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Article

Efficacy of IS Code Provisions for Design of Eccentrically Loaded Single Angle Compression Members

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Abstract: In practical applications, steel single angles are often subjected to eccentric loading when connected to a gusset plate through one leg only, resulting in complex compression behaviour. This behaviour has not been as extensively studied as that of concentrically loaded members. Various international codes of practice offer differed approaches for designing these elements. The literature indicates that the Indian Standard Code IS 800–2007 accurately predicts the axial capacity of eccentrically loaded single angle columns. However, a recent amendment to IS 800 in 2024 introduced modified design provisions. This study is the first to explore the implications of these latest design provisions by comparing them with earlier provisions and experimental strengths reported in the literature. Initially, the design strengths as per the previous and latest design provisions for various Indian Standard Angles (ISA) were presented and compared accounting for varying slenderness ratio, plate slenderness ratio (b/t), and different types of end connections and restraints. The findings reveal that the latest design provisions generally result in much higher design strengths compared to the earlier provisions, with a maximum increase of 104.84%. Upon noticing the significant variation, this study is further extended to compare with the reported data available in the literature. The nominal strengths calculated using the latest provisions were often higher than the strengths reported in the literature considered in this study, indicating potential unsafe design.

Keywords: eccentrically loaded; bending; bolted connection; single angle; flexural-torsional buckling; strength; welded connection

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1. Introduction

Single angle sections, often provided as web members in roof trusses, bracings in buildings and transmission line towers and cross frame members in plate girder bridges, are connected at the ends through one leg only and hence are subjected to eccentric loading [1,2]. In general, they are designed to resist axial force alone i.e., either tension or compression. Under compression, the members are prone to buckling failure at a load capacity well below their material yield strength and hence stability criteria often govern the design of such members. Though the angle sections look simple, their buckling behaviour under compression is complex as they can be singly/mono-symmetric or un-symmetric, in which the centroidal (x-x, y-y) and principal axes (u-u, v-v) do not coincide (Figure 1). Also, the shear centre 'SC' does not coincide with centroid 'C'. Equal leg angles are symmetric about the principal axis 'u-u', which is also the axis of symmetry. It is well known that the radius of gyration is minimum about the 'v-v' axis and hence is termed as the 'minor—principal axis', whereas the axis 'u-u' is termed as the 'major-principal axis', about which the radius of gyration is maximum.

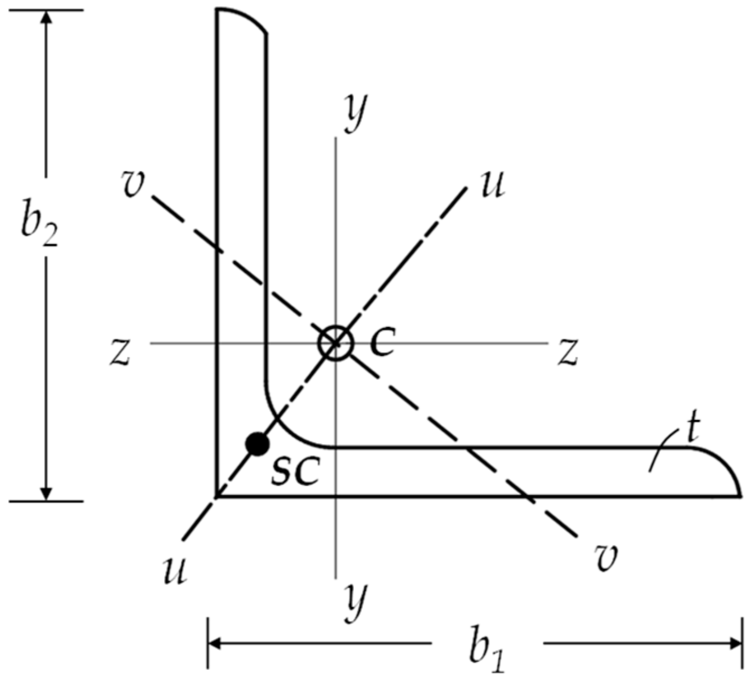


Figure 1. Angle section.

