Design of Steel Connections as per IS 800:2007 and Comparison with other Codal Provisions

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***Abstract*-- Advancement in design, fabrication and erection of steel structures has taken place with edification of technology and globalization. Limit state design method (LSM) philosophy for steel structures represents advantages in terms of more accuracy and economy over traditional design methods. Steel connection parameters such as specifications, assumptions, and design methodology based on provisions of IS 800:2007 have been presented in this paper. Extent of discussion has been particularly focused on important connections like web angle connections.**

Design steps for steel connections have been summarized on the basis of codal provisions used in different countries. A typical design example of double web angle connections has been computed using corresponding design stipulations of BS5950, AISC-LRFD and IS 800. Analytical calculations of the solved example have been presented and the results are compared using different countries code.

Index Terms-- Limit State Design, Steel Connections, IS 800, AISC LRFD, BS5950

I. INTRODUCTION

Steel structures are assemblage of different elements joined together at ends of members by using various types of steel connections. The steel connections are important elements in controlling behaviour of the whole steel structure. Behaviour of connections is complex due to influence of factors like geometric imperfections, lack of fit, residual stress, connection flexibility, geometric complexity, slipping, nonlinear load deformation characteristics etc. A variety of components such as angle cleats, end plates, stiffeners and bolts are used to transfer disperse loads from one member to other. Uses of bolts for discrete load paths are employed to transfer loads. Understanding of different types of connections is very essential for safe and economical design of the steel structure. It is vital for the connectors to develop full strength or a little higher strength compared to the members being joined for achieving an economical design.

Structural design of steel structure is based on provisions of a standard code. A standard code serves as a reference document consisting of important guidelines related to different philosophy of connection design. Standard codes incorporate comprehensive details for design of different structural components. These details include, concept of design, design specifications, design methods, safety factors and loading values etc. Here, these details are discussed in brief for different countries codal provision.

In present days, many countries have published their own standards. These codes are a product of constant research, development and past experiences of experts corresponding to respective fields. Design assumptions and specifications for steel connections have been summarized on the basis of codal provisions used in different countries. The design provisions of connections are compared with different countries codes. Design stipulations towards different types of steel connections pertinent to AISC 'Load and Resistance Factor Design' (LRFD) (2005) and BS 5950 (2000) specifications have been compared with the relevant parameters of IS 800 (LSM)(2007).

Philosophy of limit state method (LSM) was introduced in British Code CP 110(1972) (now BS 8110) and Indian concrete code IS 456:1978. Limit states design was first adopted for steel structures in Canadian code in 1974, this was followed by the British codes BS 5950 and BS 5400. In USA, the American Institute of Steel in Construction introduced LSM in the form of load resistant factor design (LRFD) in 1993. N. Subramanian (2008).

IS 800 (2007):

Due to globalization, engineering practice has not remained confined to a particular area therefore practicing people are facing problems with existing code in India. Realizing these difficulties Bureau of Indian Standards, New Delhi with faculty of Civil Engineering, Indian Institute of Technology, Madras to help and prepare draft for revision of IS 800 (2007). This work was carried out in a project mode with financial support from Institute for Steel Development and Growth (INSDAG), Kolkatta.

In India research and development in steel has done up to certain extent, so IS 800(2007) is based on the international experience. This new code is a improvement over the previous code IS 800 (1984), with new provisions on partial safety factor based limit state method of design include design against fatigue, design for fire load, design for durability, design by experimental data etc. It includes parameters like fatigue, ultimate strength, member end connections, restrains and many more having greater influence on the design considerations which makes IS 800 (2007) more complicated and time consuming for new users.

Philosophy of limit state design method incorporates a multiple safety factor format that to provides adequate serviceability at service loads, by considering all possible 'limit states'.

