INTERNATIONAL UNIVERSITY OF AFRICA
CIVIL ENGINEERING DEPARTMENT

ANALYSIS AND DESIGN OF STEEL WORKS II

GRADE 4

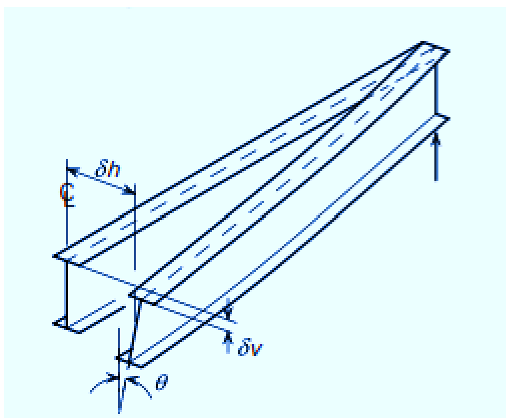
7TH SEMESTER

Un-Restrained beams

1 INTRODUCTION

An unrestrained beam (i.e. without full lateral restraint, as described in previous lecture 2 is susceptible to lateral torsional buckling. Lateral torsional buckling (LTB) is the combined lateral (sideways) *deflection and twisting of an unrestrained member subject to bending about its major axis*, as shown in Figure 1. LTB can either occur over the full length of a member or between points of intermediate lateral restraint

إن العارضة غير المقيدة (أي بدون تقييد جانبي كامل، كما هو موضح في المحاضرة 2 السابقة) تكون عرضة للانبعاج الالتوائي الجانبي. الانبعاج الالتوائي الجانبي (LTB) هو الانحراف الجانبي (الجانبي) المدمج والالتواء للعضو غير المقيد الخاضع للانحناء حول محوره الرئيسي ، كما هو مبين في الشكل 1. يمكن أن يحدث LTB على طول العضو بالكامل أو بين نقاط التقييد الجانبي المتوسط



Lateral torsional buckling occurs in unrestrained beams because the compression flange will try to buckle laterally about the beam's more flexible minor axis. The section then twists because the other flange is in tension and is reluctant to buckle. Figure 1 shows the lateral deflection δh, the vertical deflection δv and the twisting θ.

حدث التواء الالتوائي الجانبي في الحزم غير المقيدة لأن شفة الضغط ستحاول الالتواء بشكل جانبي حول المحور الصغير الأكثر مرونة للعارضة. ثم يلتوي القسم لأن الشفة الأخرى تكون في حالة توتر وتكون مترددة في الإبزيم. يوضح الشكل 1 الانحراف الجانبي δh والانحراف العمودي δv والالتواء

When a steel beam is designed, it is usual to first consider the need to provide adequate strength and stiffness against vertical bending. This leads to a member in which the stiffness in the vertical plane is much greater than that in the horizontal plane. Sections normally used as beams have the majority of their material concentrated in the flanges that are made relatively narrow so as to prevent local buckling. Open sections (i.e. I or H sections) are usually used because of the

عندما يتم تصميم عارضة فولاذية، فمن المعتاد أن نأخذ في الاعتبار أولاً الحاجة إلى توفير القوة والصلابة الكافية ضد الانحناء العمودي. يؤدي هذا إلى نقطة تكون فيها الصلابة في المستوى الرأسي أكبر بكثير من تلك الموجودة في المستوى الأفقي. المقاطع التي تستخدم عادةً كعوارض تتركز معظم موادها في الشفاه التي تكون ضيقة نسبيًا لمنع الانبعاج المحلي. عادةً ما يتم استخدام الأقسام المفتوحة (أي الأقسام I أو H) بسبب

حتاج إلى توصيل العارضة إلى الأعضاء الآخرين.   يؤدي الجمع بين كل هذه العوامل
إلى ظهور قسم تكون صلابته الالتوائية والجانبية منخفضة نسبيًا، وهو ما له تأثير كبير
على مقاومة الانبعاج للعضو غير المقيد

need to connect beams to other members. The combination of all these factors results in a section whose torsional and lateral stiffnesses are relatively low, which has a major affect on the buckling resistance of an unrestrained member

