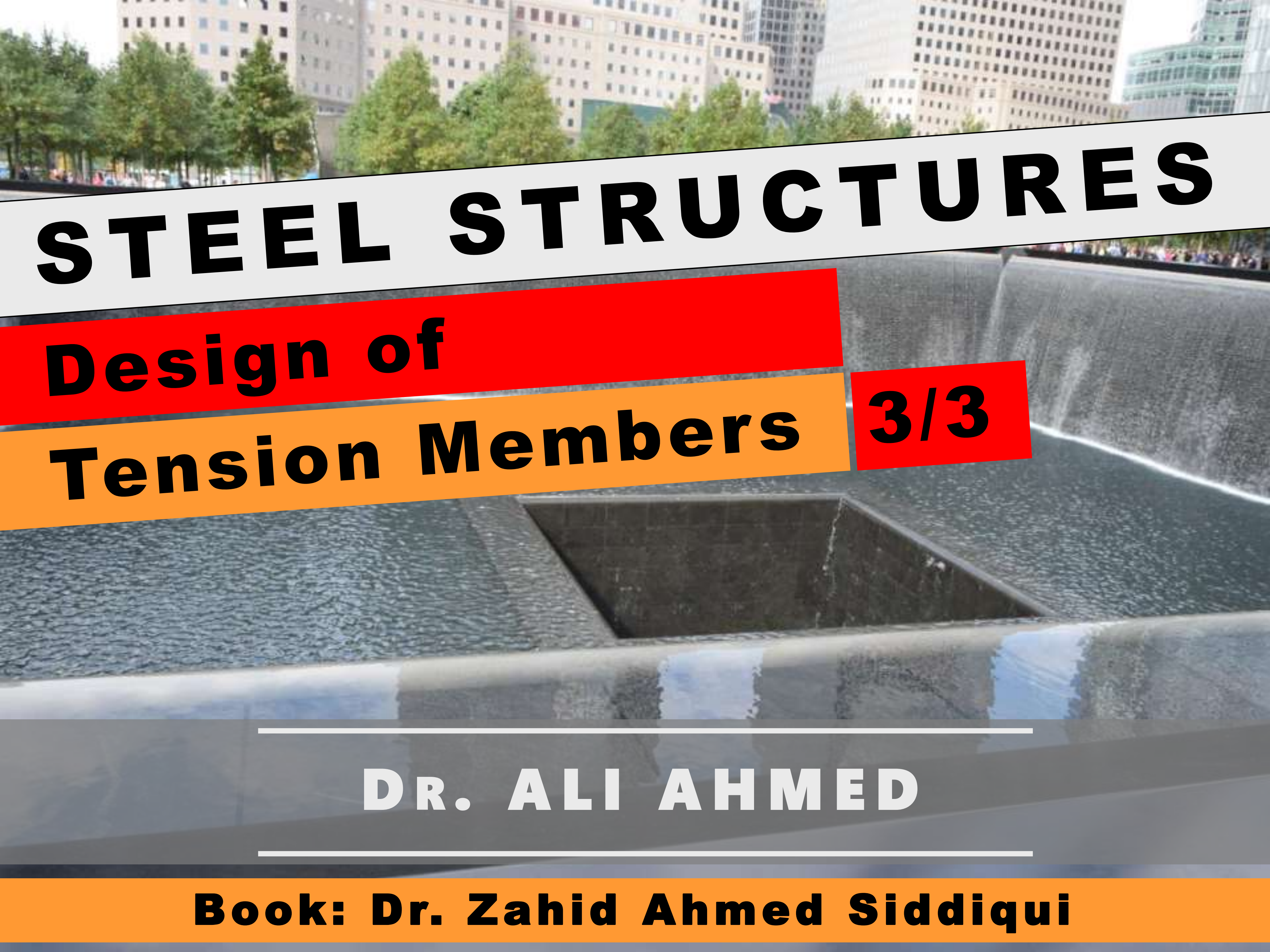

STEEL STRUCTURES

Design of Tension Members 3/3

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Book: Dr. Zahid Ahmed Siddiqui

Example 2.4: Using LRFD procedure, investigate the shear rupture failure mode for the angle **L 102 x 102 x 6.4** attached with three **20 mm diameter rivets** to a **10 mm gusset plate**, as shown in the Figure 2.14. The material is A36 steel.