In general, equal leg angle sections, when subjected to pure axial compression or when concentrically loaded, may undergo torsional-flexural buckling, TFB (also called flexural-torsional buckling, FTB). This will happen when the compressive load acts through the centroid—as in the case of angles, where both legs are connected and loaded simultaneously through gusset plates or directly connected to adjoining members. In such cases, the buckling will be by twisting about the shear centre (*SC*) coupled with bending about the major-principal axis '*u-u*'. It is also possible for buckling to be about the minor-principal axis '*v-v*' alone, which is termed as minor-axis flexural buckling, FB. The incidence of FB depends upon their length and boundary conditions (fixed/hinged) [3–6]. Latest research by Sofiani et al. [6] reported that interactive FTB and FB occurs in fixed-end angles of intermediate length. Warping fixity at the ends also influences the behaviour and strength of such members. A failure termed as 'local buckling' may also occur in the case of large plate slenderness ratio i.e., the ratio of width of leg '*b*' to thickness '*t*'—(b/t).

The behaviour of eccentrically loaded single angles, i.e., loaded through one leg only (i.e., when one leg only is connected to the gusset or directly to the adjoining member), is much more complex. This is because of the eccentricity '*e*' of the load i.e., the point of application of the load does not coincide with the centroid of the angle section (Figure 2a,b) resulting in higher stress of the connected leg. The literature available on eccentrically loaded angles is also relatively limited, compared to that of concentrically loaded angles. The structural response of eccentrically loaded single angles connected at the ends to a restraining member (gusset or adjoining member) is similar to that of a '*restrained column*'. The analytical solution accounting for inelasticity, rotational restraint at ends, and bi-axial bending due to the eccentricity of the load (beam-column behaviour) was first proposed by Usami and Galambos [7]. The end connection of the angle struts to the gusset (or directly to the adjoining member) at either end, offer rotational restraint to both in-plane and out-of-plane buckling (about the *y-y* axis and *z-z* axis, respectively). This has to be duly accounted for by providing rotational springs (R_y and R_z) as shown in Figure 3 [7]. However, in reality, the degree of rotational restraint/fixity of the restraining member (gusset or adjoining member), as shown in Figure 4, also plays a crucial role. As per the experimental research conducted at Washington University in the year 1969, it was observed that the eccentrically loaded angle has potential to predominantly undergo flexural buckling about the centroidal axis parallel to the connected leg—that is, buckling out-of-plane of the restraining member (gusset or web of adjoining member) coupled with very little twisting. Based on this fact, a design procedure for single angle web struts in trusses was proposed by Woolcock and Kitipornchai [8]. Elgaaly et al. [9] stated that failure by flexural buckling about the minor-principal axis and/or twisting was also observed from the tests conducted on three-dimensional trusses comprising single angles provided with single or double bolted end connections. Further, they found that the ratio of the failure load to the yield load capacity may not necessarily increase when the overall slenderness of angle sections is reduced [9,10]. This was revealed by the tests conducted on specimens with slenderness ranging from 65 to 210. Accordingly, design considerations, simplified procedures, and design tables were reported by various researchers [11–16]. A numerical study [17] indicated that eccentrically loaded single angles with hinged ends experienced buckling along both the major and minor principal axes. Elastic and in-elastic second-order analyses were found to result in the reliable prediction of the capacity of eccentrically loaded single angles in a truss system [18,19].