II. Comparison of Specifications

Here, general specifications for design of connections are presented here with different countries code. Specifications for connections includes spacing, design strength of bolts in shear, bearing and tension force, Bolt Subjected to Combined Shear and Tension stresses are compared as presented in Table 1 and notations of Table are as follows. Web angle connection's specifications compared in Table II.

n_n=no. of shear planes with threads intercepting shear plane
n_s=no.of shear planes without threads intercepting shear plane
A_{sb} =nominal plain shank area of bolt
β_{lg} =Reduction factor that allows for effect of large grip length
β_{lj} =Reduction factor which allows for overloading of end bolts that occur in long connection
β_{pkg}=Reduction factor to account for packing plates in excess of 6mm
A_{nb} =net tensile area at threads, may be taken as area corresponding to root diameter at thread
For ISO thread profile, as $A_{nb} = (A/4)(d\text{-}0.9382p)^2$

TABLE I
COMPARISON OF GENERAL SPECIFICATIONS

	IS:800 (2007)	BS:5950-I (2000)	AISC LRFD (2005)
Bolt Holes (d=bolts dia.)	d+1mm;d≤14mm d+2mm;d>16mm d+3mm;d>24mm d=dia. of bolts	- d+2mm;d≤24mm d+3mm;d>24mm d +6mm;Hold down bolts:	- d+2mm;d≤24mm; d+3mm;d>24mm d=dia. of rivet/bolts
Minimum Spacing	≥ 2.5 × d d=dia. of rivet/bolt	≥ 2.5 × d d=dia. of rivet/bolt	2.66 × d in direction of force
Maximum Spacing in Direction of stresses exposed. Any direction connection in exposed condition	<32*t* or 300 mm, - t= thickness of thinner plate	In the direction of stress should not exceed 14 × t Maximum spacing <16 × t ≤ 200mm t=thick. of thinner plate	12 t < 6 in. (150 mm) T=thick. of the connected part
Pitch, in Tension member, Compression member, Connecting to face of Rolled hot sections (RHS)	<16*t* or 200 mm, <12*t* or 200 mm t = thickness of thinner plate	Vertical = 70mm; bolt gauge or cross centres = 90mm or 140mm for end plates & 100mm+ beam web thick. ≥0.3 × RHS face width	Min. of 12*t* or 150 mm t = thickness of thinner plate
Min. Edge dist. Sheared or Hand Flame Cut, Rolled, Machine Flame Cut	1.7 × Hole Diameter 1.5 × Hole Diameter	1.40 x Hole Diameter 1.25 x Hole Diameter	1.75 × Hole Diameter 1.25 × Hole Diameter
Max. Edge distance to nearest line of fasteners from an edge of any unstiffened part, Exposed to corrosive influences	< 12 × $t\varepsilon$; Yield stress ratio $\varepsilon=(250/f_y)^{1/2}$ and *t* is thickness of thinner outer plate < 40 mm + 4*t*,	11 × $t\varepsilon$; $\varepsilon = (275/f_y)^{1/2}$ *t* is thickness of thinner outer plate < 40 mm + 4*t*,	<24*t* or 305 mm, *t* is thickness of thinner outer plate <14*t* or 180 mm,
Effective Areas of Bolts	A_n net tensile stress area at root ofthe threads	A_s , Area at root of threads	Area at threads, A_{nt}
Factored shear force (V_{sb})	$V_{sb} = V_{db}$; V_{db}=design strength =smaller of shear, V_{dsb} and bearing, V_{dpb}	shear capacity *P*s of a bolt should be taken as: $P_s=p_s \times A_s$	The design tension or shear strength, $\Phi \times F_n \times A_b$, Φ= 0.75 (LRFD);
Shear Capacity of Bolt (V_{dsb})	$V_{dsb} \leq V_{nsb} / \gamma_{mb}$	$P_s=p_s \times A_s$ p_s=*Shear strength of bolt*, A_s=*Shear area*	$V_u=\Phi \times A_b \times F_v$ Φ=resistance factor $F_{v=}$ Nominal strength $A_{b=}$ Nomi. area
nominal shear capacity of a bolt (V_{nsb})	$V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$ $\beta_{lj}\beta_{lg}\beta_{pkg}$	$V_{nsb} = p_s \times A_s \times$ *Reduction factors* p_s=Shear strength of bolt, A_s =Shear area	F_v = *0.6 × nominal Tension capacity of A307* =*0.65 for A325& A490* $A_{nt} = 0.785\left(d - \frac{0.9743}{n}\right)$
Net tensile area at thread A_{nb}	$A_{nb} = (A/4)\times(d\text{-}0.9382p)^2$	A_t=tensile area of bolt	