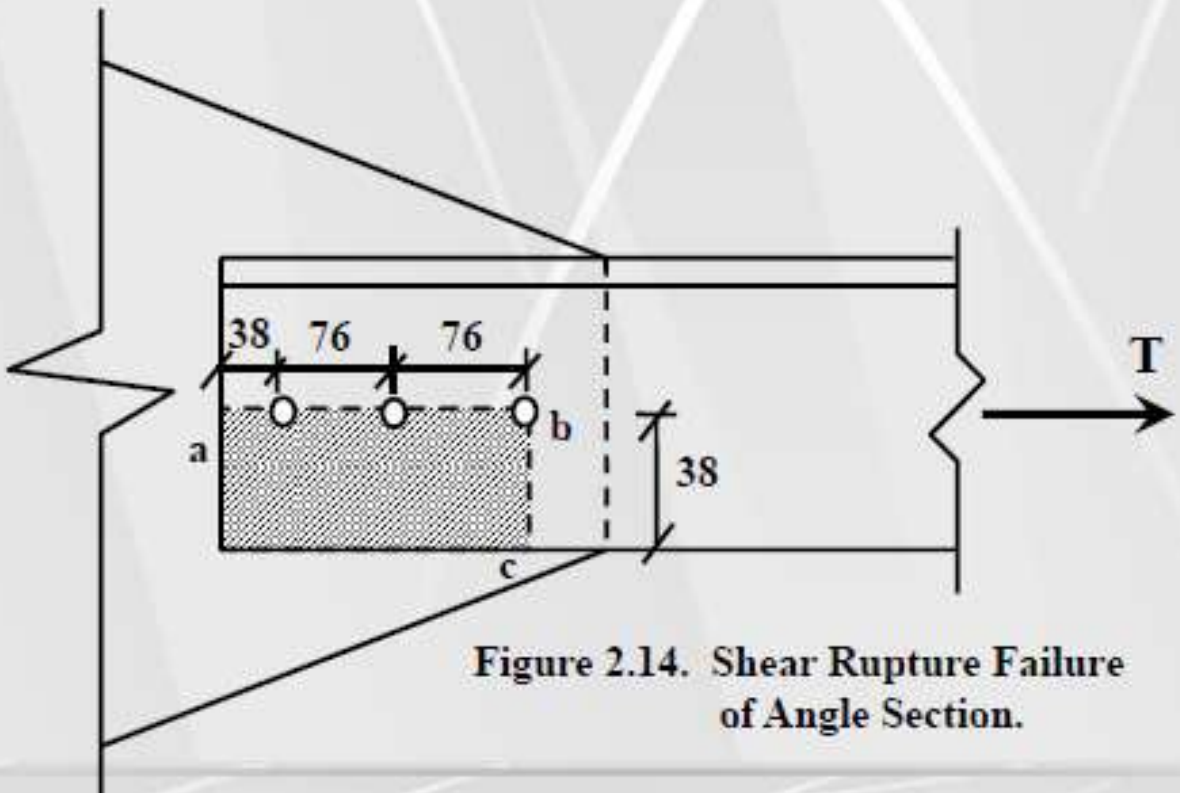


Figure 2.14. Shear Rupture Failure of Angle Section.

Solution:

Capacity of Section:

Yielding of gross section

$$\phi_t F_y A_g = 0.9 \times 250 \times 1250 / 1000 = 281.25 \text{ kN}$$

Fracture in net section

$$\phi_t F_u A_e = 0.75 \times 400 \times 0.60 \times \{1250 - (20 + 3)(6.4)\} / 1000$$

$$= 198.5 \text{ kN}$$

$$\therefore \quad \boldsymbol{\phi_t T_n = 198.5 \text{ kN}}$$

U = 0.6, For angle section with 3 fastener in a row

Block Shear Failure Along Path a–b–c:

$$A_{gv} = (76 + 76 + 38)(6.4) = 1216\ mm^2$$

$$A_{nv} = 1216 - (2.5)(20 + 3)(6.4) = 848\ mm^2$$

$$A_{nt} = 243.2 - (0.5)(20 + 3)(6.4) = 169.6\ mm^2$$

$$U_{\mathrm{bs}} = 1.0$$

$$\phi R_n = \text{lesser of}$$

1. $$\phi\,[0.6\,F_{\mathrm{u}}\,A_{\mathrm{nv}} + U_{\mathrm{bs}}\,F_{\mathrm{u}}\,A_{\mathrm{nt}}]$$

