INTERNATIONAL UNIVERSITY OF AFRICA

CIVIL ENGINEERING DEPARTMENT

ANALYSIS AND DESIGN OF STEEL WORKS II

GRADE 4

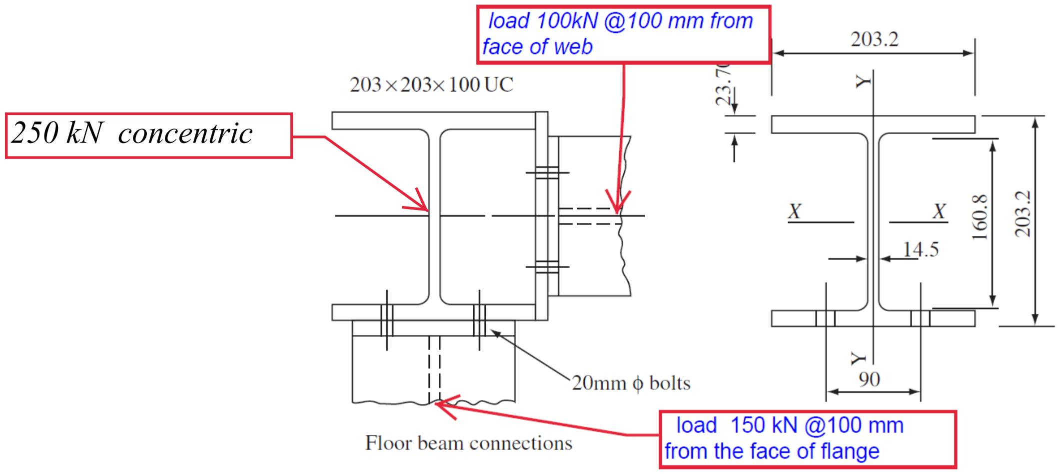
7TH SEMESTER

lec 8 axial+moment member (columns)

solved example

prepared by Dr. Talaat

A steel stanchion of 203 203 100 UC of length 8m and grade S355 grade of steel in a multi-storey building frame is under a factored concentric مركزيه *axial force of 250 kN, a factored reaction from beam of 100 kN with nominal eccentricities* لامركزيه *of 100mm from the face of the web and another factored reaction from beam of 150 kN of 100mm nominal eccentricity from the face of the flange. Check the adequacy of the stanchion. The effective length for flexural column buckling is 1.0 of its physical length and the effective length for lateral-torsional beam buckling is 0.5 of its physical length.*



Solution

SECTION PROPERTIES

$D = 228.6mm$, $B = 210.3mm$, $t = 14.5mm$, $T = 23.7mm$, $d = 160.8mm$, $I_x = 11300cm^4$,

$I_y = 3680cm^4$, $r_x = 9.44cm$, $r_y = 5.39cm$, $Z_x = 988cm^3$, $Z_y = 350cm^3$, $S_x = 1150cm^3$,

$S_y = 534cm^3$, $u = 0.852$, $x = 9.02$, $A = 127cm^2$

DESIGN LOAD

Axial load, $F_c = 250 + 100 + 150 = 500kN$

Moment about major axis, $M_x = 150 \times (228.6/2 + 100) \times 10^{-3} = 32.1kNm$

Moment about minor axis, $M_y = 100 \times (14.5/2 + 100) \times 10^{-3} = 10.7kNm$

SECTION CLASSIFICATION

Design strength, $p_y = 345N/mm^2$ for $16mm < T \leq 40mm$

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

Table 9 *T.==32 mm ≥16 mm Py= 345 N/mm²* **Table 11**

Table 9 — Design strength p_y

Steel grade	Thickness[a] less than or equal to mm	Design strength p_y N/mm²
S 275	16	275
	40	265
	63	255
	80	245
	100	235
	150	225
S 355	16	355
	40	345

Table 11 — Limiting width-to-thickness ratios for sections other than CHS and RHS