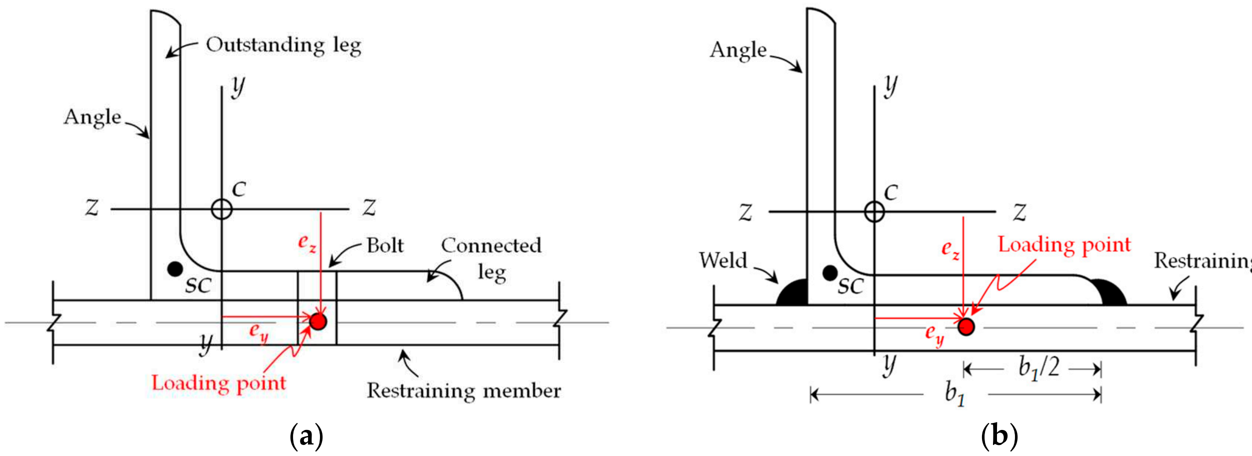


Figure 2. Loading point in eccentrically loaded angle sections: (**a**) bolted connection. (**b**) welded connection (equal weld on either side).

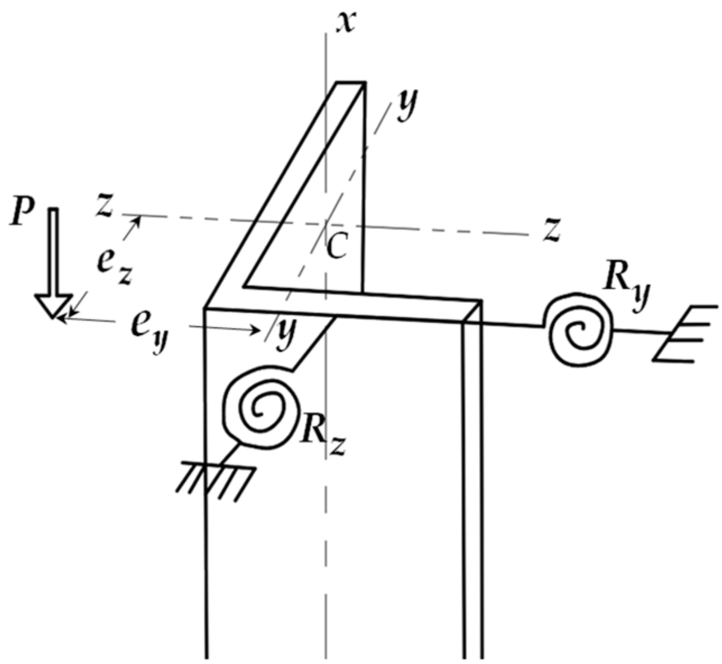


Figure 3. Single angle column idealized as beam column.

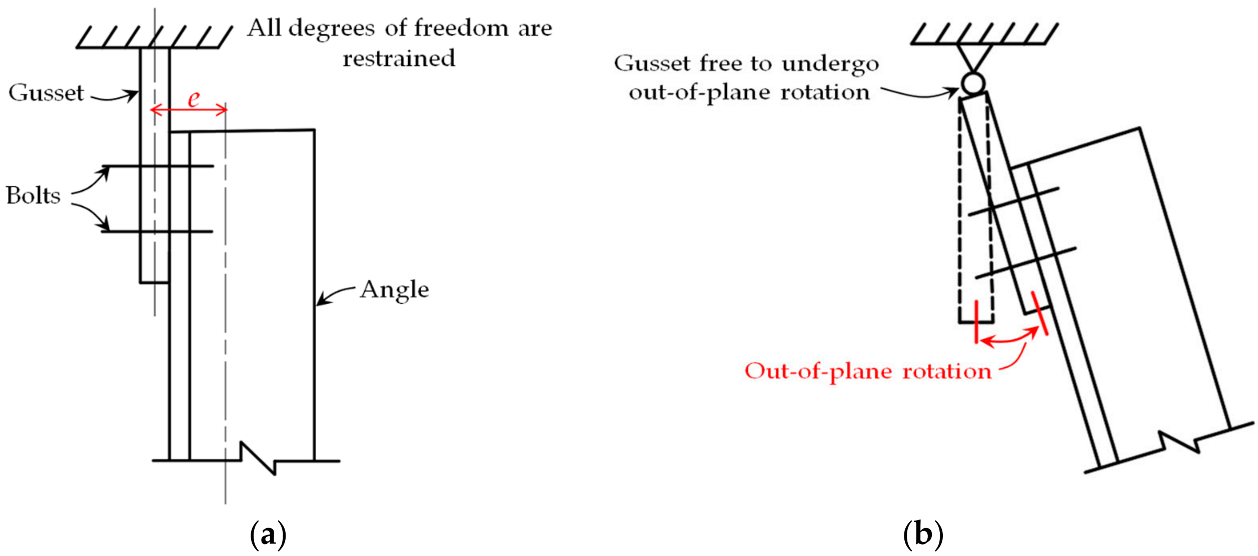


Figure 4. Rotational restraint of gusset plate: (**a**) gusset—fixed condition; (**b**) gusset—hinged condition.