$$= 0.75/1000\ [0.6 \times 400 \times 848 + 1.0 \times 400 \times 169.6]$$

$$= 203.52\ kN$$

2. $\phi\,[0.6\,F_y\,A_{gv} + U_{bs}\,F_u\,A_{nt}]$

$= 0.75/1000\,[0.6 \times 250 \times 1216 + 1.0 \times 400 \times 169.6]$

$= 187.68\,kN$

$$\boxed{\boldsymbol{\phi R_n = 187.68\,kN}}$$

Hence, block shear failure is the governing limit state and factored capacity of the member is reduced from 198.5 kN to 187.68 kN due to it.

DESIGN PROCEDURE/DESIGN FLOW CHART

Known Data:

Service or working loads, T_D, T_L, and T_W, etc. and length of member, L

↓

Find factored tension (T_u) in LRFD method and service tension (T_a) in ASD method using load combinations.

For example, $T_u = 1.2\, T_D + 1.6\, T_L$ for gravity loads alone

↓

Find A_{req} as the bigger out of that required for yielding in the gross section and fracture in the net section.

LRFD

$A_{req.}$ for riveted members

$$= \text{larger of } \frac{T_u(in\ kN)\times 1000}{0.9\ F_y} \quad \text{and} \quad \frac{T_u(in\ kN)\times 1000}{0.75\ F_u \times U \times R}$$

ASD

$A_{req.}$ for riveted members

$$= \text{larger of } \frac{T_a(in\ kN)\times 1670}{F_y} \quad \text{and} \quad \frac{T_a(in\ kN)\times 2000}{F_u \times U \times R}$$

where, any reasonably assumed value of $\boldsymbol{U}$ may be considered and $\boldsymbol{R}$ is the assumed ratio of $\boldsymbol{A_n}$ with respect to $\boldsymbol{A_g}$.

$A_{req.}$ for welded members

$$= \frac{T_u (in\ kN) \times 1000}{0.9\ F_y} \quad \text{(LRFD)},$$

$$= \frac{T_a (in\ kN) \times 1670}{F_y} \quad \text{(ASD)}$$

$b_{min.} = 3.25\text{d} + 18 \geq 50$ mm or $(2.5\text{d} + 16 \geq 50$ mm)

$b_{min.} = L/40,$ For member with $L = 2$ to 3 m

b_{min} for welded members may be kept equal to **50 mm**

Diameter of rivet **d** may be assumed as **15mm** if not known

Selection of Trial Section: It depends on the following four criteria:

A. $A_{sel} \geq A_{req.}$

B. Section should be of minimum weight and smaller size.

C. Connected leg width $\geq b_{min.}$

D. Compatibility of connections with other members is to be provided.

Check Tensile Capacity: Find actual values of $\boldsymbol{U}$ and $\boldsymbol{A_n}$ if rivet pattern and diameter of rivets are known from connection design.

LRFD

Yielding of gross section

$$\phi_t T_n = 0.90 \, F_y A_{sel} / 1000 \quad \geq \quad T_u \qquad \textbf{(OK)}$$

Fracture in net section

$$\phi_t T_n = 0.75 \, F_u U A_n / 1000 \quad \geq \quad T_u \qquad \textbf{(OK)}$$

ASD

Yielding of gross section

$$T_n / \Omega_t = F_y A_{sel} / 1670 \quad \geq \quad T_a \quad \textbf{\textit{(OK)}}$$

Fracture in net section

$$T_n / \Omega_t = F_u U A_n / 2000 \quad \geq \quad T_a \quad \textbf{\textit{(OK)}}$$

Calculate $\boldsymbol{r_x}$, $\boldsymbol{r_y}$ and $\boldsymbol{r_z}$ for built-up sections or directly note these values from tables for hot rolled sections.

Find r_{min} = smallest of r_x, r_y and r_z

Check Maximum Preferable Slenderness Ratio:

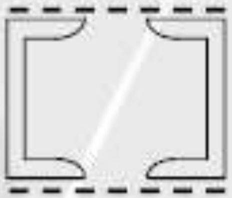
$\ell / r_{min} \leq$ **300** ***(OK)***

Otherwise, make the decision that whether the preferable limit is to be exceeded.

