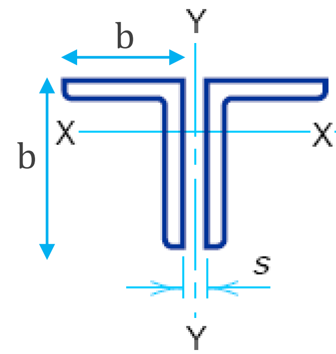
Unstiffened element:-

A. Single angle or double angle:-

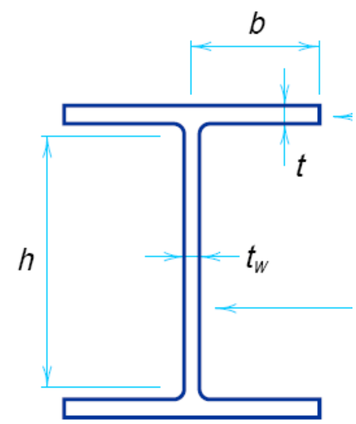


$$\lambda_r = 0.45\sqrt{\frac{E}{F_y}} > \frac{b}{t} \quad \Rightarrow \quad Q_s = 1$$

$$0.45\sqrt{\frac{E}{F_y}} < \frac{b}{t} < 0.91\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{E}{F_y}}$$

$$\frac{b}{t} > 0.91\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = \frac{0.534 * E}{F_y * \left(\frac{b}{t}\right)^2}$$

B. Flanges of W-shape and channels:-

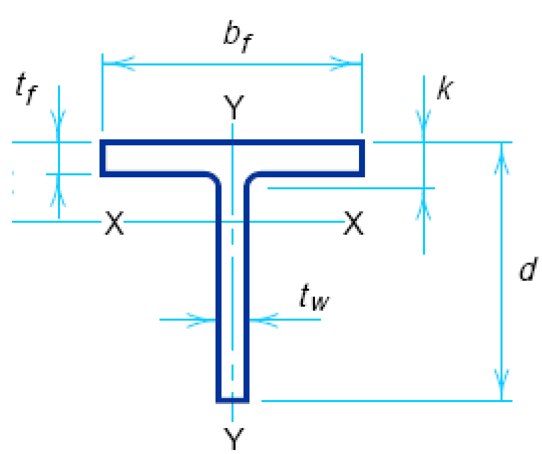


$$\lambda_r = 0.56\sqrt{\frac{E}{F_y}} > \frac{b}{t} \quad \Rightarrow \quad Q_s = 1$$

$$0.56\sqrt{\frac{E}{F_y}} < \frac{b}{t} < 1.03\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{E}{F_y}}$$

$$\frac{b}{t} > 1.03\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = \frac{0.69 * E}{F_y * \left(\frac{b}{t}\right)^2}$$

C. Stem of tee:-

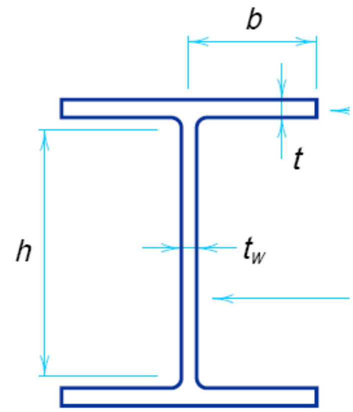


$$\lambda_r = 0.75\sqrt{\frac{E}{F_y}} < \frac{d}{t} \quad \Rightarrow \quad Q_s = 1$$

$$0.75\sqrt{\frac{E}{F_y}} < \frac{d}{t} < 1.03\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = 1.908 - 1.22\left(\frac{d}{t}\right)\sqrt{\frac{E}{F_y}}$$

$$\frac{d}{t} > 1.03\sqrt{\frac{E}{F_y}} \quad \Rightarrow \quad Q_s = \frac{0.69 * E}{F_y * \left(\frac{d}{t}\right)^2}$$

Stiffened elements:-



$$If\ \frac{h}{t_w} \geq 1.49\sqrt{\frac{E}{F_y}}$$

$$b_e = 1.91 * t_w * \sqrt{\frac{E}{f_{cr}}} * \left(1 - \left\{\left(\frac{0.34}{h/t_w}\right)\sqrt{\frac{E}{f_{cr}}}\right\}\right) < h$$

$$Q_a = \frac{\sum Area\ effective}{A} = \frac{A - (b^* * t_w)}{A} \quad ,where\ \ b^* = h - \ b_e$$

