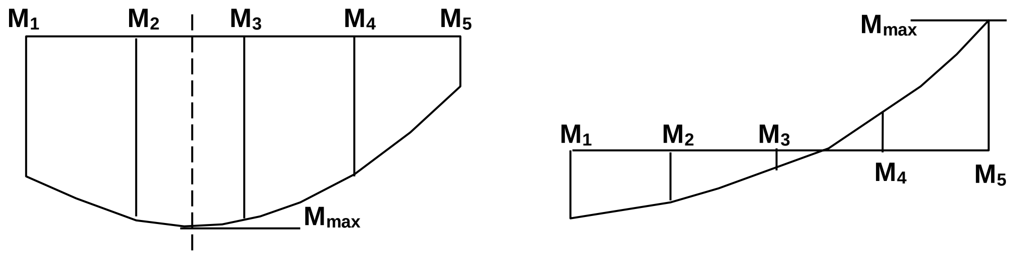
Special case (no intermediate lateral restraint)	
 **m_{LT} = 0.85**	 **m_{LT} = 0.93**
 **m_{LT} = 0.93**	 **m_{LT} = 0.74**

Table 8.4b - Equivalent uniform moment factor m_{LT} for lateral-torsional buckling of beams under non-typical loads

General case (segment between intermediate lateral restraint)
 For beams: $m_{LT} = 0.2 + \dfrac{0.15M_2 + 0.5M_3 + 0.15M_4}{M_{max}}$ but $m_{LT} \geq 0.44$ All moment are taken as positive. The moment M_2 and M_4 are the values at the quarter points, M_3 is the value at mid-length and M_{max} is the maximum moment in the segment.
For cantilevers without intermediate lateral restraint, m_{LT} = 1.00

8.3.5.3 *Equivalent slenderness for flexural-torsional buckling λ_{LT}*

The equivalent slenderness λ_{LT} should be obtained as follows.

$$\lambda_{LT} = u\, v\, \lambda \sqrt{\beta_w} \tag{8.25}$$

where

$$\lambda = \frac{L_E}{r_y} \tag{8.26}$$

L_E is the effective length for lateral-torsional buckling from clause 8.3.4

r_y is the radius of gyration about the minor y-axis

u is the buckling parameter from Appendix 8.2 or conservatively equal to 0.9 for hot-rolled sections and 1.0 for welded sections

v is the slenderness factor given by,

$$v = \frac{1}{\left(1 + 0.05(\lambda / x)^2\right)^{0.25}} \tag{8.27}$$

x is the torsional index from Appendix 8.2 or conservatively as *D/T*

D is the depth of the section

T is the thickness of the flange

β_w is the ratio defined as,

$\beta_w = 1.0$ for Class 1 plastic section and Class 2 compact section,

$$\beta_w = \frac{Z_x}{S_x} \text{ or } \frac{S_{x,eff}}{S_x} \text{ for Class 3 semi-compact sections and} \tag{8.28}$$

$$\beta_w = \frac{Z_{x,eff}}{S_x} \text{ for Class 4 slender sections} \tag{8.29}$$

Lateral-torsional buckling design of channels can be based on clause 8.3.5.3, provided that the loads pass through the shear centre and that boundary conditions apply at the shear centre.

8.4 PLATE GIRDERS

8.4.1 Design strength

For the simple design of plate girders, the webs are generally assumed to take shear, transverse and axial forces and flanges are assumed to resist moment. The material design strength in section 3 should be used when the web and the flanges are of the same steel grade.

When web and flanges are of different steel grades and the web design strength p_{yw} is greater than the flange design strength p_{yf}, p_{yf} should be used for moment, axial force and shear checking. When p_{yw} is less than p_{yf} , the steel grades for flange (p_{yf}) should be used for axial force and moment checks and the strength of the web (p_{yw}) should be used for shear and transverse force checks.

8.4.2 Minimum web thickness for serviceability

For serviceability requirements, the d/t ratio of the web in a plate girder should satisfy the following conditions:

For webs without intermediate stiffeners, $t \geq d/250$; (8.30)

For webs with transverse stiffeners only,

(a) When stiffener spacing $a > d$, $t \geq d/250$ (8.31)

(b) When stiffener spacing $a \leq d$, $t \geq d/250\sqrt{\dfrac{a}{d}}$ (8.32)

in which,

a is the spacing of stiffeners;

d is the depth of web.

Reference should be made to specialist literature for webs having both longitudinal and transverse stiffeners.

8.4.3 Minimum web thickness to avoid compression flange buckling

To prevent compression flange buckling into the web, the following conditions need to be satisfied:-

(a) For webs without intermediate transverse stiffeners or with stiffeners at spacing

$a > 1.5d$, $t \geq d/250 \times \dfrac{p_{yf}}{345}$ (8.33)

(b) For webs with intermediate transverse stiffeners at spacing $a \leq 1.5d$,

$t \geq d/250\sqrt{\dfrac{p_{yf}}{455}}$ (8.34)

where p_{yf} is the design strength of the compression flange.

8.4.4 Moment capacity of restrained girders

8.4.4.1 Web not susceptible to shear buckling

If the web depth-to-thickness ratio $d/t \leq 62\varepsilon$, shear buckling need not be considered and the methods for beam design in clause 8.2 should be followed to determine the moment capacity.

8.4.4.2 Web susceptible to shear buckling

If the web depth-to-thickness ratio $d/t > 70\varepsilon$ for hot-rolled section or $d/t > 62\varepsilon$ for welded sections, shear buckling should be considered and the following methods should be used to determine the moment resistance:

(a) Low shear load:
The low shear load condition is assumed when $V \leq 0.6\,V_w$. The girder should be designed to clause 8.2 as for rolled section beams.

(b) High shear load:
The high shear load condition is assumed when $V > 0.6\,V_w$. Provided that the flanges are not Class 4 slender, the moment capacity of the girder is equal to the moment capacity provided by flanges alone, i.e.

$$M_p = p_{yf}\ B\,T\ (D - T) \tag{8.35}$$

where V_w is the shear buckling resistance determined from clause 8.4.6 ignoring contribution from flanges.

Alternative methods given in other codes can be used when the contribution of webs to moment capacity is allowed for.

8.4.5 Effects of axial force

Additional stress caused by axial force should be added to the bending stress in flanges calculated in clause 8.4.4.2 and the resultant stress should not be larger than p_{yf}.

8.4.6 Shear buckling resistance

If the web depth-to-thickness ratio $d/t > 70\varepsilon$ for hot-rolled section or $d/t > 62\varepsilon$ for welded sections, shear buckling should be checked.

The shear buckling resistance V_w of a web with or without web stiffeners should be taken as,

$$V_w = d\ t\ q_w \tag{8.36}$$

in which

d is the depth of the web,

t is the web thickness,

q_w is the shear buckling strength of the web depending on d/t and a/d ratios and obtained from Tables 8.5a - 8.5l or Appendix 8.3.

8.4.7 Intermediate transverse web stiffeners

To resist shear buckling in clause 8.4.6, intermediate transverse stiffeners may be used. Intermediate transverse stiffeners may be provided on either one or both sides of the web.

8.4.7.1 *Spacing*
Where intermediate transverse web stiffeners are provided, their spacing should conform to clauses 8.4.2 and 8.4.3.

8.4.7.2 *Outstand of stiffeners*
The outstand of the stiffeners should conform to clause 8.4.10.1.

8.4.7.3 *Minimum stiffness*
Intermediate transverse web stiffeners not subjected to external loads or moments should have a second moment of area I_s about the centreline of the web not less than I_s given by:

for $a/d \geq \sqrt{2}$: $I_s = 0.75\ d\ t_{\min}^3$ (8.37)

for $a/d < \sqrt{2}$: $I_s = 1.5\left(d/a\right)^2 d\ t_{\min}^3$ (8.38)

where

a is the actual stiffener spacing;

d is the depth of the web;

$t_{\min}$ is the minimum required web thickness determined from clauses 8.4.2 and 8.4.3 for the actual stiffener spacing a.

8.4.7.4 *Additional stiffness for external loading*

If an intermediate transverse web stiffener is subjected to externally applied forces, the required value of I_s given in clause 8.4.7.3 should be increased by adding I_{ext} as follows:

(a) for transverse forces effectively applied in line with the web:

$$I_{ext} = 0$$

(b) for transverse forces applied eccentric to the web:

$$I_{ext} = F_x e_x D^2 / Et \qquad (8.39)$$

(c) for lateral forces, deemed to be applied at the level of the compression flange of the girder:

$$I_{ext} = 2F_h D^3 / Et \qquad (8.40)$$

where

D is the overall depth of the section;

E is the modulus of elasticity;

e_x is the eccentricity of the transverse force from the centreline of the web;

F_h is the external lateral force;

F_x is the external transverse force;

t is the web thickness.

8.4.7.5 *Buckling resistance*

Intermediate transverse web stiffeners not subjected to external forces or moments should satisfy the condition:

$$F_q \leq P_q \qquad (8.41)$$

in which F_q is the larger value, considering the two web panels on each side of the stiffener, of the compressive axial force given by:

$$F_q = V - V_{cr} \qquad (8.42)$$

where

P_q is the buckling resistance of the intermediate web stiffener determined from clause 8.4.10.6.2 except that L_E is taken as $0.7L$ and checking for bearing is not required;

V is the shear in a web panel adjacent to the stiffener;

V_{cr} is the critical shear buckling resistance (see clause 8.4.8b) of the same web panel.

Intermediate transverse web stiffeners subjected to external forces or moments should meet the conditions for load carrying web stiffeners given in clause 8.4.10.6.2. In addition, they should also satisfy the following:

- if $F_q > F_x$:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \leq 1 \qquad (8.43)$$

- if $F_q \leq F_x$:

$$\frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \leq 1 \qquad (8.44)$$

in which

$$M_s = F_x e_x + F_h D \qquad (8.45)$$

where

F_h is the external lateral force, if any;

F_x is the external transverse force;

M_{ys} is the moment capacity of the stiffener calculated by its section modulus;

P_x is the buckling resistance of a load carrying stiffener, see clause 8.4.10.6.2.

8.4.7.6 *Connection to web of intermediate stiffeners*

Intermediate transverse web stiffeners that are not subjected to external forces or moments should be connected to the web to withstand the shear between each component and the web (in kN per millimetre run) with length of not less than:-

$$t^2/(5b_s) \tag{8.46}$$

where

b_s is the outstand of the stiffener (in mm);

t is the web thickness (in mm).

If the stiffeners are subjected to external forces or moments, the resulting shear between the web and the stiffener should be added to the above value.

Intermediate transverse web stiffeners that are not subjected to external forces or moments should extend to the compression flange, but need not be connected to it. Intermediate transverse web stiffeners that are not subjected to external forces or moments may terminate clear of the tension flange. In such cases, the welds connecting the stiffener to the web should terminate not more than $4t$ clear of the tension flange.

Table 8.5a - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

1) Grade S275 steel, web thickness ≤ 16mm – design strength p_y = 275N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
60	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
65	165	165	165	165	165	165	165	165	165	165	165	165	165	165	161
70	165	165	165	165	165	165	165	165	165	165	164	162	160	158	155
75	165	165	165	165	165	165	165	165	163	160	158	156	154	152	148
80	165	165	165	165	165	165	165	162	157	154	152	150	147	146	142
85	165	165	165	165	165	165	162	156	152	149	146	144	141	139	135
90	165	165	165	165	165	163	157	151	146	143	140	138	135	133	128
95	165	165	165	165	165	158	152	146	141	137	134	132	129	127	122
100	165	165	165	165	160	154	147	140	135	131	128	126	123	120	116
105	165	165	165	164	156	149	142	135	129	125	122	120	117	115	110
110	165	165	165	160	152	144	137	130	124	120	117	115	111	109	105
115	165	165	165	156	147	139	132	124	118	114	112	110	106	105	101
120	165	165	163	152	143	135	128	119	113	110	107	105	102	100	96
125	165	165	159	148	139	130	123	114	109	105	103	101	98	96	92
130	165	165	156	145	134	125	118	110	105	101	99	97	94	93	89
135	165	165	152	141	130	121	113	106	101	97	95	93	91	89	86
140	165	162	149	137	126	116	109	102	97	94	92	90	87	86	83
145	165	159	145	133	121	112	105	98	94	91	88	87	84	83	80
150	165	156	142	129	117	109	102	95	91	88	86	84	82	80	77
155	165	153	138	125	113	105	99	92	88	85	83	81	79	78	75
160	165	150	135	121	110	102	96	89	85	82	80	79	76	75	72
165	165	147	131	117	107	99	93	86	82	80	78	76	74	73	70
170	162	144	128	114	103	96	90	84	80	77	75	74	72	71	68
175	160	141	124	110	100	93	87	81	78	75	73	72	70	69	66
180	157	138	121	107	98	90	85	79	76	73	71	70	68	67	64
185	155	135	117	104	95	88	83	77	73	71	69	68	66	65	62
190	152	132	114	102	93	86	80	75	72	69	68	66	64	63	61
195	150	129	111	99	90	84	78	73	70	67	66	65	63	62	59
200	147	126	109	97	88	81	76	71	68	66	64	63	61	60	58
205	145	123	106	94	86	79	75	70	66	64	63	61	60	59	56
210	142	120	103	92	84	78	73	68	65	63	61	60	58	57	55
215	140	117	101	90	82	76	71	66	63	61	60	59	57	56	54
220	137	115	99	88	80	74	70	65	62	60	58	57	56	55	53
225	135	112	97	86	78	72	68	63	60	58	57	56	54	53	51
230	132	110	94	84	76	71	66	62	59	57	56	55	53	52	50
235	130	107	92	82	75	69	65	61	58	56	55	54	52	51	49
240	128	105	90	80	73	68	64	59	57	55	53	52	51	50	48
245	125	103	89	79	72	66	62	58	55	54	52	51	50	49	47
250	123	101	87	77	70	65	61	57	54	53	51	50	49	48	46

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5b - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

2) Grade S275 steel, web thickness >16mm ≤ 40mm – design strength p_y = 265N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	159	159	159	159	159	159	159	159	159	159	159	159	159	159	159
60	159	159	159	159	159	159	159	159	159	159	159	159	159	159	159
65	159	159	159	159	159	159	159	159	159	159	159	159	159	159	157
70	159	159	159	159	159	159	159	159	159	159	159	158	156	154	151
75	159	159	159	159	159	159	159	159	159	156	154	152	150	148	145
80	159	159	159	159	159	159	159	157	153	150	148	147	144	142	138
85	159	159	159	159	159	159	158	152	148	145	143	141	138	136	132
90	159	159	159	159	159	158	153	147	143	139	137	135	132	130	126
95	159	159	159	159	159	154	149	142	137	134	131	129	126	124	120
100	159	159	159	159	156	150	144	137	132	128	126	124	120	118	113
105	159	159	159	159	152	145	139	132	127	123	120	118	114	112	108
110	159	159	159	156	148	141	134	127	121	117	114	112	109	107	103
115	159	159	159	152	144	136	130	122	116	112	110	108	104	103	99
120	159	159	158	149	140	132	125	117	111	108	105	103	100	98	95
125	159	159	155	145	136	127	120	112	107	103	101	99	96	94	91
130	159	159	152	141	132	123	116	108	103	99	97	95	92	91	87
135	159	159	148	137	127	119	111	104	99	96	93	92	89	87	84
140	159	158	145	134	123	114	107	100	95	92	90	88	86	84	81
145	159	155	142	130	119	110	104	97	92	89	87	85	83	81	78
150	159	152	139	126	115	107	100	93	89	86	84	82	80	79	76
155	159	149	135	122	111	103	97	90	86	83	81	80	77	76	73
160	159	147	132	119	108	100	94	87	83	81	79	77	75	74	71
165	159	144	129	115	105	97	91	85	81	78	76	75	73	72	69
170	158	141	125	112	102	94	88	82	78	76	74	73	71	69	67
175	156	138	122	108	99	91	86	80	76	74	72	71	69	67	65
180	153	135	119	105	96	89	83	78	74	72	70	69	67	66	63
185	151	132	115	102	93	86	81	76	72	70	68	67	65	64	61
190	149	129	112	100	91	84	79	74	70	68	66	65	63	62	60
195	146	127	109	97	89	82	77	72	68	66	65	63	62	61	58
200	144	124	107	95	86	80	75	70	67	65	63	62	60	59	57
205	141	121	104	92	84	78	73	68	65	63	61	60	59	58	55
210	139	118	102	90	82	76	71	67	64	61	60	59	57	56	54
215	137	115	99	88	80	74	70	65	62	60	59	58	56	55	53
220	134	112	97	86	78	73	68	64	61	59	57	56	55	54	52
225	132	110	95	84	77	71	67	62	59	57	56	55	53	52	50
230	130	108	93	82	75	70	65	61	58	56	55	54	52	51	49
235	127	105	91	81	73	68	64	60	57	55	54	53	51	50	48
240	125	103	89	79	72	67	63	58	56	54	52	52	50	49	47
245	123	101	87	77	70	65	61	57	54	53	51	50	49	48	46
250	120	99	85	76	69	64	60	56	53	52	50	49	48	47	45

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5c - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

3) Grade S355 steel, web thickness ≤ 16mm – design strength p_y = 355N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	213	213	213	213	213	213	213	213	213	213	213	213	213	213	213
60	213	213	213	213	213	213	213	213	213	213	213	213	209	208	203
65	213	213	213	213	213	213	213	213	212	208	206	204	200	198	193
70	213	213	213	213	213	213	213	209	204	200	197	195	191	189	184
75	213	213	213	213	213	213	209	201	196	191	188	186	182	180	174
80	213	213	213	213	213	209	202	194	187	183	180	177	173	170	164
85	213	213	213	213	211	202	195	186	179	174	171	168	164	161	155
90	213	213	213	213	205	195	187	178	171	166	162	159	155	152	146
95	213	213	213	209	198	189	180	170	163	157	153	151	146	144	138
100	213	213	213	203	192	182	173	162	155	149	146	143	139	137	131
105	213	213	211	197	185	175	165	154	147	142	139	136	132	130	125
110	213	213	206	192	179	168	158	147	140	136	133	130	126	124	119
115	213	213	201	186	173	161	151	141	134	130	127	124	121	119	114
120	213	213	195	180	166	154	145	135	129	124	121	119	116	114	109
125	213	208	190	174	160	148	139	130	124	119	117	115	111	109	105
130	213	204	185	169	154	142	134	125	119	115	112	110	107	105	101
135	213	199	180	163	148	137	129	120	114	111	108	106	103	101	97
140	213	195	175	157	143	132	124	116	110	107	104	102	99	98	94
145	213	190	170	151	138	128	120	112	107	103	101	99	96	94	91
150	209	186	165	146	133	123	116	108	103	100	97	95	93	91	88
155	206	181	159	142	129	119	112	105	100	96	94	92	90	88	85
160	202	177	154	137	125	116	109	101	97	93	91	89	87	85	82
165	198	173	150	133	121	112	105	98	94	91	88	87	84	83	80
170	195	168	145	129	117	109	102	95	91	88	86	84	82	80	77
175	191	164	141	125	114	106	99	93	88	85	83	82	79	78	75
180	187	159	137	122	111	103	97	90	86	83	81	80	77	76	73
185	184	155	133	119	108	100	94	88	83	81	79	77	75	74	71
190	180	151	130	115	105	97	91	85	81	79	77	75	73	72	69
195	176	147	127	113	102	95	89	83	79	77	75	73	71	70	67
200	173	143	123	110	100	93	87	81	77	75	73	72	70	68	66
205	169	140	120	107	97	90	85	79	75	73	71	70	68	67	64
210	165	136	117	104	95	88	83	77	74	71	69	68	66	65	63
215	162	133	115	102	93	86	81	75	72	69	68	67	65	64	61
220	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60
225	155	127	110	98	89	82	77	72	69	66	65	64	62	61	58
230	151	124	107	95	87	80	76	70	67	65	63	62	60	59	57
235	148	122	105	93	85	79	74	69	66	64	62	61	59	58	56
240	145	119	103	91	83	77	72	67	64	62	61	60	58	57	55
245	142	117	101	90	82	76	71	66	63	61	59	58	57	56	54
250	139	115	99	88	80	74	70	65	62	60	58	57	56	55	53

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5d - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

4) Grade S355 steel, web thickness >16mm ≤ 40mm – design strength p_y = 345N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	207	207	207	207	207	207	207	207	207	207	207	207	207	207	207
60	207	207	207	207	207	207	207	207	207	207	207	207	205	203	199
65	207	207	207	207	207	207	207	207	207	204	201	200	196	194	190
70	207	207	207	207	207	207	207	205	200	196	193	191	187	185	180
75	207	207	207	207	207	207	205	197	192	188	185	183	179	177	171
80	207	207	207	207	207	205	198	190	184	180	176	174	170	168	162
85	207	207	207	207	206	198	191	182	176	172	168	165	161	159	153
90	207	207	207	207	200	192	184	175	168	163	160	157	152	150	144
95	207	207	207	205	194	185	177	167	160	155	151	149	144	142	136
100	207	207	207	199	188	178	170	160	152	147	144	141	137	135	129
105	207	207	206	194	182	172	163	152	145	140	137	134	131	128	123
110	207	207	202	188	176	165	156	145	138	134	131	128	125	123	118
115	207	207	197	182	170	159	149	139	132	128	125	123	119	117	113
120	207	207	192	177	164	152	143	133	127	123	120	118	114	112	108
125	207	204	187	171	158	146	137	128	122	118	115	113	110	108	104
130	207	200	182	166	152	140	132	123	117	113	111	109	105	104	100
135	207	195	177	160	146	135	127	118	113	109	106	105	102	100	96
140	207	191	172	155	141	130	122	114	109	105	103	101	98	96	92
145	207	187	167	149	136	126	118	110	105	102	99	97	95	93	89
150	205	183	162	144	131	122	114	106	102	98	96	94	91	90	86
155	201	178	157	140	127	118	111	103	98	95	93	91	88	87	84
160	198	174	152	135	123	114	107	100	95	92	90	88	86	84	81
165	194	170	147	131	119	111	104	97	92	89	87	86	83	82	78
170	191	165	143	127	116	107	101	94	90	87	85	83	81	79	78
175	187	161	139	124	113	104	98	91	87	84	82	81	78	77	74
180	184	157	135	120	109	101	95	89	85	82	80	78	76	75	72
185	180	153	131	117	106	99	93	86	82	80	78	76	74	73	70
190	177	149	128	114	104	96	90	84	80	77	76	74	72	71	68
195	173	145	125	111	101	94	88	82	78	76	74	72	70	69	66
200	170	141	122	108	98	91	86	80	76	74	72	71	69	67	65
205	166	138	119	106	96	89	84	78	74	72	70	69	67	66	63
210	163	134	116	103	94	87	82	76	73	70	68	67	65	64	62
215	159	131	113	101	92	85	80	74	71	68	67	66	64	63	60
220	156	128	111	98	90	83	78	73	69	67	65	64	62	61	59
225	152	125	108	96	88	81	76	71	68	65	64	63	61	60	58
230	149	123	106	94	86	79	74	69	66	64	62	61	60	59	56
235	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55
240	143	118	101	90	82	76	71	67	63	61	60	59	57	56	54
245	140	115	99	88	80	74	70	65	62	60	59	58	56	55	53
250	137	113	97	87	79	73	69	64	61	59	57	56	55	54	52