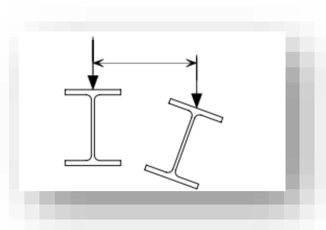
It should be recognized that during the construction phase the member may be unrestrained. *Hence, although the construction load may be less than the final design load, checks on the adequacy of the member should be carried out for the construction phase loading, treating the beam as unrestrained.*

Lateral torsional buckling is only possible where the beam has a less stiff minor axis (i.e. $I_x > I_y$). Hence, circular and square hollow sections need not be designed for lateral torsional buckling.

Situations where lateral torsional buckling has to be taken into account include gantry girders, runway beams and members supporting walls and cladding.

2 FACTORS INFLUENCING BUCKLING RESISTANCE

- The length of the unrestrained span, i.e. the distance between points at which lateral deflection is prevented.
- The lateral bending stiffness of the section.
- The torsional stiffness of the section.
- The conditions of the restraint provided by the end connections.
- The position of application of the applied load and whether or not it is free to to move with the member as it buckles see blow



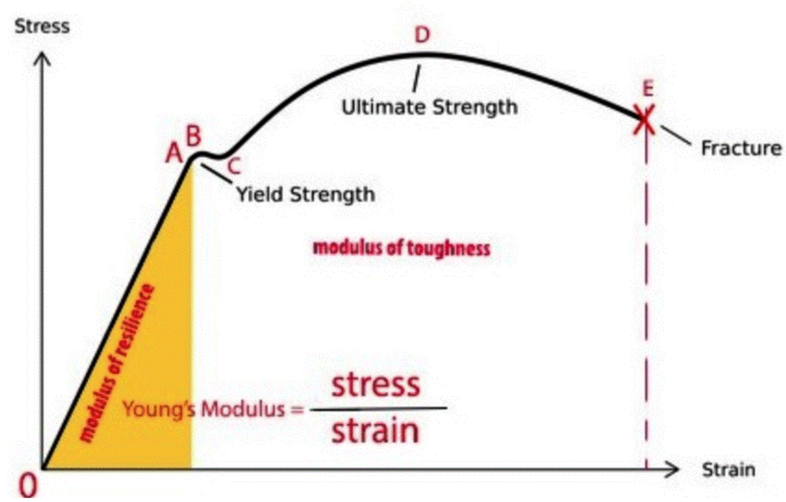
All the factors above are brought together in a single parameter λ_{LT}, the 'equivalent slenderness' of the beam

3 Behavior of beams

- Short stocky members will attain the full plastic moment M_P.
- Long slender members will fail at moments approximately equal to the elastic critical moment Mcr. This is a theoretical value that takes no account of imperfections and residual stress.

Beams of intermediate slenderness fail through a combination of elastic and plastic buckling. Imperfections and residual stresses are most significant in this region.

- الكمرات القصيره والسميكه سوف يصلون إلى MP العزم البلاستيكي الكامل.
- ¬ الكمرات الطويلة النحيلة سوف تفشل في لحظات تساوي تقريبا اللحظة الحرجة المرنة Mcr. هذه قيمة نظرية لا تأخذ في الاعتبار العيوب والضغوط المتبقية.
- تفشل الكمرات ذات النحافة المتوسطة من خلال مزيج من السلوك المرن والبلاستيكي. تعتبر العيوب والضغوط المتبقية هي الأكثر أهمية في هذه المنطقة.