Chen and Wang [20] proposed modified design equations based on the equivalent slenderness concept to accurately predict the capacity of high-strength steel angles also accounting for compression, bi-axial bending, and torsion under eccentric loading. It was also stated that the failure axis (axis of bending) was neither the geometrical (centroidal) axis parallel to the connected leg of the angle section (or restraining member) nor the minor-principal axis. In fact, the bending of the angle section at its mid-height was about an axis (*Q*-*Q*) oriented at an angle with respect to the geometrical axis parallel to the connected leg as shown in Figure 5. The thickness of the end gusset (or restraining member) and the width-to-thickness ratio of the connected leg of the angle section also influenced the capacity of the angle section significantly and were accounted for in the proposed design equations for accurately predicting their axial capacity.

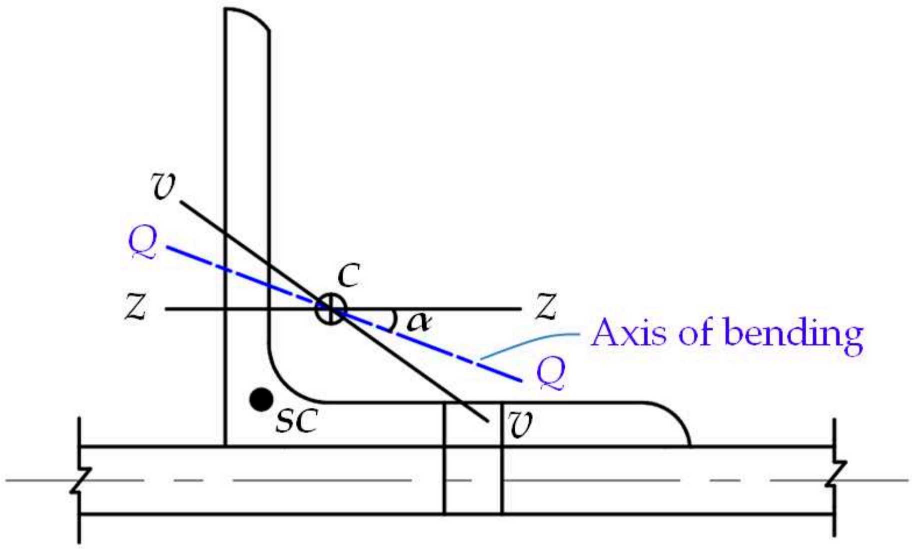


Figure 5. Axis of bending at mid-height of column.

While recent studies [21,22] have assessed the structural response of eccentrically loaded single angles with bolted end connections, limited findings have been reported on welded end connections [23,24]. Based on the experimental and numerical studies carried out by Temple and Sakla [23,24] reported in 1998, it was observed that the axis of bending was oriented at an angle with respect to the geometrical axis parallel to the connected leg at mid-height. It was also noted that the weld length, thickness of the gusset, initial imperfection, and material properties of the angle sections significantly influence the capacity of the eccentrically loaded single angles. Hence, it may be concluded that the capacity of eccentrically loaded angles in general is influenced by the type of end connection, degree of rotational restraint offered by both end connection and restraining member, width–thickness ratio of the connected leg of the angle, thickness of the restraining member (end gusset or adjoining member element), axis of bending, overall slenderness, initial imperfection and material strength of the angle section.

Different international codes of practice adopt differed approaches for designing eccentrically loaded single angles. Recently, Bashar and Amanat [25] carried out a numerical investigation on the behaviour of eccentrically loaded single angle struts of a truss system, which was originally tested experimentally by Elgaaly [9]. They compared the numerical strengths with the design strengths predicted using the provisions of various national codes of practice. It must to be noted that though the offset of web angles with respect to the top and bottom chord angle sections was accounted, the end connection was not modelled explicitly in the numerical investigation, and also the load was applied concentrically onto the considered web angle. The design provisions of IS 800–2007 [26] resulted in a correlation coefficient of 0.94, conservatively predicting the axial capacity in the order of 88% of the test strength. These results were the best among all the various codes of practice considered and hence was recommended for safe and reliable design. An amendment to the IS 800–2007 [27] was released recently (in 2024), in which modified design provisions for eccentrically loaded single angles were incorporated.