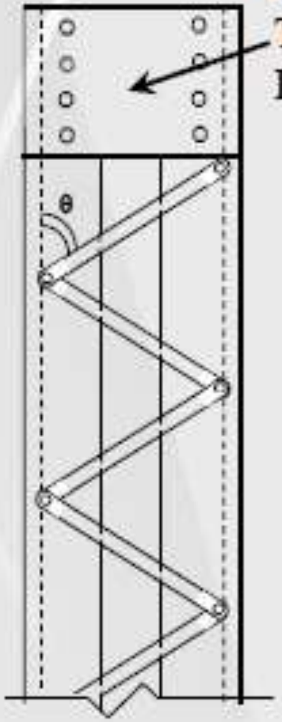
Check Fatigue Strength:

If loading cycles > 20,000 $\Rightarrow$ increase the section accordingly

Design Lacing: Decide spacing of stay plates or arrangement and sizes of lacing in case of built-up sections.



For finding spacing of stay plates, maximum slenderness ratio of individual elements may be equated to the maximum allowed slenderness ratio that is 300.



End Tie Plate Size:

Minimum length $= {}^{2}/_{3}\ \boldsymbol{s}$

Minimum thickness $= \boldsymbol{s\ /\ 50}$

where ***s*** is the distance between the lines of welds or fasteners on the two components of the built-up section.

The longitudinal spacing of welds or fasteners at tie plates should not exceed 150 mm.

↓

Design the Connections:

↓

Check Block Shear Failure: The block shear strength must be checked at the connection, if the connection details are available.

↓

Write the final selection very clearly

<u>Example 2.5:</u> Calculate the factored load capacity of a double channel section member of **A36** steel according to AISC LRFD Specification. The member is **5m long** and consists of $\mathbf{2C_s\ 200 \times 20.5}$, with flanges turned out and with clear gap of 100 mm. Assume that there can be as many as **two 15 mm rivets at any one cross-section** (one in each flange). $\boldsymbol{U \approx 0.80}$.

<u>Solution:</u>

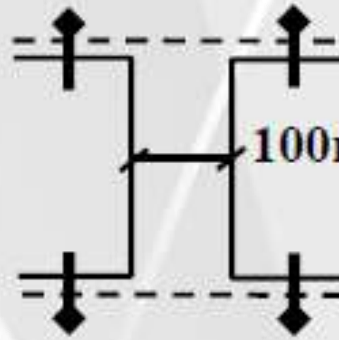
$A_g = 2 \times 2610 = 5220\ \text{mm}^2$; $U = 0.80$;

$L = 5$ m ; clear gap = 100 mm, $\phi_t T_n = ?$

$$A_n = A_g - n(d+3)t_f$$

$$= 2 \times [2610 - 2\,(15+3) \times 9.9]$$

$$= 4507.2 \text{ mm}^2$$



$\boldsymbol{\phi_t T_n}$ = **lesser of**

1. $\phi_t F_y A_g / 1000$

$$= 0.90 \times 250 \times (2 \times 2610) / 1000 = 1174.5 \text{ kN}$$

2. $\phi_t F_u \boldsymbol{U} A_n / 1000$

$$= 0.75 \times 400 \times \mathbf{0.80} \times 4507.2/1000 = 1081.7 \text{ kN}$$

$\boldsymbol{\phi_t T_n = 1081.7\ kN}$

I_y *about individual centroid* $= 63.7 \times 10^4\ mm^4$,
Centroid location, $x = 14\ mm$, $r_x = 75.9\ mm$

$$I_y\ \text{(built–up)} = [63.7 \times 10^4 + 2610\,(50 + 14)^2] \times 2$$
$$= 2265.5 \times 10^4\ mm^4$$

$$\boldsymbol{r_y = \sqrt{\frac{2265.5 \times 10^4}{2 \times 2610}} = 65.88\ \text{mm}}$$

$$r_{\min} = r_y = 65.88\ \text{mm}$$

$$L / r = 5000 / 65.88 = 75.9 < 300\ \textbf{\textit{(OK)}}$$

$$b_f = 59mm > b_{min} = 2.5(15)+16 = 53.5\ mm\ \textbf{\textit{(OK)}}$$

Loading cycles are assumed to be less than 20,000 and hence no reduction in strength due to fatigue is considered.