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5e - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

5) Grade S460 steel, web thickness $\leq$ 16mm – design strength p_y = 460N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	276	276	276	276	276	276	276	276	276	275	272	269	265	263	256
60	276	276	276	276	276	276	276	275	268	263	259	256	252	249	242
65	276	276	276	276	276	276	273	263	256	250	246	243	238	235	228
70	276	276	276	276	276	272	262	251	244	238	233	230	225	222	214
75	276	276	276	276	273	261	252	240	231	225	221	217	211	208	199
80	276	276	276	276	263	251	241	228	219	213	208	204	198	195	187
85	276	276	276	268	254	241	230	217	207	200	195	192	186	183	176
90	276	276	276	260	245	231	219	205	195	189	184	181	176	173	166
95	276	276	269	251	235	221	208	194	185	179	175	172	167	164	157
100	276	276	261	243	226	211	198	184	176	170	166	163	158	156	150
105	276	276	254	234	216	201	188	176	167	162	158	155	151	148	142
110	276	269	246	226	207	191	180	168	160	155	151	148	144	141	136
115	276	263	239	217	198	183	172	160	153	148	144	142	138	135	130
120	276	256	231	208	189	176	165	154	147	142	138	136	132	130	125
125	276	250	224	200	182	169	158	148	141	136	133	130	127	124	120
130	273	243	216	192	175	162	152	142	135	131	128	125	122	120	115
135	268	237	208	185	168	156	147	137	130	126	123	121	117	115	111
140	262	230	201	178	162	150	141	132	126	121	119	116	113	111	107
145	257	224	194	172	157	145	136	127	121	117	114	112	109	107	103
150	252	217	187	167	152	140	132	123	117	113	111	109	105	104	100
155	246	210	181	161	147	136	128	119	113	110	107	105	102	100	96
160	241	204	176	156	142	132	124	115	110	106	104	102	99	97	93
165	235	197	170	151	138	128	120	112	107	103	101	99	96	94	91
170	230	192	165	147	134	124	116	108	103	100	98	96	93	92	88
175	225	186	160	143	130	120	113	105	100	97	95	93	90	89	85
180	219	181	156	139	126	117	110	102	98	94	92	91	88	86	83
185	214	176	152	135	123	114	107	100	95	92	90	88	86	84	81
190	208	172	148	131	120	111	104	97	93	89	87	86	83	82	79
195	203	167	144	128	117	108	101	95	90	87	85	84	81	80	77
200	198	163	140	125	114	105	99	92	88	85	83	81	79	78	75
205	193	159	137	122	111	103	96	90	86	83	81	79	77	76	73
210	188	155	134	119	108	100	94	88	84	81	79	78	75	74	71
215	184	152	131	116	106	98	92	86	82	79	77	76	74	72	70
220	180	148	128	114	103	96	90	84	80	77	75	74	72	71	68
225	176	145	125	111	101	94	88	82	78	76	74	72	70	69	66
230	172	142	122	109	99	92	86	80	76	74	72	71	69	68	65
235	168	139	120	106	97	90	84	78	75	72	71	69	67	66	64
240	165	136	117	104	95	88	82	77	73	71	69	68	66	65	62
245	161	133	115	102	93	86	81	75	72	69	68	67	65	64	61
250	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5f - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

6) Grade S460 steel, web thickness >16mm ≤ 40mm – design strength p_y = 440N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	264	264	264	264	264	264	264	264	264	264	263	261	257	254	249
60	264	264	264	264	264	264	264	264	259	255	251	248	244	242	235
65	264	264	264	264	264	264	264	255	248	243	239	236	231	229	222
70	264	264	264	264	264	263	254	244	237	231	227	224	219	216	209
75	264	264	264	264	264	253	244	233	225	219	215	212	206	203	195
80	264	264	264	264	255	244	234	222	214	208	203	199	194	190	183
85	264	264	264	260	246	234	224	211	202	196	191	187	182	179	172
90	264	264	264	252	237	225	214	201	191	185	180	177	172	169	162
95	264	264	260	244	229	215	204	190	181	175	171	168	163	160	154
100	264	264	253	236	220	206	194	180	172	166	162	159	155	152	146
105	264	264	246	228	211	196	184	172	164	158	155	152	147	145	139
110	264	261	239	220	202	187	176	164	156	151	148	145	141	138	133
115	264	255	232	212	194	179	168	157	150	145	141	139	135	132	127
120	264	248	225	204	185	172	161	150	143	139	135	133	129	127	122
125	264	242	218	196	178	165	155	144	138	133	130	127	124	122	117
130	264	236	211	188	171	158	149	139	132	128	125	123	119	117	112
135	259	230	204	181	165	153	143	134	127	123	120	118	115	113	108
140	254	224	196	174	159	147	138	129	123	119	116	114	111	109	104
145	249	218	189	168	153	142	133	124	119	115	112	110	107	105	101
150	244	212	183	163	148	137	129	120	115	111	108	106	103	101	97
155	239	206	177	158	143	133	125	116	111	107	105	103	100	98	94
160	234	199	172	153	139	129	121	113	107	104	101	100	97	95	91
165	229	193	166	148	135	125	117	109	104	101	98	97	94	92	89
170	224	187	162	144	131	121	114	106	101	98	95	94	91	90	86
175	219	182	157	140	127	118	111	103	98	95	93	91	88	87	84
180	214	177	153	136	124	114	107	100	96	92	90	89	86	85	81
185	209	172	148	132	120	111	105	97	93	90	88	86	84	82	79
190	204	168	145	129	117	108	102	95	90	88	85	84	81	80	77
195	199	163	141	125	114	106	99	92	88	85	83	82	79	78	75
200	194	159	137	122	111	103	97	90	86	83	81	80	77	76	73
205	189	155	134	119	108	100	94	88	84	81	79	78	75	74	71
210	184	152	131	116	106	98	92	86	82	79	77	76	74	72	70
215	180	148	128	114	103	96	90	84	80	77	75	74	72	71	68
220	176	145	125	111	101	94	88	82	78	76	74	72	70	69	66
225	172	142	122	109	99	92	86	80	76	74	72	71	69	68	65
230	168	139	119	106	97	90	84	78	75	72	71	69	67	66	64
235	165	136	117	104	95	88	82	77	73	71	69	68	66	65	62
240	161	133	114	102	93	86	81	75	72	69	68	66	64	63	61
245	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60
250	155	127	110	98	89	82	77	72	69	67	65	64	62	61	58

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5g - Shear buckling strength q_w (N/mm^2) of a web (for $t \le 50mm$)

7) Grade S550 steel, web thickness ≤ 50mm – design strength p_y = 550N/mm²															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	330	330	330	330	330	330	330	328	320	314	309	306	300	297	289
60	330	330	330	330	330	330	325	313	304	297	293	289	283	279	270
65	330	330	330	330	330	322	311	298	288	281	276	272	265	261	252
70	330	330	330	330	323	309	297	283	272	265	259	255	248	243	234
75	330	330	330	327	310	296	283	267	256	248	242	238	231	227	218
80	330	330	330	316	298	282	269	252	240	232	227	223	216	213	204
85	330	330	325	304	286	269	255	237	226	219	213	210	204	200	192
90	330	330	316	293	274	256	240	224	214	207	202	198	192	189	182
95	330	330	306	282	261	243	228	212	202	196	191	188	182	179	172
100	330	323	296	271	249	230	216	202	192	186	181	178	173	170	163
105	330	315	286	260	237	219	206	192	183	177	173	170	165	162	156
110	330	306	276	249	226	209	197	183	175	169	165	162	157	155	149
115	330	297	266	238	216	200	188	175	167	162	158	155	150	148	142
120	325	289	256	228	207	192	180	168	160	155	151	148	144	142	136
125	318	280	246	219	199	184	173	161	154	149	145	143	138	136	131
130	311	272	236	210	191	177	166	155	148	143	140	137	133	131	126
135	304	263	227	202	184	171	160	149	142	138	134	132	128	126	121
140	297	255	219	195	178	165	154	144	137	133	130	127	124	122	117
145	290	246	212	188	171	159	149	139	133	128	125	123	119	117	113
150	283	238	205	182	166	154	144	134	128	124	121	119	115	113	109
155	276	230	198	176	160	149	140	130	124	120	117	115	112	110	105
160	269	223	192	171	155	144	135	126	120	116	113	111	108	106	102
165	262	216	186	166	151	140	131	122	117	113	110	108	105	103	99
170	255	210	181	161	146	135	127	119	113	109	107	105	102	100	96
175	248	204	175	156	142	132	124	115	110	106	104	102	99	97	93
180	240	198	171	152	138	128	120	112	107	103	101	99	96	95	91
185	234	193	166	148	134	124	117	109	104	100	98	96	94	92	88
190	228	188	162	144	131	121	114	106	101	98	95	94	91	90	86
195	222	183	157	140	127	118	111	103	99	95	93	91	89	87	84
200	216	178	154	137	124	115	108	101	96	93	91	89	87	85	82
205	211	174	150	133	121	112	106	98	94	91	89	87	84	83	80
210	206	170	146	130	118	110	103	96	92	89	86	85	82	81	78
215	201	166	143	127	116	107	101	94	89	86	84	83	80	79	76
220	197	162	140	124	113	105	98	92	87	85	82	81	79	77	74
225	192	158	136	121	110	102	96	90	85	83	81	79	77	76	73
230	188	155	134	119	108	100	94	88	84	81	79	77	75	74	71
235	184	152	131	116	106	98	92	86	82	79	77	76	74	72	70
240	180	148	128	114	104	96	90	84	80	77	76	74	72	71	68
245	177	145	125	111	101	94	88	82	78	76	74	73	71	69	67
250	173	143	123	109	99	92	87	81	77	74	73	71	69	68	65

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5h - Shear buckling strength q_w (N/mm^2) of a web (for 50mm < $t \leq$ 100mm)

8) Grade S550 steel, web thickness >50mm ≤ 100mm – design strength p_y = 530N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	318	318	318	318	318	318	318	318	311	306	301	298	293	290	282
60	318	318	318	318	318	318	316	305	296	290	286	282	276	273	265
65	318	318	318	318	318	313	303	290	281	275	270	266	260	256	247
70	318	318	318	318	314	301	290	276	266	259	254	250	243	239	229
75	318	318	318	318	302	288	276	262	251	244	238	233	226	223	214
80	318	318	318	307	291	276	263	247	236	228	223	219	212	209	201
85	318	318	317	297	279	263	250	233	222	215	210	206	200	197	189
90	318	318	307	286	267	251	236	220	210	203	198	194	189	186	178
95	318	318	298	276	256	238	223	208	199	192	187	184	179	176	169
100	318	314	288	265	244	226	212	198	189	182	178	175	170	167	160
105	318	306	279	255	233	215	202	188	180	174	170	167	162	159	153
110	318	298	270	244	222	206	193	180	172	166	162	159	154	152	146
115	318	290	260	233	212	197	185	172	164	159	155	152	148	145	140
120	316	282	251	223	203	188	177	165	157	152	148	146	142	139	134
125	310	274	241	214	195	181	170	158	151	146	142	140	136	134	128
130	303	266	232	206	188	174	163	152	145	140	137	135	131	128	123
135	296	258	223	199	181	167	157	147	140	135	132	130	126	124	119
140	290	250	215	192	174	161	152	141	135	130	127	125	121	119	115
145	283	242	208	185	168	156	146	136	130	126	123	121	117	115	111
150	276	233	201	179	163	151	142	132	126	122	119	117	113	111	107
155	270	226	194	173	157	146	137	128	122	118	115	113	110	108	104
160	263	219	188	168	153	141	133	124	118	114	111	109	106	104	100
165	256	212	183	162	148	137	129	120	114	111	108	106	103	101	97
170	250	206	177	158	144	133	125	116	111	107	105	103	100	98	94
175	243	200	172	153	139	129	121	113	108	104	102	100	97	95	92
180	236	194	167	149	136	126	118	110	105	101	99	97	94	93	89
185	230	189	163	145	132	122	115	107	102	99	96	95	92	90	87
190	223	184	159	141	128	119	112	104	99	96	94	92	89	88	84
195	218	179	155	137	125	116	109	101	97	94	91	90	87	86	82
200	212	175	151	134	122	113	106	99	94	91	89	87	85	84	80
205	207	171	147	131	119	110	104	97	92	89	87	85	83	81	78
210	202	167	144	128	116	108	101	94	90	87	85	83	81	80	76
215	197	163	140	125	114	105	99	92	88	85	83	81	79	78	75
220	193	159	137	122	111	103	97	90	86	83	81	79	77	76	73
225	189	155	134	119	108	100	94	88	84	81	79	78	75	74	71
230	185	152	131	117	106	98	92	86	82	79	77	76	74	73	70
235	181	149	128	114	104	96	90	84	80	78	76	74	72	71	68
240	177	146	126	112	102	94	88	82	79	76	74	73	71	70	67
245	173	143	123	109	100	92	87	81	77	74	73	71	69	68	66
250	170	140	121	107	98	90	85	79	75	73	71	70	68	67	64

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5i - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 50mm)

9) Grade S690 steel, web thickness ≤ 50mm – design strength p_y = 690N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	414	414	414	414	414	414	402	386	375	367	360	356	348	343	332
60	414	414	414	414	413	397	382	365	353	344	337	331	323	318	306
65	414	414	414	414	396	378	363	344	330	320	313	307	298	293	282
70	414	414	414	400	379	360	343	323	308	297	290	285	277	272	262
75	414	414	411	385	362	341	323	301	287	278	271	266	258	254	244
80	414	414	397	369	344	322	303	282	269	260	254	249	242	238	229
85	414	414	383	353	327	304	285	266	253	245	239	235	228	224	215
90	414	404	369	338	310	287	269	251	239	231	226	222	215	212	203
95	414	392	355	322	293	272	255	238	227	219	214	210	204	201	193
100	414	380	341	306	278	258	242	226	215	208	203	200	194	191	183
105	412	368	327	291	265	246	231	215	205	198	194	190	185	182	174
110	402	356	313	278	253	235	220	205	196	189	185	181	176	173	166
115	392	344	299	266	242	224	211	196	187	181	177	174	169	166	159
120	382	331	287	255	232	215	202	188	179	174	169	166	161	159	153
125	372	319	275	245	223	206	194	181	172	167	163	160	155	152	146
130	363	307	265	235	214	198	186	174	166	160	156	154	149	147	141
135	353	296	255	227	206	191	179	167	160	154	151	148	144	141	136
140	343	285	246	219	199	184	173	161	154	149	145	143	138	136	131
145	333	275	237	211	192	178	167	156	149	144	140	138	134	131	126
150	323	266	229	204	186	172	161	151	144	139	135	133	129	127	122
155	313	257	222	197	180	166	156	146	139	134	131	129	125	123	118
160	303	249	215	191	174	161	151	141	135	130	127	125	121	119	114
165	294	242	208	185	169	156	147	137	131	126	123	121	117	116	111
170	285	235	202	180	164	152	142	133	127	122	120	117	114	112	108
175	277	228	197	175	159	147	138	129	123	119	116	114	111	109	105
180	269	222	191	170	155	143	135	125	120	116	113	111	108	106	102
185	262	216	186	165	151	139	131	122	116	113	110	108	105	103	99
190	255	210	181	161	147	136	127	119	113	110	107	105	102	100	96
195	248	205	176	157	143	132	124	116	110	107	104	102	99	98	94
200	242	200	172	153	139	129	121	113	108	104	102	100	97	95	92
205	236	195	168	149	136	126	118	110	105	102	99	97	95	93	89
210	231	190	164	146	133	123	115	108	103	99	97	95	92	91	87
215	225	186	160	142	130	120	113	105	100	97	95	93	90	89	85
220	220	181	156	139	127	117	110	103	98	95	92	91	88	87	83
225	215	177	153	136	124	115	108	100	96	93	90	89	86	85	81
230	211	174	150	133	121	112	105	98	94	91	88	87	84	83	80
235	206	170	146	130	118	110	103	96	92	89	86	85	82	81	78
240	202	166	143	127	116	107	101	94	90	87	85	83	81	79	76
245	198	163	140	125	114	105	99	92	88	85	83	81	79	78	75
250	194	160	138	122	111	103	97	90	86	83	81	80	78	76	73

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5j - Shear buckling strength q_w (N/mm^2) of a web (for 50mm < $t \leq$ 100mm)

10) Grade S690 steel, web thickness >50mm ≤ 100mm – design strength p_y = 650N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	390	390	390	390	390	390	385	370	360	352	347	342	335	331	321
60	390	390	390	390	390	380	367	351	340	331	325	320	312	308	297
65	390	390	390	390	379	363	349	332	319	310	303	298	290	285	273
70	390	390	390	383	364	346	331	312	299	289	282	277	269	264	254
75	390	390	390	369	348	329	312	293	279	269	263	258	251	247	237
80	390	390	380	355	332	312	294	274	261	253	247	242	235	231	222
85	390	390	367	340	316	295	277	258	246	238	232	228	221	218	209
90	390	386	355	326	301	278	261	244	232	225	219	215	209	206	197
95	390	375	342	312	285	264	247	231	220	213	208	204	198	195	187
100	390	364	329	297	270	250	235	219	209	202	197	194	188	185	178
105	390	353	316	283	257	238	224	209	199	192	188	184	179	176	169
110	385	342	303	270	246	228	214	199	190	184	179	176	171	168	162
115	376	331	291	258	235	218	204	191	182	176	172	168	164	161	155
120	367	320	278	247	225	209	196	183	174	168	164	161	157	154	148
125	358	309	267	238	216	200	188	175	167	162	158	155	150	148	142
130	349	298	257	228	208	193	181	169	161	155	152	149	145	142	137
135	340	287	247	220	200	185	174	162	155	150	146	143	139	137	132
140	331	277	238	212	193	179	168	157	149	144	141	138	134	132	127
145	321	267	230	205	186	173	162	151	144	139	136	134	130	128	123
150	312	258	223	198	180	167	157	146	139	135	131	129	125	123	118
155	303	250	215	192	174	162	152	141	135	130	127	125	121	119	115
160	294	242	209	186	169	156	147	137	131	126	123	121	118	116	111
165	285	235	202	180	164	152	142	133	127	122	120	117	114	112	108
170	277	228	196	175	159	147	138	129	123	119	116	114	111	109	105
175	269	221	191	170	154	143	134	125	119	115	113	111	107	106	102
180	261	215	185	165	150	139	131	122	116	112	110	108	104	103	99
185	254	209	180	160	146	135	127	118	113	109	107	105	102	100	96
190	247	204	176	156	142	132	124	115	110	106	104	102	99	97	94
195	241	199	171	152	139	128	121	112	107	104	101	99	96	95	91
200	235	194	167	148	135	125	118	110	104	101	99	97	94	92	89
205	229	189	163	145	132	122	115	107	102	99	96	94	92	90	87
210	224	184	159	141	129	119	112	104	100	96	94	92	90	88	85
215	219	180	155	138	126	116	109	102	97	94	92	90	87	86	83
220	214	176	152	135	123	114	107	100	95	92	90	88	85	84	81
225	209	172	148	132	120	111	104	97	93	90	88	86	84	82	79
230	204	168	145	129	118	109	102	95	91	88	86	84	82	80	77
235	200	165	142	126	115	107	100	93	89	86	84	82	80	79	76
240	196	161	139	124	113	104	98	91	87	84	82	81	78	77	74
245	192	158	136	121	110	102	96	89	85	82	81	79	77	76	73
250	188	155	134	119	108	100	94	88	84	81	79	77	75	74	71

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5k - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

11) Grade Q235 steel, web thickness ≤ 16mm – design strength p_y = 215N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	129	129	129	129	129	129	129	129	129	129	129	129	129	129	129
60	129	129	129	129	129	129	129	129	129	129	129	129	129	129	129
65	129	129	129	129	129	129	129	129	129	129	129	129	129	129	129
70	129	129	129	129	129	129	129	129	129	129	129	129	129	129	129
75	129	129	129	129	129	129	129	129	129	129	129	129	128	127	124
80	129	129	129	129	129	129	129	129	129	129	127	126	124	123	120
85	129	129	129	129	129	129	129	129	127	125	123	122	119	118	115
90	129	129	129	129	129	129	129	126	123	121	119	117	115	114	111
95	129	129	129	129	129	129	127	122	119	117	115	113	111	110	106
100	129	129	129	129	129	128	124	119	115	113	111	109	107	105	102
105	129	129	129	129	129	125	120	115	111	109	106	105	102	101	97
110	129	129	129	129	127	122	117	111	107	105	102	101	98	96	92
115	129	129	129	129	124	118	113	108	104	101	98	97	94	92	88
120	129	129	129	127	121	115	110	104	100	97	94	92	90	88	85
125	129	129	129	124	118	112	107	100	96	92	90	89	86	85	81
130	129	129	129	122	115	109	103	97	92	89	87	85	83	81	78
135	129	129	127	119	112	105	100	93	89	86	84	82	80	78	75
140	129	129	125	116	109	102	96	90	85	83	81	79	77	75	73
145	129	129	122	114	106	99	93	86	82	80	78	76	74	73	70
150	129	129	120	111	103	96	90	84	80	77	75	74	72	70	68
155	129	128	117	108	100	92	87	81	77	74	73	71	69	68	65
160	129	126	115	105	97	90	84	78	75	72	70	69	67	66	63
165	129	124	113	103	94	87	81	76	72	70	68	67	65	64	61
170	129	122	110	100	91	84	79	74	70	68	66	65	63	62	60
175	129	120	108	97	88	82	77	72	68	66	64	63	61	60	58
180	129	117	105	94	86	80	75	70	66	64	63	61	60	59	56
185	129	115	103	92	84	77	73	68	64	62	61	60	58	57	55
190	127	113	100	89	81	75	71	66	63	61	59	58	56	55	53
195	126	111	98	87	79	73	69	64	61	59	58	57	55	54	52
200	124	109	96	85	77	72	67	63	60	58	56	55	54	53	51
205	122	107	93	83	75	70	65	61	58	56	55	54	52	51	49
210	120	105	91	81	74	68	64	60	57	55	54	53	51	50	48
215	119	103	89	79	72	66	62	58	55	54	52	51	50	49	47
220	117	101	87	77	70	65	61	57	54	52	51	50	49	48	46
225	115	99	85	75	69	64	60	56	53	51	50	49	48	47	45
230	113	97	83	74	67	62	58	54	52	50	49	48	47	46	44
235	112	94	81	72	66	61	57	53	51	49	48	47	46	45	43
240	110	92	79	71	64	60	56	52	50	48	47	46	45	44	42
245	108	90	78	69	63	58	55	51	49	47	46	45	44	43	41
250	107	89	76	68	62	57	54	50	48	46	45	44	43	42	40