4 Design requirements

4.1 General

The Code states that an unrestrained beam must be checked for local moment capacity of the section and also for buckling resistance. However, lateral torsional buckling need not be checked for the following situations:

- Circular or square hollow sections or solid bars.
- Section bending only about the minor axis.
- I, H or channel sections when the equivalent slenderness λ_{LT} is less than a limiting slenderness value λ_{L0}

4.2 Moment capacity Step 1

The section classification and moment capacity of the section should be determined and checked ***IN THE SAME WAY AS FOR RESTRAINED BEAMS*** i.e

$$\boxed{M_x \leq M_{cx}}$$

where:

Mx is the maximum major axis moment in the segment under consideration

Mcx is the major axis moment capacity of the cross-section

Any reductions for high shear forces should be included in this check

4.3 Buckling resistance Step 2

The buckling resistance of the member between either the ends of the member or any intermediate restraints, a 'segment', should be checked as:

$$\boxed{M_x \leq M_b/m_{LT}}$$

where:

Mx is the maximum major axis moment in the segment under consideration
Mb is the buckling resistance moment
mLT is the equivalent uniform moment factor

Buckling resistance moment M_b

The buckling resistance moment Mb is dependent on the section classification of the member and a bending strength pb that depends on the slenderness of the beam. *Mb* is calculated as follows:

For Class 1 plastic $Mb = pb\ Sx$

For Class 2 compact sections $Mb = pb\ Sx$

For Class 3 semi-compact sections $Mb = pb\ Zx$

For Class 4 slender sections $Mb = pb\ Zx,eff$

where:
pb is the bending strength ***how to Get it***
Sx is the section plastic modulus
Zx is the section elastic modulus
Zx,eff is the section effective elastic modulus

Bending strength *pb*

The value of the bending strength *pb* is obtained from Tables 16 and 17 of BS 5950-1 and depends on the value of the equivalent slenderness λLT and the design strength *py*.
For I and H sections, the equivalent slenderness is given by:

$$\lambda_{LT} = u\ v\ \lambda\ (\beta_w)^{0.5}$$

where:

u is a buckling parameter obtained from section property tables
v is a slenderness factor obtained from Table 19 of BS 5950-1 and depends on λ/x
x is the torsional index, obtained from section property tables
λ is the slenderness, taken as LE/ry
L_E is the effective length between points of restraint
r_y is the radius of gyration about the minor axis

Universal Beams to BS4 Part1 1993 - Dimensions & Properties

Designation	Mass Per metre	Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for Local Buckling		Second Moment of Area		Radius of Gyration		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of section
				Web	Flange			Flange	Web	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y					
		h	b	s	t	r	d	b/2t	d/s	I_x	I_y	r_x	r_y	Z_x	Z_y	S_x	S_y	u	x	H	J	A
	kg/m	mm	mm	mm	mm	mm	mm			cm^4	cm^4	cm	cm	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
1016x305x487	487	1036.1	308.5	30	54.1	30	867.9	2.85	28.9	1021400	26720	41	6.6	19720	1732	23200	2800	0.867	21.1	64.4	4299	620
1016x305x437	437	1025.9	305.4	26.9	49	30	867.9	3.12	32.3	909900	23450	40	6.5	17740	1535	20760	2469	0.868	23.1	55.9	3185	557
1016x305x393	393	1016	303	24.4	43.9	30	868.2	3.45	35.6	807700	20500	40	6.4	15900	1353	18540	2168	0.868	25.5	48.4	2330	500
1016x305x349	349	1008.1	302	21.1	40	30	868.1	3.77	41.1	723100	18460	40	6.4	14350	1223	16590	1941	0.872	27.9	43.3	1718	445
1016x305x314	314	1000	300	19.1	35.9	30	868.2	4.18	45.5	644200	16230	40	6.4	12880	1082	14850	1713	0.872	30.7	37.7	1264	400
1016x305x272	272	990.1	300	16.5	31	30	868.1	4.84	52.6	554000	14000	40	6.4	11190	934	12830	1470	0.873	35	32.2	835	347
1016x305x249	249	980.2	300	16.5	26	30	868.2	5.77	52.6	481300	11750	39	6.1	9821	784	11350	1245	0.861	39.9	26.8	582	317
1016x305x222	222	970.3	300	16	21.1	30	868.1	7.11	54.3	408000	9546	38	5.8	8409	636	9807	1020	0.85	45.7	21.5	390	283
914x419x388	388	921	420.5	21.4	36.6	24.1	799.6	5.74	37.4	719600	45440	38	9.6	15630	2161	17670	3341	0.885	26.7	88.9	1734	494