The factored tensile capacity is 1081.7 kN

Example 2.6:

Select a **W-section** to resist a dead tensile load of 1020 kN and a service tensile live load of 680 kN using A36 steel and AISC LRFD Specification. The member is to be 9m long and is to be connected through its flanges only. Assume that there can be as many as four 20mm rivets at any one cross-section (two in each flange). Fasteners per line are at least three and b_f of the W-section may be assumed to be lesser than $^2/_3\, d$ for the initial calculation of shear lag factor.

Solution:

$$T_D = 1020 \text{ kN} \;\; ; \; T_L = 680 \text{ kN} \; ; \; L = 9 \text{ m}$$

$$T_u = 1.2\,T_D + 1.6\,T_L = 2312 \text{ kN}$$

$$A_{req} = \text{larger of}$$

$$\frac{T_u \times 1000}{0.9F_y} = \frac{2312 \times 1000}{0.9 \times 250} = 10{,}276 \text{ mm}^2$$

and

$$\frac{T_u}{0.75F_u \times U \times R} = \frac{2312 \times 1000}{0.75 \times 400 \times 0.85^2} = 10{,}667 \text{ mm}^2$$

$$\boxed{A_{req} = 10{,}667 \text{ mm}^2}$$

$b_{min} = 3.25\,d + 18 = 3.25(20) + 18 \cong 83$ mm

(the web does not have bolts).

Approx. minimum flange width required = 83×2

= 166 mm

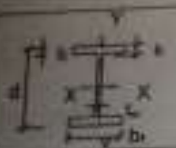
Options for selection of section:

W200 x 86 $A = 11{,}000\ \text{mm}^2$

W310 x 86 $A = 11{,}000\ \text{mm}^2$

W410 x 85 $A = 10{,}800\ \text{mm}^2$

Weight is relatively lesser for this section but the depth is excessively large.



W SHAPES
Dimensions

SI Designation	Standard Designation	Area A (mm^2)	Depth d (mm)	Flange Width b_f (mm)	Flange Thickness t_f (mm)	Web Thickness t_w (mm)	k (mm)	Compact Section Criteria $b_f/2t_f$	Compact Section Criteria h/t_w	Axis X-X I_x ($mm^4 \times 10^4$)	Axis X-X S_x ($mm^3 \times 10^3$)	Axis X-X r_x (mm)	Axis Y-Y I_y ($mm^4 \times 10^4$)	Axis Y-Y S_y ($mm^3 \times 10^3$)	Axis Y-Y r_y (mm)
W 200 x 100	W 8 x 67	12,700	229	210	23.7	14.5	37.6	4.4	11.1	11,300	990	94.6	3,600	351	53.8
x 86	x 58	11,000	222	209	20.6	13.0	30.5	5.1	12.4	9,490	852	92.7	3,130	300	53.1
x 71	x 48	9,100	216	206	17.4	10.2	27.4	5.9	15.9	7,660	708	91.7	2,530	246	52.8
x 59	x 40	7,550	210	205	14.2	9.14	24.2	7.2	17.6	6,080	582	89.7	2,040	200	51.8
x 52	x 35	6,650	206	204	12.6	7.87	22.6	8.1	20.5	5,290	511	89.2	1,770	174	51.6
x 46.1	x 31	5,880	203	203	11.0	7.24	21.1	9.2	22.3	4,580	451	88.1	1,540	152	51.3
W 310 x 86	W 12 x 58	11,000	310	254	16.3	9.14	31.5	7.8	27.0	19,800	1,280	134	4,450	351	63.8
x 79	x 53	10,100	307	254	14.6	8.76	30.0	8.7	28.1	17,700	1,160	133	3,990	310	63.0
W 310 x 74	W 12 x 50	9,420	310	205	16.3	9.40	29.0	6.3	26.8	16,300	1,050	132	2,340	226	49.8
x 67	x 45	8,450	307	204	14.6	8.51	27.4	7.0	29.6	14,500	946	131	2,080	203	49.6
x 60	x 40	7,550	302	203	13.1	7.49	25.9	7.8	33.6	12,800	844	130	1,840	180	49.3
W 310 x 52	W 12 x 35	6,650	318	167	13.2	7.62	20.8	6.3	36.2	11,900	747	133	1,020	122	39.1
x 44.5	x 30	5,670	312	166	11.2	6.60	18.8	7.4	41.8	9,900	633	132	845	102	38.6
x 38.7	x 26	4,940	310	165	9.65	5.84	17.3	8.5	47.2	8,500	547	131	720	87.5	38.4
W 410 x 85	W 16 x 57	10,800	417	181	18.2	10.9	28.4	5.0	33.0	31,600	1,510	171	1,790	198	40.6
x 75	x 50	9,480	414	180	16.0	9.65	26.2	5.6	37.4	27,400	1,330	170	1,550	172	40.4
x 67	x 45	8,580	409	179	14.4	8.76	24.6	6.2	41.1	24,400	1,190	169	1,370	153	39.9
x 60	x 40	7,610	406	178	12.8	7.75	23.0	6.9	46.5	21,600	1,060	168	1,200	135	39.9
x 53	x 36	6,840	404	178	10.9	7.49	21.1	8.1	48.1	18,600	926	165	1,020	115	38.6
W 410 x 46.1	W 16 x 31	5,890	404	140	11.2	6.99	21.4	6.3	51.6	15,600	773	163	516	73.6	29.7
x 38.8	x 26	4,960	399	140	8.8	6.35	19.0	8.0	56.0	12,500	629	159	399	57.2	28.4