Hence, through this study, an effort has been made to assess the implications of the latest modifications [27] on the design adequacy of eccentrically loaded single angle compression members as compared with previous provisions [26]. Initially, the design strengths were obtained for equal leg Indian Standard Angle (ISA) 50 × 50 × 6 [28] both as per the latest and previous design provisions. Later, the reported data available in literature [21,22] was compared with both the previous and latest provisions. Based on this holistic comparison, salient findings and recommendations have been brought out, which will be greatly helpful to both practicing engineers and researchers. The scope of this study is limited to non-slender angle sections with no local buckling failures.

2. Design Provisions of IS 800–2007 [26]

As per clause 7.5.1.2, the flexural-torsional buckling (FTB) strength of a single angle loaded eccentrically (through one leg) is to be evaluated considering equivalent slenderness ratio, 'λ_e'. The stated provisions were based on the numerical work carried out by Sambasiva Rao et al. [29] at IIT Madras, India, considering both bolted and welded end connections.

$$\lambda e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_{\varphi}^2}. \tag{1}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}. \tag{2}$$

$$\lambda_{\varphi} = \frac{\left(\frac{b_1+b_2}{2t}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}. \tag{3}$$

where

l = centre–centre length of the supporting member

r_{vv} = radius of gyration about minor-principal axis

b_1, b_2 = width of the connected and outstanding legs, respectively

t = thickness of the leg of angle section

ε = yield stress ratio = $\sqrt{\frac{250}{f_y}}$

f_y = yield stress or strength
E = modulus of elasticity = 200 GPa = 2.0 × 10^5 MPa

$$\phi = 0.5\left[1 + \alpha\,(\lambda - 0.2) + \lambda_e^2\right] \tag{4}$$

The imperfection factor α is to be taken as 0.49, as buckling class '*c*' has to be considered for angle sections (as per clause 7.1.2.2.)

$$\text{Stress reduction factor, } \chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_e^2}} \tag{5}$$

$$\text{Design compressive stress, } f_{cd} = \chi\, f_y/\gamma_{m0} \tag{6}$$

$$\text{Design compressive strength, } P_d = A \times f_{cd} \tag{7}$$

As per code [26], the b/t and $(b_1 + b_2)/t$ ratios of the angle sections should not exceed 15.7 ε and 25 ε in order to avoid local buckling. The constants k_1, k_2, and k_3 account for different end connection fixities (as mentioned in Table 1), which were obtained by carrying out a multivariate regression analysis of the obtained data from the reported numerical investigation and available test results in the literature [29]. Bolted connection was classified into two or more bolts and single bolt cases (Figures 6a and 6b, respectively). In general, providing two or more bolts offers greater rotational restraint over the single bolt connection, whether in-plane of the gusset or out-of-plane. Though welded connection (Figure 6c) was not mentioned separately, it was usually considered equivalent to that of two bolt case.

Table 1. Constants k_1, k_2, and k_3 as per IS 800–2007 [26].

No. of Bolts at Each end Connection	Gusset/Connecting Member Fixity	k_1	k_2	k_3
≥2	Fixed	0.20	0.35	20
	Hinged	0.70	0.60	5
1	Fixed	0.75	0.35	20
	Hinged	1.25	0.50	60