Compression element			Ratio[a]	Limiting value[b] Class 1 plastic	Class 2 compact	Class 3 semi-compact
Outstand element of compression flange	Rolled section		b/T	9ε	10ε	15ε
	Welded section		b/T	8ε	9ε	13ε
Internal element of compression flange	Compression due to bending		b/T	28ε	32ε	40ε
	Axial compression		b/T	Not applicable		
Web of an I-, H- or box section[c]	Neutral axis at mid-depth		d/t	80ε	100ε	120ε
	Generally[d]	If r_1 is negative:	d/t		$\frac{100\varepsilon}{1+r_1}$	
		If r_1 is positive:	d/t	$\frac{80\varepsilon}{1+r_1}$ but $\geq 40\varepsilon$	$\frac{100\varepsilon}{1+1.5r_1}$ but $\geq 40\varepsilon$	$\frac{120\varepsilon}{1+2r_2}$ but $\geq 40\varepsilon$

Plastic limiting value of b/T for outstand flange of an H-section is 9ε

$$\frac{b}{T} = \frac{210.3}{2 \times 23.7} = 4.44 \leq 9 \times 0.89 = 8.01$$

$\therefore$ flange is plastic

$$\frac{d}{t} = \frac{160.8}{14.5} = 11.1 < \frac{80 \times 0.89}{} = 71.2$$

$\therefore$ web is plastic

$\therefore$ the section is Class 1 plastic

MOMENT CAPACITY

Moment capacity,

$$M_{cx} = p_y S_x \leq 1.2 p_y Z_x$$

$$= 345 \times 1150 \times 10^3 \leq 1.2 \times 345 \times 988 \times 10^3$$

$$= 396.8kNm \leq 409.0kNm$$

$$> M_x \quad (OK)$$

$$M_{cy} = p_y S_y \leq 1.2 p_y Z_y$$

$$= 345 \times 534 \times 10^3 = 184.2kNm$$

but $> 1.2 \times 345 \times 350 \times 10^3 = 144.9kNm$

$$\therefore M_{cy} = 144.9kNm > M_y \quad (OK)$$

Check for section in local buckling

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

F_c=250 kN *A_g=127 cm2* *P_y =345 N/mm^2* *Mx=*

M_x=32.1 kN.m *M_y=10.7 kN/mm^2*

Mcx =396.8 kN.m *Mcy=144.9 kN.m*

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{500 \times 10^3}{12700 \times 345} + \frac{32.1}{396.8} + \frac{10.7}{144.9} = 0.27 \leq 1 \quad (OK)$$

Check of section in Global buckling

MEMBER BUCKLING RESISTANCE

Equivalent moment factor, $m_x \quad m_y \quad m_{LT} = 1$ for simple construction

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

1-General equation

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

2-Buckling About The Minor Axis

*In the general equation we can take mx and my =1 **very very very** conservative assumption but acceptable see Table 26*

Then general eq

$$\frac{500}{934.7} + \frac{1x32.1x10^3}{345x988} + \frac{1x10.7x10^3}{345x350} = 0.771 < 1 \qquad safe\ ok\ eqauation\ 1\ satisfied$$

Buckling in the *minor* axis

- Get m_{LT} *from table 18 **very very very** conservative =1*

For Class 1 plastic	*Mb = pb Sx*
For Class 2 compact sections	*Mb = pb Sx*

our case section os calssified as plastic

- where:

pb is the bending strength ***how to Get it***

Sx is the section plastic modulus

- *Zx* is the section elastic modulus

- 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.