Trial section: **<u>W200 x 86</u>**

$A = 11{,}000\ mm^2$

$b_f = 209\ mm$, $t_f = 20.6\ mm$

$r_x = 92.7\ mm$, $r_y = 53.3\ mm$

$t_w = 13\ mm$

Projected flange $= 209 / 2$

$= 104.5\ mm > b_{min}$ ***(OK)***

Capacity Check:

$$b_f / d = 209 / 222 = 0.941 > {}^2/_3$$

In the absence of the detailed connection details, the AISC specification and the table in **Reference-1** **(Page 98)** gives the efficiency factor as $\boldsymbol{U = 0.9}$.

$$\boldsymbol{A_n = A_g - n\,(d + 3)\,t_f}$$
$$= 11{,}000 - 4(20 + 3)(20.6) = 9{,}105 \text{ mm}^2$$

$$\boldsymbol{\phi_t T_n = 0.90\, F_y A_{sel}}$$
$$= 0.9 \text{ x } 250 \text{ x } 11{,}000/1000 = 2{,}475 \text{ kN} > T_u$$

$$\boldsymbol{\phi_t T_n = 0.75\, F_u \,\mathrm{U}\, A_n}$$
$$= 0.75 \text{ x } 400 \text{ x } 0.9 \text{ x } 9{,}105/1000$$
$$= 2{,}458 \text{ kN} > T_u \qquad \textbf{(\textit{OK})}$$

$$r_{min} = \text{smaller of } r_x \text{ and } r_y = 53.3 \text{ mm}$$

$$L / r_{min} = \frac{9 \times 1000}{53.3} = 168.8 < 300 \qquad \textbf{\textit{(OK)}}$$

Loading cycles are assumed lesser than 20,000, if not given.

Design connections. Block shear cannot be checked until the connection design is available.

<u>**Final Selection: W200 x 86**</u>

GENERAL CONSIDERATIONS FOR SELECTION OF SECTIONS

The type of connections used for the structure often affects the choice of member type.

It is very difficult to apply bolts/rivets between some steel sections and the required gusset plates, while the same sections may easily be welded to the gusset plates.

For example, plate-members are to be welded to other members in most the cases when the two plates are lying perpendicular to each other.

The designer should select members such that connections to other members in the structure are easy.

More parts of the section as far as possible are connected at the end to improve joint efficiency and to obtain a compact arrangement.

Most commonly, W-sections have gusset plates on both sides of the section connected with the flanges.

Filler plates are to be used if depths of the joining sections are different.

Gusset plates present within the two angles connect double angles.

DESIGN OF ZERO FORCE MEMBER

Sometimes **zero-force members** are required for internal stability of frame, for minor loads like fans, false ceiling, etc., for future changes in loading, and for temperature effects.

These may also the used to reduce effective lengths of other members.

Section is selected for these members keeping in view the following:

1. Preferably slenderness ratio equal to limiting maximum value for compression members is maintained, which is equal to 200.

   Using this criterion if the size becomes excessive, slenderness ratio of tension members may be provided.

   However, if still the section is excessively bigger, a section comparable with other truss members may be used.