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5l - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

12) Grade Q235 steel, web thickness >16mm ≤ 40mm – design strength p_y = 205N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	123	123	123	123	123	123	123	123	123	123	123	123	123	123	123
60	123	123	123	123	123	123	123	123	123	123	123	123	123	123	123
65	123	123	123	123	123	123	123	123	123	123	123	123	123	123	123
70	123	123	123	123	123	123	123	123	123	123	123	123	123	123	123
75	123	123	123	123	123	123	123	123	123	123	123	123	123	123	120
80	123	123	123	123	123	123	123	123	123	123	123	121	120	118	116
85	123	123	123	123	123	123	123	123	122	120	119	118	116	114	112
90	123	123	123	123	123	123	123	122	119	117	115	114	112	110	107
95	123	123	123	123	123	123	123	118	115	113	111	110	108	106	103
100	123	123	123	123	123	123	120	115	112	109	107	106	104	102	99
105	123	123	123	123	123	120	116	111	108	105	103	102	100	98	95
110	123	123	123	123	122	117	113	108	104	102	100	98	96	94	90
115	123	123	123	123	120	114	110	105	101	98	96	94	91	90	86
120	123	123	123	123	117	111	107	101	97	94	92	90	88	86	83
125	123	123	123	120	114	108	104	98	93	90	88	87	84	83	79
130	123	123	123	118	111	105	100	94	90	87	85	83	81	79	76
135	123	123	123	115	108	102	97	91	86	84	82	80	78	76	73
140	123	123	120	113	106	99	94	87	83	81	79	77	75	74	71
145	123	123	118	110	103	96	91	84	80	78	76	75	72	71	68
150	123	123	116	108	100	93	88	82	78	75	73	72	70	69	66
155	123	123	114	105	97	90	85	79	75	73	71	70	68	67	64
160	123	121	111	102	94	87	82	76	73	70	69	67	66	64	62
165	123	119	109	100	92	85	80	74	71	68	67	65	64	62	60
170	123	118	107	97	89	82	77	72	69	66	65	63	62	61	58
175	123	116	105	95	86	80	75	70	67	64	63	62	60	59	57
180	123	114	102	92	84	78	73	68	65	63	61	60	58	57	55
185	123	112	100	90	82	76	71	66	63	61	59	58	57	56	53
190	123	110	98	87	79	74	69	64	61	59	58	57	55	54	52
195	121	108	96	85	77	72	67	63	60	58	56	55	54	53	51
200	120	106	93	83	75	70	66	61	58	56	55	54	52	51	49
205	118	104	91	81	74	68	64	60	57	55	54	53	51	50	48
210	116	102	89	79	72	66	62	58	55	54	52	51	50	49	47
215	115	100	87	77	70	65	61	57	54	52	51	50	49	48	46
220	113	98	85	75	68	63	60	55	53	51	50	49	48	47	45
225	112	96	83	74	67	62	58	54	52	50	49	48	46	46	44
230	110	94	81	72	65	61	57	53	51	49	48	47	45	45	43
235	108	92	79	70	64	59	56	52	49	48	47	46	44	44	42
240	107	90	78	69	63	58	55	51	48	47	46	45	44	43	41
245	105	88	76	68	61	57	53	50	47	46	45	44	43	42	40
250	104	87	74	66	60	56	52	49	46	45	44	43	42	41	39

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5m - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

13) Grade Q345/Q355 steel, web thickness ≤ 16mm – design strength p_y = 305N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	183	183	183	183	183	183	183	183	183	183	183	183	183	183	183
60	183	183	183	183	183	183	183	183	183	183	183	183	183	183	182
65	183	183	183	183	183	183	183	183	183	183	183	182	179	178	174
70	183	183	183	183	183	183	183	183	182	179	177	175	172	170	166
75	183	183	183	183	183	183	183	180	176	172	170	168	165	163	159
80	183	183	183	183	183	183	181	174	169	166	163	161	158	156	151
85	183	183	183	183	183	181	175	168	163	159	156	154	150	148	143
90	183	183	183	183	183	176	169	162	156	152	149	147	143	141	135
95	183	183	183	183	178	170	163	155	150	145	142	140	136	134	128
100	183	183	183	182	173	165	158	149	143	139	135	133	129	127	122
105	183	183	183	177	168	159	152	143	137	132	129	126	123	121	116
110	183	183	183	173	163	154	146	137	130	126	123	121	117	115	111
115	183	183	180	168	158	148	140	131	124	120	117	115	112	110	106
120	183	183	176	163	153	143	134	125	119	115	113	111	107	106	101
125	183	183	172	159	147	137	129	120	115	111	108	106	103	101	97
130	183	182	167	154	142	132	124	115	110	106	104	102	99	97	94
135	183	179	163	150	137	127	119	111	106	103	100	98	95	94	90
140	183	175	159	145	132	123	115	107	102	99	97	95	92	91	87
145	183	172	155	140	128	118	111	104	99	95	93	91	89	87	84
150	183	168	151	136	123	114	107	100	95	92	90	88	86	84	81
155	183	165	147	131	119	111	104	97	92	89	87	86	83	82	79
160	181	161	143	127	116	107	101	94	89	87	84	83	81	79	76
165	178	157	139	123	112	104	98	91	87	84	82	80	78	77	74
170	175	154	135	120	109	101	95	88	84	81	79	78	76	75	72
175	172	150	131	116	106	98	92	86	82	79	77	76	74	72	70
180	169	147	127	113	103	95	89	83	80	77	75	74	72	70	68
185	166	143	124	110	100	93	87	81	77	75	73	72	70	68	66
190	163	140	120	107	97	90	85	79	75	73	71	70	68	67	64
195	161	136	117	104	95	88	83	77	73	71	69	68	66	65	62
200	158	133	114	102	93	86	81	75	72	69	68	66	64	63	61
205	155	129	112	99	90	84	79	73	70	68	66	65	63	62	59
210	152	126	109	97	88	82	77	71	68	66	64	63	61	60	58
215	149	123	106	95	86	80	75	70	67	64	63	62	60	59	57
220	146	121	104	92	84	78	73	68	65	63	61	60	59	58	55
225	143	118	102	90	82	76	72	67	64	62	60	59	57	56	54
230	140	115	99	88	80	75	70	65	62	60	59	58	56	55	53
235	137	113	97	87	79	73	69	64	61	59	57	56	55	54	52
240	134	111	95	85	77	71	67	63	60	58	56	55	54	53	51
245	131	108	93	83	76	70	66	61	58	57	55	54	53	52	50
250	129	106	91	81	74	69	64	60	57	55	54	53	52	51	49

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5n - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

14) Grade Q345/Q355 steel, web thickness >16mm ≤ 40mm – design strength p_y = 295N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	177	177	177	177	177	177	177	177	177	177	177	177	177	177	177
60	177	177	177	177	177	177	177	177	177	177	177	177	177	177	177
65	177	177	177	177	177	177	177	177	177	177	177	177	175	174	170
70	177	177	177	177	177	177	177	177	177	175	173	171	168	167	163
75	177	177	177	177	177	177	177	176	172	168	166	164	161	160	155
80	177	177	177	177	177	177	176	170	165	162	159	158	154	152	148
85	177	177	177	177	177	177	171	164	159	156	153	151	147	145	141
90	177	177	177	177	177	171	165	158	153	149	146	144	141	138	133
95	177	177	177	177	174	166	160	152	147	143	140	137	134	131	126
100	177	177	177	177	169	161	154	146	140	136	133	131	127	125	120
105	177	177	177	173	164	156	149	140	134	130	127	124	121	119	114
110	177	177	177	169	159	151	143	134	128	124	121	119	115	113	109
115	177	177	175	164	154	145	138	128	122	118	116	113	110	108	104
120	177	177	171	160	149	140	132	123	117	113	111	109	106	104	100
125	177	177	168	155	145	135	127	118	113	109	106	104	101	100	96
130	177	177	164	151	140	130	122	114	108	105	102	100	97	96	92
135	177	174	160	147	135	125	117	109	104	101	98	97	94	92	89
140	177	171	156	142	130	120	113	105	101	97	95	93	91	89	86
145	177	168	152	138	126	116	109	102	97	94	92	90	87	86	83
150	177	164	148	134	121	112	106	98	94	91	89	87	84	83	80
155	177	161	144	129	117	109	102	95	91	88	86	84	82	80	77
160	176	158	140	125	114	105	99	92	88	85	83	82	79	78	75
165	174	154	136	121	110	102	96	89	85	83	81	79	77	76	73
170	171	151	132	118	107	99	93	87	83	80	78	77	75	73	70
175	168	147	129	114	104	96	91	84	80	78	76	75	72	71	68
180	165	144	125	111	101	94	88	82	78	76	74	72	70	69	67
185	163	141	122	108	98	91	86	80	76	74	72	71	68	67	65
190	160	137	118	105	96	89	83	78	74	72	70	69	67	66	63
195	157	134	115	103	93	87	81	76	72	70	68	67	65	64	61
200	154	131	112	100	91	84	79	74	70	68	66	65	63	62	60
205	152	127	110	98	89	82	77	72	69	66	65	64	62	61	58
210	149	124	107	95	87	80	75	70	67	65	63	62	60	59	57
215	146	121	105	93	85	78	74	69	65	63	62	61	59	58	56
220	143	119	102	91	83	77	72	67	64	62	60	59	58	57	54
225	141	116	100	89	81	75	70	66	63	61	59	58	56	55	53
230	138	113	98	87	79	73	69	64	61	59	58	57	55	54	52
235	135	111	96	85	77	72	67	63	60	58	57	56	54	53	51
240	132	109	94	83	76	70	66	62	59	57	55	54	53	52	50
245	129	107	92	82	74	69	65	60	57	56	54	53	52	51	49
250	127	104	90	80	73	67	63	59	56	54	53	52	51	50	48

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5o - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

15) Grade Q390 steel, web thickness ≤ 16mm – design strength p_y = 345N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	207	207	207	207	207	207	207	207	207	207	207	207	207	207	207
60	207	207	207	207	207	207	207	207	207	207	207	207	205	203	199
65	207	207	207	207	207	207	207	207	207	204	201	200	196	194	190
70	207	207	207	207	207	207	207	205	200	196	193	191	187	185	180
75	207	207	207	207	207	207	205	197	192	188	185	183	179	177	171
80	207	207	207	207	207	205	198	190	184	180	176	174	170	168	162
85	207	207	207	207	206	198	191	182	176	172	168	165	161	159	153
90	207	207	207	207	200	192	184	175	168	163	160	157	152	150	144
95	207	207	207	205	194	185	177	167	160	155	151	149	144	142	136
100	207	207	207	199	188	178	170	160	152	147	144	141	137	135	129
105	207	207	206	194	182	172	163	152	145	140	137	134	131	128	123
110	207	207	202	188	176	165	156	145	138	134	131	128	125	123	118
115	207	207	197	182	170	159	149	139	132	128	125	123	119	117	113
120	207	207	192	177	164	152	143	133	127	123	120	118	114	112	108
125	207	204	187	171	158	146	137	128	122	118	115	113	110	108	104
130	207	200	182	166	152	140	132	123	117	113	111	109	105	104	100
135	207	195	177	160	146	135	127	118	113	109	106	105	102	100	96
140	207	191	172	155	141	130	122	114	109	105	103	101	98	96	92
145	207	187	167	149	136	126	118	110	105	102	99	97	95	93	89
150	205	183	162	144	131	122	114	106	102	98	96	94	91	90	86
155	201	178	157	140	127	118	111	103	98	95	93	91	88	87	84
160	198	174	152	135	123	114	107	100	95	92	90	88	86	84	81
165	194	170	147	131	119	111	104	97	92	89	87	86	83	82	78
170	191	165	143	127	116	107	101	94	90	87	85	83	81	79	76
175	187	161	139	124	113	104	98	91	87	84	82	81	78	77	74
180	184	157	135	120	109	101	95	89	85	82	80	78	76	75	72
185	180	153	131	117	106	99	93	86	82	80	78	76	74	73	70
190	177	149	128	114	104	96	90	84	80	77	76	74	72	71	68
195	173	145	125	111	101	94	88	82	78	76	74	72	70	69	66
200	170	141	122	108	98	91	86	80	76	74	72	71	69	67	65
205	166	138	119	106	96	89	84	78	74	72	70	69	67	66	63
210	163	134	116	103	94	87	82	76	73	70	68	67	65	64	62
215	159	131	113	101	92	85	80	74	71	68	67	66	64	63	60
220	156	128	111	98	90	83	78	73	69	67	65	64	62	61	59
225	152	125	108	96	88	81	76	71	68	65	64	63	61	60	58
230	149	123	106	94	86	79	74	69	66	64	62	61	60	59	56
235	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55
240	143	118	101	90	82	76	71	67	63	61	60	59	57	56	54
245	140	115	99	88	80	74	70	65	62	60	59	58	56	55	53
250	137	113	97	87	79	73	69	64	61	59	57	56	55	54	52

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5p - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

16) Grade Q390 steel, web thickness >16mm ≤ 40mm – design strength p_y = 330N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	198	198	198	198	198	198	198	198	198	198	198	198	198	198	198
60	198	198	198	198	198	198	198	198	198	198	198	198	198	197	193
65	198	198	198	198	198	198	198	198	198	197	195	193	190	188	184
70	198	198	198	198	198	198	198	198	193	190	187	185	182	180	175
75	198	198	198	198	198	198	198	191	186	182	179	177	174	172	167
80	198	198	198	198	198	198	192	184	179	175	172	169	166	163	158
85	198	198	198	198	198	192	185	177	171	167	164	161	157	155	149
90	198	198	198	198	194	186	179	170	164	159	156	153	149	147	141
95	198	198	198	198	188	180	172	163	157	152	148	145	141	139	133
100	198	198	198	193	183	174	166	156	149	144	141	138	134	132	127
105	198	198	198	188	177	167	159	149	142	137	134	131	128	126	121
110	198	198	195	182	171	161	152	142	135	131	128	125	122	120	115
115	198	198	190	177	165	155	146	136	129	125	122	120	117	115	110
120	198	198	186	172	160	149	140	130	124	120	117	115	112	110	106
125	198	197	181	167	154	143	134	125	119	115	112	110	107	105	101
130	198	193	177	162	148	137	129	120	115	111	108	106	103	101	97
135	198	189	172	156	143	132	124	116	110	107	104	102	99	98	94
140	198	185	167	151	138	127	120	112	106	103	100	99	96	94	90
145	198	181	163	146	133	123	116	108	103	99	97	95	92	91	87
150	198	177	158	141	128	119	112	104	99	96	94	92	89	88	84
155	195	173	153	136	124	115	108	101	96	93	91	89	86	85	82
160	192	169	149	132	120	112	105	98	93	90	88	86	84	82	79
165	188	165	144	128	117	108	102	95	90	87	85	84	81	80	77
170	185	161	140	124	113	105	99	92	88	85	83	81	79	78	74
175	182	157	136	121	110	102	96	89	85	82	80	79	77	75	72
180	179	153	132	118	107	99	93	87	83	80	78	77	74	73	70
185	175	149	129	114	104	96	91	84	80	78	76	75	72	71	68
190	172	145	125	111	101	94	88	82	78	76	74	73	71	69	67
195	169	142	122	108	99	91	86	80	76	74	72	71	69	68	65
200	166	138	119	106	96	89	84	78	74	72	70	69	67	66	63
205	162	135	116	103	94	87	82	76	73	70	69	67	65	64	62
210	159	131	113	101	92	85	80	74	71	69	67	66	64	63	60
215	156	128	111	98	90	83	78	73	69	67	65	64	62	61	59
220	152	125	108	96	88	81	76	71	68	65	64	63	61	60	58
225	149	123	106	94	86	79	74	69	66	64	62	61	60	59	56
230	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55
235	143	117	101	90	82	76	71	66	63	61	60	59	57	56	54
240	140	115	99	88	80	74	70	65	62	60	59	58	56	55	53
245	137	113	97	86	79	73	68	64	61	59	57	56	55	54	52
250	134	110	95	85	77	71	67	62	60	58	56	55	54	53	51

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5q - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

17) Grade Q420 steel, web thickness ≤ 16mm – design strength p_y = 375N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	225	225	225	225	225	225	225	225	225	225	225	225	225	225	222
60	225	225	225	225	225	225	225	225	225	225	223	221	218	216	211
65	225	225	225	225	225	225	225	225	221	217	214	212	208	206	201
70	225	225	225	225	225	225	225	218	212	208	205	202	198	196	190
75	225	225	225	225	225	225	218	209	203	198	195	192	188	186	180
80	225	225	225	225	225	218	210	201	194	189	186	183	178	176	169
85	225	225	225	225	220	210	202	192	185	180	176	173	168	166	159
90	225	225	225	224	213	203	194	184	176	171	167	164	159	156	150
95	225	225	225	218	206	195	186	175	167	162	158	155	150	148	142
100	225	225	225	211	199	188	178	167	159	154	150	147	143	141	135
105	225	225	220	205	192	180	170	159	151	146	143	140	136	134	129
110	225	225	214	199	185	173	162	151	144	140	136	134	130	128	123
115	225	225	208	192	178	166	155	145	138	133	130	128	124	122	117
120	225	221	203	186	171	158	149	139	132	128	125	123	119	117	113
125	225	217	197	180	164	152	143	133	127	123	120	118	114	112	108
130	225	212	192	174	158	146	137	128	122	118	115	113	110	108	104
135	225	207	186	167	152	141	132	123	118	114	111	109	106	104	100
140	225	202	180	161	147	136	128	119	113	110	107	105	102	100	96
145	222	197	175	156	142	131	123	115	109	106	103	101	99	97	93
150	218	192	169	150	137	127	119	111	106	102	100	98	95	94	90
155	214	188	164	146	132	123	115	107	102	99	97	95	92	91	87
160	210	183	158	141	128	119	112	104	99	96	94	92	89	88	84
165	206	178	154	137	124	115	108	101	96	93	91	89	87	85	82
170	202	173	149	133	121	112	105	98	93	90	88	87	84	83	79
175	198	168	145	129	117	109	102	95	91	88	86	84	82	80	77
180	194	164	141	125	114	106	99	92	88	85	83	82	79	78	75
185	190	159	137	122	111	103	97	90	86	83	81	80	77	76	73
190	186	155	133	119	108	100	94	88	84	81	79	77	75	74	71
195	182	151	130	116	105	98	92	85	81	79	77	75	73	72	69
200	178	147	127	113	103	95	89	83	79	77	75	74	71	70	68
205	174	144	124	110	100	93	87	81	77	75	73	72	70	69	66
210	170	140	121	107	98	91	85	79	76	73	71	70	68	67	64
215	166	137	118	105	95	88	83	77	74	71	70	68	66	65	63
220	162	134	115	103	93	86	81	76	72	70	68	67	65	64	61
225	159	131	113	100	91	85	79	74	71	68	67	65	63	62	60
230	155	128	110	98	89	83	78	72	69	67	65	64	62	61	59
235	152	125	108	96	87	81	76	71	68	65	64	63	61	60	57
240	149	123	106	94	86	79	74	69	66	64	62	61	60	59	56
245	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55
250	143	118	101	90	82	76	71	67	63	61	60	59	57	56	54

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5r - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < t ≤ 40mm)

18) Grade Q420 steel, web thickness >16mm ≤ 40mm – design strength p_y = 355N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	213	213	213	213	213	213	213	213	213	213	213	213	213	213	213
60	213	213	213	213	213	213	213	213	213	213	213	213	209	208	203
65	213	213	213	213	213	213	213	213	212	208	206	204	200	198	193
70	213	213	213	213	213	213	213	209	204	200	197	195	191	189	184
75	213	213	213	213	213	213	209	201	196	191	188	186	182	180	174
80	213	213	213	213	213	209	202	194	187	183	180	177	173	170	164
85	213	213	213	213	211	202	195	186	179	174	171	168	164	161	155
90	213	213	213	213	205	195	187	178	171	166	162	159	155	152	146
95	213	213	213	209	198	189	180	170	163	157	153	151	146	144	138
100	213	213	213	203	192	182	173	162	155	149	146	143	139	137	131
105	213	213	211	197	185	175	165	154	147	142	139	136	132	130	125
110	213	213	206	192	179	168	158	147	140	136	133	130	126	124	119
115	213	213	201	186	173	161	151	141	134	130	127	124	121	119	114
120	213	213	195	180	166	154	145	135	129	124	121	119	116	114	109
125	213	208	190	174	160	148	139	130	124	119	117	115	111	109	105
130	213	204	185	169	154	142	134	125	119	115	112	110	107	105	101
135	213	199	180	163	148	137	129	120	114	111	108	106	103	101	97
140	213	195	175	157	143	132	124	116	110	107	104	102	99	98	94
145	213	190	170	151	138	128	120	112	107	103	101	99	96	94	91
150	209	186	165	146	133	123	116	108	103	100	97	95	93	91	88
155	206	181	159	142	129	119	112	105	100	96	94	92	90	88	85
160	202	177	154	137	125	116	109	101	97	93	91	89	87	85	82
165	198	173	150	133	121	112	105	98	94	91	88	87	84	83	80
170	195	168	145	129	117	109	102	95	91	88	86	84	82	80	77
175	191	164	141	125	114	106	99	93	88	85	83	82	79	78	75
180	187	159	137	122	111	103	97	90	86	83	81	80	77	76	73
185	184	155	133	119	108	100	94	88	83	81	79	77	75	74	71
190	180	151	130	115	105	97	91	85	81	79	77	75	73	72	69
195	176	147	127	113	102	95	89	83	79	77	75	73	71	70	67
200	173	143	123	110	100	93	87	81	77	75	73	72	70	68	66
205	169	140	120	107	97	90	85	79	75	73	71	70	68	67	64
210	165	136	117	104	95	88	83	77	74	71	69	68	66	65	63
215	162	133	115	102	93	86	81	75	72	69	68	67	65	64	61
220	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60
225	155	127	110	98	89	82	77	72	69	66	65	64	62	61	58
230	151	124	107	95	87	80	76	70	67	65	63	62	60	59	57
235	148	122	105	93	85	79	74	69	66	64	62	61	59	58	56
240	145	119	103	91	83	77	72	67	64	62	61	60	58	57	55
245	142	117	101	90	82	76	71	66	63	61	59	58	57	56	54
250	139	115	99	88	80	74	70	65	62	60	58	57	56	55	53

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5s - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