Column formulas:-

$$\emptyset P_n = 0.85 * \ A_g \ * F_{cr}$$

If $\lambda_c * \sqrt{Q} > 1.5 \Rightarrow$ (Elastic buckling, long column)

$$F_{cr} = \frac{0.877}{{\lambda_c}^2} * F_y$$

If $\lambda_c * \sqrt{Q} \ \leq \ 1.5 \Rightarrow$ (Inelastic buckling, short and intermediate column)

$$F_{cr} = Q * (0.658)^{Q*{\lambda_c}^2} * F_y$$

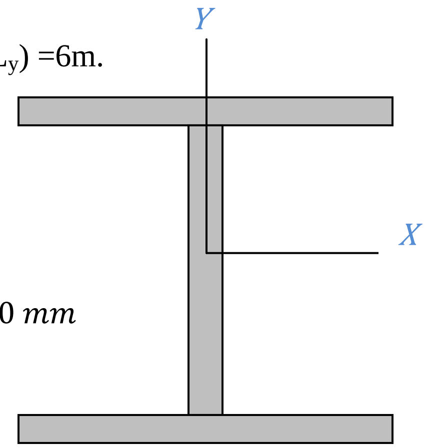
Where

$$\lambda_c = \frac{KL/r}{\pi} * \sqrt{\frac{F_y}{E}}$$

Example 1)

Determine the factor tensile strength of W shape 360X64, (KL_x) =9 m, (KL_y) =6m.

F_y=345 Mpa.



Given:

$$A_g = 8140\ mm^2, r_x = 148\ mm \quad , r_y = 48.2\ mm,$$

$$b = \frac{b_f}{2} = 101.5\ mm \ , \qquad t_f = 13.5\ mm\ , t_w = 7.7\ mm\ , h = 280\ mm$$

Solution:-

1. <u>Check the local buckling</u>

a) **<u>Unstiffened element:-</u>**

$$\lambda_r = 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{200000}{345}} = 13.48$$

$$\frac{b}{t} = \frac{101.5}{13.5} = 7.52$$

$$\lambda_r = 0.56\sqrt{\frac{E}{F_y}} > \frac{b}{t} \quad \Rightarrow \quad Q_s = 1 \ \ (no\ local\ buckling)$$

b) **<u>Stiffened elements:-</u>**

$$1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{200000}{345}} = 35.87$$

$$\frac{h}{t_w} = \frac{280}{7.7} = 36.36$$

$$\frac{h}{t_w} \geq 1.49\sqrt{\frac{E}{F_y}}$$

$$b_e = 1.91 * t_w * \sqrt{\frac{E}{f_{cr}}} * \left(1 - \left\{\left(\frac{0.34}{h/t_w}\right)\sqrt{\frac{E}{f_{cr}}}\right\}\right) < h$$

$$Q_a = \frac{\sum Area\ effective}{A} = \frac{A - (b^* * t_w)}{A} \quad ,where\ \ b^* = h - \ b_e$$

$$Q = Q_a * Q_s = Q_a * 1 = Q_a$$

Frist assume $Q_a = 1$

$$(KL_x/r_x) = \frac{9000}{148} = 60.81\ mm$$

$$\left(KL_y/r_y\right) = \frac{6000}{48.2} = 124.48\ mm$$

Buckling @ Y-Y axis is critical:

$$\lambda_c = \frac{KL_y/r_y}{\pi} * \sqrt{\frac{F_y}{E}} = \frac{124.48}{\pi} * \sqrt{\frac{345}{200000}} = 1.645 > 1.5$$

$$F_{cr} = \frac{0.877}{{\lambda_c}^2} * F_y = \frac{0.877}{1.645^2} * 345 = 111.7174\ Mpa$$

$$b_e = 1.91 * 7.7 * \sqrt{\frac{200000}{111.60}} * \left(1 - \left\{\left(\frac{0.34}{280/7.7}\right)\sqrt{\frac{200000}{111.60}}\right\}\right) = 376.16\ mm > h$$

$$\emptyset P_n = 0.85 * A_g * F_{cr} = 0.85 * 8140 * 111.60 * 10^{-3} = 772.973\ Kn$$

<u>By using LRFD table:-</u>

At (KL_y) = 6 m⇒ $\emptyset P_n = 773\ Kn$
Since (KL_y) < (KL_x) check the buckling strength about major axis.

$$(KL)_{eq} = \frac{9}{3.07} = 2.932\ m$$

If $(KL)_{eq} < KL_y$ ∴ Major buckling is not problem ⇒ $\emptyset P_n = 773\ Kn$

