International Journal of Latest Research in Engineering and Technology (IJLRET)
ISSN: 2454-5031
www.ijlret.com || Volume 03 - Issue 05 || May 2017 || PP. 17-23

Comparative Study of Tension Members Designed As Per IS 800: 1984 and IS 800: 2007

Vishal P. Jamnekar[1]
[1](*Assistant Engineer Gr.-II, Water Resources Department (Special Project), Amravati, Maharashtra, India)*

Abstract: The most commonly used methods for structural steel design in India are working stress method which is based on IS 800: 1984 and Limit state method which is based on IS 800: 2007.In Indian context most of the steel structures were designed on the basis of working stress method but now a days the latest standards introduced by IS 800: 2007are in use. Thus the approach has been made to make the comparative study of both codes and to evaluate the economy achieved using IS 800: 2007. An illustrative study has been carried out on tension member subject to axial load. Total 47 members have been analyised. Thus from the results obtained it is observed that working stress method of analysis underestimates the actual strength of members. Also it is observed that the strength of member largely depends upon its length of outstand, pitch and gauge distances.
Keywords: Tension members, IS 800: 2007, IS 800: 1984, design strength, Rupture strength.

I. INTRODUCTION

The Steel tension or compression members are most probably used in unified steel structures which act efficiently under the action of loads. These members or structural elements are subjected to direct axial tension or compression depending on its functional utility. Due to the application of these forces, there is substantial elongation or compression of structural steel members. The members and connections are so arranged that the eccentricity in connections and bending stresses in the members are not developed. As there is no eccentric force, stresses in the members is assumed to be uniformly distributed over the net sectional area of the member[1]. A member in pure tension can be stressed up to and beyond the yield limit and does not buckle locally. Hence the design is not affected by the type of section used i.e., Plastic, Compact or Semi-compact.

Most of the tension members occur in trusses, bridges, transmission towers, transmission lines, wind bracing, steel bracings in multi-storey buildings and industrial sheds etc. tension members in the form of high strength steel can be utilised in strengthen the existing structures. Since the entire cross sectional area is subjected to uniform stresses thus tension members can carry load in most efficiently way. The strength of these members are influenced by various factors such as; overall length of member, size and numbers of bolts/ rivets required for connections, gross area and net area effects etc. hence the study of tension member under the effect of axial load is necessary. The codes published by the Bureau of Indian Standards for design of steel structures are IS800:1984 and IS800:2007. Earlier for designing steel structures Working Stress Method (WSM) is used which is in accordance to IS800:1984[2]. Now designing is done using Limit State Method (LSM) as per IS800:2007[3]. Thus in this research paper an effort has been taken to focus on the strength and economy achieved by designing the tension members subjected to axial loads using IS:800 :1984 and IS:800 : 2007.

1.1 Behavior of cross-section of tension members

Generally tension members are designed using rolled sections, bars or flats. When more area is needed or connection design is required, it is possible to combine profiles or to build up a specific section using plates. Flats are generally not used because of their high flexibility; for good practice the slenderness should be limited to 300 for principal members or 400 for secondary members but obviously this rule does not apply to round bars. In general, rolled sections are preferred and the use of compound sections is reserved for larger loads or to resist bending moments in addition to tension.

The load-deformation behavior of members subjected to uniform tensile stress is similar to the load-deflection behavior of the corresponding basic material. The typical stress-strain behavior of mild steel under axial tensile load is shown in Fig. 1. The upper yield point is merged with the lower yield point for convenience. The material shows a linear elastic behavior in the initial region (O to A). The material undergoes sufficient yielding in portion A to B. Further deformation leads to an increase in resistance, where the material strain hardens (from B to C). The material reaches its ultimate stress at point C. The stress decreases with increase in further deformation and breaks at D. The high strength steel members do not exhibit the well defined yield point and the yield region (Fig. 1.1). For such materials, the 0.2 percent proof stress is usually taken as the yield stress

International Journal of Latest Research in Engineering and Technology (IJLRET)
ISSN: 2454-5031
www.ijlret.com || Volume 03 - Issue 05 || May 2017 || PP. 17-23

(E). From this curve it can be concluded that the high strength steel is not much ductile hence there is possibility of causing failure due to yielding, rupture and shear failure.