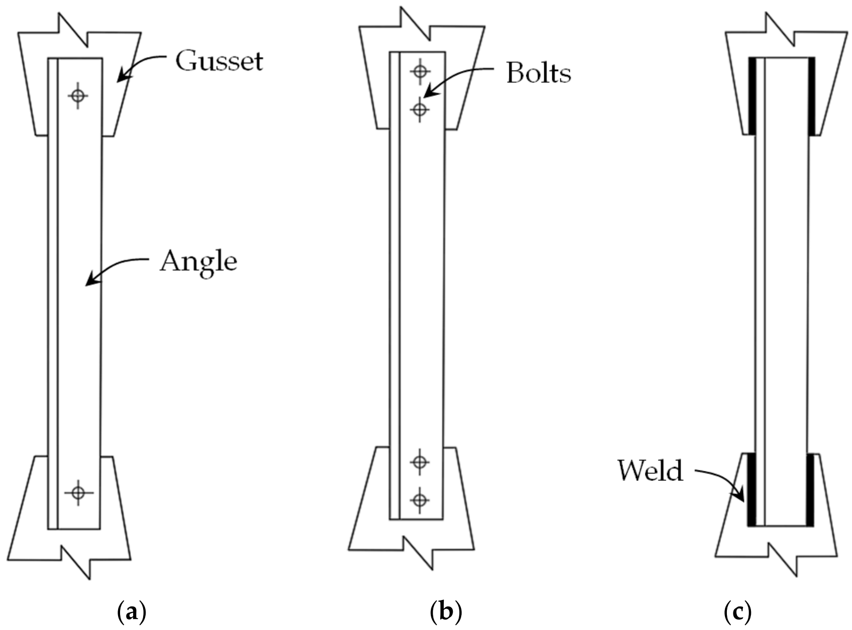


Figure 6. Classification of end connection: (**a**) single bolt; (**b**) two or more bolts; (**c**) welded.

3. Design Provisions of IS 800–2007 Amendment 2 [27]

As per Amendment 2 to IS 800–2007, for the single angles loaded eccentrically, the combined effect of both flexural torsional buckling and bending has to be accounted for in the design. The design compressive strength in such cases can be determined using the following method, instead of adopting a more precise second order analysis and design for combined bending and compression.

$$f_{cde} = K_f \chi_{aa} f_y / \gamma_{m0} \tag{8}$$

K_f = modification factor to account for eccentric end connection = $k_1 + k_2\ \lambda_{aa} + k_3\ \lambda_\phi$ (9)

$$\lambda_{\text{aa}} = \frac{\left(\frac{l_{aa}}{r_{aa}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \tag{10}$$

where λ_ϕ is same as expressed in Equation (3)

l_{aa} = centre-to-centre length of lateral support preventing translation of member perpendicular to a-a axis

r_{aa} = radius of gyration of the angle member about the a-a axis

k_1, k_2 and k_3 = refer to Table 2

Table 2. Constants k_1, k_2, and k_3 as per IS 800–2007 Amendment 2.

End Connection	Gusset/Connecting Member Fixity	k_1	k_2	k_3
Fully welded or connected with two or more bolts	Fixed	0.798	0.563	−2.072
	Hinged	0.401	0.420	−1.040
Single bolt	Fixed	0.418	0.547	−1.400
	Hinged	0.374	0.415	−2.072

$$\phi = 0.5\left[1 + \alpha\left(\lambda_{\text{aa}} - 0.2\right) + \lambda_{aa}^2\right] \tag{11}$$

The imperfection factor α is to be taken as 0.34, as buckling class '*b*' has to considered for angle sections.

$$\text{Stress reduction factor } \chi_{aa} = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{aa}^2}} \tag{12}$$

Upon substituting the obtained χ_{aa} in Equation (8) and thereafter substituting f_{cde} in place of f_{cd} in Equation (7), the design strength P_d of the angle section is obtained.

4. Major Changes Incorporated in Amendment 2

Upon careful examination of the design provisions of both IS 800–2007 [26] and Amendment 2 to IS 800–2007 [27], the following major changes were observed:

(1) It was stated that both flexural-torsional buckling FTB and bending effects were considered in the latest design provisions, contrary to FTB alone considered earlier.

(2) In the design calculations, the radius of gyration of the minor-principal axis is no longer required, and the emphasis is on the centroidal axis parallel to the connected leg or the plane of end gusset (designated as the *a-a* axis)—that is, considering out-of-plane buckling (buckling in the direction perpendicular to the plane of the gusset or structural system). Thus, it presumes much greater rotational restraint offered by the restraining member against in-plane buckling i.e., buckling of the angle about axis perpendicular to the plane of gusset [8,29,30]. But, the latest tests [22] reported that in-plane buckling and combined in-plane and out-of-plane buckling were also possible. Based on these experiments and numerical analysis, a new set of equations

were proposed to determine the rotational stiffness for both in-plane and out-of-plane buckling [31–34].

(3) The buckling class has been upgraded from curve '*c*' to '*b*', thereby resulting in a lower imperfection factor of 0.34 (to that of 0.49 adopted previously, corresponding to buckling class '*c*'), as adopted in EN 1993-1-1: EC3 [35].

(4) A set of new values were presented for constants k_1, k_2, and k_3.