2. Connected legs should have a preferable width greater than or equal to the minimum width required for proper connection.

3. If the zero force member is a top or bottom chord member, continue the same section as present in the adjoining panel.

MEMBERS UNDER STRESS REVERSAL

The maximum factored tensile and compressive forces acting at different time instants due to different load combinations may be represented by the following notation:

T_u = magnitude of ultimate tensile factored force.

P_u = magnitude of factored compressive force.

There are three possibilities of design based on the relative magnitudes of T_u and P_u, as explained in the following cases:

Case 1.

$T_u < P_u$ and Welded Connections

OR $T_u < 0.75P_u$ and Riveted / Bolted Connections

*Neglect the tensile force and design the member as **pure compression member.***

Case 2.

$P_u < 10\%$ of T_u

and $(KL/r)_{max} = 300$

*The compressive force may be ignored and the member is designed as a **pure tension member**.*

Case 3.

$$T_u > (1 + 0.015\,L^2)\,P_u$$

Where, $\mathbf{L}$ *= length of member in meters.*

The member may be designed for a tension of T_u. However, during the capacity check, it is made sure that the compression capacity $\phi_c P_n$ is greater than or equal to P_u.

It is better to keep the slenderness ratio up to **200** for these members.

Case 4.

If the conditions of Case-1 and Case-3 are not satisfied, the section is to be designed for P_u as a compression member. It is checked later that $\phi_t T_n$ is greater than or equal to T_u.

Note: The factored force may be replaced with the service force in case of allowable stress design (ASD).

Example 2.7

Design the member of a roof truss using **LRFD** procedure carrying a factored compressive force (P_u) of 450 kN and a factored tensile force (T_u) of 840 kN; L = **6m**. Built-up section consisting of two channels back to back with a total width of 300 mm is to be used. Check the member under stress reversal.

Welded connections are to be used.

Solution:

$P_u = 450$ kN

$T_u = 840$ kN

$$(1 + 0.015\,L^2)\,P_u = (1.54)\,(450) = 693 \text{ kN}$$

$$T_u > (1 + 0.015\,\mathrm{L}^2)\,P_u$$

$\therefore$ Design first as a tension member and then check for P_u (Case 2).

For welded connections,

$$A_{req} = \frac{T_u \times 1000}{0.9F_y} = \frac{840 \times 1000}{0.9 \times 250} = 3733 \text{ mm}^2$$

$$A_{req} \text{ for one channel } = \frac{3733}{2} = 1867 \text{ mm}^2$$

b_{min} = 50 mm for welded connections

Options available: 1. **C 150 × 15.6**

2. **MC 310 × 15.8**

x_p = plastic neutral axis (PNA) location

SI Designation	Standard Designation	Area A mm²	Depth d mm	Flange Width b_f mm	Flange Thickness t_f mm	Web Thickness t_w mm	k mm	x mm	e_o mm	x_p mm	Elastic and Plastic Properties, Axis X-X: I ×10⁶ mm⁴	Axis X-X: Z ×10³ mm³	Axis X-X: S ×10³ mm³	Axis X-X: r mm	Axis Y-Y: I ×10⁶ mm⁴	Axis Y-Y: Z ×10³ mm³	Axis Y-Y: S ×10³ mm³	Axis Y-Y: r mm
C 180 × 22	C 7 × 14.75	2,790	178	58	9.3	10.6	22	13.3	13.7	6.48	1010	138	114	66.0	48.7	23.4	11.5	14.5
× 18.2	× 12.25	2,320	178	55	9.3	8.0	22	13.7	16.4	5.16	887	117	100	69.1	40.3	20.6	10.2	14.8
× 14.6	× 9.8	1,850	178	53	9.3	5.3												
C 150 × 19.3	C 6 × 13	2,470	152	54	8.7	11.1	21	13.1	9.7	8.05	724	119	95.0	54.1	43.7	22.3	10.5	13.3
× 15.6	× 10.5	1,990	152	51	8.7	8	21	12.7	12.3	6.48	633	101	82.9	56.4	36.0	18.8	9.24	13.4
× 12.2	× 8.2	1,550	152	48	8.7	5.1	21	13.0	15.2	5.03	545	84.1	71.8	59.4	28.8	16.3	8.06	13.6
MC 310 × 15.8	MC 12 × 10.6	2,000	305	38	7.8	4.8	27	6.83	7.21	3.28	2310	190	151	107	15.9	10.5	5.08	8.92
MC 250 × 61.2	MC 10 × 41.1	7,810	254	110	14.6	20.2	32	27.7	21.9	15.3	6580	637	516	91.7	658	143	80.0	29.0
× 50	× 33.6	6,370	254	104	14.6	14.6	32	27.4	26.9	12.4	5790	547	456	95.3	549	123	71.8	29.5
× 42.4	× 28.5	5,400	254	100	14.6	10.8	32	28.4	30.7	10.5	5290	485	415	98.8	475	112	65.9	29.7