Reduction factor for *Long Joints* (β_{lj}); element containing more than two bolts	A_{nb} =78-80% of gross area β_{lj} = 1.075 – l_j / (200 d) but 0.75 ≤ β_{lj} ≤ 1.0	L_j > 500mm (5500-L_j)/5000 L_j =Length of joints	β_{lj} = 1.2 – 0.002 (L /w)≤1.0 L=actual length of end loaded weld
Reduction factorfor *Large Grip Lengths* $l_g > 5 \times d$ of bolts	$\beta_{lg} = 8d/(3d+l_g)$; Where $l_q<8\times d$ & $\beta_{lq}<\beta_{lj}$,	$(8d)/(3d+T_g)$ T_g=thickness of grip	No. increased by 1% for each 2mm increased in grip
Reduction factor for Packing Plates(β_{pkg})	β_{pkg} =(1-0.0125t_{pkg}) where , t_{pkg} = thickness of thicker packingin mm	$(9d)/(8d+3*t_{pa})$ t_{pa}=thickness of pack	[1-0.0154(t-6)]; t=total thickness of filler plates≤19mm

A. *Double Angle Web Cleat Connections*

Connection consists of a pair of angle cleats that are usually bolted to supported beam web in shop and beam assembly is then bolted to supporting member on site. Double web angle connection as shown in Fig. 1

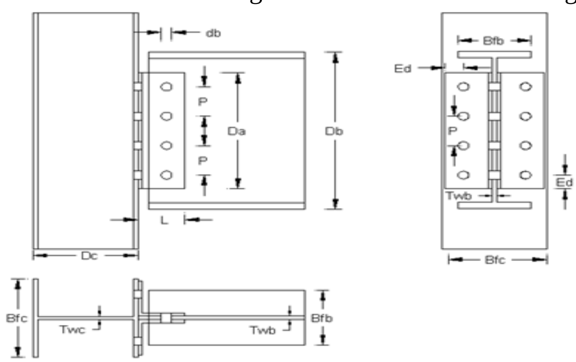

Fig. 1 DETAILS OF DOUBLE WEB ANGLE CONNECTION

The step wise procedure and design codal provision using different countries code are presented as shown in Table 2.

TABLE 2
COMPARISON OF DESIGN PROVISIONS FOR WEB ANGLE CONNECTIONS

	IS:800 (2007)	BS:5950-I (2000)	AISC LRFD (2005)
Design philosop-hy	Limit State design Method	Limit State design Method	Load and Resistance Factor Design
Steel Grade	Fy 250	S275	f_y=50ksi, f_u=65ksi f_y=36ksi, f_u=58ksi
Bolt class	M20 of 8.8 grade	M20 of 8.8 grade	¾ inch A325-N
1. Design of connection to Beam web			
Shear capacity of bolt connecting cleats to beam web	$V_{dsb} \le V_{nsb} / \gamma_{mb}$ $V_{nsb} = f_u/\sqrt{3}\ (n_n A_{nb} + n_s A_{sb}) \beta_{lj} \beta_{lg} \beta_{pkg}$	The shear capacity of a single bolt $P_s = p_s A_s$ p_s=Strength of bolts A_s=Area of bolts	$V_u = \Phi . A_b . F_v$ Φ=resistance factor, F_v= Nominal strength, A_b =Nominal area at major thread dia
Pitch, in Tension member, Compression member,	<16t or 200 mm, <12t or 200 mm t =thickness of thinner plate	Vertical = 70mm; bolt gauge = 90mm or 140mm for end plates & 100mm+ beam web thickness (t_w);	Min. of 12t or 150 mm t = thickness of thinner plate
Horizontal Shear force due to eccentricity	$H = V_x e_x r_i / \Sigma r_i^2$ e_x=eccentricity of column face to bolt	$F_{sm} = F_v\, a/Z_{bg}$ $Z_{bg} = n(n+1)p/6$	-
Vertical Shear force per bolts	$v=V/n$ n=No of bolts	$F_{sv} = F_v/n$ n=No of bolts	$F_{sv} = R_u /n$ n=No of bolts
Resultant due to direct shear and moment	$R=\sqrt{(H^2+v^2)}$ < Bolt shear strength	$F_s = \sqrt{(F_{sv}^2+F_{sm}^2)}$ $< 2 P_s$	*From table [1]* t_w=web thick. of web $\Phi R_n > R_u$
2. Design of Connection to column flange			
Slip resistance per bolt	$V_{nsf} = \mu_f\ n_e K_h\ 0.8 A_{sb}\ 0.70 f_{ub}$	$P_{sL} = 1.1 K_s \mu P_0$ P_0=min. shank tension	slip resistance, ΦR_n $Rn = \mu D_u h_{sc} T_b N_s$