Table 19 — Slenderness factor v for sections with two plain flanges

λ/x	Unequal flanges, larger flange in compression (Compression / Tension)				Equal flanges	Unequal flanges, smaller flange in compression (Compression / Tension)			
	η				η	η			
	0.9	**0.8**	**0.7**	**0.6**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**
0.5	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20
1.0	0.80	0.83	0.87	0.92	0.99	1.10	1.27	1.53	2.11
1.5	0.80	0.82	0.86	0.91	0.97	1.08	1.24	1.48	1.98
2.0	0.78	0.81	0.85	0.89	0.96	1.06	1.20	1.42	1.84

βw = 1.0 for Class 1 and Class 2 sections

βw = Zx/Sx for Class 3 sections when Zx is used to calculate Mb

βw = $Zx,eff/Sx$ for Class 4 sections.

For a quick and conservative design of rolled I or H sections with equal flanges, u may be taken as 0.9, v may be taken as 1.0 and x may be taken as D/T, where D is the depth of the section and T is the thickness of the flange.

5 Effective length

5.1 Beams without intermediate lateral restraints

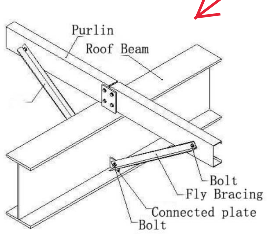
The simple model of lateral torsional buckling on which the Codes rules are based, assumes that the ends of the member are effectively pinned in both the vertical and horizontal planes. The type of restraint provided in practice at the ends of the member needs to be considered and this is done by use of an effective length; the effective length may be greater than or less than the actual length of the member between restraints.

Values of effective length L are given in BS 5950-1 Table 13 for beams and Table 14 for cantilevers. Part of Table 13 is reproduced here.

Table 13 — Effective length L_E for beams without intermediate restraint

Conditions of restraint at supports		Loading condition: Normal	Loading condition: Destabilizing
Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as given in **4.2.2**.	Both flanges fully restrained against rotation on plan.	$0.7L_{LT}$	$0.85L_{LT}$
	Compression flange fully restrained against rotation on plan.	$0.75L_{LT}$	$0.9L_{LT}$
	Both flanges partially restrained against rotation on plan.	$0.8L_{LT}$	$0.95L_{LT}$
	Compression flange partially restrained against rotation on plan.	$0.85L_{LT}$	$1.0L_{LT}$
	Both flanges free to rotate on plan.	$1.0L_{LT}$	$1.2L_{LT}$
Compression flange laterally unrestrained. Both flanges free to rotate on plan.	Partial torsional restraint against rotation about longitudinal axis provided by connection of bottom flange to supports.	$1.0L_{LT} + 2D$	$1.2L_{LT} + 2D$
	Partial torsional restraint against rotation about longitudinal axis provided only by pressure of bottom flange onto supports.	$1.2L_{LT} + 2D$	$1.4L_{LT} + 2D$

D is the overall depth of the beam.



ايبام بدون قيود جانبية

النموذج البسيط للانبعاج الالتوائي الجانبي يفترض أن نهايات العضو مثبتة بشكل فعال في كل من االمستويات الرأسية والأفقية. وعليه يتاثر الطول الفعال بطريقه تثبيت طرفي العضو تحت الدراسه وعليه قد يكون الطول الفعال أكبر أو أقل من الطول الفعلي للعضو بين طرفي التثبيت

.

For most beams, the effective length will be less than or equal to the actual length. However, if the member is torsionally unrestrained at the end, or the load is destabilising then the effective length may be greater than the actual length. This is reflected in the values given in Tables 13 and 14 of the code.