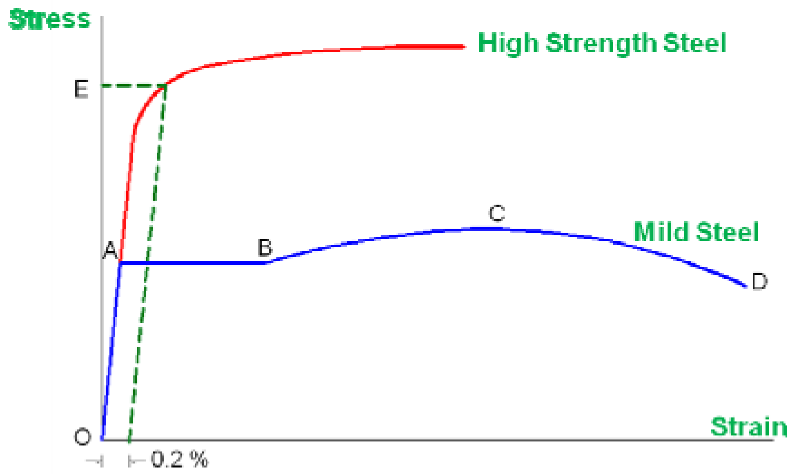


Fig. 1.1: Behavior of tension member

1.2 Objectives of the study
The main objective of this study is to;
1. Understand the basic difference in design methodology adopted in IS 800: 1984 and IS 800: 2007.
2. Determination of improvement in strength of structural steel members design as per IS 800: 2007.

Economy achieved by using IS 800: 2007.
The detail design procedure of tension members using permissible stress design method as per IS 800: 1984 and limit state design method as per IS 800: 2007 have been discussed. This design procedure is explained with the help of example and comparative study has been carried out.

II. DESIGN OF TENSION MEMBER

The following procedure is adopted in designing axial loaded tension members.

2.1 Tension member subjected to axial load designed using IS 800: 1984
The following procedure is adopted in designing axial loaded tension member.

1. The net area (A_{net}) to carry the design load (P) is obtained by using equation;
$$P = \sigma_{at} A_{net} \quad (1)$$
2. The net area calculated thus, is increased suitably to compute the gross sectional area. Then select the suitable section which will have cross sectional area matching or little more with the computed gross sectional area.
3. The number of rivets required to make the connection is calculated.these are arranged in a suitable pattern and the net area of section provided is calculated. This should be more than the net area calculated using eq.(1). In this case the net area is calculated as;
$$A_{net} = A_1 + kA_2 \quad (2)$$
$$k = 3A_1/(3A_1 + A_2) \quad (3)$$
4. The slenderness ratio of the member is checked as per the IS code specifications.

2.2 Tension member subjected to axial load designed using IS 800: 2007
Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads upto the ultimate load, at this stage they may fail by rupture at a critical section. The design strength of the tension member shall satisfy following requirements;

The factored design tension (T) should be less that the design strength of the member (Td). The design strength of member under axial tension is lowest of the following;

5. Design strength governed due to yielding of gross section (Tdg)
6. Design strength governed due to rupture at critical section (Tdn)
7. Design strength governed due to block shear failure (Tdb)

2.2.1 Strength Governed Due To Yielding Of Gross Section

The design strength of member under axial tension (Tdg) as governed by yielding of gross section, is given by

$$T_{dg} \quad = \quad (A_g\, f_y)/\gamma_{m0} \tag{4}$$

Where,

γ_{m0} = Partial factor of safety for failure in tension by yielding [3]