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

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

Effective length, $L_E = 0.5L_{LT} = 0.5\times 8 = 4m$

Slenderness ratio, $\lambda = \dfrac{L_E}{r_y} = \dfrac{4000}{53.9} = 74.2$

$$\frac{\lambda}{x} = \frac{74.2}{9.02} = 8.2 \quad \textit{see table 19 to get v}$$

$v=0.691$

$\beta_w = 1.0$ for Class 1 plastic section

Equivalent slenderness, $\lambda_{LT} = uv\lambda\sqrt{\beta_w} = 0.852\times 0.691\times 74.2\times 1 = 43.7$

Determine pb using λLT and the design strength py *Table 16 & table 17*

Buckling strength, $p_b = 305.9\ N/mm^2$

Buckling resistance, $M_b = p_b S_x = 305.9\times 1150\times 10^3 = 351.8kNm$

Equivalent uniform moment factor, $m_{LT} = 1.0$

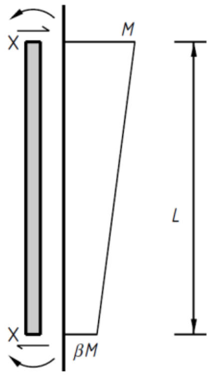
$m_{LT}M_x = 1.0\times 32.7 = 32.7kNm \le M_b$ (OK)

$$\frac{F_c}{P_{cy}} + \frac{m_{LT}M_{LT}}{M_b} + \frac{m_y \overline{M}_y}{M_{cy}} = \frac{500}{934.7} + \frac{32.1}{351.8} + \frac{10.7\times 10^3}{345\times 350} = 0.73 \le 1$$

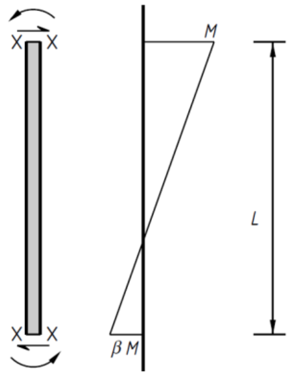
Table 26 — Equivalent uniform moment factor m for flexural buckling

Segments with end moments only (values of m from the formula for the general case)

β positive



β negative



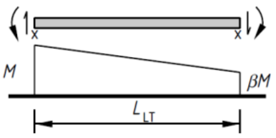
β	m
1.0	1.00
0.9	0.96
0.8	0.92
0.7	0.88
0.6	0.84
0.5	0.80
0.4	0.76
0.3	0.72
0.2	0.68
0.1	0.64
0.0	0.60
− 0.1	0.58
− 0.2	0.56
− 0.3	0.54
− 0.4	0.52
− 0.5	0.50
− 0.6	0.48
− 0.7	0.46
− 0.8	0.44
− 0.9	0.42
− 1.0	0.40

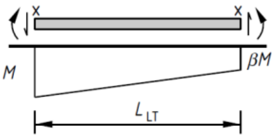
Segments between intermediate lateral restraints

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)

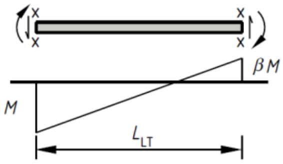
β positive

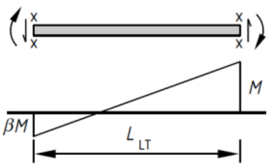




X Lateral restraint

β negative





β	m_{LT}
1.0	1.00
0.9	0.96
0.8	0.92
0.7	0.88
0.6	0.84
0.5	0.80
0.4	0.76
0.3	0.72
0.2	0.68
0.1	0.64
0.0	0.60
− 0.1	0.56
− 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

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

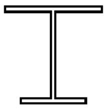
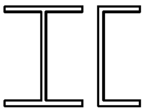
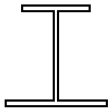
λ/x	Unequal flanges, larger flange in compression  Compression Tension				Equal flanges 	Unequal flanges, smaller flange in compression  Compression Tension			
	η				η	η			
	0.9	**0.8**	**0.7**	**0.6**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**
0.5	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20
1.0	0.80	0.83	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
2.5	0.77	0.80	0.83	0.88	0.93	1.03	1.16	1.35	1.70
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

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	$\boxed{302}$	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