$F_y = 345$ MPa

COLUMNS
W shapes
Design axial strength in kilonewtons ($\phi = 0.85$)

Designation	W360													
kg/m	122		110		101		91		79		72		64	
F_y	250	345	250	345	250	345	250	345	250	345	250	345	250	345*
Effective length KL (m) with respect to least radius of gyration r_y														
0	3290	4550	2980	4110	2740	3780	2470	3400	2150	2960	1940	2670	1730	2390
2.25	3080	4140	2780	3740	2560	3440	2300	3090	1920	2540	1730	2280	1540	2040
2.50	3030	4050	2740	3660	2520	3370	2260	3020	1870	2450	1680	2200	1500	1960
2.75	2980	3950	2690	3570	2470	3280	2220	2950	1810	2350	1630	2110	1460	1880
3.00	2920	3850	2640	3480	2430	3200	2180	2870	1760	2250	1580	2020	1410	1800
3.25	2860	3740	2580	3380	2380	3110	2130	2780	1700	2140	1530	1920	1360	1710
3.50	2800	3630	2530	3280	2320	3010	2080	2700	1640	2040	1470	1830	1310	1620
3.75	2730	3510	2470	3170	2270	2910	2030	2610	1570	1930	1410	1730	1250	1530
4.00	2660	3380	2400	3060	2210	2810	1980	2510	1510	1820	1350	1620	1200	1440
4.25	2590	3260	2340	2950	2150	2700	1920	2410	1440	1700	1290	1520	1150	1350
4.50	2510	3130	2270	2830	2080	2590	1870	2320	1370	1590	1230	1420	1090	1260
4.75	2440	3000	2200	2710	2020	2480	1810	2220	1300	1490	1160	1320	1030	1170
5.00	2360	2870	2130	2590	1950	2370	1750	2120	1230	1380	1100	1230	978	1090
5.50	2200	2600	1990	2360	1820	2150	1630	1920	1100	1170	979	1040	867	920
6.00	2040	2340	1840	2120	1680	1930	1500	1720	966	987	860	876	761	773
6.50	1870	2090	1700	1890	1550	1720	1380	1530	841	841	746	746	659	659
7.00	1710	1840	1550	1670	1410	1520	1260	1340	725	725	644	644	568	568
7.50	1550	1610	1410	1460	1280	1320	1140	1170	632	632	561	561	495	495
8.00	1400	1410	1270	1280	1150	1160	1020	1030	555	555	493	493	435	435
8.50	1250	1250	1140	1140	1030	1030	911	911	492	492	436	436	385	385
9.00	1120	1120	1010	1010	918	918	813	813	439	439	389	389	344	344
9.50	1000	1000	909	909	824	824	729	729	394	394	349	349	308	308
10.00	905	905	820	820	744	744	658	658						
10.50	821	821	744	744	675	675	597	597						
11.00	748	748	678	678	615	615	544	544						
Properties														
u	2.85	2.68	2.82	2.62	2.80	2.56	2.74	2.44	3.20	2.70	3.12	2.56	2.97	2.37
P_{wo} (kN)	683	942	570	787	512	706	439	606	435	600	376	519	327	452
P_{wi} (kN/mm)	3.25	4.49	2.85	3.93	2.63	3.62	2.38	3.28	2.35	3.24	2.15	2.97	1.93	2.66
P_{wb} (kN)	1160	1370	781	918	610	717	452	531	437	513	335	394	241	283
P_{fb} (kN)	662	914	557	769	471	650	378	522	397	548	321	442	256	354
L_p (m)	3.14	2.67	3.14	2.67	3.12	2.65	3.09	2.63	2.43	2.07	2.41	2.06	2.40	2.04
L_r (m)	13.0	9.01	12.0	8.47	11.3	8.04	10.5	7.59	8.53	6.16	7.98	5.85	7.49	5.57
A (mm^2)	15500		14000		12900		11600		10100		9110		8140	
$I_x/10^6$ (mm^4)	365		331		302		267		227		201		178	
$I_y/10^6$ (mm^4)	61.5		55.7		50.6		44.8		24.2		21.4		18.9	
r_y (mm)	63.0		63.1		62.6		62.1		48.9		48.5		48.2	
Ratio r_x/r_y	2.43		2.44		2.44		2.45		3.07		3.07		3.07	
$P_{ex}(KL)^2/10^3$ (kN-m^2)	716		655		596		529		449		399		352	
$P_{ey}(KL)^2/10^3$ (kN-m^2)	121		110		100		88.3		47.7		42.3		37.3	