(5) A new modification factor K_f has been introduced, which accounts for the influence of end connection fixity on the slenderness ratios λ_{aa} and λ_{ϕ}.

5. Design Strength Comparison: IS 800–2007 [26] vs. IS 800–2007 (Amendment 2) [27]

The design strengths of eccentrically loaded non-slender single angle ISA 50 × 50 × 6 [28] as per the IS 800–2007 [26] and its latest amendment [27] were presented in Tables 3–6 for column lengths varying from 0.5 m to 3.0 m and different end connections. The yield stress, f_y was assumed as 250 MPa. The ratio f_{cd}/f_y and design strength 'P_d' were also presented. Subscripts 1 and 2 represent the design strengths obtained as per [26] and [27], respectively. The plots for f_{cd}/f_y vs. λ for different end connection fixities were shown in Figures 7–10. The design strengths [26,27] for various other angle sections (for L = 1.5 m) were also presented in Tables 7 and 8 corresponding to two bolt—fixed and one bolt—hinged cases, respectively. Figure 11 depicts the graphical representation for the case of 2 bolts—fixed case.

Table 3. Two bolt/welded—fixed case.

L (m)	IS 800:2007 [26]				IS 800:2007 Amendment 2 [27]				P_{d2}/P_{d1}
	L/r_{vv}	λ_e	f_{cd}/f_y	P_{d1} (kN)	L/r_{aa}	λ_{aa}	f_{cd}/f_y	P_{d2} (kN)	
0.50	52.08	0.70	0.66	93.20	33.11	0.37	0.69	98.37	1.06
0.75	78.13	0.80	0.60	85.15	49.67	0.56	0.72	101.62	1.19
1.00	104.17	0.93	0.53	75.41	66.23	0.75	0.70	100.07	1.33
1.25	130.21	1.06	0.46	65.18	82.78	0.93	0.66	93.33	1.43
1.50	156.25	1.21	0.39	55.53	99.34	1.12	0.59	83.50	1.50
1.75	182.29	1.36	0.33	47.06	115.89	1.30	0.52	73.38	1.56
2.00	208.33	1.52	0.28	39.92	132.45	1.49	0.45	64.39	1.61
2.50	260.42	1.84	0.21	29.21	165.56	1.86	0.36	50.64	1.73
2.75	286.46	2.00	0.18	25.25	182.12	2.05	0.32	45.49	1.80
3.00	312.50	2.17	0.15	22.00	198.68	2.24	0.29	41.21	1.87

Table 4. Two bolt/welded—hinged case.

L (m)	IS 800:2007 [26]				IS 800:2007 Amendment 2 [27]				P_{d2}/P_{d1}
	L/r_{vv}	λ_e	f_{cd}/f_y	P_{d1} (kN)	L/r_{aa}	λ_{aa}	f_{cd}/f_y	P_{d2} (kN)	
0.50	52.08	0.97	0.50	71.61	33.11	0.37	0.39	55.63	0.78
0.75	78.13	1.10	0.44	62.58	49.67	0.56	0.42	59.55	0.95
1.00	104.17	1.25	0.37	52.87	66.23	0.75	0.42	60.29	1.14
1.25	130.21	1.43	0.31	43.88	82.78	0.93	0.40	57.47	1.31
1.50	156.25	1.61	0.26	36.25	99.34	1.12	0.37	52.35	1.44
1.75	182.29	1.81	0.21	30.05	115.89	1.30	0.33	46.48	1.55
2.00	208.33	2.01	0.18	25.10	132.45	1.49	0.29	41.48	1.65
2.50	260.42	2.43	0.13	18.01	165.56	1.86	0.23	33.28	1.85
2.75	286.46	2.64	0.11	15.48	182.12	2.05	0.21	30.14	1.95
3.00	312.50	2.86	0.09	13.42	198.68	2.24	0.19	27.49	2.05

Table 5. One bolt—fixed case.