5.2 Destabilising loads

Destabilizing loads are loads that are applied to the beam above the shear centre and are free to move with the beam as it deflects laterally and twists (see Figure .4). Such loads increase the twist on the beam and induce additional stresses. Therefore, destabilising loads reduce the resistance of a member to lateral torsional buckling and to account for this the effective length is increased as shown in Table 13. Also the equivalent uniform moment factor *mLT* should be taken as 1.0. Theoretically, the effective length could be decreased if the load was applied below the shear center but the code makes no allowance for this

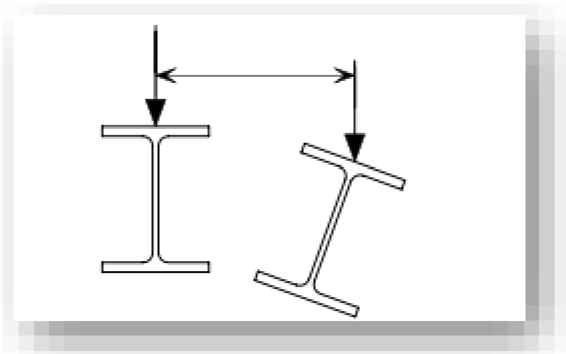


Figure 4 Destabilizing loads

5.3 Beams with intermediate lateral restraints

A segment, i.e. a length of beam between intermediate lateral restraints, can be designed as a member without intermediate restraints. The effective length of the segment should be taken as $1.0\ L_{LT}$ for normal loading conditions and $1.2\ L_{LT}$ for destabilizing loads, where L_{LT} is the length of the relevant segment between restraints.

6 Equivalent uniform moment factor m_{LT}

$$M_x \leq M_b/m_{LT}$$

The values of the bending strength given in Tables 16 and 17 of BS 5950-1 are for a beam subject to uniform moment, as shown in Figure .6. In members that are subject to non-uniform moments the compressive force in the flange varies along the beam and the beam is likely to be able to sustain a higher peak value of bending moment than if the moment were uniform.

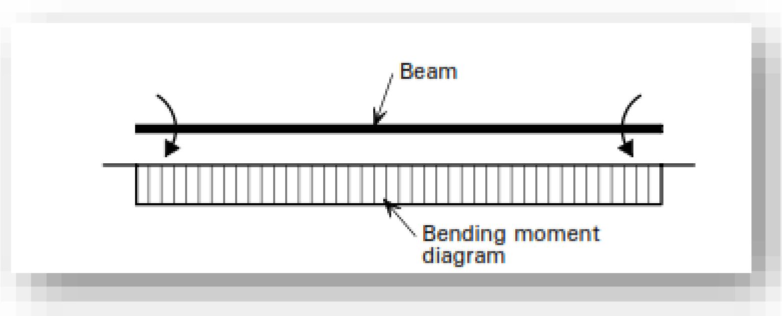


Figure.6 Member subject to uniform moment

The equivalent uniform moment factor m_{LT} takes account of the shape of the bending moment diagram between restraints and is obtained from Table 18 BS 5950-1

The first part of the table deals with linear moment gradients, i.e. members with no load between restraints.

The second part deals with specific cases of members that are subject to transverse loading

The third part provides a general formulae from which m_{LT} may be calculated for more complex cases such as continuous beams. The general formulae may be used to derive the values of m_{LT} in the first two parts of the table.

$$m_{LT} = 0.2 + \frac{0.15M_2 + 0.5M_3 + 0.15M_4}{M_{max}} \quad \text{but} \quad m_{LT} \geq 0.44$$

DON'T USE THE FORMULA USE THE TABLES

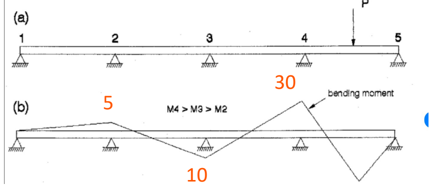
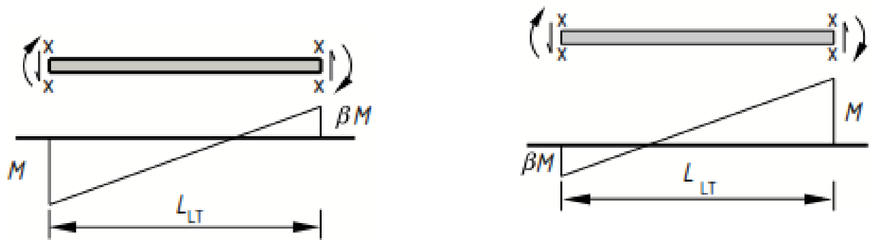