2.2.2 Strength Governed Due To Rupture at critical Section

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength (Tdn), as governed by rupture at net section is given by

$$T_{dn} \quad = \quad 0.9\, f_u A_{nc}/\gamma_{ml} \; + \; \beta A_{go} f_y/\gamma_{m0} \tag{5}$$

Where

$$\beta = 1.4 - 0.076\,(w/t)\,(fy/fu)\,(bs/Lc) \le (fu\ \gamma_{m0} / fy\ \gamma_{m1}) \ge 0.7 \tag{6}$$

Where

w = outstand leg width

bs = shear lag width, as shown in Fig. 2.1

Lc = Length of the end connection, i.e., distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction

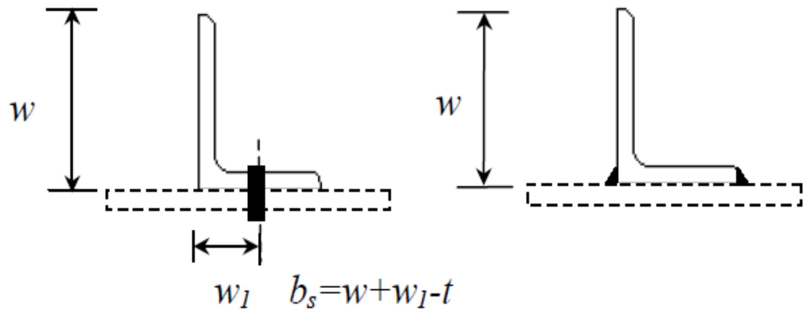


Fig. 2.1: Angles with end connections

For preliminary sizing, the rupture strength of net section may be approximately taken as

$$T_{dn} = \alpha A_n\, fu/\gamma_{m1} \tag{7}$$

Where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

2.2.3 Strength Governed Due To Block Shear Failure

A tension member may fail along end connection due to block shear. The corresponding design strength can be evaluated using the following equations. If the centroid of bolt pattern is not located between the heel of the angle and the centerline of the connected leg, the connection shall be checked for block shear strength given by

$$T_{db} = (A_{vg}\, fy/(\sqrt{3}\gamma_{m0}) + 0.9\, fu\, A_{tn}/\gamma_{m1}) \tag{8}$$

or

$$T_{db} = (0.9\ fu\, A_{vn}/(\sqrt{3}\gamma_{m1}) + fy\, A_{tg}/\gamma_{m0}) \tag{9}$$

III. DESIGN EXAMPLE

A single unequal angle 100 x 75 x 10 mm is connected to a 10 mm thick gusset plate at the ends with 6 numbers of 16 mm diameter bolts to transfer tension as shown in Fig. 13. Determine the design tensile strength of the angle if the gusset is connected to the 100 mm leg. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa.

3.1 Design as per IS 800: 1984

Solution:

Step1. Axial Stress Calculations

$$\sigma_{at} = 0.6\, f_y = 0.6 \mathrm{x} 250 = 150\ \mathrm{N/mm^2}$$

Step2. Net Area Calculations

$$A_{net} = A_1 + kA_2$$

$$A_1 = \left(200 - \frac{10}{2} - 17.5\right) \times 10 = 775\ \mathrm{mm^2}$$

$$A_2 = \left(75 - \frac{10}{2}\right) \times 10 = 700\ \mathrm{mm^2}$$

$$k = \frac{(3 \times 775)}{[(3 \times 775) + 700]} = 0.76$$

$$A_{net} = 775 + (0.76 \mathrm{\ x\ } 700) = 1307\ \mathrm{mm^2}$$

Step3. Load Carrying Capacity

$$P = 150 \mathrm{\ x\ } 1307 = 196.050\ \mathrm{kN}$$

Step4. Design Load Carrying Capacity

$$P = 1.5 \mathrm{\ x\ } 196.050\ \mathrm{kN} = 294.075\ \mathrm{kN}$$

Strength of Single Angle is, 294.075 kN.