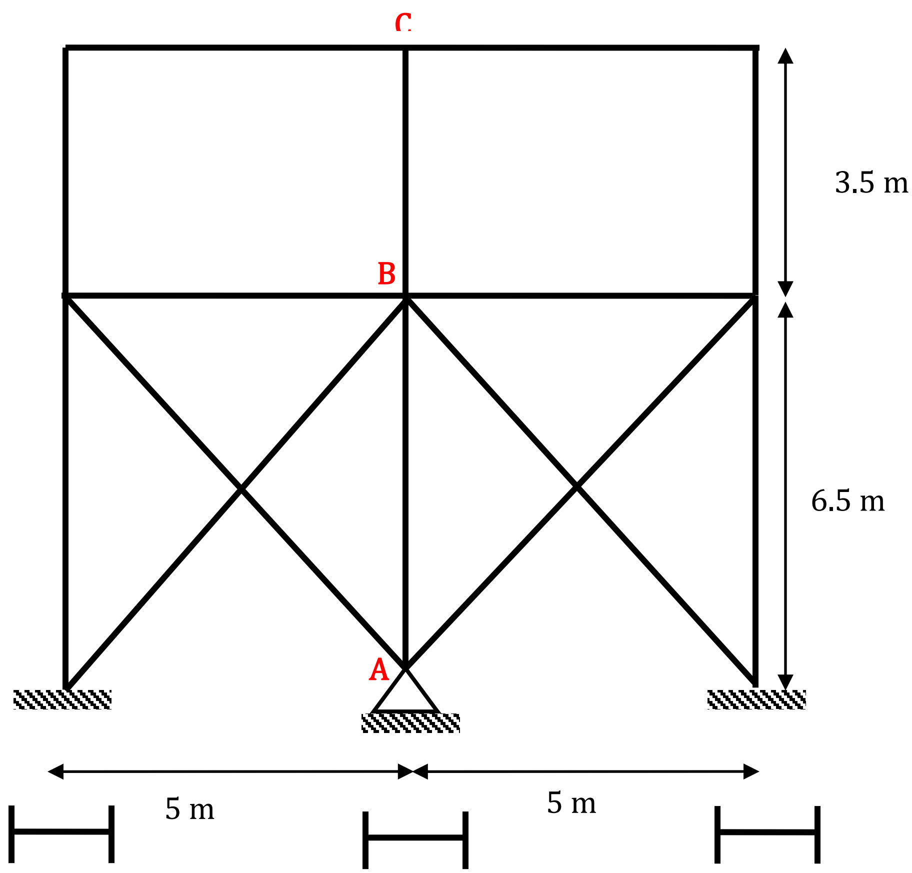
Example 2:

a) Compute the bucklng length factor for two story for column (A-B) and (B-C) ,if the columns and beams have section 310X79 ,360X101 respectively.
b) Design the column (A-B) if P_d=500 Kn,P_L=300 Kn ,the steel grade 250 Mpa, assume(K_x L_x =K_y L_y =6.5 m).

$$for\ columns\ (310X79)\ \ I_x = 177 * 10^6\ mm^4$$

$$for\ beams\ (360X101)\ \ I_x = 302 * 10^6\ mm^4$$

$$G = 10\ (\ hinge\ support)\ ,\qquad G = 1\ (\ fixed\ support)$$



Solution:

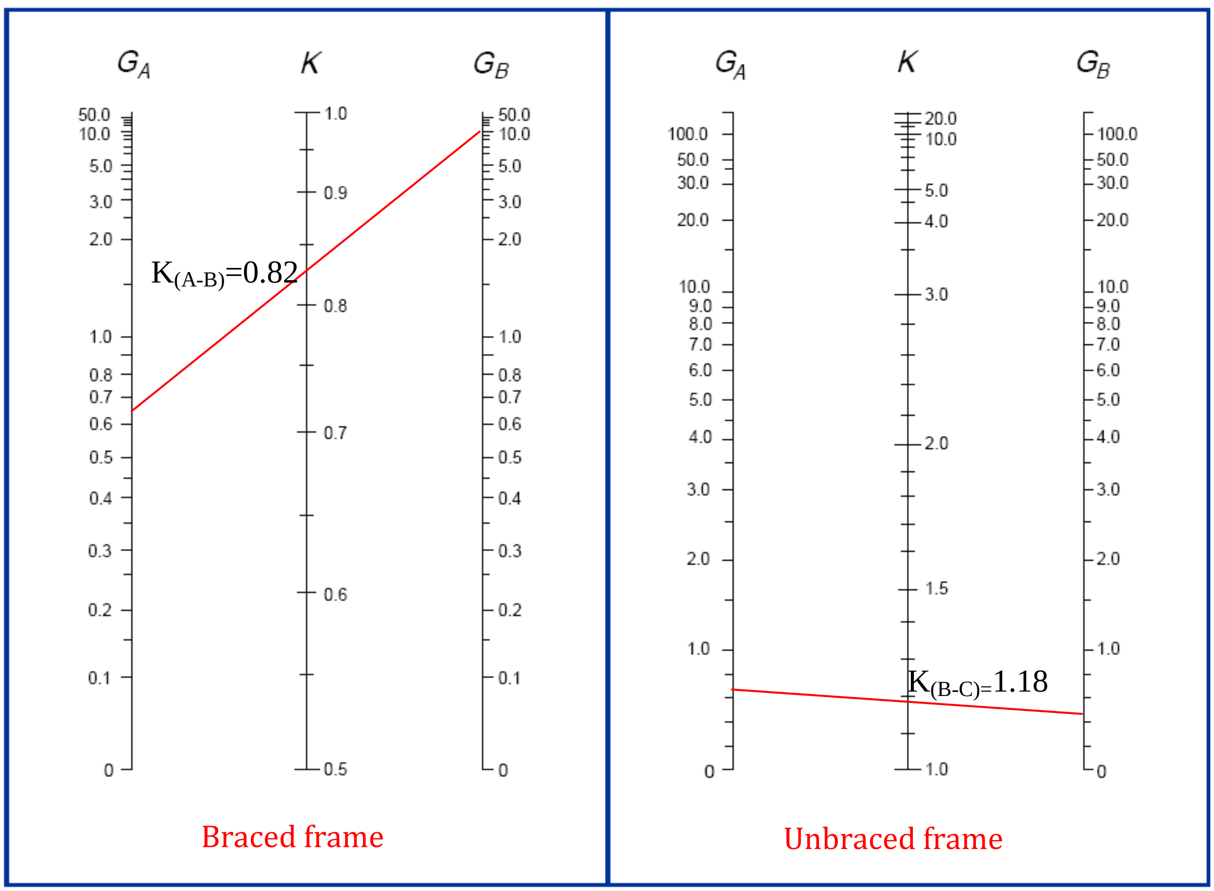
<u>Column (A-B) is braced</u> $G_A = 10\ (hinge\ support)$

$$G_B = \frac{\sum\left(\frac{I}{L}\right)_C}{\sum\left(\frac{I}{L}\right)_B} = \frac{\left(\frac{177*10^6}{3500}\right)+\left(\frac{177*10^6}{6500}\right)}{\left(\frac{302*10^6}{5000}\right)+\frac{302*10^6}{5000}} = 0.644$$

<u>Column (B-C) is unbraced</u>

$$G_B = \frac{\sum\left(\frac{I}{L}\right)_C}{\sum\left(\frac{I}{L}\right)_B} = \frac{\left(\frac{177*10^6}{3500}\right)+\left(\frac{177*10^6}{6500}\right)}{\left(\frac{302*10^6}{5000}\right)+\frac{302*10^6}{5000}} = 0.644$$

$$G_C = \frac{\sum\left(\frac{I}{L}\right)_C}{\sum\left(\frac{I}{L}\right)_B} = \frac{\left(\frac{177*10^6}{3500}\right)}{\left(\frac{302*10^6}{5000}\right)+\frac{302*10^6}{5000}} = 0.419$$



Braced frame

Unbraced frame

The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as b)

$$P_u = 1.2(500) + 1.6(300) = 1080\ KN$$

$$\emptyset P_n \geq P_u$$

W360X91 $\emptyset P_n = 1380\ Kn$ weight =0.893Kn/m

W310X79 $\emptyset P_n = 1220\ Kn$ weight =0.775 Kn/m

W250X73 $\emptyset P_n = 1150 Kn$ weight = 0.716 Kn/m

W200X100 $\emptyset P_n = 1240\ Kn$ weight = 0.981 Kn/m

Choose the lightest section:-

W250X73 $\emptyset P_n = 1150\ Kn$

Check the buckling about major axis:-

At (KLy) = 6.5 m$\Rightarrow$ $\emptyset P_n = 1150\ Kn$

Since (KLy) = (KLx) ∴ Major buckling is not problem $\Rightarrow \emptyset P_n = 1150 Kn$