		μ= slip factor varies from 0.2 to 0.5	(a) For standard size holes h_{sc} = 1
Horizontal Shear force due toeccentricity	$H=V_x g r_i/\Sigma r_i^2$ g=eccentricity due to guage	$F_{sm}=F_v\, a/Z_{bg}$ $Z_{bq}=n(n+1)p/6$	-
Resultant force on outermost bolt due to direct shear and moment	$R=\sqrt{(H^2+v^2)}$ < Bolt capacity	$F_s=\sqrt{(F_{sv}^2+F_{sm}^2)}$ $< 2\,P_s$	*From table[1]* t_f=thickness of flange of beam, $\Phi R_n > R_u$
3. Connecting element capacity			
Shear capacity of the leg of the angle cleat	$V/2\; t\, f_y/(\sqrt{3}\,\gamma_{m0})$ t=thickness of web angle, f_y =250N/mm^2 γ_{m0} =1.25	P_v = min (0.6 p_y A_v , 0.7 $p_y K_e$ $A_{v.net}$) A_v=0.9(2e_1+(n –1)p)t_c $A_{v.net}$= A_v – n D_h t_c	$\Phi\, R_n = \Phi$ [0.6 f_u A_{nv} + f_y A_{gt}] *design strength,*ΦR_n $R_n = 0.60F_y\, A_g$ Φ= 1.00
Bearing resistance on cleat	$V_{sb} \leq V_{npb} / \gamma_{mb}$ V_{npb} = 2.5 $d\, t\, f_u$	$P_{bs} = d\, t_c\, p_{bs}$ $< 0.5\, e\, t_c\, p_{bs}$ p_{bs}=460 N/mm^2	-
Cleat Bending moment	$M_{req}=V/2\times g/2$ g=guage distance	$M_x= V/2\times g/2$ g=guage distance	$M_x= V/2\times e_1$ e_1=eccentricity
Moment capacity of cleat	$M_{obt}=1.2\times f_y\times Z/\gamma_{m0}$ f_y=250 γ_{m0} =1 $M_{obt} > M_{req}$	$M_{obt}=1.2\times p_y\times Z_x$ p_y=275 $M_{obt} > M_x$	$M_c = p_y\times Z$ p_y=36 ksi $M_c > M_x$

III. DESIGN EXAMPLES

Here, effort is made to solve problem using different countries codal provision for same geometry and component size of connection. Double web angle connection is considered with configuration as shown in Fig 1. Capacity in a connection is found out with one vertical line of bolts. Here, in this problem connection configuration are predefined to resist shear force of 120 kN.

Strengths of following component are checked for design of double web angle connection.

1. Bolts group in beam web
2. Bolts group in column flange
3. Angle cleats in shear
4. Angle cleats in bending

Detailing of beam column connection configuration is shown in Fig. 1. and size parameters presented in Table 4 for example using four different codes.

A. IS 800 (1984)

1. Bolts group in beam web

Design strength of 20 mm dia. rivets

In single shear = $\frac{\pi}{4} d^2 F_s$ = 36.29 kN

In Double shear =2 x 36.29 = 72.57 kN

Bearing resistance on 8.9 mm thick web of beam

$V_{npb} = \Phi\, t_{wb}\, f_u$ = (21.5 x 8.9 x 300) = 57.41kN

Bearing resistance on 9 mm thick flange of column

$V_{npb} = \Phi\, t_{fc}\, f_u$ = (21.5 x 9.0 x 300) = 58.05 kN

Rivets which connects web of beam with angles are in double shear and bearing.

Rivets value = 57.41 kN

No of rivets to be provided = 4

Beam web strength = 57.41 × 4 = 229.6 ≈ 230 kN

Rivets which connects flange of column with angles are in single shear and bearing.