19) Grade Q460 steel, web thickness ≤ 16mm – design strength p_y = 410N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	246	246	246	246	246	246	246	246	246	246	246	246	244	242	236
60	246	246	246	246	246	246	246	246	246	242	239	236	232	230	224
65	246	246	246	246	246	246	246	242	236	231	228	225	221	219	212
70	246	246	246	246	246	246	241	232	226	221	217	214	210	207	200
75	246	246	246	246	246	241	232	222	215	210	206	203	198	195	188
80	246	246	246	246	242	232	223	213	205	200	195	192	187	184	176
85	246	246	246	246	234	224	214	203	195	189	185	181	176	173	166
90	246	246	246	239	226	215	205	193	185	178	174	171	166	163	157
95	246	246	246	232	218	207	196	184	175	169	165	162	157	155	149
100	246	246	241	225	211	198	187	174	166	161	157	154	149	147	141
105	246	246	234	218	203	190	178	166	158	153	149	146	142	140	134
110	246	246	228	210	195	181	170	158	151	146	142	140	136	134	128
115	246	242	221	203	187	173	162	151	144	140	136	134	130	128	123
120	246	236	215	196	179	166	156	145	138	134	131	128	124	122	118
125	246	231	209	189	172	159	149	139	133	128	125	123	120	118	113
130	246	225	202	182	165	153	144	134	128	123	120	118	115	113	109
135	246	220	196	175	159	147	138	129	123	119	116	114	111	109	105
140	241	214	190	168	153	142	133	124	119	115	112	110	107	105	101
145	237	209	183	163	148	137	129	120	114	111	108	106	103	101	97
150	232	203	177	157	143	133	124	116	111	107	104	103	100	98	94
155	228	198	171	152	138	128	120	112	107	104	101	99	96	95	91
160	223	192	166	147	134	124	117	109	104	100	98	96	93	92	88
165	219	187	161	143	130	121	113	106	101	97	95	93	91	89	86
170	214	181	156	139	126	117	110	102	98	94	92	90	88	86	83
175	210	176	152	135	123	114	107	99	95	92	90	88	85	84	81
180	205	171	147	131	119	110	104	97	92	89	87	85	83	82	78
185	201	166	143	127	116	107	101	94	90	87	85	83	81	79	76
190	196	162	140	124	113	105	98	92	87	84	82	81	79	77	74
195	192	158	136	121	110	102	96	89	85	82	80	79	77	75	72
200	187	154	133	118	107	99	93	87	83	80	78	77	75	73	71
205	182	150	129	115	105	97	91	85	81	78	76	75	73	72	69
210	178	146	126	112	102	95	89	83	79	76	75	73	71	70	67
215	174	143	123	110	100	92	87	81	77	75	73	72	69	68	66
220	170	140	121	107	98	90	85	79	75	73	71	70	68	67	64
225	166	137	118	105	95	88	83	77	74	71	70	68	66	65	63
230	162	134	115	103	93	86	81	76	72	70	68	67	65	64	61
235	159	131	113	100	91	85	79	74	71	68	67	65	64	63	60
240	156	128	110	98	89	83	78	73	69	67	65	64	62	61	59
245	152	126	108	96	88	81	76	71	68	66	64	63	61	60	58
250	149	123	106	94	86	80	75	70	66	64	63	62	60	59	56

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5t - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

20) Grade Q460 steel, web thickness >16mm ≤ 40mm – design strength p_y = 390N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	234	234	234	234	234	234	234	234	234	234	234	234	234	233	228
60	234	234	234	234	234	234	234	234	234	233	230	228	224	222	217
65	234	234	234	234	234	234	234	233	227	223	220	218	214	211	206
70	234	234	234	234	234	234	233	224	218	213	210	207	203	201	195
75	234	234	234	234	234	232	224	215	208	204	200	197	193	190	183
80	234	234	234	234	233	224	216	206	199	194	190	187	182	179	172
85	234	234	234	234	226	216	207	197	189	184	180	177	172	169	162
90	234	234	234	230	219	208	199	188	180	174	170	167	162	159	153
95	234	234	234	224	211	200	191	179	171	165	161	158	153	151	145
100	234	234	232	217	204	192	182	170	162	157	153	150	146	143	138
105	234	234	226	210	197	184	174	162	154	149	146	143	139	136	131
110	234	234	220	204	189	177	166	154	147	142	139	136	132	130	125
115	234	233	214	197	182	169	158	148	141	136	133	130	127	125	120
120	234	228	208	190	175	162	152	141	135	130	127	125	121	119	115
125	234	223	202	184	167	155	146	136	130	125	122	120	117	115	110
130	234	218	196	177	161	149	140	131	125	120	118	115	112	110	106
135	234	213	190	170	155	144	135	126	120	116	113	111	108	106	102
140	233	207	184	164	150	139	130	121	116	112	109	107	104	102	98
145	228	202	179	159	144	134	126	117	112	108	105	103	100	99	95
150	224	197	173	153	140	129	121	113	108	104	102	100	97	96	92
155	220	192	167	148	135	125	117	110	104	101	99	97	94	92	89
160	216	187	162	144	131	121	114	106	101	98	95	94	91	90	86
165	212	182	157	139	127	118	110	103	98	95	93	91	88	87	83
170	207	177	152	135	123	114	107	100	95	92	90	88	86	84	81
175	203	172	148	131	120	111	104	97	93	89	87	86	83	82	79
180	199	167	144	128	116	108	101	94	90	87	85	83	81	80	76
185	195	162	140	124	113	105	98	92	88	85	83	81	79	77	74
190	191	158	136	121	110	102	96	89	85	82	80	79	77	75	72
195	186	154	133	118	107	99	93	87	83	80	78	77	75	73	71
200	182	150	129	115	105	97	91	85	81	78	76	75	73	72	69
205	178	146	126	112	102	95	89	83	79	76	75	73	71	70	67
210	174	143	123	110	100	92	87	81	77	75	73	71	69	68	66
215	169	140	120	107	97	90	85	79	75	73	71	70	68	67	64
220	166	136	118	105	95	88	83	77	74	71	69	68	66	65	63
225	162	133	115	102	93	86	81	75	72	70	68	67	65	64	61
230	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60
235	155	128	110	98	89	83	77	72	69	67	65	64	62	61	59
240	152	125	108	96	87	81	76	71	67	65	64	63	61	60	57
245	149	122	106	94	85	79	74	69	66	64	62	61	59	58	56
250	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5u - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

21) Grade Q550 steel, web thickness $\leq$ 16mm – design strength p_y = 520N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	312	312	312	312	312	312	312	312	307	301	297	294	289	286	279
60	312	312	312	312	312	312	312	300	292	286	282	279	273	270	262
65	312	312	312	312	312	309	299	287	278	271	266	263	257	253	244
70	312	312	312	312	309	297	286	273	263	256	251	247	240	237	227
75	312	312	312	312	298	285	273	259	249	241	236	231	224	221	212
80	312	312	312	303	287	273	260	245	234	226	221	217	210	207	199
85	312	312	312	293	276	260	247	231	220	213	208	204	198	195	187
90	312	312	303	283	264	248	234	218	208	201	196	192	187	184	177
95	312	312	294	272	253	236	221	206	197	190	186	182	177	174	167
100	312	310	285	262	242	224	210	196	187	181	176	173	168	165	159
105	312	302	276	252	231	213	200	187	178	172	168	165	160	158	151
110	312	294	266	242	220	204	191	178	170	164	160	157	153	150	145
115	312	286	257	231	210	195	183	170	163	157	153	151	146	144	138
120	312	279	248	221	201	187	175	163	156	151	147	144	140	138	132
125	305	271	239	212	193	179	168	157	150	145	141	139	135	132	127
130	299	263	230	204	186	172	162	151	144	139	136	133	129	127	122
135	292	255	221	197	179	166	156	145	138	134	131	128	125	123	118
140	286	247	213	190	173	160	150	140	134	129	126	124	120	118	114
145	279	239	206	183	167	154	145	135	129	125	122	119	116	114	110
150	273	231	199	177	161	149	140	131	125	121	118	115	112	110	106
155	266	224	193	171	156	144	136	126	121	117	114	112	109	107	103
160	260	217	187	166	151	140	131	123	117	113	110	108	105	103	99
165	253	210	181	161	147	136	127	119	113	110	107	105	102	100	96
170	247	204	176	156	142	132	124	115	110	106	104	102	99	97	94
175	240	198	171	152	138	128	120	112	107	103	101	99	96	95	91
180	234	192	166	148	134	124	117	109	104	100	98	96	93	92	88
185	228	187	161	144	131	121	114	106	101	98	95	94	91	89	86
190	221	182	157	140	127	118	111	103	98	95	93	91	89	87	84
195	216	178	153	136	124	115	108	101	96	93	90	89	86	85	82
200	210	173	149	133	121	112	105	98	93	90	88	87	84	83	79
205	205	169	146	130	118	109	103	96	91	88	86	85	82	81	78
210	200	165	142	126	115	107	100	93	89	86	84	82	80	79	76
215	196	161	139	124	112	104	98	91	87	84	82	81	78	77	74
220	191	157	136	121	110	102	96	89	85	82	80	79	76	75	72
225	187	154	133	118	107	100	93	87	83	80	78	77	75	74	71
230	183	151	130	115	105	97	91	85	81	79	77	75	73	72	69
235	179	147	127	113	103	95	89	83	80	77	75	74	72	70	68
240	175	144	124	111	101	93	88	82	78	75	74	72	70	69	66
245	172	141	122	108	99	91	86	80	76	74	72	71	69	68	65
250	168	139	119	106	97	90	84	78	75	72	71	69	67	66	64

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5v - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

22) Grade Q550 steel, web thickness >16mm ≤ 40mm – design strength p_y = 500N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	300	300	300	300	300	300	300	300	298	293	289	286	281	278	272
60	300	300	300	300	300	300	300	292	284	279	275	271	266	263	255
65	300	300	300	300	300	300	290	279	271	265	260	256	251	247	239
70	300	300	300	300	300	289	278	266	257	250	245	242	235	232	223
75	300	300	300	300	290	277	266	253	243	236	231	227	220	216	208
80	300	300	300	295	279	266	254	240	229	222	216	212	206	203	195
85	300	300	300	285	269	254	242	226	216	209	204	200	194	191	183
90	300	300	294	275	258	243	229	214	204	197	192	189	183	180	173
95	300	300	286	265	247	231	217	202	193	187	182	179	174	171	164
100	300	300	277	256	237	220	206	192	183	177	173	170	165	162	156
105	300	294	269	246	226	209	196	183	175	169	165	162	157	155	148
110	300	286	260	236	216	200	187	175	167	161	157	154	150	147	142
115	300	279	251	227	206	191	179	167	159	154	150	148	143	141	136
120	300	271	243	217	198	183	172	160	153	148	144	142	137	135	130
125	297	264	234	208	190	176	165	154	147	142	138	136	132	130	125
130	290	256	226	200	182	169	159	148	141	136	133	131	127	125	120
135	284	249	217	193	176	163	153	142	136	131	128	126	122	120	115
140	278	242	209	186	169	157	147	137	131	127	124	121	118	116	111
145	272	234	202	180	163	151	142	133	126	122	119	117	114	112	108
150	266	227	195	174	158	146	137	128	122	118	115	113	110	108	104
155	260	219	189	168	153	142	133	124	118	114	112	110	106	105	101
160	254	212	183	163	148	137	129	120	115	111	108	106	103	101	97
165	248	206	177	158	144	133	125	117	111	107	105	103	100	98	94
170	242	200	172	153	139	129	121	113	108	104	102	100	97	95	92
175	235	194	167	149	135	125	118	110	105	101	99	97	94	93	89
180	229	189	163	145	132	122	115	107	102	98	96	94	92	90	87
185	223	184	158	141	128	119	111	104	99	96	94	92	89	88	84
190	217	179	154	137	125	116	109	101	96	93	91	89	87	85	82
195	212	174	150	134	122	113	106	99	94	91	89	87	85	83	80
200	206	170	146	130	119	110	103	96	92	89	86	85	82	81	78
205	201	166	143	127	116	107	101	94	89	86	84	83	80	79	76
210	196	162	139	124	113	105	98	92	87	84	82	81	79	77	74
215	192	158	136	121	110	102	96	89	85	82	80	79	77	75	73
220	187	154	133	118	108	100	94	87	83	81	79	77	75	74	71
225	183	151	130	116	105	98	92	85	81	79	77	75	73	72	69
230	179	148	127	113	103	95	90	84	80	77	75	74	72	71	68
235	176	145	125	111	101	93	88	82	78	75	74	72	70	69	66
240	172	142	122	109	99	92	86	80	76	74	72	71	69	68	65
245	168	139	120	106	97	90	84	78	75	72	71	69	67	66	64
250	165	136	117	104	95	88	82	77	73	71	69	68	66	65	62

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5w - Shear buckling strength q_w (N/mm^2) of a web (for $t \leq$ 16mm)

23) Grade Q690 steel, web thickness ≤ 16mm – design strength p_y = 630N/mm^2															
d/t	**Stiffener spacing ratio a/d**														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	378	378	378	378	378	378	376	362	352	345	340	335	329	325	315
60	378	378	378	378	378	371	359	344	333	325	319	314	307	303	292
65	378	378	378	378	371	355	341	325	313	305	298	293	285	281	269
70	378	378	378	375	356	339	324	307	294	285	278	272	265	260	250
75	378	378	378	361	341	323	307	288	274	265	259	254	247	243	233
80	378	378	371	347	326	306	290	270	257	249	243	238	231	228	219
85	378	378	359	333	311	290	272	254	242	234	228	224	218	214	206
90	378	377	347	320	296	274	257	240	229	221	216	212	206	202	194
95	378	367	335	306	280	259	244	227	217	209	204	201	195	192	184
100	378	356	323	292	266	247	231	216	206	199	194	191	185	182	175
105	378	346	311	279	253	235	220	206	196	189	185	182	176	173	167
110	376	335	298	266	242	224	210	196	187	181	177	173	168	166	159
115	367	325	286	254	231	214	201	188	179	173	169	166	161	158	152
120	359	314	274	244	222	205	193	180	171	166	162	159	154	152	146
125	350	304	263	234	213	197	185	173	165	159	155	153	148	146	140
130	341	293	253	225	205	190	178	166	158	153	149	147	142	140	135
135	333	283	243	217	197	183	171	160	152	147	144	141	137	135	130
140	324	272	235	209	190	176	165	154	147	142	139	136	132	130	125
145	316	263	227	202	184	170	160	149	142	137	134	132	128	126	121
150	307	254	219	195	177	164	154	144	137	133	129	127	123	121	117
155	298	246	212	189	172	159	149	139	133	128	125	123	119	117	113
160	290	238	205	183	166	154	145	135	129	124	121	119	116	114	109
165	281	231	199	177	161	149	140	131	125	121	118	116	112	110	106
170	272	224	193	172	157	145	136	127	121	117	114	112	109	107	103
175	265	218	188	167	152	141	132	123	118	114	111	109	106	104	100
180	257	212	183	162	148	137	129	120	114	111	108	106	103	101	97
185	250	206	178	158	144	133	125	117	111	108	105	103	100	98	95
190	244	201	173	154	140	130	122	114	108	105	102	100	97	96	92
195	237	196	169	150	136	126	119	111	106	102	100	98	95	93	90
200	231	191	164	146	133	123	116	108	103	99	97	95	93	91	87
205	226	186	160	143	130	120	113	105	100	97	95	93	90	89	85
210	220	182	157	139	127	117	110	103	98	95	92	91	88	87	83
215	215	177	153	136	124	115	108	100	96	93	90	89	86	85	81
220	210	173	149	133	121	112	105	98	94	90	88	87	84	83	80
225	206	169	146	130	118	110	103	96	91	88	86	85	82	81	78
230	201	166	143	127	116	107	101	94	89	87	84	83	81	79	76
235	197	162	140	124	113	105	99	92	88	85	83	81	79	77	74
240	193	159	137	122	111	103	96	90	86	83	81	79	77	76	73
245	189	156	134	119	109	101	94	88	84	81	79	78	76	74	71
250	185	153	131	117	106	99	93	86	82	80	78	76	74	73	70

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

Table 8.5x - Shear buckling strength q_w (N/mm^2) of a web (for 16mm < $t \leq$ 40mm)

24) Grade Q690 steel, web thickness >16mm ≤ 40mm – design strength p_y = 615N/mm^2															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	369	369	369	369	369	369	369	356	346	339	334	330	323	320	310
60	369	369	369	369	369	365	353	338	328	320	314	310	303	298	288
65	369	369	369	369	364	349	336	320	309	301	294	290	282	277	266
70	369	369	369	368	350	334	319	302	290	281	274	269	261	257	247
75	369	369	369	355	335	318	303	284	271	262	256	251	244	240	231
80	369	369	365	341	321	302	286	267	254	246	240	235	229	225	216
85	369	369	353	328	306	287	269	251	239	231	226	222	215	212	203
90	369	369	341	315	292	271	254	237	226	218	213	209	203	200	192
95	369	360	330	302	277	256	241	224	214	207	202	198	193	189	182
100	369	350	318	289	263	244	229	213	203	197	192	188	183	180	173
105	369	340	306	275	250	232	218	203	194	187	183	179	174	171	165
110	369	330	294	263	239	221	208	194	185	179	174	171	166	164	157
115	361	320	283	251	229	212	199	185	177	171	167	164	159	156	150
120	353	310	271	241	219	203	191	178	169	164	160	157	152	150	144
125	344	300	260	231	210	195	183	171	163	157	153	151	146	144	138
130	336	290	250	222	202	187	176	164	156	151	148	145	141	138	133
135	328	279	241	214	195	180	169	158	151	146	142	140	136	133	128
140	319	269	232	206	188	174	163	152	145	140	137	135	131	129	123
145	311	260	224	199	181	168	158	147	140	136	132	130	126	124	119
150	303	251	216	193	175	162	152	142	136	131	128	126	122	120	115
155	294	243	209	186	170	157	148	138	131	127	124	122	118	116	112
160	286	235	203	181	164	152	143	133	127	123	120	118	114	112	108
165	278	228	197	175	159	148	139	129	123	119	116	114	111	109	105
170	269	222	191	170	155	143	135	125	120	116	113	111	108	106	102
175	261	215	186	165	150	139	131	122	116	112	110	108	105	103	99
180	254	209	180	160	146	135	127	118	113	109	107	105	102	100	96
185	247	204	176	156	142	132	124	115	110	106	104	102	99	97	93
190	241	198	171	152	138	128	120	112	107	103	101	99	96	95	91
195	235	193	167	148	135	125	117	109	104	101	98	97	94	92	89
200	229	188	162	144	131	122	114	107	102	98	96	94	91	90	86
205	223	184	158	141	128	119	112	104	99	96	94	92	89	88	84
210	218	179	155	138	125	116	109	102	97	94	91	90	87	86	82
215	213	175	151	134	122	113	106	99	95	91	89	88	85	84	80
220	208	171	148	131	120	111	104	97	92	89	87	86	83	82	79
225	203	167	144	128	117	108	102	95	90	87	85	84	81	80	77
230	199	164	141	126	114	106	99	93	88	85	83	82	80	78	75
235	195	160	138	123	112	104	97	91	87	84	82	80	78	77	74
240	191	157	135	120	110	101	95	89	85	82	80	78	76	75	72
245	187	154	133	118	107	99	93	87	83	80	78	77	75	73	71
250	183	151	130	116	105	97	91	85	81	79	77	75	73	72	69

Note : For other steel grades not covered in Table 8.5, refer to Appendix 8.3.

8.4.8 End anchorage

End anchorage is not required under either one of the following conditions:

a) Shear capacity, but not shear buckling resistance, governs the design as,

$$V_c = V_w \tag{8.47}$$

b) Sufficient shear buckling resistance is available without forming the tension field action as,

$$V \le V_{cr} \tag{8.48}$$

in which V_{cr} is the critical shear buckling resistance without tension field given by,

If $V_w = V_c$, $\quad V_{cr} = V_c$ (8.49)

If $V_c > V_w > 0.72\,V_c$, $\quad V_{cr} = (9V_w - 2V_c)/7$ (8.50)

If $V_w \le 0.72\,V_c$, $\quad V_{cr} = (V_w / 0.9)^2 / V_c$ (8.51)

in which V is the maximum shear force, V_w is the simple shear buckling resistance in clause 8.4.6 and V_c is the shear capacity.

When neither of the above conditions is satisfied, literature on design of plate girders should be referred to.

8.4.9 Panels with openings

For design of panels with an opening of any dimension larger than 10% of the minimum panel dimension, reference to specialist literature should be made. The panel should not be used as an anchor panel and the adjacent panel should be designed as an end panel.

8.4.10 Web capacity and stiffeners design

Stiffeners should be provided for unstiffened webs subjected to local loads or reactions as follows. Only the intermediate stiffeners in clause 8.4.7 and load bearing and load carrying stiffeners are covered in the Code. For design of other types of stiffeners, specialist literature should be referred to.

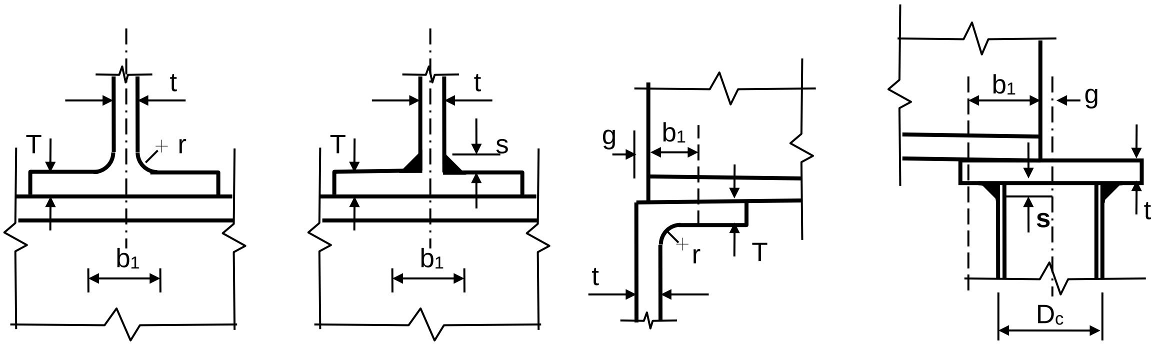
8.4.10.1 Maximum outstand of web stiffeners

Except for the outer edge of a web stiffener stiffened continuously, its outstand from the face of the web should not exceed $19\,\varepsilon\,t_s$.

If the outstand of a stiffener is larger than $13\,\varepsilon\,t_s$ but smaller than $19\,\varepsilon\,t_s$ its design should be based on an effective cross-section with an outstand of $13\,\varepsilon\,t_s$.

8.4.10.2 Stiff bearing length

The stiff bearing length b_1 is used in the width of stress bearing area and should be taken as the length of support that cannot deform appreciably in bending. To determine b_1, the dispersion of load through a steel bearing is shown in Figure 8.3. Dispersion at 45° through packs may be assumed provided that they are firmly fixed in position.



$b_1 = t + 1.6r + 2T$ $\quad$ $b_1 = t + 1.6s + 2T$ $\quad$ $b_1 = t + T + 0.8r - g$ $\quad$ $b_1 = 0.5D_c + t + 0.8s - g$

Figure 8.3 - Stiff bearing length

8.4.10.3 *Eccentricity*

When a load or reaction is applied eccentrically from the centreline of the web, or when the centroid of the stiffener does not lie on the centreline of the web, the resulting eccentricity of loading should be allowed for in design.