3.2 Design as per IS 800: 2007

Solution:

Step1. Strength Governed Due To Yielding Of Gross Section

$$A_g = \left[\left(100 - \frac{10}{2}\right) + \left(75 - \frac{10}{2}\right)\right] \times 10 = 1650\ \mathrm{mm^2}$$

$$T_{dg} = \frac{1650 \times 250}{[1.25]} = 375 \times 10^3\ \mathrm{N}$$

Step2. Strength Governed Due To Rupture at critical Section

$$A_{nc} = \left(100 - \frac{10}{2} - 18\right) \times 10 = 770\ \mathrm{mm^2}$$

$$A_{go} = \left(75 - \frac{10}{2}\right) \times 10 = 700\ \mathrm{mm^2}$$

$$w = 75\ \mathrm{mm}$$

$$b_s = (75 + 60 - 10) = 125\ \mathrm{mm}$$

$$L_v = [(5 \times 50) + 30] = 280\ \mathrm{mm}$$

$$L_c = \big[[(5 \times 50) + 30] - (5.5 \times 18)\big] = 181\ \mathrm{mm}$$

$$\beta = 1.4 - 0.076\left(\frac{75}{10}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{125}{181}\right) = 1.160$$

$$T_{dn} = \frac{(0.9 \times 410 \times 770)}{(1.25)} + \frac{(1.160 \times 700 \times 250)}{(1.1)} = 411.845 \times 10^3\ \mathrm{N}$$

Step3. Strength Governed Due To Block Shear Failure

Design strength due to block shear (T_{db}) is minimum of T_{db1} and T_{db2}

$$A_{vg} = [(5 \times 50) + 30] \times 10 = 2800\ \mathrm{mm^2}$$

$$A_{vn} = \big[[(5 \times 50) + 30] - (5.5 \times 18)\big] \times 10 = 1810\ \mathrm{mm^2}$$

$$A_{tg} = [40 \times 10] = 400\ \mathrm{mm^2}$$

$$A_{tn} = [(40 - (0.5 \times 18)] \times 10 = 310\ \mathrm{mm^2}$$

$$T_{db1} = \frac{(2800 \times 250)}{(\sqrt{3} \times 1.1)} + \frac{(0.9 \times 410 \times 310)}{(1.25)} = 458.917 \times 10^3\ \mathrm{N}$$

$$T_{db\,2} = \frac{(0.9 \times 410 \times 1810)}{(\sqrt{3} \times 1.25)} + \frac{(250 \times 400)}{(1.1)} = 399.394 \times 10^3 \text{ N}$$

The block shear strength is $T_{db} = 399.394 \times 10^3$ N

Step4. Design tensile strength Td

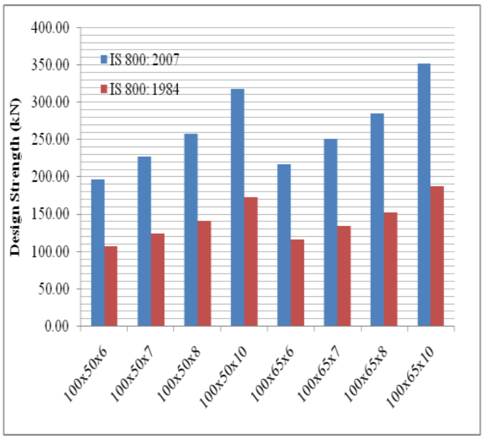
Strength of Single Angle is least of above three values, thus $\mathbf{T_d}$ **= 375.00 kN**