Table 18 — Equivalent uniform moment factor m_{LT} for lateral-torsional buckling *(continued overleaf)*

Segments with end moments only (values of m_{LT} from the formula for the general case)	β	m_{LT}
β positive	1.0	1.00
	0.9	0.96
	0.8	0.92
	0.7	0.88
	0.6	0.84
	0.5	0.80
	0.4	0.76
	0.3	0.72
	0.2	0.68
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

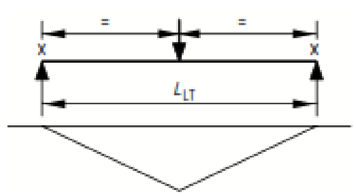
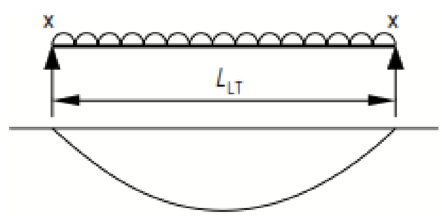
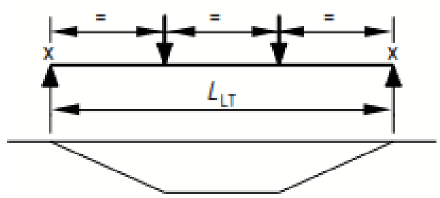
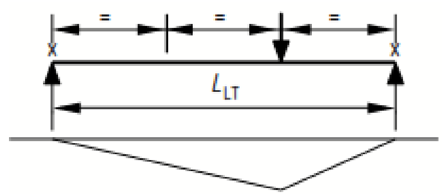
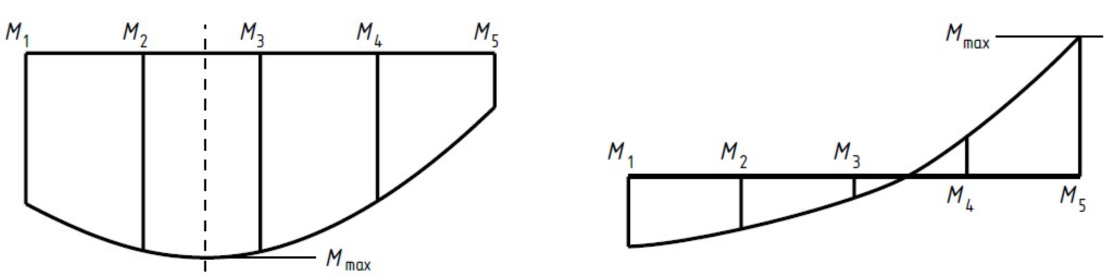
Specific cases (no intermediate lateral restraint)	
 $m_{LT} = 0.850$	 $m_{LT} = 0.925$
 $m_{LT} = 0.925$	 $m_{LT} = 0.744$

Table 18 —Equivalent uniform moment factor m_{LT} for lateral-torsional buckling *(continued)*

General case (segments between intermediate lateral restraints)



For beams: $$m_{LT} = 0.2 + \frac{0.15M_2 + 0.5M_3 + 0.15M_4}{M_{max}} \quad \text{but} \quad m_{LT} \geq 0.44$$

All moments are taken as positive. The moments M_2 and M_4 are the values at the quarter points, the moment M_3 is the value at mid-length and M_{max} is the maximum moment in the segment.

For cantilevers without intermediate lateral restraint: $m_{LT} = 1.00$.

7 Summary of design procedure

1. Select section and steel grade
2. Determine design strength py — Table 9
3. Determine the section classification — Tables 11, 12
4. For Class 1 and Class 2 sections use the gross section properties **Sxx**
5. For Class 3 semi-compact sections calculate the effective Elastic modulus **Zxx** Cl. 3.6
6. For Class 4 slender sections calculate the effective elastic modulus **avoid**
7. Calculate local moment capacity allowing for shear, as for a restrained beam ***Low shear or high shear if high shear reduce capacity of moment***
8. Determine actual unrestrained length and effective length
9. Calculate slenderness $\lambda = Le/ry$
10. Determine the slenderness factor v using λ/x Table 19
11. Calculate βw
    βw = 1.0 for Class 1 and Class 2 sections
    βw = Zx/Sx for Class 3 sections when Zx is used to calculate Mb
    βw = Zx,eff/Sx for Class 4 sections.
12. Calculate the equivalent slenderness λLT
13. Determine pb using λLT and the design strength py *Table 16 & table 17*
14. Calculate the buckling resistance moment Mb appropriate for the section classification
15. Determine the equivalent uniform moment factor mLT Table 18
16. Check the maximum applied moment against the value of Mb/mLT