8.4.10.4 *Hollow sections*

Where concentrated loads are applied to hollow sections, consideration should be given to local stresses and deformations. The section should be reinforced or stiffened as necessary.

8.4.10.5 *Bearing capacity of webs*

8.4.10.5.1 Bearing capacity of unstiffened webs

Bearing stiffeners should be provided where the local compressive force F_x applied through a flange by loads or reactions exceeds the bearing capacity P_{bw} of the unstiffened web at the web-to-flange connection given by:

$$P_{bw} = (b_1 + nk)\, t\, p_{yw} \tag{8.52}$$

in which

at the ends of a member:

$$n = 2 + 0.6 b_e / k \quad \leq \; 5 \tag{8.53}$$

at other locations

- n = 5
- For rolled I- or H-sections: $k = T + r$ (8.54)
- For welded I- or H-sections: $k = T$ (8.55)

where

b_1 is the stiff bearing length, see clause 8.4.10.2;

b_e is the distance to the nearer end of the member from the end of the stiff bearing;

p_{yw} is the design strength of the web;

r is the root radius;

T is the flange thickness;

t is the web thickness.

8.4.10.5.2 Bearing capacity of stiffened web

Bearing stiffeners should be designed for the applied force F_x minus the bearing capacity P_{bw} of the unstiffened web. The capacity P_s of the stiffener should be obtained from:

$$P_s = A_{s.net} p_y \tag{8.56}$$

in which $A_{s.net}$ is the net cross-sectional area of the stiffener, allowing for cope holes for welding.

If the web and the stiffener have different design strengths, the smaller value should be used to calculate both the web capacity P_{bw} and the stiffener capacity P_s .

8.4.10.6 *Buckling resistance of webs*

8.4.10.6.1 Buckling resistance of unstiffened web

Load carrying web stiffeners should be provided where the local compressive force F_x applied through a flange by a load or reaction exceeds the buckling resistance of the web. P_x should be calculated as follows.

When the flange through which the load or reaction is applied is effectively restrained against both:

a) rotation relative to the web;

b) lateral movement relative to the other flange;

then provided that the distance a_e from the load or reaction to the nearer end of the member is at least 0.7d, the buckling resistance of the unstiffened web should be taken as P_x below:

$$P_x = \frac{25\,\varepsilon\,t}{\sqrt{(b_1 + nk)d}} P_{bw} \tag{8.57}$$

where

d is the depth of the web;

P_{bw} is the bearing capacity of the unstiffened web at the web-to-flange connection, from clause 8.4.10.5.1.

When the distance a_e from the load or reaction to the nearer end of the member is less than $0.7d$, the buckling resistance P_x of the web should be taken as:

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\,\varepsilon\,t}{\sqrt{(b_1 + nk)d}} P_{bw} \tag{8.58}$$

When the condition a) or b) is not met, the buckling resistance of the web should be reduced to P_{xr} given by:

$$P_{xr} = \frac{0.7d}{L_E} P_x \tag{8.59}$$

in which L_E is the effective length of the web, acting as a compression member or a part of a compression member.

8.4.10.6.2 Buckling resistance of load carrying stiffeners

Load carrying web stiffeners should be added where the local compressive stress f_{ed} on the compression edge of a web, due to loads or reactions applied through a flange between the web stiffeners already provided, exceeds the compressive strength for edge loading p_{ed}.

For this check, the stress f_{ed} on the compression edge of a web panel of depth d between two transverse stiffeners of spacing a should be calculated as follows:

(a) points loads and distributed loads of load-span shorter than the smaller panel dimension a or d should be divided by the smaller panel dimension;

(b) for a series of equally spaced and similar point loads, divide the largest load by the spacing, or by the smaller panel dimension if this is less;

(c) add the intensity (force/unit length) of any other distributed loads;

(d) divide the sum of (a), (b) or (c) by the web thickness t.

The compressive strength for edge loading p_{ed} should be calculated as follows:

- if the compression flange is restrained against rotation relative to the web:

$$p_{ed} = \left[2.75 + \frac{2}{(a/d)^2}\right] \frac{E}{(d/t)^2} \tag{8.60}$$

- if the compression flange is not restrained against rotation relative to the web:

$$p_{ed} = \left[1.0 + \frac{2}{(a/d)^2}\right] \frac{E}{(d/t)^2} \tag{8.61}$$

The load or reactions F_x on a load carrying stiffener should not exceed the buckling resistance P_x of the stiffener, given by:

$$P_x = A_s p_c \tag{8.62}$$

The effective area A_s of the load carrying stiffener should be taken as that of a cruciform cross-section made up from the effective area of the stiffeners themselves (see clause 8.4.10.1) together with an effective width of web on each side of the centreline of the stiffeners with limit of 15 times the web thickness t.

The compressive strength p_c should be determined from clause 8.7.6 using strut curve "c" (see Table 8.8) and the radius of gyration of the complete cruciform area A_s of the stiffener about its axis parallel to the web.

The design strength p_y should be taken as the lower value for the web or the stiffeners. The reduction of 20 N/mm^2 for welded sections in clause 8.7.6 should be applied when the stiffeners themselves are welded sections.

Provided that the flange through which the load or reaction is applied is effectively restrained against lateral movement relative to the other flange, the effective length L_E should be taken as follows:

(a) flanges are restrained against rotation in the plane of the stiffener by other structural elements:

L_E = 0.7 times the length L of the stiffener clear between flanges;

(b) flanges are not restrained in (a) above:

L_E = 1.0 times the length L of the stiffener clear between flanges.

If the load or reaction is applied to the flange by a compression member and the effective lateral restraint is provided at this point, the stiffener should be designed as part of the compression member applying the load and the connection should be checked for the effects of strut action.

If the stiffener also acts as an intermediate transverse stiffener to resist shear buckling, it should be checked for the effect of combined loads in accordance with clause 8.4.7.5.

Load carrying stiffeners should also be checked as bearing stiffeners, see clause 8.4.10.5.2.

8.4.11 Other types of stiffeners

Specialist literature should be referred to for design and use of various other types of stiffeners such as tension stiffeners, diagonal stiffeners, intermediate transverse web stiffeners, torsion stiffeners etc.

8.4.12 Connections between web stiffeners and webs

Stiffeners can be connected to webs by fitted bolts, preloaded bolts and welds. This connection should be designed to transmit a load equal to the least of the following:

a) The sum of forces at both ends when the forces are in the same direction.
b) The larger of the forces when they are in opposite directions.
c) The capacity of the stiffeners.

8.4.13 Connections between web stiffeners and flanges

8.4.13.1 Stiffeners in compression

Web stiffeners required to resist compression should be either (a) fitted against the loaded flange or (b) connected to it by continuous welds, fitted bolts or preloaded bolts designed for non-slip under factored loads.

The stiffener should be fitted against or connected to both flanges when any one of the following applies.

a) A load is directly above a support.
b) The stiffener forms the end stiffener of a stiffened web.
c) The stiffener acts as a torsional stiffener (refer to specialist literature for design of torsional stiffeners).

8.4.13.2 Stiffeners in tension

Web stiffeners required to resist tension should be connected to the flange transmitting the load or the reaction using continuous welds, fitted bolts or preloaded bolts designed to be non-slip under factored loads. This connection should be designed to resist the least of the applied load, reaction or the capacity of the stiffeners according to clause 8.4.10.5.2.

8.4.13.3 Length of web stiffeners

Bearing stiffeners or tension stiffeners performing a single function may be curtailed at a length such that the capacity P_{us} of the unstiffened web beyond the end of the stiffener is not less than that part of the applied load or reaction carried by the stiffener. P_{us} can be calculated as,

$$P_{us} = (b_1 + w)\, t\, p_{yw} \tag{8.63}$$

where

b_1 is the stiff bearing length in clause 8.4.10.2;

w is the length obtained by a dispersion at 45° to the level where the stiffener curtailed;

p_{yw} is the design strength of web.

8.5 BUCKLING RESISTANCE MOMENT FOR SINGLE ANGLE MEMBERS

An angle bent about an axis parallel to its leg should be checked against lateral-torsional buckling. It can be checked using the following method, or alternatively, and for unequal angle sections, by methods given in specialist literature or by carrying out buckling analysis.

For equal angle with $b/t \le 15\varepsilon$ and bent about x-axis, the resistance moment is given by,

$M_b = 0.8 p_y Z_x$ for heel of angle in compression (8.64)

$M_b = p_y Z_x \left(\dfrac{1350\varepsilon - L_E / r_v}{1625\varepsilon} \right) \le 0.8 p_y Z_x$ for heel of angle in tension (8.65)

where

L_E is the effective length from clause 8.3.4 using L_v as the distance between restraints against buckling about v-axis;

r_v is the radius of gyration about the v-axis;

Z_x is the smaller section modulus about x-axis.

8.6 TENSION MEMBERS

8.6.1 Tension capacity

The tension capacity P_t of a member is generally taken as,

$$P_t = p_y A_e \tag{8.66}$$

in which A_e is the sum of effective areas a_e of all elements in the cross-section in clause 9.3.4.4.

8.6.2 Members with eccentric connections

Members with eccentric connections can generally be considered as in uniaxial or biaxial bending and designed using clause 8.8. However, angles, channels or T-sections with eccentric end connections may be designed using a reduced tension capacity as follows.

8.6.3 Single and double angle, channel and T-sections

For a single angle section connected through one leg, a single channel through the web or a single T-section through the flange, the tension capacity should be obtained as,

for bolted connections, $P_t = p_y\,(A_e - 0.5 a_2)$ (8.67)

for welded connections, $P_t = p_y\,(A_e - 0.3 a_2)$ (8.68)

where

$a_2 = A_g - a_1$; (8.69)

A_g is the sum of gross cross-sectional area defined in clause 9.3.4.1 ;

a_1 is the gross area of the connected leg, taken as the product of thickness and the leg length of an angle, the depth of a channel or the flange width of a T-section.

For angles connected separately on the same side of a gusset or member, checking in clause 8.8 should also be applied.

8.6.4 Double angle, channel and T-sections with intermediate connections

For double angles, channels and T-sections connected on both sides of a gusset plate and interconnected by bolts or welds by at least 2 battens or solid packing pieces within their length, their tension capacity should be obtained as follows.

For bolted connections, $P_t = p_y \left(A_e - 0.25 a_2\right)$ (8.70)

For welded connections, $P_t = p_y \left(A_e - 0.15 a_2\right)$ (8.71)

8.7 COMPRESSION MEMBERS

8.7.1 Segment length

A segment length L of a compression member in any plane is defined as the length between the points at which the member is restrained against translation in that plane.

8.7.2 Effective length in general

Use of the effective length method for design shall satisfy the condition in clause 6.6. The effective length L_E of a compression member should be taken as the equivalent length of a pin-ended member with identical buckling resistance. When necessary, an imaginarily extended member length is required for estimation of this effective length. A general equivalent length factor is indicated in Table 8.6. Refer to clause 6.6.4 for slenderness ratio limits.

Except for angles, channels and T-sections designed to clause 8.7.9, the effective length L_E of a compression member or its buckling effects should be determined from a buckling analysis or other recognised methods. The guidelines given below may be used to determine the effective length factor for simple columns whose boundary conditions which can be reliably approximated in design.

a) A restraining member carrying more than 90% of its capacity to clause 8.9 should not be considered as being capable of providing a lateral directional restraint.

b) The effective length of continuous columns in multi-storey frames depends on conditions of restraint in the relevant plane, directional and rotational restraints, connection stiffness and member stiffness. Clause 6.6.3 should be referred to for determination of effective length factor.

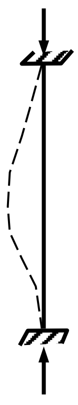
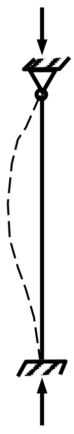
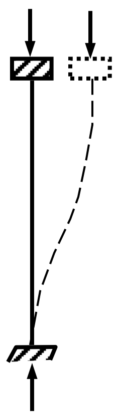
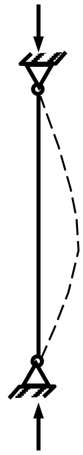
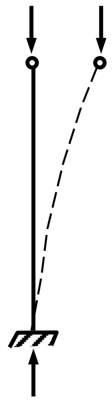
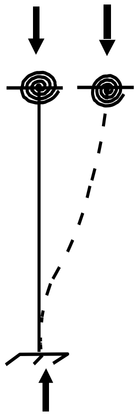
c) Columns under a variable axial force along their lengths and varying sectional properties should be designed using an analytical method, a second-order analysis or an advanced plastic analysis.

d) The effective length recommended for design differs from the theoretical values in Table 8.6 because of uncertainty in boundary assumptions.

e) The buckling resistance of sloping members with moment-resisting connections cannot be determined by the effective length method or the moment amplification method, because of possible snap-through buckling. Second-order or advanced analysis should be used in this case.

For more accurate assessments of buckling resistance of compression members, elastic critical load analysis, second-order P-Δ-δ analysis or advanced analysis should be used. The P-Δ–δ analysis should be carried out as an alternative to the effective length method for the analysis and design of structures with ultimate load affected considerably by buckling. Columns with both ends pinned in a sway frame or leaning should not be designed by the effective length method.

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.				

8.7.3 Restraints

A restraint should have adequate strength and stiffness to limit movement of the restrained point in position, direction or both. A positional restraint should have adequate stiffness to prevent significant lateral displacement and adequate strength to resist a minimum of 1% of the axial force in the member being restrained. When the bracing member is restraining more than one member, a reduction factor should be applied to the resistance force requirement of the restraint as follows:-

$$k_r = \sqrt{0.2 + \frac{1}{N_r}} \leq 1 \qquad (8.72)$$

in which N_r is the number of parallel members being restrained.

8.7.4 Slenderness

The slenderness of a compression member should be taken as the effective length divided by the radius of gyration about the axis considered for buckling.

8.7.5 Compression resistance

The compression resistance P_c of a member should be obtained from,

For Class 1 plastic, Class 2 compact and Class 3 semi-compact cross sections,

$$P_c = A_g p_c \qquad (8.73)$$

For Class 4 slender cross-sections,

$$P_c = A_{eff} p_{cs} \quad (8.74)$$

in which

A_{eff} is the effective cross-sectional area in clause 7.6;

A_g is the sum of gross sectional area in clause 9.3.4.1;

p_c is the compressive strength in clause 8.7.6;

p_{cs} is the value of p_c obtained using a reduced slenderness of $\lambda\sqrt{\frac{A_{eff}}{A_g}}$ where λ is the slenderness ratio calculated from the radius of gyration of the gross sectional area and effective length.

8.7.6 Compressive strength

The compressive strength p_c should be based on the type of section, design strength, slenderness and a suitable buckling curve a_0, a, b, c or d that should be selected from Table 8.7. The value p_c for these buckling curves should be obtained from Tables 8.8(a_0), 8.8(a) to 8.8(h) and Figure 8.4 or, alternatively, by the formulae in Appendix 8.4.

For welded I, H or box sections, p_y should be reduced by 20 N/mm^2 and p_c should then be determined on the basis of this reduced p_y.

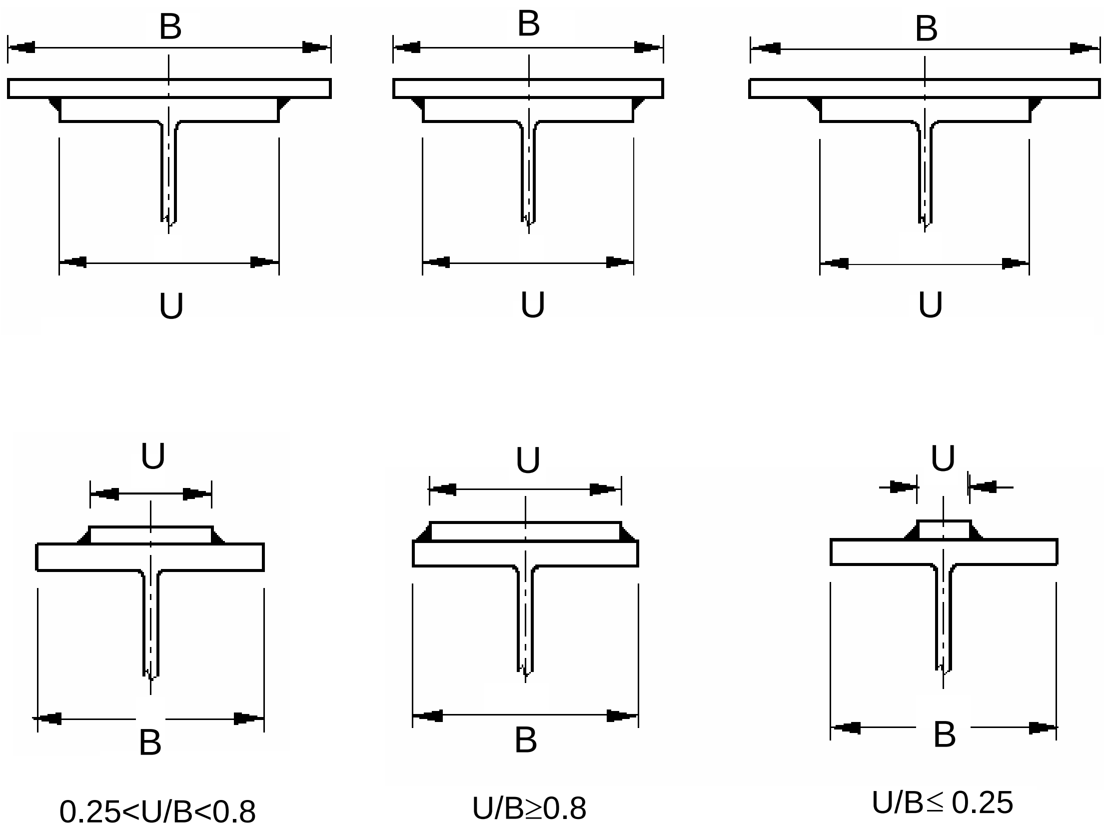


Figure 8.4 - Rolled I- or H-sections with welded flange plates

Table 8.7 - Designation of buckling curves for different section types

Type of section	Maximum thickness (see note1)	Axis of buckling	
		x-x	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 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 λ < 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

Note : For other steel grades, refer to Appendix 8.4.

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

1) Values of p_c in N/mm^2 with λ < 110 for strut curve a															
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	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

Note : For other steel grades, refer to Appendix 8.4.

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

2) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve a															
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
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

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel

3) Values of p_c in N/mm^2 with λ < 110 for strut curve b

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	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

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel

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)														
λ	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

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel

5) Values of p_c in N/mm^2 with λ < 110 for strut curve b						
λ	**Steel grade and design strength p_y (N/mm^2)**					
	S550			**S690**		
	490	530	550	630	650	690
15	486	525	544	621	641	679
20	477	515	534	609	628	665
25	467	504	522	595	613	650
30	456	491	509	579	596	631
35	443	477	494	560	576	608
40	428	460	476	537	551	580
42	422	452	467	526	540	567
44	415	444	459	514	528	553
46	408	436	449	502	514	538
48	400	427	440	489	500	522
50	391	417	429	475	485	505
52	383	407	418	460	470	488
54	373	396	406	445	454	470
56	364	385	394	429	437	452
58	354	373	382	414	421	434
60	344	361	369	398	404	416
62	333	349	357	382	388	398
64	323	337	344	367	372	381
66	312	326	332	352	356	364
68	302	314	319	337	341	348
70	292	302	307	323	327	333
72	281	291	295	310	313	319
74	271	280	284	297	300	305
76	262	270	273	285	287	292
78	252	259	263	273	275	279
80	243	250	252	262	264	268
82	234	240	243	251	253	257
84	226	231	234	241	243	246
86	218	223	225	232	233	236
88	210	214	216	223	224	227
90	202	206	208	214	216	218
92	195	199	201	206	207	209
94	188	192	193	199	200	201
96	182	185	186	191	192	194
98	176	179	180	184	185	187
100	170	172	174	178	179	180
102	164	167	168	171	172	174
104	159	161	162	165	166	168
106	153	156	157	160	160	162
108	148	150	151	154	155	156

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel (cont'd)

5) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve b						
λ	**Steel grade and design strength p_y (N/mm^2)**					
	S550			**S690**		
	490	530	550	630	650	690
110	144	146	146	149	150	151
112	139	141	142	144	145	146
114	135	136	137	140	140	141
116	131	132	133	135	136	137
118	127	128	129	131	132	132
120	123	124	125	127	127	128
122	119	121	121	123	124	124
124	116	117	118	119	120	121
126	112	114	114	116	116	117
128	109	110	111	113	113	114
130	106	107	108	109	110	110
135	99	100	100	102	102	103
140	93	93	94	95	95	96
145	87	87	88	89	89	89
150	81	82	82	83	83	84
155	76	77	77	78	78	79
160	72	73	73	74	74	74
165	68	68	69	69	70	70
170	64	65	65	65	66	66
175	61	61	61	62	62	62
180	58	58	58	59	59	59
185	55	55	55	56	56	56
190	52	52	52	53	53	53
195	49	50	50	50	50	51
200	47	47	47	48	48	48
210	43	43	43	44	44	44
220	39	39	39	40	40	40
230	36	36	36	36	37	37
240	33	33	33	34	34	34
250	31	31	31	31	31	31
260	28	28	29	29	29	29
270	26	26	26	27	27	27
280	25	25	25	25	25	25
290	23	23	23	23	23	23
300	21	22	22	22	22	22
310	20	20	20	20	20	20
320	19	19	19	19	19	19
330	18	18	18	18	18	18
340	17	17	17	17	17	17
350	16	16	16	16	16	16

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel

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

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel (cont'd)

6) Values of p_c in N/mm² with $\lambda \geq 110$ for strut curve c															
λ	Steel grade and design strength p_y (N/mm²)														
	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

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel

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)**					
	S550			**S690**		
	490	530	550	630	650	690
15	484	522	541	617	636	673
20	470	506	525	598	616	652
25	455	490	507	577	595	629
30	439	472	489	554	571	603
35	421	452	468	529	544	573
40	402	431	445	500	513	539
42	394	421	435	487	500	524
44	385	412	425	474	486	509
46	377	402	414	461	472	493
48	368	392	403	447	457	477
50	359	381	392	432	442	460
52	349	370	380	418	427	443
54	340	359	369	404	412	427
56	330	348	357	389	396	410
58	320	337	345	375	381	394
60	310	326	334	361	367	378
62	301	315	322	347	352	362
64	291	305	311	333	338	347
66	281	294	300	320	325	333
68	272	284	289	308	312	319
70	263	273	278	295	299	306
72	254	264	268	284	287	293
74	245	254	258	272	276	281
76	237	245	249	262	265	270
78	228	236	240	252	254	259
80	221	228	231	242	244	248
82	213	219	222	232	235	239
84	206	212	214	224	226	229
86	199	204	207	215	217	220
88	192	197	199	207	209	212
90	185	190	192	200	201	204
92	179	184	186	192	194	197
94	173	177	179	186	187	189
96	167	171	173	179	180	183
98	162	166	167	173	174	176
100	157	160	162	167	168	170
102	152	155	156	161	162	164
104	147	150	151	156	157	158
106	142	145	146	151	152	153
108	138	141	142	146	147	148

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel (cont'd)

5) Values of p_c in N/mm^2 with $\lambda \geq 110$ for strut curve c						
λ	**Steel grade and design strength p_y (N/mm^2)**					
	S550			**S690**		
	490	530	550	630	650	690
110	134	136	137	141	142	143
112	130	132	133	137	137	139
114	126	128	129	132	133	134
116	122	124	125	128	129	130
118	119	120	121	124	125	126
120	115	117	118	121	121	122
122	112	114	114	117	118	119
124	109	110	111	114	114	115
126	106	107	108	110	111	112
128	103	104	105	107	108	109
130	100	101	102	104	105	106
135	93	95	95	97	98	98
140	88	89	89	91	91	92
145	82	83	84	85	86	86
150	77	78	79	80	80	81
155	73	74	74	75	75	76
160	69	69	70	71	71	72
165	65	66	66	67	67	67
170	61	62	62	63	63	64
175	58	59	59	60	60	60
180	55	56	56	57	57	57
185	52	53	53	54	54	54
190	50	50	51	51	51	52
195	47	48	48	49	49	49
200	45	46	46	46	47	47
210	41	42	42	42	42	43
220	38	38	38	39	39	39
230	35	35	35	36	36	36
240	32	32	32	33	33	33
250	30	30	30	30	30	30
260	28	28	28	28	28	28
270	26	26	26	26	26	26
280	24	24	24	24	24	24
290	22	22	22	23	23	23
300	21	21	21	21	21	21
310	20	20	20	20	20	20
320	18	19	19	19	19	19
330	17	17	18	18	18	18
340	16	16	17	17	17	17
350	16	16	16	16	16	16

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel

7) Values of p_c in N/mm^2 with λ < 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

Note : For other steel grades, refer to Appendix 8.4.