2. Bolts group in column flange

Rivets value = min of 36.3 kN and 58.05 kN = 36.3 kN

No of rivets to be provided = 8 nos

Column flange strength = 36.30×8 = 290.3 ≈ 290 kN

3. Angle cleats in shear

The rectangular section of width as 2 x 8 and Depth of angle as 295 mm

Shear capacity of angle

= (2/3 x 100 x 295 x 2 x 8)/1000 = 315 kN

4. Angle cleats in bending

Moment = $f_s Z$

= 100x2x8x295x295/6000= 23.2 kNm

Capacity of angle cleats in bending

= M_c / e_1 = 23.2/0.065 = 357 kN

B. IS 800 (2007)

Detailing of connection is shown in Fig 1 and table 3

1. Bolts group in beam web

End reaction is transferred by shear and bearing from web of beam to web bolts and to angles cleats. Cleat angles are transferred to bolts at junction of supported members and to supporting members mainly by shear, tension and compression.

Thickness of column flange t_f = 9.0 mm

Shear capacity of M20 Bolt in single shear = 52.6 kN

Bearing capacity on web of beam (web thickness of ISMB 400 = 8.9 mm)

=2.5 x 20 x 8.9 x 410 / (1.25 X 1000)

= 146 kN

Strength of bolt =52.6 kN and no of bolts in web of beam = 4 nos.

Capacity of bolts in beam web = 420.8 kN

Thicker angle reduces flexibility of connection and may introduce end moments. Gauges are provided from 100 mm to 140 mm in connection to make joint flexible.

Using web cleats ISA 90 x 90 x 8, with a gauge distance =55+55+8.9=118.9 mm

Use 4 M20 bolts with a vertical pitch of 75 mm

Beam is designed as simply supported and supporting members are designed for eccentric beam reaction.

Assuming shear to be acting on face of column, due to eccentricity, some horizontal shear forces will be acting on bolt group in addition to shear due to reaction, e=65 mm.

Horizontal shear force on bolt due to moment of eccentricity

$$= \frac{v_x e_x r_i}{\Sigma r_i^2} \quad (1) \qquad =31.2 \text{ kN}$$

Vertical shear force per bolt = 120 / 4 = 30.0 kN

Resultant shear force = $\sqrt{(31.2^2+30^2)}$ = 43.3 kN < 52.6 kN

Hence connection is safe

Capacity = (120 / 43.3) x 52.30

= 145.77 kN ≈ 146 kN.

2. Bolts group in column flange

Connection to column flange,

Assuming μ=.48,

Slip resistance per bolt = 105.37 kN.

Bearing resistance on 8 mm cleat per bolt = 128 kN

Hence, Bolt strength = 52.6 kN

4 No of bolts with vertical pitch of 75 mm,

Gauge distance=59.45mm From Centerline of beam web.

Shear transfer between beam web and angle cleats may be assumed to take place on face of beam web.

Horizontal shear force on bolt due to moment due to eccentricity as per eqn. 1 = 14.3 kN

Alternatively, if center of pressure is assumed 25 mm below top of cleat, then

Horizontal shear force = 9.509 < 14.3 kN

Vertical shear per bolt=120/8 =15 kN

Resultant shear Force = $\sqrt{(14.3^2+15^2)}$ = 20.7 < 52.6

Hence connection is safe

Column flange strength = (52.6/20.7)x120=304.93≈305 kN.

3. Angle cleats in shear

Determine length of web angle based on number of bolts, pitch and normally not less than 0.6 to 0.75 times depth of beam.

Shear capacity of angle cleat

= $V\, t\, f_y /(\sqrt{3}\, \gamma_{m0})$= 120 × 2 × 16 × 250 / (√3 × 1.25)

= 221.75 ≈ 222 kN > 120 kN

Shear capacity of angle cleat

= f_y × Area of cleat

= 250×295×2×8/(√3×1.25×1000) kN. = 545 kN

4. Angle cleats in bending

Take two angles of ISA90×90×8 of length 295 mm.

Check Bending at bolt line of connection to column flange.

Moment Capacity

= 1.2× 250 × 8 × 295 × 295 / (6 × 1000) = 34.81 kNm

Angle cleats in bending

= M/e_1 =34.810 /0.060 =536 kN

C. *BS 5950 (2000)*

Detailing of beam column connection configuration is shown in Fig 1 and table 3

Design steps involved in design of web angle connection are as follows.