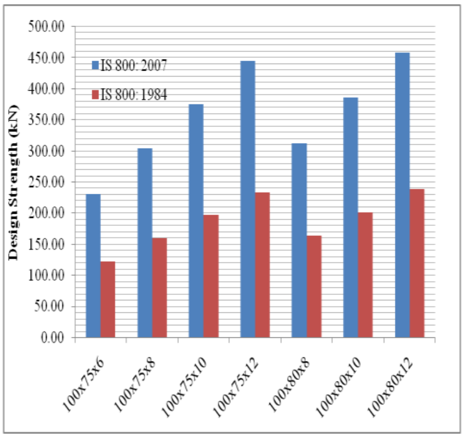
IV. COMPARATIVE STUDIES

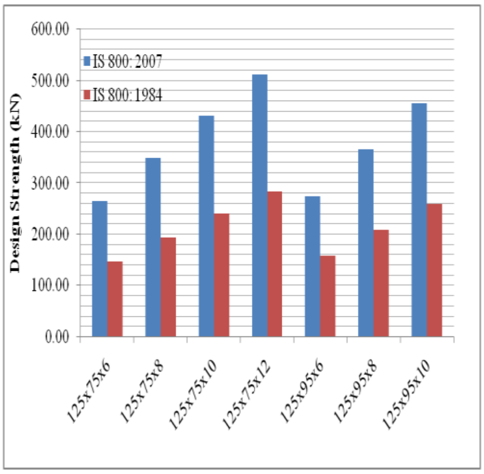
In this study the design strength of total 47 tension members designed as per IS 800: 1984 and IS 800: 2007 has been carried out. The design strength of these various members has been compared and is shown in Fig. 1 to Fig. 6. Also the study has been carried out to determine the increase in load carrying capacity of members designed as per the above mentioned design codes.

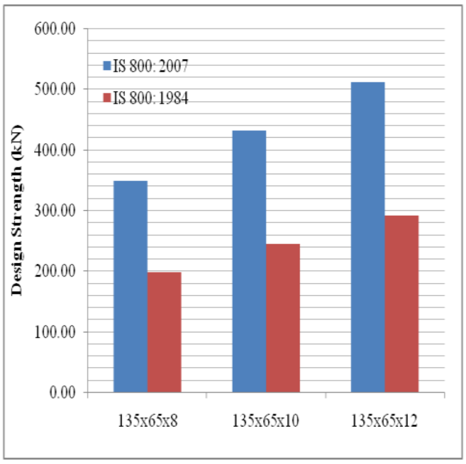
4.1 Comparison of Load Carrying Capacity Vs Different Sections

The analysis is carried out on various tension members and graphs were plotted. From Fig. 1 to Fig. 6 it is clear that there is substantial increase in load carrying capacity of members designed as per IS 800: 2007 as compare to IS 800: 1984. From Fig. 1 and Fig. 2, it is clear that the maximum increase in strength is achieved in ISA 100x80x12 mm which is about 47.96 %. From Fig. 1 to Fig. 6; it can be observed that 'the saw tooth pattern' of improvement in strength is achieved in each set of angle sections. Thus from this 'saw tooth pattern' it can be predicted that 'as the length of outstanding leg increases' reverse cyclic pattern of strength improvement is followed in both codes. For ISA 125x75x12, 135x65x12, 150x75x12 and 200x100x10 the maximum improvement in strengths is found to be 44.62%, 43.25%, 42.71% and 35.51% respectively. Also the least improvement in strength is observed in ISA 200x150x20 mm which is found to be about 12.28 %.









International Journal of Latest Research in Engineering and Technology (IJLRET)
ISSN: 2454-5031
www.ijlret.com || Volume 03 - Issue 05 || May 2017 || PP. 17-23