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

i) S275 ~ S460 steel (cont'd)

8) Values of p_c in N/mm^2 with $\lambda \geq 110$ 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
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

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel

5) 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)					
	S550			S690		
	490	530	550	630	650	690
15	481	518	537	611	629	666
20	461	496	514	584	602	637
25	440	474	490	557	573	606
30	419	450	466	527	542	572
35	397	426	440	496	509	536
40	374	400	413	463	475	498
42	365	390	402	449	460	482
44	356	379	391	435	446	466
46	346	369	380	421	431	450
48	337	358	368	407	416	434
50	327	347	357	393	402	418
52	317	336	345	379	387	403
54	308	326	334	366	373	387
56	298	315	323	352	359	372
58	289	305	312	339	346	357
60	280	294	301	327	332	343
62	271	284	291	314	319	329
64	262	275	281	302	307	316
66	253	265	271	291	295	304
68	245	256	261	279	284	291
70	237	247	252	269	273	280
72	229	238	243	258	262	269
74	221	230	234	249	252	258
76	214	222	226	239	242	248
78	206	214	218	230	233	238
80	200	207	210	222	224	229
82	193	200	203	214	216	220
84	186	193	196	206	208	212
86	180	186	189	198	201	204
88	174	180	183	191	193	197
90	169	174	176	185	186	190
92	163	168	170	178	180	183
94	158	163	165	172	174	177
96	153	157	159	166	168	170
98	148	152	154	161	162	165
100	144	148	149	155	157	159
102	139	143	145	150	152	154
104	135	139	140	145	147	149
106	131	134	136	141	142	144
108	127	130	132	136	137	139

Note : For other steel grades, refer to Appendix 8.4.

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

ii) S550 ~ S690 steel (cont'd)

5) 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)					
λ	S550			S690		
	490	530	550	630	650	690
110	123	126	128	132	133	135
112	120	123	124	128	129	131
114	116	119	120	124	125	127
116	113	116	117	121	121	123
118	110	112	113	117	118	119
120	107	109	110	114	114	116
122	104	106	107	110	111	112
124	101	103	104	107	108	109
126	98	100	101	104	105	106
128	96	98	99	101	102	103
130	93	95	96	99	99	100
135	87	89	90	92	93	94
140	82	84	84	86	87	88
145	77	78	79	81	81	82
150	73	74	74	76	77	77
155	69	70	70	72	72	73
160	65	66	66	68	68	69
165	61	62	63	64	64	65
170	58	59	59	61	61	61
175	55	56	56	57	58	58
180	52	53	53	54	55	55
185	50	51	51	52	52	52
190	48	48	48	49	49	50
195	45	46	46	47	47	47
200	43	44	44	45	45	45
210	40	40	40	41	41	41
220	36	37	37	37	38	38
230	33	34	34	34	34	35
240	31	31	31	32	32	32
250	29	29	29	29	29	30
260	27	27	27	27	27	27
270	25	25	25	25	25	26
280	23	23	23	24	24	24
290	22	22	22	22	22	22
300	20	20	20	21	21	21
310	19	19	19	19	19	20
320	18	18	18	18	18	18
330	17	17	17	17	17	17
340	16	16	16	16	16	16
350	15	15	15	15	15	15

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with $\lambda < 110$ for strut curve a					
	Steel grade and design strength (N/mm²)				
λ	Q235	Q345/Q355	Q390	Q420	Q460
	215	305	345	375	410
15	215	305	345	375	409
20	215	303	342	371	405
25	212	299	338	366	400
30	210	296	333	362	395
35	208	292	329	357	389
40	205	287	324	350	382
42	204	285	321	348	378
44	203	283	319	345	375
46	201	281	316	342	371
48	200	279	313	338	367
50	199	277	310	335	363
52	198	274	307	331	358
54	196	271	303	327	353
56	195	268	300	322	347
58	193	265	295	317	341
60	191	262	291	312	334
62	190	259	286	306	327
64	188	255	281	300	320
66	186	251	276	293	312
68	184	247	270	287	303
70	182	242	265	279	295
72	180	237	258	272	286
74	178	233	252	265	277
76	175	227	245	257	268
78	173	222	239	249	259
80	170	217	232	241	251
82	168	211	225	234	242
84	165	206	219	226	234
86	162	200	212	219	225
88	159	195	205	212	218
90	156	189	199	205	210
92	153	184	193	198	203
94	150	179	187	191	196
96	147	173	181	185	189
98	144	168	175	179	182
100	141	163	169	173	176
102	137	158	164	167	170
104	134	154	159	162	165
106	131	149	154	157	159
108	128	145	149	152	154

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with $\lambda \geq$ **110** for strut curve a					
λ	Steel grade and design strength (N/mm²)				
	Q235	Q345/Q355	Q390	Q420	Q460
	215	305	345	375	410
110	125	140	144	147	149
112	122	136	140	142	144
114	119	132	136	138	140
116	116	129	132	134	135
118	113	125	128	129	131
120	111	121	124	126	127
122	108	118	120	122	123
124	105	115	117	118	120
126	103	111	114	115	116
128	100	108	110	112	113
130	98	105	107	108	110
135	92	98	100	101	102
140	86	92	94	95	95
145	81	86	88	88	89
150	77	81	82	83	84
155	73	76	77	78	79
160	69	72	73	73	74
165	65	68	69	69	70
170	62	64	65	65	66
175	58	61	61	62	62
180	55	58	58	59	59
185	53	55	55	56	56
190	50	52	53	53	53
195	48	50	50	50	51
200	46	47	48	48	48
210	42	43	43	44	44
220	38	39	40	40	40
230	35	36	36	36	37
240	32	33	33	34	34
250	30	31	31	31	31
260	28	28	29	29	29
270	26	26	27	27	27
280	24	25	25	25	25
290	23	23	23	23	23
300	21	22	22	22	22
310	20	20	20	20	20
320	19	19	19	19	19
330	18	18	18	18	18
340	17	17	17	17	17
350	16	16	16	16	16

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with λ < 110 for strut curve b							
	Steel grade and design strength (N/mm²)						
λ	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	520	630
15	215	305	345	375	409	515	621
20	215	301	339	368	401	505	609
25	211	295	332	360	393	494	595
30	207	289	325	353	384	482	579
35	202	283	318	344	374	468	560
40	198	276	310	335	364	452	537
42	196	273	306	331	359	445	526
44	194	270	302	326	354	437	514
46	193	267	298	322	349	429	502
48	191	263	294	317	343	420	489
50	189	260	290	312	337	411	475
52	187	256	286	307	331	401	460
54	185	253	281	302	325	390	445
56	182	249	276	296	318	380	429
58	180	245	271	290	311	368	414
60	178	241	266	284	304	357	398
62	176	236	261	278	296	346	382
64	173	232	255	271	289	334	367
66	171	227	249	265	281	322	352
68	168	223	244	258	273	311	337
70	166	218	238	251	265	300	323
72	163	213	232	244	257	289	310
74	161	208	226	237	249	278	297
76	158	204	220	230	241	268	285
78	155	199	214	224	234	258	273
80	153	194	208	217	226	248	262
82	150	189	202	210	219	239	251
84	147	184	196	204	212	230	241
86	144	179	190	198	205	221	232
88	141	174	185	191	198	213	223
90	138	169	179	185	192	206	214
92	136	165	174	180	185	198	206
94	133	160	169	174	179	191	199
96	130	156	164	169	173	184	191
98	127	151	159	163	168	178	184
100	124	147	154	158	162	172	178
102	121	143	149	153	157	166	171
104	119	139	145	149	152	160	165
106	116	135	141	144	148	155	160
108	113	131	137	140	143	150	154

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with $\lambda \geq$ **110** for strut curve b							
λ	Steel grade and design strength (N/mm²)						
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	520	630
110	111	128	133	136	139	145	149
112	108	124	129	132	134	140	144
114	106	121	125	128	130	136	140
116	103	117	122	124	126	132	135
118	101	114	118	120	123	128	131
120	99	111	115	117	119	124	127
122	96	108	112	114	116	120	123
124	94	105	109	111	112	117	119
126	92	103	106	107	109	113	116
128	90	100	103	105	106	110	113
130	88	97	100	102	103	107	109
135	83	91	94	95	97	100	102
140	78	86	88	89	90	93	95
145	74	81	83	84	85	87	89
150	70	76	78	79	80	82	83
155	66	72	73	74	75	77	78
160	63	68	69	70	71	72	74
165	60	64	65	66	67	68	69
170	57	61	62	62	63	65	65
175	54	58	59	59	60	61	62
180	51	55	56	56	57	58	59
185	49	52	53	53	54	55	56
190	47	49	50	51	51	52	53
195	45	47	48	48	49	50	50
200	43	45	46	46	46	47	48
210	39	41	42	42	42	43	44
220	36	38	38	38	39	39	40
230	33	35	35	35	36	36	36
240	31	32	32	32	33	33	34
250	28	30	30	30	30	31	31
260	26	27	28	28	28	28	29
270	25	26	26	26	26	26	27
280	23	24	24	24	24	25	25
290	22	22	22	23	23	23	23
300	20	21	21	21	21	23	23
310	19	20	20	20	20	20	20
320	18	18	19	19	19	19	19
330	17	17	17	18	18	18	18
340	16	16	17	17	17	17	17
350	15	16	16	16	16	16	16

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with λ < 110 for strut curve c							
λ	Steel grade and design strength (N/mm²)						
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	520	630
15	215	305	345	374	408	512	617
20	214	299	336	364	396	497	598
25	208	290	326	353	384	481	577
30	202	281	315	341	371	464	554
35	196	272	305	329	357	445	529
40	190	262	293	316	343	424	500
42	187	258	288	311	337	415	487
44	185	254	284	305	330	405	474
46	182	250	279	300	324	396	461
48	180	246	274	294	317	386	447
50	177	241	268	288	310	375	432
52	174	237	263	282	303	365	418
54	172	232	258	276	296	354	404
56	169	228	252	270	289	344	389
58	166	223	247	263	282	333	375
60	164	219	241	257	274	322	361
62	161	214	236	251	267	312	347
64	158	210	230	244	260	301	333
66	155	205	224	238	252	291	320
68	152	200	219	231	245	281	308
70	150	195	213	225	238	271	295
72	147	191	207	219	231	261	284
74	144	186	202	212	223	252	272
76	141	181	196	206	217	243	262
78	139	177	191	200	210	234	252
80	136	172	185	194	203	226	242
82	133	168	180	188	197	218	232
84	130	163	175	183	191	210	224
86	128	159	170	177	185	203	215
88	125	155	165	172	179	196	207
90	122	151	161	167	173	189	200
92	120	147	156	162	168	183	192
94	117	143	151	157	163	176	186
96	114	139	147	152	158	170	179
98	112	135	143	148	153	165	173
100	109	132	139	143	148	159	167
102	107	128	135	139	144	154	161
104	105	125	131	135	139	149	156
106	102	121	127	131	135	144	151
108	100	118	124	128	131	140	146

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with $\lambda \geq$ **110** for strut curve c							
λ	Steel grade and design strength (N/mm²)						
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	520	630
110	98	115	120	124	127	136	141
112	96	112	117	120	124	131	137
114	93	109	114	117	120	127	132
116	91	106	111	114	117	124	128
118	89	104	108	111	114	120	124
120	87	101	105	108	110	117	121
122	85	98	102	105	107	113	117
124	84	96	100	102	104	110	114
126	82	93	97	99	102	107	110
128	80	91	95	97	99	104	107
130	78	89	92	94	96	101	104
135	74	84	87	88	90	94	97
140	70	79	81	83	85	88	91
145	66	74	77	78	80	83	85
150	63	70	72	74	75	78	80
155	60	66	68	69	71	73	75
160	57	63	65	66	67	69	71
165	54	60	61	62	63	65	67
170	52	57	58	59	60	62	63
175	49	54	55	56	57	59	60
180	47	51	52	53	54	56	57
185	45	49	50	50	51	53	54
190	43	46	47	48	49	50	51
195	41	44	45	46	46	48	49
200	39	42	43	44	44	46	46
210	36	39	40	40	40	42	42
220	33	36	36	37	37	38	39
230	31	33	33	34	34	35	36
240	29	30	31	31	31	32	33
250	27	28	29	29	29	30	30
260	25	26	27	27	27	28	28
270	23	24	25	25	25	26	26
280	22	23	23	23	24	24	24
290	20	21	22	22	22	22	23
300	19	20	20	20	21	21	21
310	18	19	19	19	19	20	20
320	17	18	18	18	18	19	19
330	16	17	17	17	17	17	18
340	15	16	16	16	16	16	17
350	14	15	15	15	15	16	16

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with λ < 110 for strut curve d							
λ	Steel grade and design strength (N/mm²)						
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	520	630
15	215	305	345	374	407	509	611
20	214	296	332	359	390	488	584
25	205	283	318	343	373	466	557
30	197	271	304	328	356	443	527
35	188	259	290	312	339	419	496
40	180	247	275	297	321	394	463
42	177	242	270	290	314	384	449
44	174	237	264	284	306	373	435
46	171	232	258	277	299	363	421
48	168	227	252	271	291	353	407
50	164	222	247	264	284	342	393
52	161	217	241	258	277	332	379
54	158	213	235	251	269	321	366
56	155	208	229	245	262	311	352
58	152	203	224	238	255	301	339
60	149	198	218	232	247	291	327
62	146	193	212	226	240	281	314
64	143	189	207	219	233	272	302
66	140	184	201	213	226	262	291
68	137	179	196	207	220	253	279
70	134	175	190	201	213	244	269
72	132	170	185	195	207	236	258
74	129	166	180	190	200	228	249
76	126	162	175	184	194	220	239
78	123	157	170	179	188	212	230
80	121	153	165	174	182	205	222
82	118	149	161	169	177	198	214
84	115	145	156	164	171	191	206
86	113	142	152	159	166	185	198
88	110	138	148	154	161	179	191
90	108	134	144	150	156	173	185
92	105	131	139	145	152	167	178
94	103	127	136	141	147	162	172
96	101	124	132	137	143	156	166
98	99	121	128	133	138	151	161
100	96	117	125	129	134	147	155
102	94	114	121	126	131	142	150
104	92	111	118	122	127	138	145
106	90	109	115	119	123	134	141
108	88	106	112	116	120	130	136

Note : For other steel grades, refer to Appendix 8.4.

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

Values of p_c in N/mm² with $\lambda \geq$ **110** for strut curve d							
λ	Steel grade and design strength (N/mm²)						
	Q235	Q345/Q355	Q390	Q420	Q460	Q550	Q690
	215	305	345	375	410	550	690
110	86	103	109	113	116	126	132
112	84	101	106	110	113	122	128
114	82	98	103	107	110	118	124
116	81	96	101	104	107	115	121
118	79	93	98	101	104	112	117
120	77	91	95	98	101	109	114
122	76	89	93	96	99	106	110
124	74	87	91	93	96	103	107
126	72	85	89	91	94	100	104
128	71	83	86	89	91	97	101
130	69	81	84	87	89	95	99
135	66	76	79	81	84	89	92
140	62	72	75	77	79	83	86
145	59	68	71	72	74	78	81
150	56	64	67	68	70	74	76
155	54	61	63	65	66	69	72
160	51	58	60	61	62	66	68
165	49	55	57	58	59	62	64
170	47	52	54	55	56	59	61
175	45	50	51	52	53	56	57
180	43	47	49	50	51	53	54
185	41	45	47	47	48	50	52
190	39	43	44	45	46	48	49
195	37	41	42	43	44	46	47
200	36	40	41	41	42	44	45
210	33	36	37	38	38	40	41
220	31	33	34	35	35	37	37
230	28	31	32	32	33	34	34
240	26	29	29	30	30	31	32
250	25	27	27	28	28	29	29
260	23	25	25	26	26	27	27
270	22	23	24	24	24	25	25
280	20	22	22	22	23	23	24
290	19	20	21	21	21	22	22
300	18	19	19	20	20	20	21
310	17	18	18	18	19	19	19
320	16	17	17	17	18	18	18
330	15	16	16	16	17	17	17
340	14	15	15	16	16	16	16
350	14	14	15	15	15	15	15

Note : For other steel grades, refer to Appendix 8.4.

8.7.7 Eccentric connections

The effect of eccentric connections should be considered explicitly using the formulae for combined loads given in clause 8.9. Angles, channels and T-sections can also be designed using clause 8.7.9.

8.7.8 Simple construction

Pattern loads need not be considered in the design of simple structures as defined in clause 6.1. For column design, all beams are assumed fully loaded and simply supported on columns.

The nominal moment on columns due to simply supported beams should be calculated as follows:

(a) For beams resting on a cap plate, the reaction should be taken as acting on the face of the column or edge of the packing towards the centre of the beam packing used.
(b) For a roof truss resting on a cap plate, eccentricity can be neglected if the centre of reaction is at the centre of the column.
(c) 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.
(d) For other cases not covered above, the actual eccentricities should be used.

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

All equivalent moment factors in columns should be taken as unity. The column should be checked against the combined load condition using clause 8.9, with the effective length determined from clause 6.6.3 and taking the effective slenderness for lateral-torsional buckling λ_{LT} as,

$$\lambda_{LT} = \frac{0.5L}{r_y} \tag{8.75}$$

where

L is the length of the column between lateral supports or the storey height; and
r_y is the radius of gyration about the minor axis.

8.7.9 Effective length of sections in triangulated structures and trusses

Angles, channels and T-sections are normally connected eccentrically and with different degrees of connection stiffness by welding or by one, two or more bolts. Buckling curve “c” in Table 8.7 should be used and the effective lengths of the sections must be carefully determined as follows or by a rational analysis such as a second-order analysis. The use of the following formulae is based on the assumption that the two ends of the members are effectively restrained against translational movement.

For out-of-plane buckling of chord members, the effective length L_E can be taken as the distance between lateral restraints unless other values can be justified by a buckling or second-order analysis. For chord members and out-of-plane buckling of web members, the effective length L_E is taken as the member length unless a smaller value can be justified by a buckling or second-order analysis.

For web members, buckling about principal axes and axes parallel to the legs should be considered. For angle sections connected by two or more bolts, the slenderness ratio should be calculated from the following:

For buckling about v-v axis, $\lambda_{eff,v} = 0.35 \times 85.8\varepsilon + 0.7\lambda_v$ or λ_v whichever is larger

For buckling about x-x axis, $\lambda_{eff,x} = 0.5 \times 85.8\varepsilon + 0.7\lambda_x$ or λ_x whichever is larger (8.76)

For buckling about y-y axis, $\lambda_{eff,y} = 0.5 \times 85.8\varepsilon + 0.7\lambda_y$ or λ_y whichever is larger

in which $\varepsilon = \sqrt{\frac{275}{p_y}}$ and λ_{eff} is the effective slenderness ratio. λ_v, λ_x and λ_y are respectively the slenderness ratios about the minor v-axis and the x- and y-axes of the angle sections.

For a single bolt connection, 80% of the axial force compression resistance of the double bolt connection should be used.

For short members, the effect of load eccentricity should be considered analytically. Alternatively, the buckling strength of these sections can be designed as for other columns using the combined axial force and moment equation in clause 8.9 or by a second-order analysis allowing for eccentric connections and member imperfections such as using an equivalent member imperfection giving the same buckling strength curve “c” of Table 8.7.

Hollow sections used as members in trusses can be connected either by welding or by bolting. The in-plane effective length can be taken as the distance between connection nodes unless other values can be justified by a buckling or second-order analysis. The out-of-plane buckling resistance can be greatly enhanced by consideration of the torsional stiffness of hollow sections. This effect can be considered in a second-order or an advanced analysis. Curve “a” in Table 8.7 for hot rolled hollow sections or curve “c” for cold-formed hollow sections should be used for checking buckling strength.

8.8 TENSION MEMBERS UNDER COMBINED AXIAL FORCE AND MOMENTS

The cross-section capacity of a tension member under biaxial moment should be checked using the following equation:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \qquad (8.77)$$

in which,

F_t is the design axial tension at critical location;

M_{cx} is the moment capacity about the major axis from clause 8.2.2;

M_{cy} is the moment capacity about the minor axis from clause 8.2.2;

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;

P_t is the tension capacity given in clause 8.6.1.

Alternatively, for members of Class 1 plastic or Class 2 compact and symmetrical I and rectangular hollow cross-sections, a more exact check can be carried out by formulae in literature or a sectional strength analysis based on an assumption as follows. Area in web, and partly in flanges if web area is not sufficient to resist axial compression, is allocated to take the design axial compression and remaining area in webs and flanges is to resist moment in order to obtain the reduced moment capacities under axial compression. Under this analysis, Equation 8.77, with first term for axial compression ignored due to its inclusion in moment capacities, should be satisfied.

For more exact design of asymmetric and mono-symmetrical sections, maximum stresses due to moments about principal axes may not occur at the same location and the maximum stress of the whole cross section can be determined from the stresses computed from axial compression and individual moments with their respective moduli.