1. Bolts group in beam web

Capacity of Bolts group in beam web

For 120 kN reaction, moment on these bolts =120 × 0.055 = 6.6 kNm

Using vector sum method to determine force on most heavily loaded bolts,

I of bolts group = 2 ($37.5^2 + 112.5^2$) = 28125 mm^4

Z for further bolts = 28125 / 112.5 = 250.0 mm^3

Force on outer most bolts due to vertical shear =120/4=30 kN

Horizontal force on outermost bolts due to moments

= 6.6 x 1000 /250.0 = 26.4 kN.

Resultant = $(30.0^2 + 26.4^2)^{0.5}$ = 39.9 kN.

A_s as the tensile area since shear plane stresses passes through threads, capacity per bolts in double shear

= 2 x 375 x 28125 = 210.9 kN.

Capacity per bolt in bearing in 6.4 mm beam web

=20 x 6.4 x 460 = 58.88 kN.

Since cleat thickness is 8 mm, Bearing in this will be less critical

Since capacity per bolts exceeds load on most heavily loaded bolt,

Group is satisfactory.

Capacity = (120/40.0)x58.88=176.64 kN.

2. Bolts group in column flange

Capacity per bolt in single shear = 375 x 28125 = 105.5 kN.

Capacity per bolt in bearing in 8 mm cleat

= 20 x 8 x 460 = 73.6 kN.

Therefore capacity of bolt group = 8 x 73.6 = 588.8 kN.

3. Angle cleats in shear

Capacity of Angle cleats in shear

Shear capacity

= (0.6 x 275 x (0.9 x 2 x 8 x 295) =700.92 kN.

4. Angle cleats in bending

Shear capacity of angle cleats=700.92 kN.

Gross I for cleats = 2 x 8 x 295^3 / 12 = 3423.0 cm^4

Less holes = 2 x 2 x 8 x 21.5 x ($37.5^2+112.5^2$) = 980.20 cm^4

Net I for cleats = 2442.8 cm^4

Z for cleats =2442.8 / 147.5 = 165.6 cm^3

$M_c = p_y Z$=275 x 165.6 /1000 = 45.5 kN

Force due to eccentricity in terms of reactions capacity

Mc/e_1 = 45.5/0.065 = 700 kN.

D. *AISC-LRFD (2005)*

Detailing connection is shown in Fig 1 and data is given in table 3

Design steps are as follows.

1. Bolts group in beam web

Beam W16 x 31

Depth of D_b =15.88”, Width of flange B_{fb} =5.53”

Thickness of flange t_{fb} =0.44” , Thick. of web t_{wb} =0.28”

Column W8x31

Depth of D_c = 8”, Width of flange B_{fc} =8”

t_{fc} =0.44”, t_{wc} =0.28”

Beam column stresses, f_y=50 ksi

f_u= 65 ksi (1ksi = 6.895 Mpa)

Bolt of ¾ inch A325-N

Angle 3½” x 3½” x 0.312”,

Angle stresses, f_y=36 ksi, f_u=58 ksi

Shear force R_u = 120/4.448 = 26.98 ≈ 27 kips

Design bolts and angles

For ¾ inch diameter A325-N Bolts and angle material with F_y=36ksi and f_u=58ksi, select 4 rows of bolts and 0.312” angle thickness.