17. Calculate the deflections and check against appropriate limit.

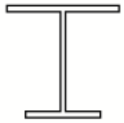
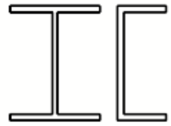
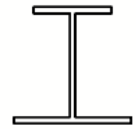
Table 16 — Bending strength p_b (N/mm²) for rolled sections

λ_{LT}	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	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

Table 17 — Bending strength p_b (N/mm²) for welded sections

λ_{LT}	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	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

Table 19 — Slenderness factor ν for sections with two plain flanges

λ/x	Unequal flanges, larger flange in compression				Equal flanges	Unequal flanges, smaller flange in compression			
	 Compression Tension					Compression Tension 			
	η				η	η			
	0.9	**0.8**	**0.7**	**0.6**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**
0.5	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20
1.0	0.80	0.83	0.87	0.92	0.99	1.10	1.27	1.53	2.11
1.5	0.80	0.82	0.86	0.91	0.97	1.08	1.24	1.48	1.98
2.0	0.78	0.81	0.85	0.89	0.96	1.06	1.20	1.42	1.84
2.5	0.77	0.80	0.83	0.88	0.93	1.03	1.16	1.35	1.70
3.0	0.76	0.78	0.82	0.86	0.91	1.00	1.12	1.29	1.57
3.5	0.74	0.77	0.80	0.84	0.89	0.97	1.07	1.22	1.46
4.0	0.73	0.75	0.78	0.82	0.86	0.94	1.03	1.16	1.36
4.5	0.71	0.73	0.76	0.80	0.84	0.91	0.99	1.11	1.27
5.0	0.70	0.72	0.75	0.78	0.82	0.88	0.95	1.05	1.20
5.5	0.68	0.70	0.73	0.76	0.79	0.85	0.92	1.01	1.13
6.0	0.67	0.69	0.71	0.74	0.77	0.82	0.89	0.97	1.07
6.5	0.65	0.67	0.70	0.72	0.75	0.80	0.86	0.93	1.02
7.0	0.64	0.66	0.68	0.70	0.73	0.78	0.83	0.89	0.97
7.5	0.63	0.65	0.67	0.69	0.72	0.76	0.80	0.86	0.93
8.0	0.62	0.63	0.65	0.67	0.70	0.74	0.78	0.83	0.89
8.5	0.60	0.62	0.64	0.66	0.68	0.72	0.76	0.80	0.86
9.0	0.59	0.61	0.63	0.64	0.67	0.70	0.74	0.78	0.83
9.5	0.58	0.60	0.61	0.63	0.65	0.68	0.72	0.76	0.80
10.0	0.57	0.59	0.60	0.62	0.64	0.67	0.70	0.74	0.78
11.0	0.55	0.57	0.58	0.60	0.61	0.64	0.67	0.70	0.73
12.0	0.54	0.55	0.56	0.58	0.59	0.61	0.64	0.66	0.70
13.0	0.52	0.53	0.54	0.56	0.57	0.59	0.61	0.64	0.66
14.0	0.51	0.52	0.53	0.54	0.55	0.57	0.59	0.61	0.63
15.0	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61
16.0	0.48	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59
17.0	0.47	0.48	0.49	0.49	0.50	0.52	0.53	0.55	0.57
18.0	0.46	0.47	0.47	0.48	0.49	0.50	0.52	0.53	0.55
19.0	0.45	0.46	0.46	0.47	0.48	0.49	0.50	0.52	0.53
20.0	0.44	0.45	0.45	0.46	0.47	0.48	0.49	0.50	0.51

NOTE For T-sections see **B.2.8**.