Tension members should be checked against lateral-torsional buckling by considering the design moment alone.

8.9 COMPRESSION MEMBERS UNDER COMBINED AXIAL FORCE AND MOMENTS

Compression members should be checked for cross-sectional capacity and member buckling resistance as follows:

8.9.1 Cross-section capacity

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

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1 \tag{8.78}$$

in which M_x and M_y are the design moments about the x- and y-axes. M_{cx} and M_{cy} are the moment capacities about the x- and y-axes.

For Class 4 slender cross-sections, the effective area A_{eff} should be used in place of the gross sectional area.

Alternatively, for members of Class 1 plastic or Class 2 compact and symmetrical I and rectangular hollow cross-sections, a more exact check can be carried out by formulae in literature or the sectional strength analysis described in clause 8.8.

For more exact design of asymmetric and mono-symmetrical sections, clause 8.8 should be referred.

8.9.2 Member buckling resistance

For Class 4 slender section, the effective moduli $Z_{x,eff}$ and $Z_{y,eff}$ should be used in place of the elastic moduli Z_x and Z_y in the following cases. When using the P-Δ-δ analysis allowing for initial imperfection and beam buckling in clause 6.8.3 a check for member buckling resistance in this clause is not required.

The resistance of the member of a sway frame as defined in clause 6.3 can be checked using,

$$\frac{F_c}{P_c} + \frac{m_x \overline{M}_x}{M_{cx}} + \frac{m_y \overline{M}_y}{M_{cy}} \le 1 \tag{8.79}$$

$$\frac{F_c}{\overline{P}_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} \le 1 \tag{8.80}$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} \le 1 \tag{8.81}$$

in which,

- F_c is the design axial compression at the critical location;
- 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-δ effect about the major x-axis (see below);
- M_y is the maximum design moment amplified for the P-Δ-δ effect about the minor y-axis (see below);
- $\overline{M}_x$ is the maximum first-order linear design moment about the major x-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-δ effect about major x-axis governing M_b (see below);
- $\overline{P}_c$ 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.

The effects of moment amplification are automatically considered in a second-order P-Δ-δ analysis. Alternatively, λ_{cr} can be found by an elastic buckling analysis or by Equation 6.1 and used to multiply the maximum moment by the following amplification factor.

$$\frac{\lambda_{cr}}{\lambda_{cr}-1} = \text{larger of } \frac{1}{1-\dfrac{F_V \delta_N}{F_N h}} \text{ and } \frac{1}{1-\dfrac{F_c L_E^{2}}{\pi^2 EI}} \qquad (8.82)$$

M_{cx} is the elastic moment capacity $p_y Z_x$ about the major principal x-axis;

M_{cy} is the elastic moment capacity $p_y Z_y$ about the minor principal y-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 (see Table 8.4);

P_{cx} is the compression resistance under sway mode and about the x-axis using clause 8.7.5;

P_{cy} is the compression resistance under sway mode and about the y-axis using clause 8.7.5;

P_c is the smaller of P_{cx} and P_{cy}.

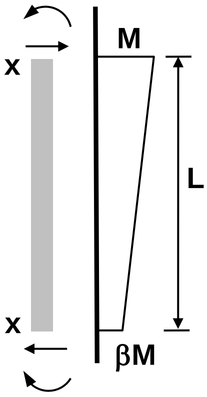
For non-sway frames defined in clause 6.3, non-sway axial force resistance can be used for the computation of P_{cx} and P_{cy} and checking using Equations 8.80 and 8.81 is adequate. For more exact analysis, $\overline{P}_c$ in equation 8.80 can be replaced by P_c using the computed effective length. The P-δ amplification factor below should be used for non-sway frames.

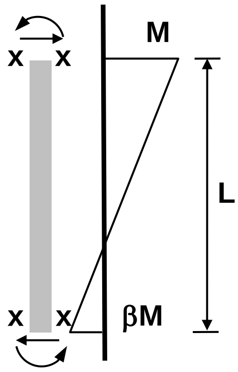
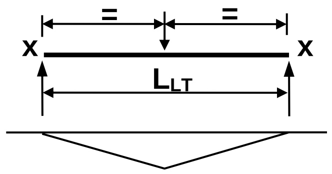
$$\frac{1}{1-\dfrac{F_c L_E^{2}}{\pi^2 EI}} \qquad (8.83)$$

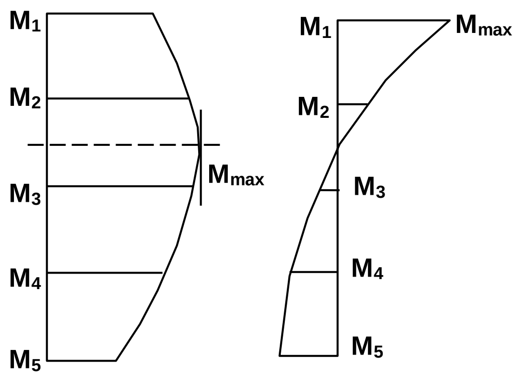
The design moment for connections in sway and ultra-sway frames must include the P-Δ effect.

When second-order P-Δ-δ analysis is used, the amplification is considered in the analysis and amplification at design stage is not needed.

Table 8.9 - Moment equivalent factor m for flexural buckling

<table>
<tr><th colspan="2">Segment with end moment only (value of m from the formula of general case)</th><th>β</th><th>m</th></tr>
<tr><td rowspan="21">β positive


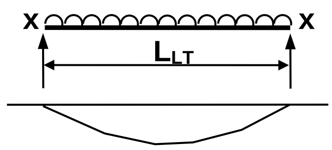
X - lateral restraint</td><td rowspan="21">β negative

</td><td>1.0</td><td>1.00</td></tr>
<tr><td>0.9</td><td>0.96</td></tr>
<tr><td>0.8</td><td>0.92</td></tr>
<tr><td>0.7</td><td>0.88</td></tr>
<tr><td>0.6</td><td>0.84</td></tr>
<tr><td>0.5</td><td>0.80</td></tr>
<tr><td>0.4</td><td>0.76</td></tr>
<tr><td>0.3</td><td>0.72</td></tr>
<tr><td>0.2</td><td>0.68</td></tr>
<tr><td>0.1</td><td>0.64</td></tr>
<tr><td>0.0</td><td>0.60</td></tr>
<tr><td>-0.1</td><td>0.58</td></tr>
<tr><td>-0.2</td><td>0.56</td></tr>
<tr><td>-0.3</td><td>0.54</td></tr>
<tr><td>-0.4</td><td>0.52</td></tr>
<tr><td>-0.5</td><td>0.50</td></tr>
<tr><td>-0.6</td><td>0.48</td></tr>
<tr><td>-0.7</td><td>0.46</td></tr>
<tr><td>-0.8</td><td>0.44</td></tr>
<tr><td>-0.9</td><td>0.42</td></tr>
<tr><td>-1.0</td><td>0.40</td></tr>
<tr><th colspan="4">Segment between intermediate lateral restraints</th></tr>
<tr><th>Specific cases</th><th colspan="3">General case</th></tr>
<tr><td>

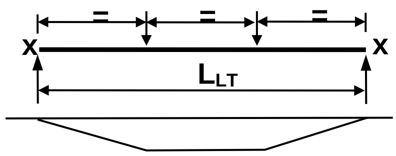
m = 0.90</td><td colspan="3" rowspan="4">

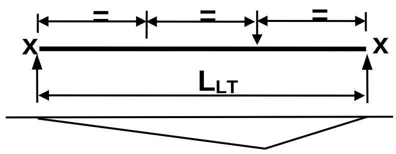
$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{max}} \quad \text{but } m \geq \frac{0.8M_{24}}{M_{max}}$$

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.</td></tr>
<tr><td>

m = 0.95</td></tr>
<tr><td>

m = 0.95</td></tr>
<tr><td>

m = 0.80</td></tr>
</table>

8.9.3 Member buckling resistance – (Alternative to 8.9.2)

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{F_c}{A_g p_{cx}} + k_{xx}\frac{M_x}{M_b} + k_{xy}\frac{M_y}{M_{cy}} \leq 1 \qquad (8.82a)$$

$$\frac{F_c}{A_g p_{cy}} + k_{yx}\frac{M_x}{M_b} + k_{yy}\frac{M_y}{M_{cy}} \leq 1 \qquad (8.82b)$$

where F_c, M_x and M_y are the design values of the compression force and the maximum moments about the major (x-x) axis and the minor (y-y) axis along the member, respectively

M_b is the buckling resistance moment in clause 8.3.5.2
M_{cy} is the moment capacity about the minor axis from clause 8.2.2
p_{cx},p_{cy} are the axial strength under column buckling about the major (x-x) axis, and the minor (y-y) axis
k_{xx}, k_{xy}, k_{yx}, k_{yy} are the interaction factors given in Table 8.10.

Refer to Table 8.10 for members not susceptible to torsional deformations, or to Table 8.11 for members susceptible to torsional deformations.

Table 8.10 Interaction factors for combined axial compression and bending

Interaction factors	Type of sections	Design assumptions	
		Elastic cross-sectional properties Class 3, Effective cross-sectional properties Class 4	Plastic cross-sectional properties Class 1, Class 2
k_{xx}	I-sections RHS	$C_{mx}\left(1+0.6\bar{\lambda}_x\frac{F_c}{P_{cx}}\right) \leq C_{mx}\left(1+0.6\frac{F_c}{P_{cx}}\right)$	$C_{mx}\left(1+\left(\bar{\lambda}_x-0.2\right)\frac{F_c}{P_{cx}}\right) \leq C_{mx}\left(1+0.8\frac{F_c}{P_{cx}}\right)$
k_{xy}	I-sections RHS	k_{yy}	$0.6k_{yy}$
k_{yx}	I-sections RHS	$0.8k_{xx}$	$0.6k_{xx}$
k_{yy}	I-sections	$C_{my}\left(1+0.6\bar{\lambda}_y\frac{F_c}{P_{cy}}\right) \leq C_{my}\left(1+0.6\frac{F_c}{P_{cy}}\right)$	$C_{my}\left(1+\left(2\bar{\lambda}_y-0.6\right)\frac{F_c}{P_{cy}}\right) \leq C_{my}\left(1+1.4\frac{F_c}{P_{cy}}\right)$
	RHS		$C_{my}\left(1+\left(\bar{\lambda}_y-0.2\right)\frac{F_c}{P_{cy}}\right) \leq C_{my}\left(1+0.8\frac{F_c}{P_{cy}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending M_x, the coefficient k_{yx} may be $k_{yx}=0$.			

Note:

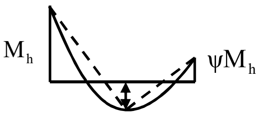
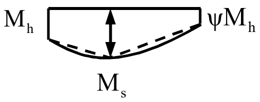
$$\bar{\lambda}_x=\frac{\lambda_x}{\pi\sqrt{\frac{E}{p_y}}}$$

$$\bar{\lambda}_y=\frac{\lambda_y}{\pi\sqrt{\frac{E}{p_y}}}$$

Table 8.11 Interaction factors kij for members susceptible to torsional deformations

Interaction factors	Design assumptions	
	Elastic cross-sectional properties Class 3, Effective cross-sectional properties Class 4	Plastic cross-sectional properties Class 1, Class 2
k_{xx}	$k_{yy}$from Table 8.10)	$k_{yy}$from Table 8.10
k_{xy}	$k_{yz}$from Table 8.10)	$k_{yz}$from Table 8.10
k_{yx}	$\left[1-\frac{0.05\bar{\lambda}_y}{(C_{mLT}-0.25)}\frac{F_c}{P_{cy}}\right] \leq \left[1-\frac{0.05}{(C_{mLT}-0.25)}\frac{F_c}{P_{cy}}\right]$	$\left[1-\frac{0.1\bar{\lambda}_y}{(C_{mLT}-0.25)}\frac{F_c}{P_{cy}}\right] \geq \left[1-\frac{0.1}{(C_{mLT}-0.25)}\frac{F_c}{P_{cy}}\right]$ for $\bar{\lambda}_y < 0.4$: $k_{zy} = 0.6 + \bar{\lambda}_y \leq 1 - \frac{0.1\bar{\lambda}_y}{(C_{mLT}-0.25)}\frac{F_c}{P_{cy}}$
k_{yy}	k_{yy} from Table 8.10	k_{yy} from Table 8.10

Table 8.12 Equivalent uniform moment factors, C_m in Tables 8.10 and 8.11

<table>
<tr><th rowspan="2">Moment diagram</th><th rowspan="2" colspan="2">Range</th><th colspan="2">C_{mx} and C_{my} and C_{mLT}</th></tr>
<tr><th>Uniform loading</th><th>Concentrated load</th></tr>
<tr><td></td><td colspan="2">$-1 \le \psi \le 1$</td><td colspan="2">$0.6 + 0.4\psi \ge 0.4$</td></tr>
<tr><td rowspan="3">
$\alpha_h = M_s / M_h$</td><td>$0 \le \alpha_s \le 1$</td><td>$-1 \le \psi \le 1$</td><td>$0.2 + 0.8\alpha_s \ge 0.4$</td><td>$0.2 + 0.8\alpha_s \ge 0.4$</td></tr>
<tr><td rowspan="2">$-1 \le \alpha_s \le 0$</td><td>$0 \le \psi \le 1$</td><td>$0.1 - 0.8\alpha_s \ge 0.4$</td><td>$-0.8\alpha_s \ge 0.4$</td></tr>
<tr><td>$-1 \le \psi \le 0$</td><td>$0.1(1-\psi) - 0.8\alpha_s \ge 0.4$</td><td>$0.2(-\psi) - 0.8\alpha_s \ge 0.4$</td></tr>
<tr><td rowspan="3">
$\alpha_h = M_h / M_s$</td><td>$0 \le \alpha_h \le 1$</td><td>$-1 \le \psi \le 1$</td><td>$0.95 + 0.05\alpha_h$</td><td>$0.90 + 0.10\alpha_h$</td></tr>
<tr><td rowspan="2">$-1 \le \alpha_h \le 0$</td><td>$0 \le \psi \le 1$</td><td>$0.95 + 0.05\alpha_h$</td><td>$0.90 + 0.10\alpha_h$</td></tr>
<tr><td>$-1 \le \psi \le 0$</td><td>$0.95 + 0.05\alpha_h(1+2\psi)$</td><td>$0.90 + 0.10\alpha_h(1+2\psi)$</td></tr>
<tr><td colspan="5">For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{mx} = 0.9$ or $C_{my} = 0.9$ respectively.</td></tr>
<tr><td colspan="5">C_{mx} , C_{my} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

Moment factor | Bending axis | Points braced in direction
C_{mx} | x-x | y-y
C_{my} | y-y | x-x
C_{mLT} | x-x | x-x</td></tr>
</table>

8.10 TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF COMPRESSION MEMBERS

Single angle, double angle, tee sections and cruciform compression members with small thickness-to-length ratios may buckle in a torsional or a flexural-torsional mode. A check should be carried out by considering the interaction of torsional buckling mode with the flexural buckling mode about the principal axes. Design checks should be carried out as given in clause 11.5.4.

8.11 PORTAL FRAMES

8.11.1 General

Either elastic or plastic analysis may be adopted for the design of single-storey frames with rigid moment-resisting joints, see clauses 6.5, 6.6, 6.7, and 6.8. All load combinations should be covered. For the load case involving the gravity load only, notional horizontal forces should be applied to check the adequacy of in-plane stability. The out-of-plane stability of all frame members under all load cases should be ensured by the provision of appropriate lateral and torsional restraints.

8.11.2 Elastic design

In the elastic analysis of portal frames, the members should be designed in accordance with clauses 8.8 and 8.9. The out-of-plane stability should be checked considering the interaction of the torsional buckling mode with the flexural buckling mode about the principal axes, see clause 8.3.

For non-sway frames and independently braced frames, the in-plane member buckling resistances should be checked using clause 8.9.

In global elastic analysis where P-Δ effects have not been calculated, the ultimate design loads must be multiplied by the required load factor λ_r obtained from clause 8.11.4 to check against the member resistance.

8.11.3 Plastic design

Plastic analysis can be used when static loading, rather than fatigue, is the main design concern. The material should be Class 1 to ensure the development of plastic hinge and redistribution of moment.

The plastic load factor obtained from a first-order global analysis with ultimate design loads should satisfy:

$$\lambda_p \geq \lambda_r \tag{8.84}$$

Member resistance is checked by multiplying ultimate design loads by λ_r.

8.11.4 In-plane stability

8.11.4.1 General

The in-plane stability of portal frames under each load combination must be checked when the member sizes have been determined. One of the following methods should be used except for the case of tied portal frames:

a) the sway-check method plus snap through stability check in clause 8.11.4.2.
b) the amplified moments method given in clause 8.11.4.3.
c) a second order analysis, see clause 8.11.4.4.

Tied portal frames should be checked as recommended in clause 8.11.4.5.

8.11.4.2 Sway-check method plus snap through stability

8.11.4.2.1 General

For untied portal frames which satisfying the following conditions, simplified sway check method may be used:

a) The span L does not exceed 5 times the mean height h of the columns;
b) The height h_r of the ridge above the apex does not exceed 0.25 times the span L;
c) If the portal frame is asymmetric, h_r should satisfy the criterion $\left(\frac{h_r}{s_a}\right)^2 + \left(\frac{h_r}{s_b}\right)^2 \leq 5$, where s_a and s_b are the horizontal distance from the apex to columns, see Figure 8.5.

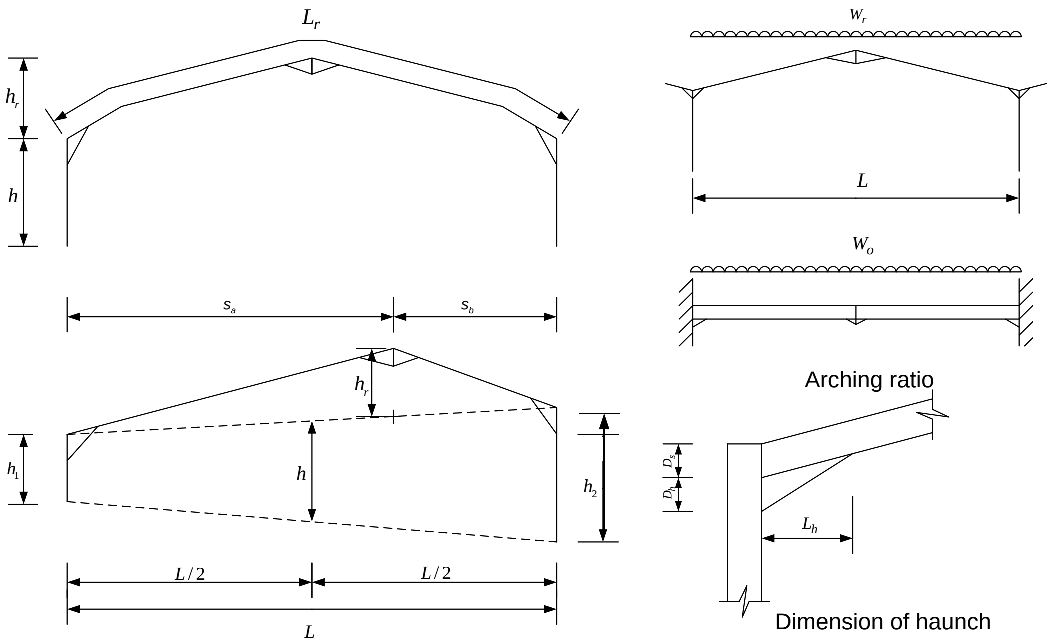


Figure 8.5 - Geometry of portal frames

When these geometric limitations are satisfied, linear elastic analysis should be used to determine the deflection at the top of the columns caused by a notional lateral force applied generally at the top of the columns in the plane of the frame. The notional horizontal force is calculated in accordance with clause 2.5.8. If significant proportions of vertical loads are applied at levels below the top of the columns, the corresponding notional horizontal forces should be applied at the same levels where the vertical loads are applied.

8.11.4.2.2 Gravity loads

The gravity load cases include load combination 1 in clause 4.3 and load combination 1 in Table 13.3 of clause 13.7.3 where vertical crane loads are present. Any stiffening effects provided by cladding, roof plan bracing, and roof sheeting should be ignored in the calculation of notional horizontal deflection. If the calculated deflection δ_i is less than $h_i/1000$ then the required plastic collapse load factor λ_r may be taken as 1.0.

A simplified formula may be used to check the stability if there are no valley beams, crane gantries or significant concentrated load larger than those from purlins and in cases where the deflections are otherwise difficult to determine:

$$\frac{L_b}{D} \leq \frac{44L}{\Omega h}\left(\frac{\rho}{4 + \rho L_r / L}\right)\left(\frac{275}{p_{yr}}\right) \tag{8.85}$$

in which

$$L_b = L - \left(\frac{2D_h}{D_s + D_h}\right)L_h \tag{8.86}$$

$$\rho = \left(\frac{2\,I_c}{I_r}\right)\left(\frac{L}{h}\right) \quad \text{for a single bay frame;} \tag{8.87}$$

$$\rho = \left(\frac{I_c}{I_r}\right)\left(\frac{L}{h}\right) \quad \text{for a multi-bay frame;} \tag{8.88}$$

and Ω is the arching ratio, given by:

$$\Omega = W_r / W_o \tag{8.89}$$

where

- D is the cross-section depth of the rafter;
- D_h is the additional depth of the haunch, see Figure 8.5;
- D_s is the depth of the rafter, allowing for its slope, see Figure 8.5;
- h is the mean column height;
- I_c is the in-plane second moment of area of the column (taken as zero if the column is not rigidly connected to the rafter, or if the rafter is supported on a valley beam);
- I_r is the in-plane second moment of area of the rafter;
- L is the span of the bay;
- L_b is effective span of the bay;
- L_h is the length of a haunch;
- L_r is the total developed length of the rafters, see Figure 8.5;
- p_{yr} is the design strength of the rafters in N/mm^2;
- W_o is the value of W_r for plastic failure of the rafters as a fixed-ended beam of span L;
- W_r is the total factored vertical load on the rafters of the bay;

If the two columns or the two rafters of a bay differ, the mean value of I_c / I_r should be used.

If the haunches at each side of the bay are different, the mean value of L_b should be used.

8.11.4.2.3 Horizontal loads

When checking load combination 2 and load combination 3 of clause 4.3, where wind loads and other significant horizontal loads are present, the stiffening effects provided by cladding, plan bracing and roof-sheeting should be taken into account to calculate the notional horizontal deflections δ_i.

The value of λ_r for load cases involving horizontal loads may be determined from the following simple rules provided that the frame is stable under gravity loading, i.e. the $h/1000$ criteria or the formula is satisfied for gravity loading. (Equation 6.9 in clause 6.6.2)

$$\lambda_r = \frac{\lambda_{sc}}{(\lambda_{sc} - 1)} \tag{8.90}$$

in which λ_{sc} is the smallest value for each column determined by equation 6.1 in clause 6.3.2.2.