From table 10.1 of AISC manual (2001)
Design strength ΦR_n = 127 kips > 27 kips
Check for supported beam web
Beam material stresses, f_y=50 ksi,
f_u=65 ksi , t_{wb} =0.28" and L_{eh} = 1½"
From table 10.1 of AISC manual (2001)
Design strength Φ R_n =351 x 0.28 = 98.28 kips > 27 kips
2. Bolts group in column flange
Check for supporting column flange
Column material stresses, f_y=50 ksi, f_u=65 ksi , t_{fc} =0.44" and L_{eh} = 1½"
From table 10.1 of AISC manual (2001)
Design strength Φ R_n = 351 x 0.44 = 154.44 kips > 27 kips
3. Angle cleats in shear
Shear capacity of angle cleats= $\Phi R_n = \Phi\,[0.6\, f_u\, A_{nv} + f_y\, A_{gt}]$
From table 9-4a and 9-4b (pp10-145) of AISC manual (2001)=[227kips+51.6kips]x0.3149=87.74 kips > 27 kips
4. Angle cleats in bending
Capacity of angle cleats in bending
Gross I for cleats = 2 x 0.3149 X 11.61^3 / 12 = 82.13 in^4
Less holes = 2 x 2 x 0.3149 x ¾ x (1.47^2+4.43^2) = 23.54 in^4
Net I for cleats = 58.68 in^4
Z for cleats =58.68 / 5.81 = 10.10 in^3
$M_c = p_y Z$=36 x 10.10 = 363.6 kips. In
Force due to eccentricity in terms of reactions capacity
M_c/e_1 = 363.5/2.5 = 145.4 kips > 27 kips
Comparisons for capacities of different component using different codes are presented in Table 4.

Table 3
DETAILS OF SECTION PROPERTIES AND CONNECTION CONFIGURATION PROVISION

	IS 800 (2007)	BS 5950 (2000)	AISC LRFD (2005)
Design philosophy	Limit State design Method	Limit State design Method	Load and Resistance Factor Design
Steel Grade	Fy 250	S275	f_y=50 ksi,f_u= 65 ksi f_y=36 ksi,f_u= 58 ksi
Bolt class	M20 8.8	M20 8.8	¾ inchA325-N
Beam	ISMB400	UB406 x 140 x 39	W16 x 31
Depth of D_b	400	406	15.88"(403mm)
Width of flange B_{fb}	140	140	5.53" (140mm)
Thick. of flange t_{fb}	16	8.6	0.44" (11mm)
Thick. of web t_{wb}	8.9	6.4	0.28" (7.1mm)
Column	ISHB200	UC203 x 203 x 46	W8 x 31
Depth of D_b	200	203	8" (203 mm)
Width of flange B_{fc}	200	203	8" (203 mm)
Thick. of flange t_{fc}	9	11	0.44" (11 mm)
Thick. of web t_{wc}	7.8	7.2	0.28" (7.1 mm)
Angle section	90 x 90 x 8mm	90 x 90 x 8mm	3½" x 3½" x 0.312"
Pitch	75	75	3.0" (76.2 mm)
Edge distance	35	35	1.3" (33.2 mm)

Table 4
COMPARISON OF COMPONENT STRENGTH WITH DIFFERENT CODES

Codal provision	IS 800 1984	IS 800 2007	BS (2000)	AISC (2005)	AISC (2005)
Component Strength	(kN)	(kN)	(kN)	(kN)	kips
Bolts group in beam web	230 157%	146 100%	177 121%	565 386%	127 386%
Bolts group in column flange	290 95%	305 100%	589 193%	687 225%	154.4 225%
Angle cleats in shear	315 58%	545 100%	701 128%	395 72%	88.74 72%
Angle cleats in bending	357 66%	536 100%	700 131%	647 121%	145.4 121%

Summary and Conclusions

The following conclusions are made from the comparative study of connections specification, provisions and analytical work conducted.

- Connection component strength is varying with multiple safety factors.
- Design steps for steel connections have been summarized using different countries codal provisions.
- In Limit state design method, specifications for connections design are found likely similar in different countries codal provision.
- Evaluated component strength of connection using different countries codal provisions are presented in Table 4.

III. References

[1] AISC-2005 Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings, American Institute of Steel Construction, INC, Chicago, Illinois.
[2] AISC-LRFD-325(2001), AISC Manual of steel construction, American Institute of Steel Construction,3rd edition.
[3] BS 5950-2000: Part1 "Structural Use of Steelwork in Buildings" Code of practice for design in simple and continuous construction: hot rolled sections, British Standards Institution, London.
[4] IS 800-2007, *"Code of Practice for General Construction in Steel"*, Third Revision, Bureau of Indian Standard, New-Delhi.
[5] IS 800-1984, *"Code of Practice for General Construction in Steel"*, First Revision, Bureau of Indian Standard, New-Delhi.
[6] N. Subramanian, *"Design of Steel Structures"* based on the limit state method of design as per the latest Indian standard code IS800:2007, Oxford University Press, New Delhi, 2008.