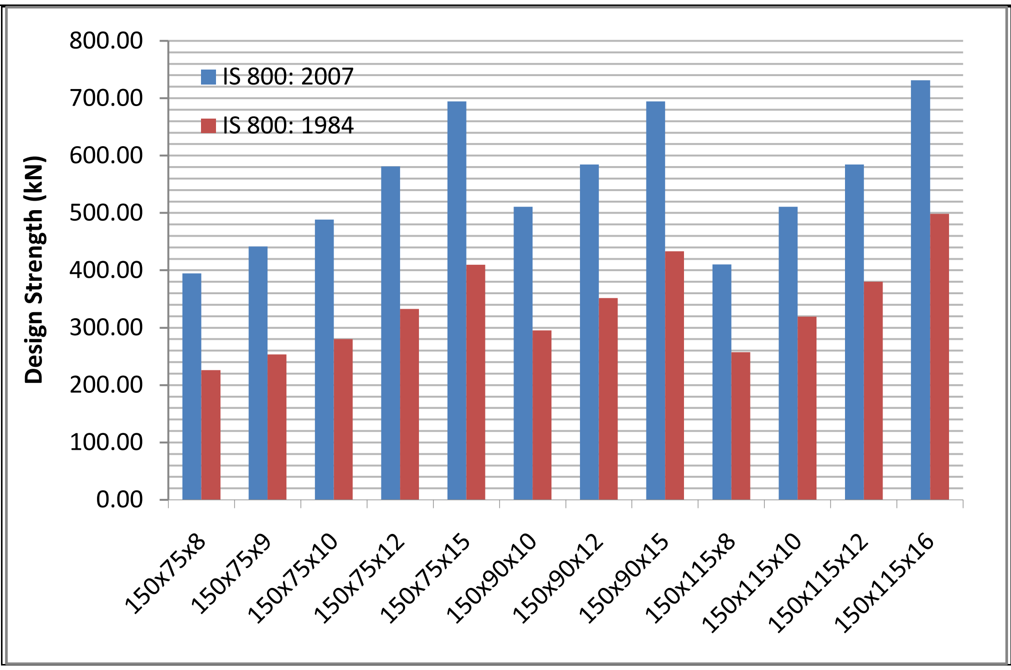


Fig. 5: Design Strength of ISA Sections

4.2 Failure Test Results

From the Fig. 6, it is clear that none of the set of angle sections failed in 'Rupture' whereas most of the set failed in 'Yielding' which is about 55% of total specimens. It is also observed that 45 % of the set failed in 'Block Shear'. As the outstanding length of ISA increases it's yielding strength decreases and failure takes place due to block shear.

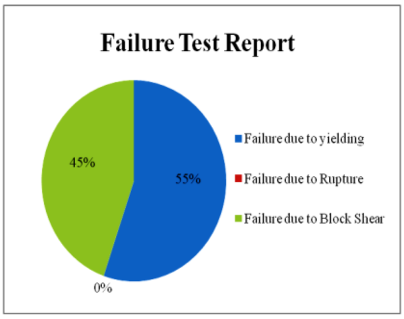


Fig. 6: Failure Result Analysis

V. CONCLUSION

1. Load carrying capacity of tension members designed as per IS 800: 2007 is much more substantial as compare to the members designed as per IS 800: 1984.
2. If members are designed as per IS 800: 2007, economy can be achieved upto 40 to 45 %.
3. The load carrying capacity of tension members depends upon the size of section, pitch distance and number of bolts used. The load carrying capacity increases upto certain limits after which it follows the similar load increment pattern.
4. Instead of using heavy angle sections, light sections are required to be preferred since from this study it is observed that the improvement in strength of similar size of section with different length of outstand were having their strength more or less equal.

REFERENCE

[1] Mohan Gupta and L. M. Gupta, 2004, Evaluation of stress distribution in bolted steel angles under tension, Electronic Journal of Structural Engineering, 2(4).

[2] IS 800: 1984, Code of practice for general construction in steel, Bureau of Indian Standards, New Delhi.

[3] IS 800: 2007, Code of practice for general construction in steel, Bureau of Indian Standards, New Delhi.

[4] Fathy A. Abdelfattah and Mohamed S. Soliman, Shear lag in bolted tension members at elastic and yielding loading conditions, Alexandria Engineering Journal, Vol. 42 (2003), No.4, 463-474.

[5] C. S. Nair, Dr. S. Sreekumar, Investigation on the Bahaviour of Rolled Steel Tension Members, Department of Civil Engineering, College of Enginering, Trivendrum, Volume 90, pp 32-37, August 2009.

[6] R. G. Driver, G.Y. Grondin and G. L. Kulak, A unified approach to design for block shear, Department of Civil and Environmental Engineering, University of Alberta, Canada.