If $\lambda_{sc} < 5$, second order analysis should be used.

If there are no valley beams, crane gantries or significant concentrated loads larger than those from purlins, and where the deflections are difficult to determine a simplified formula may be used to check the stability:

$$\frac{L_b}{D} \le \frac{220DL}{\Omega h L_b}\left(\frac{\rho}{4 + \rho\, L_r / L}\right)\left(\frac{275}{p_{yr}}\right) \tag{8.91}$$

If the axial forces are tensile in all rafters and columns under wind loads, the required load factor λ_r should be taken as 1.0.

8.11.4.2.4 Snap-through stability

For internal bays of multi-bay frames the possibility of snap through stability should be considered and checked by the following formula:

$$\frac{L_b}{D} \le \frac{22(4 + L/h)}{4(\Omega - 1)}\left(1 + \frac{I_c}{I_r}\right)\left(\frac{275}{p_{yr}}\right)\tan 2\theta \qquad (8.92)$$

where θ is the slope of the rafters for a symmetrical frame and $\tan^{-1}(2h_r/L)$ for asymmetric frames.

8.11.4.3 Amplified moment method

When the geometric limitations of clause 6.4.2 to use the sway check method are not met, the amplified moment method may be adopted. The elastic critical load factor λ_{cr} should be calculated from the eigenvalue analysis instead of using the approximate formula (Equation 6.1) in clause 6.3.2.2. If λ_{cr} for the relevant load case can be calculated, and is not less than 5, the required load factor can be determined.

If $\lambda_{cr} \ge 10$ then λ_r may be taken as 1.0;

If $10 > \lambda_{cr} \ge 5$ then λ_r may be calculated by

$$\lambda_r = \frac{0.9\lambda_{cr}}{(\lambda_{cr} - 1)} \qquad (8.93)$$

If $\lambda_{cr} < 5$ then the amplified moment method should not be used as the lateral stiffness of the portal frame is too low. In such a case the lowest elastic critical load factor is generally obtained using elastic critical load analysis.

8.11.4.4 Second order analysis

Where the above methods are not appropriate, a full second order analysis, either elastic or elastic-plastic, should be carried out. In this case, the required load factor λ_r should be taken as 1.0.

8.11.4.5 Tied portals

Tied portals should be treated with extreme caution to ensure stability of the slender rafters that can result from this method of design. A full second order elastic or elastic-plastic analysis should be carried out. Tied portals may usually have shallow rafters. The axial forces in rafters increase inversely proportional to the actual slopes of the rafters. In addition, the curvature of the deformed rafter will increase apex drop. The analysis software routines adopted should be able to account for such nonlinearities.

8.11.5 Out-of-plane stability

8.11.5.1 General

The out-of-plane stability of the frame should be ensured by making the frame effectively non-sway out-of-plane. This will imply the use of bracing or very stiff portal frame. The out-of-plane stability of all frame members should be ensured by the provision of appropriate lateral and torsional restraints under all load cases.

8.11.5.2 Torsional restraints

Torsional restraint prevents twisting of the section. The most effective way is to provide lateral restraints to both flanges of a section. The use of a “fly” brace to provide torsional restraint is shown in Figure 8.6.

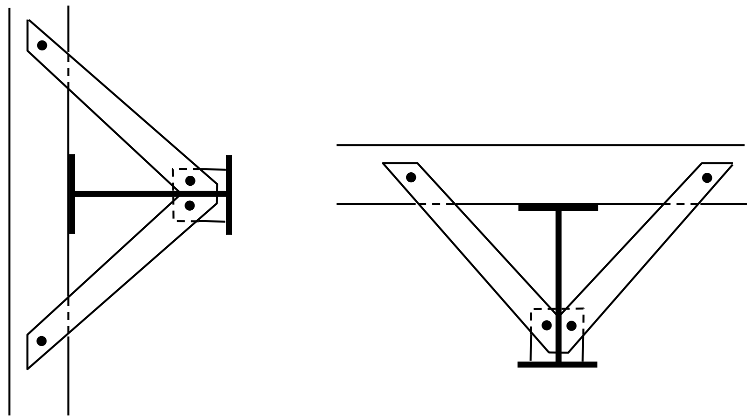


Figure 8.6 - Example of using fly brace for column and rafter

At the point of contraflexure in a portal frame rafter, the section may be assumed to be torsionally restrained by a virtual lateral restraint to the bottom flange if the purlins and their connections to the top flange of the rafter are capable of providing torsional restraint to the top flange of the rafter.

Torsional restraint of the top flange may be assumed to exist if the following conditions are satisfied:

a) The rafter is an I-section with $D/B \geq 1.2$, where D is the depth and B is the flange width;

b) For haunched rafters D_h is not greater than 2D;

c) Every length of purlin has at least two bolts in each purlin-to-rafter connection;

d) The depth of the purlin section is not less than 0.25 times the depth D of the rafter.

Lateral restraint of the bottom flange should not be assumed at the point of contraflexure under other restraint conditions, unless a lateral restraint is actually provided at that point.

8.11.5.3 Location of torsional restraints

8.11.5.3.1 General

Torsional restraints should be provided in accordance with the following requirements:

(1) All plastic hinges locations should be torsionally restrained at both flanges. Where it is not practical to do this at the hinge location, a restraint may be provided within a distance D/2 of the hinge location.

(2) The distance between the plastic hinge and the next torsional restraint (restraint to both flanges) can be calculated by two methods:

a) A conservative method which does not allow for the shape of the moment diagram between the plastic hinge and the next torsional restraint.

b) An approximate method which allows for shape of the moment diagram.

The lateral restraints at both flanges and/or virtual restraints at bottom flange (see Figure 8.6) should extend up to or beyond the point of contraflexure.

8.11.5.3.2 Conservative method

The distance between a plastic hinge and the next torsional restraint L_m should not exceed L_u determined by:

$$L_u = \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}\right)^2\right]^{1/2}} \tag{8.94}$$

where

f_c is the compressive stress (in N/mm²) due to axial force;

p_y is the design strength (in N/mm²);

r_y is the radius of gyration about the minor axis;

x is the torsional index, see Appendix 8.2.

If the member has unequal flanges the radius of gyration about the minor axis r_y should be taken as the lesser of the values for the compression flange(s).

8.11.5.3.3 Approximate method allowing for moment gradient

For I section members with uniform cross section with equal flanges and $D/B \geq 1.2$ where f_c does not exceed 80 N/mm²:

$$L_m = \phi L_u \quad (8.95)$$

In which case L_u is given in equation 8.94 of clause 8.11.5.3.2 and ϕ is given as following:

For $1 \geq \beta \geq \beta_u$: $\phi = 1$ (8.96)

For $\beta_u > \beta > 0$: $\phi = 1 - (1 - KK_0)(\beta_u - \beta)/\beta_u$ (8.97)

For $0 \geq \beta > -0.75$: $\phi = K(K_0 - 4(1 - K_0)\beta/3)$ (8.98)

For $\beta \leq -0.75$: $\phi = K$ (8.99)

where β is the end moment ratio, and

For steel grade of design strength between 200 to 300 MPa,

$$\beta_u = 0.44 + \frac{x}{270} - \frac{f_c}{200} \quad (8.100)$$

For steel grade of design strength greater than 300 MPa and less than 460 MPa,

$$\beta_u = 0.47 + \frac{x}{270} - \frac{f_c}{250} \quad (8.101)$$

For other steel grades, P-Δ-δ and advanced analyses should be used.

Coefficient K_0 and K can be calculated as follows:

$$K_0 = (180 + x)/300 \quad (8.102)$$

For $20 \leq x \leq 30$, $K = 2.3 + 0.03x - x\, f_c/3000$ (8.103)

For $30 < x \leq 50$, $K = 0.8 + 0.08x - (x - 10) f_c/2000$ (8.104)

8.11.5.3.4 Segments with one flange restrained

Where one flange is restrained between torsional restraints (i.e. restraints to both flanges) the distance between torsional restraints may be increased provided that:

a) adjacent to a plastic hinge location, the spacing of intermediate lateral restraints should not exceed L_m as given in clause 8.11.5.3.3; and

b) the distance between the intermediate lateral restraints would be adequate if they were attached to the compression flange. Member buckling resistance should be checked in accordance with clause 8.9 or L_m from clause 8.11.5.3.3.

In addition to conditions a) and b), when the following conditions are satisfied, a simplified method may be used:

a) The member is an I-section with $D/B \geq 1.2$;

b) For haunched rafters $D_h \leq 2D_s$;

c) For haunched segments the haunch flange is not smaller than the member flange;

d) The steel grade of design strength between 200 to 460 MPa.

For other steel grades, P-Δ-δ elastic analysis or advanced analysis should be used.

The spacing L_y between restraints to the compression flange should not exceed the limiting spacing L_s given as follows:

For steel grade of design strength between 200 to 300 MPa,

$$L_s = \frac{620\, r_y}{K_1 \left[72 - (100/x)^2\right]^{0.5}} \qquad (8.105)$$

For steel grade of design strength greater than 300 MPa and less than 460 MPa,

$$L_s = \frac{645\, r_y}{K_1 \left[94 - (100/x)^2\right]^{0.5}} \qquad (8.106)$$

Where

r_y is the minor axis radius of the un-haunched rafter (is the minimum value of the radius of gyration within the length of the segment (at the top of the haunch));

x is the maximum value of torsional index within the segment, see Appendix 8.2. Note that at the bottom of the haunch x can be approximated with D/T .

K_1 has the following value:

For an un-haunched segment	$K_1 = 1.00$
For a haunch with D_h/D_s = 1	$K_1 = 1.25$
For a haunch with D_h/D_s = 2	$K_1 = 1.40$
For a haunch generally	$K_1 = 1 + 0.25(D_h/D_s)^{2/3}$ (8.107)

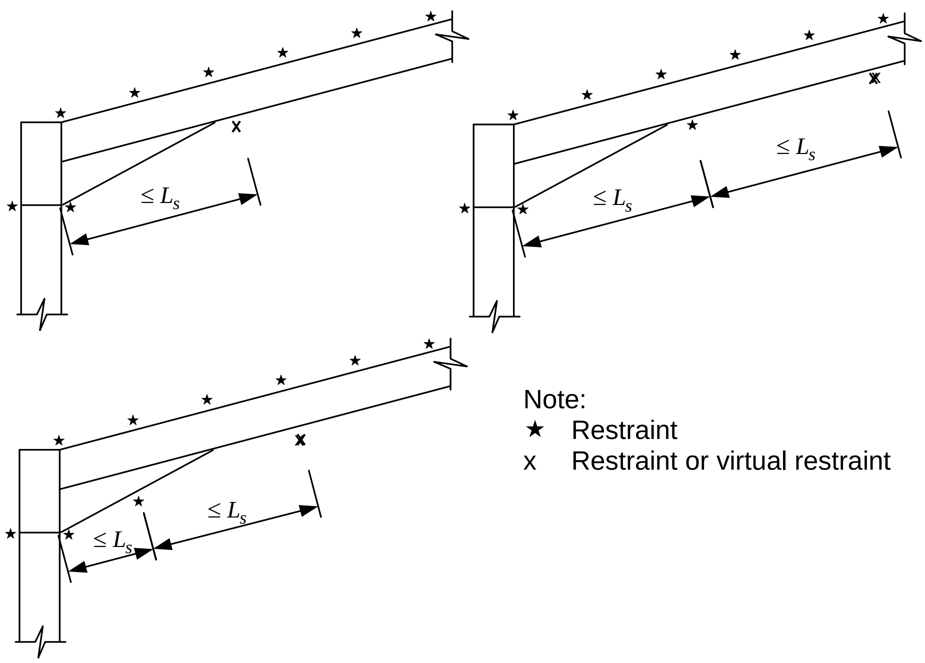


Figure 8.7 - Haunch restraints

8.12 LACED AND BATTENED STRUTS

Lighter structural elements can be fabricated to form laced and battened struts to increase their resistance. Laced columns normally have transverse members arranged in triangulated manner and battened columns have the transverse members placed perpendicular to the longitudinal axis of the compound columns.

8.12.1 Laced struts

A laced strut consisting of two or more main components may be designed using a second-order analysis with allowance for component and strut imperfections or as a single integral member if the following conditions are met:

a) The main components should be effectively restrained against buckling by a lacing system of flats or sections.

b) The lacing should comprise an effectively restrained system on each face and the lacing should not vary throughout the length of the member.

c) Except for tie panels in f) below, double and single intersection lacing systems mutually opposed in direction on opposite sides of two main components should not be combined with members or diaphragms perpendicular to the longitudinal axis of the strut unless all forces resulting from deformation of the strut members are allowed for.

d) Single lacing systems mutually opposed in direction on opposite sides of two main components should not be used unless the resulting torsional effects are allowed for.

e) All lacings should be inclined at an angle between 40° and 70° to the axis of the member.

f) Tie panels should be provided at the ends of the lacing systems, at points where the lacing is interrupted and at connections with other members. Tie panels may take the form of battens conforming to clause 8.12.2 below or cross braced panels of equivalent rigidity may be used. In either case, the tie panels should be designed to carry the loads for which the lacing system is designed.

g) The slenderness λ_c of the main components about their minimum radius of gyration between consecutive points where the lacing is attached should not exceed 50. If the overall slenderness of the member is less than 1.4 λ_c the design should be based on a slenderness of $1.4\lambda_c$.

h) The effective length of a lacing should be taken as the distance between the inner end welds or bolts for single intersection lacing and as 0.7 times of this distance for double intersection lacing connected by welds or bolts at the intersection. The slenderness of a lacing should not exceed 180.

i) The lacings and their connections should be designed to carry forces induced by a transverse shear at any point in the length of the member equal to 2.5% of the axial force in the member, divided equally amongst all transverse lacing systems in parallel planes. For members carrying moments due to eccentricity of loading, applied end moments or lateral loading, the lacing should be proportioned to resist the shear due to bending in addition to 2.5% of the axial force.

8.12.2 Battened struts

A battened strut consisting of two or more main components may be designed using a second-order analysis with allowance for component and strut imperfections or as a single integral member if the following conditions are met.

a) The main component should be effectively restrained against buckling by a system of battens consisting of plates or sections, so connected to the main components to form an effectively rigid-jointed frame.

b) Battens should be positioned opposite each other in each plane at ends of the members and at points where it is laterally restrained. Intermediate battens should be positioned opposite each other and be spaced and proportioned uniformly throughout the length of a member.

c) The slenderness λ_c of a main component based on minimum radius of gyration between end welds or end bolts of adjacent battens should not exceed 50. The slenderness λ_b of the battened strut about the axis perpendicular to the plane of the battens should be calculated as,

$$\lambda_b = \sqrt{\lambda_m^2 + \lambda_c^2} \tag{8.108}$$

in which λ_m is the ratio $\frac{L_E}{r}$ of the whole member about that axis.

d) If λ_b is less than $1.4\lambda_c$, the design should be based on a slenderness of $1.4\lambda_c$.

e) The thickness of the plate battens should not be less than 1/50 of the minimum distance between welds or bolts. The slenderness of sections used as battens should not exceed 180. The width of an end batten along the axis of the main components should not be less than the distance between centroids of the main members and not less than half this distance for intermediate battens. Further, the width of any batten should be not less than twice the width of the narrow main component.

f) The battens and their connections and the main components should be designed to carry the forces and moments induced by transverse shear at any point in the length of the member equal to 2.5% of the axial force in the member. For members carrying moments due to eccentricity of loading, applied end moments or lateral loads, the battens should be proportioned to resist the shear due to bending in addition to 2.5% of the axial force.

Appendix 8.1

The bending buckling strength p_b for resistance to lateral-torsional buckling should be taken as the smaller root of the following equation.

$$(p_E - p_b)(p_y - p_b) = \eta_{LT} p_E p_b \qquad \text{(A8.1)}$$

Hence

$$p_b = \frac{p_E p_y}{\phi_{LT} + \sqrt{\left(\phi_{LT}^2 - p_E p_y\right)}} \qquad \text{(A8.2)}$$

where,

$$p_E = \frac{\pi^2 E}{\lambda_{LT}^2} \qquad \text{(A8.3)}$$

$$\phi_{LT} = \frac{p_y + (\eta_{LT} + 1) p_E}{2} \qquad \text{(A8.4)}$$

p_y is the design strength,

λ_{LT} is the equivalent slenderness

η_{LT} is the Perry factor taken as follows.

For rolled sections,

$$\eta_{LT} = \alpha_{LT}(\lambda_{LT} - \lambda_{L0})/1000;\ \eta_{LT} \geq 0\ ; \qquad \text{(A8.5)}$$

For welded sections,

$$\text{For } \lambda_{LT} \leq \lambda_{L0}\ ; \eta_{\mathrm{LT}} = 0 \qquad \text{(A8.6)}$$

$$\text{For } \lambda_{L0} < \lambda_{LT} \leq 2\lambda_{L0}\ ; \eta_{\mathrm{LT}} = \frac{2\alpha_{LT}(\lambda_{LT} - \lambda_{L0})}{1000} \qquad \text{(A8.7)}$$

$$\text{For } 2\lambda_{L0} < \lambda_{LT} \leq 3\lambda_{L0}\ ; \eta_{\mathrm{LT}} = \frac{2\alpha_{LT}\lambda_{L0}}{1000} \qquad \text{(A8.8)}$$

$$\text{For } \lambda_{LT} > 3\lambda_{L0}\ ; \eta_{\mathrm{LT}} = \frac{\alpha_{LT}(\lambda_{LT} - \lambda_{L0})}{1000} \qquad \text{(A8.9)}$$

$$\lambda_{L0} = 0.4\sqrt{\frac{\pi^2 E}{p_y}} \qquad \text{(A8.10)}$$

$$\alpha_{LT} = 7.0 \qquad \text{(A8.11)}$$

α_{LT} is applicable to all steel grades at sources.

Appendix 8.2

For uniform I, H and channel sections with equal flanges, the equivalent slenderness λ_{LT} should be obtained as follows.

$$\lambda_{LT} = uv\lambda\sqrt{\beta_w} \qquad \text{(A8.12)}$$

where

$$\lambda = \frac{L_E}{r_y} \qquad \text{(A8.13)}$$

L_E is the effective length for lateral-torsional buckling from clause 8.3.4

r_y is the radius of gyration about the minor y-axis

u is the buckling parameter

$$= \left(\frac{4S_x^2\gamma}{A^2h_s^2}\right)^{0.25} \text{ for equal flanged I and H sections} \qquad \text{(A8.14)}$$

$$= \left(\frac{I_yS_x^2\gamma}{A^2H}\right)^{0.25} \text{ for equal flanged channels} \qquad \text{(A8.15)}$$

v is the slenderness factor given by,

$$\frac{1}{\left(1+0.05(\lambda/x)^2\right)^{0.25}} \qquad \text{(A8.16)}$$

x is the torsional index

$$= 0.566h_s\sqrt{\frac{A}{J}} \text{ for equal flanged I and H sections} \qquad \text{(A8.17)}$$

$$= 1.132\sqrt{\frac{AH}{I_yJ}} \text{ for equal flanged channels} \qquad \text{(A8.18)}$$

Alternatively, u and x in Equations A8.14, A8.15, A8.17 and A8.18 can be obtained for rolled and welded I, H or channel sections with equal flanges as,

$x = \frac{D}{T}$ and u=0.9 for rolled I, H or channel sections with equal flanges.

$x = \frac{D}{T}$ and u=1.0 for welded I, H or channel sections with equal flanges.

D is the depth of the section

H is the warping constant for equal flanged channel sections

$$= \frac{h_s^2(B-t/2)^3T}{12}\ \frac{2h_st+3(B-t/2)T}{h_st+6(B-t/2)T} \qquad \text{(A8.19)}$$

h_s is the distance between shear centres of flanges

J is the torsional constant

S_x is the plastic modulus about the major axis

t is the web thickness

T is the flange thickness

$$\gamma = 1-\frac{I_y}{I_x} \qquad \text{(A8.20)}$$

β_w is the ratio defined in Equations 8.28 and 8.29.

Buckling strength of other sections should be determined by a recognized buckling analysis.

Appendix 8.3

The shear buckling strength of web in an I-section beams or plate girders may be obtained as follows:

For welded I-section,

For $\lambda_w \leq 0.8$, $\quad q_w = p_v$ (A8.21)

For $0.8 < \lambda_w < 1.25$, $\quad q_w = (\frac{13.48 - 5.6\lambda_w}{9})p_v$ (A8.22)

For $\lambda_w \geq 1.25$, $\quad q_w = 0.9p_v / \lambda_w$ (A8.23)

For hot-rolled I-sections,

$\lambda_w \leq 0.9$, $q_w = p_v$ (A8.24)

$\lambda_w > 0.9$, $q_w = 0.9p_v / \lambda_w$ (A8.25)

in which

$p_v = 0.6p_{yw}$ (A8.26)

$$\lambda_w = \sqrt{\frac{p_v}{q_e}} \quad \text{(A8.27)}$$

For $a/d \leq 1$, $q_e = \left[0.75 + \frac{1}{(a/d)^2}\right]\left[\frac{1000}{d/t}\right]^2$ in N/mm² (A8.28)

For $a/d > 1$, $q_e = \left[1 + \frac{0.75}{(a/d)^2}\right]\left[\frac{1000}{d/t}\right]^2$ in N/mm² (A8.29)

λ_w is applicable to all steel grades at source.

Appendix 8.4

The compressive buckling strength p_c of a column for resistance to flexural buckling should be taken as the smaller root of the following equation.

$$(p_E - p_c)(p_y - p_c) = \eta p_E p_c \quad \text{(A8.30)}$$

Hence

$$p_c = \frac{p_E p_y}{\phi_C + \sqrt{(\phi_C^2 - p_E p_y)}} \quad \text{(A8.31)}$$

where,

$$p_E = \frac{\pi^2 E}{\lambda^2} \quad \text{(A8.32)}$$

$$\phi_C = \frac{p_y + (\eta + 1)p_E}{2} \quad \text{(A8.33)}$$

p_y is the design strength,

λ is the slenderness in clause 8.7.4

η is the Perry factor taken as,

$$\eta = \alpha\ (\lambda - \lambda_0)/1000;\ \eta \geq 0 \quad \text{(A8.34)}$$

in which the limit slenderness should be taken as $\lambda_0 = 0.2\sqrt{\frac{\pi^2 E}{p_y}}$ (A8.35)

and the Robertson constant should be taken as,

Curve (a_0), α=1.8 ;
Curve (a), α=2.0 ;
Curve (b), α=3.5 ;
Curve (c), α=5.5 ;
Curve (d), α=8.0 ;

When using other steel grades, it may be necessary to determine the Perry factor or member bow imperfection for a manual or a second-order analysis. Perry factors above can be used for member design or member bow imperfection in Table 6.1 can be used for second-order analysis. The Perry factor can be related to the member bow imperfection e_0 as,

$$\eta = \frac{e_0 \bar{y}}{r^2}$$

in which $\bar{y}$ is the maximum distance from centroidal axis of a section and r is the radius of gyration.