Trial section: 2C_s 150 × 15.6 (Figure 2.16)

A **= 1990 mm²**

d = 152mm , b_f = 51mm , x = 12.7 mm

$I_x = 633 \times 10^4$ mm^4 ; $I_y = 36.0 \times 10^4$ mm^4

r_x = 56.4 mm ; r_y = 13.4 mm

Capacity Check:

$$\phi_t T_n = 0.9\, F_y\, A_{sel}$$

$$= 0.9 \times 250 \times (2 \times 1990) / 1000$$

$$= 895.5\ kN > T_u \qquad \textbf{(OK)}$$

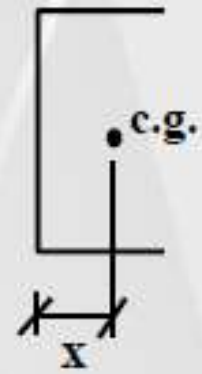


Figure 2.16. Location of centroid for a channel Section.

$$\phi_t T_n = 0.75\, F_u\, \mathrm{U}\, A_n$$

$$= 0.75 \times 400 \times 1.0 \times (2 \times 1990)/1000$$

$$= 1194\ kN > T_u \qquad \textbf{(OK)}$$

Approximate r_x *and* r_y: (using Reference-1, **Page 102**)

$$r_x = 0.36\,h = 0.36(152) = 55 \text{ mm}$$

$$r_y = 0.60\,b = 0.60(300 - 2\times51) = 118.8 \text{ mm}$$

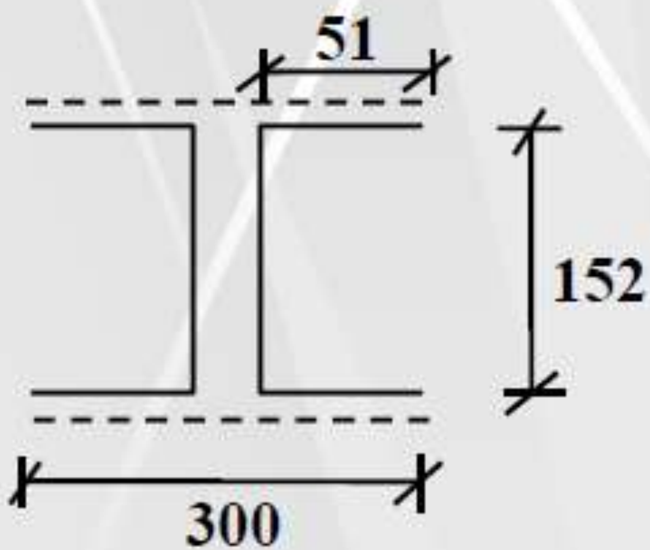


Figure 2.17.Built-Up Section Made By Two Channels.

Exact r_x & r_y (preferable and a must for final trial):

Referring to Figure 2.17,

$r_x = 56.4$ mm as for a single section

$I_y = 2 \times 36.0 \times 10^4 + 2 \times 1990\,(150 - 51 + 12.7)^2$

$= 5038 \times 10^4\ \text{mm}^4$

$$r_y = \sqrt{\frac{5038 \times 10^4}{2 \times 1990}} = 112.5\ \text{mm}$$

$r_{min} = 56.4$ mm

$$L / \mathrm{r}_{\mathrm{min.}} = \frac{6 \times 1000}{56.4} = 106.4 < 200 \quad (OK)$$

- ***Design of Lacing:***

- ***Check For Compressive Strength:***

  These parts will be completed after doing the next chapter.

Loading cycles are assumed less than 20,000

- **Design Connections**

ASSIGNMENT FOR TENSION MEMBER DESIGN