L (m)	IS 800:2007 [26]				IS 800:2007 Amendment 2 [27]				P_{d2}/P_{d1}
	L/r_{vv}	λ_e	f_{cd}/f_y	P_{d1} (kN)	L/r_{aa}	λ_{aa}	f_{cd}/f_y	P_{d2} (kN)	
0.50	52.08	1.02	0.48	68.01	33.11	0.37	0.42	59.32	0.87
0.75	78.13	1.09	0.44	62.94	49.67	0.56	0.46	65.55	1.04
1.00	104.17	1.19	0.40	56.86	66.23	0.75	0.48	67.90	1.19
1.25	130.21	1.30	0.36	50.46	82.78	0.93	0.46	65.87	1.31
1.50	156.25	1.42	0.31	44.27	99.34	1.12	0.43	60.83	1.37
1.75	182.29	1.55	0.27	38.63	115.89	1.30	0.39	54.85	1.42
2.00	208.33	1.69	0.24	33.65	132.45	1.42	0.35	49.18	1.46
2.50	260.42	1.98	0.18	25.70	165.56	1.86	0.28	40.01	1.56
2.75	286.46	2.14	0.16	22.59	182.12	2.05	0.26	36.44	1.61
3.00	312.50	2.29	0.14	19.96	198.68	2.24	0.24	33.40	1.67

Table 6. One bolt—hinged case.

L (m)	IS 800:2007 [26]				IS 800:2007 Amendment 2 [27]				P_{d2}/P_{d1}
	L/r_{vv}	λ_e	f_{cd}/f_y	P_{d1} (kN)	L/r_{aa}	λ_{aa}	f_{cd}/f_y	P_{d2} (kN)	
0.50	52.08	1.40	0.32	45.26	33.11	0.37	0.28	40.43	0.89
0.75	78.13	1.47	0.29	41.48	49.67	0.56	0.32	45.55	1.10
1.00	104.17	1.57	0.27	37.81	66.23	0.75	0.34	47.82	1.26
1.25	130.21	1.69	0.24	33.64	82.78	0.93	0.33	46.85	1.39
1.50	156.25	1.82	0.21	29.63	99.34	1.12	0.31	43.59	1.47
1.75	182.29	1.97	0.18	25.98	115.89	1.30	0.28	39.54	1.52
2.00	208.33	2.13	0.16	22.76	132.45	1.49	0.25	35.63	1.57
2.50	260.42	2.46	0.12	17.55	165.56	1.86	0.21	29.20	1.66
2.75	286.46	2.64	0.11	15.49	182.12	2.05	0.19	26.67	1.72
3.00	312.50	2.82	0.10	13.73	198.68	2.24	0.17	24.51	1.79

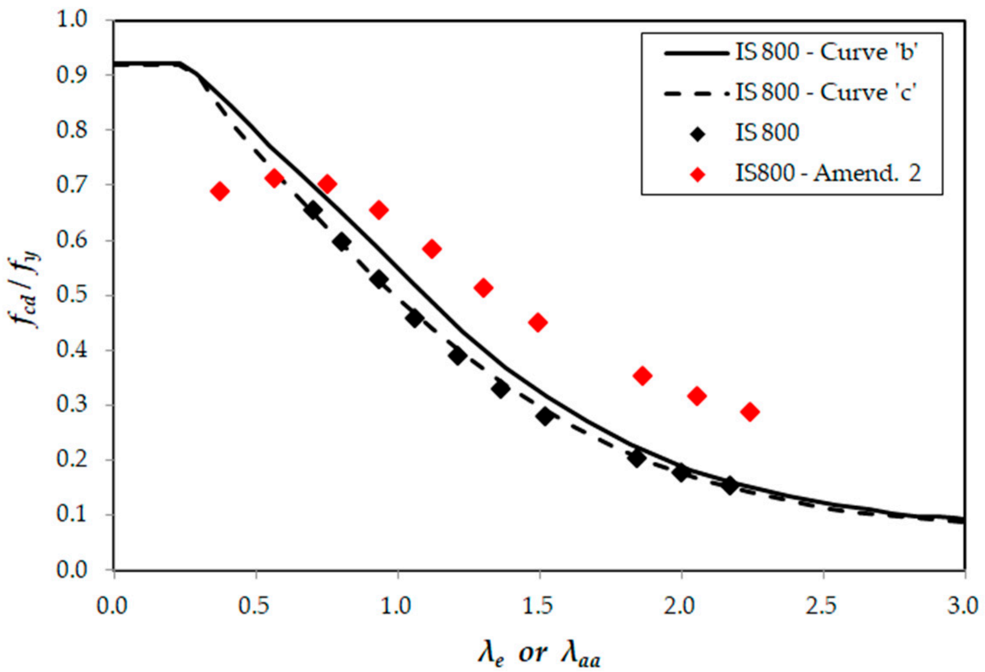


Figure 7. f_{cd}/f_y vs. λ plot for two bolt/welded—fixed case.

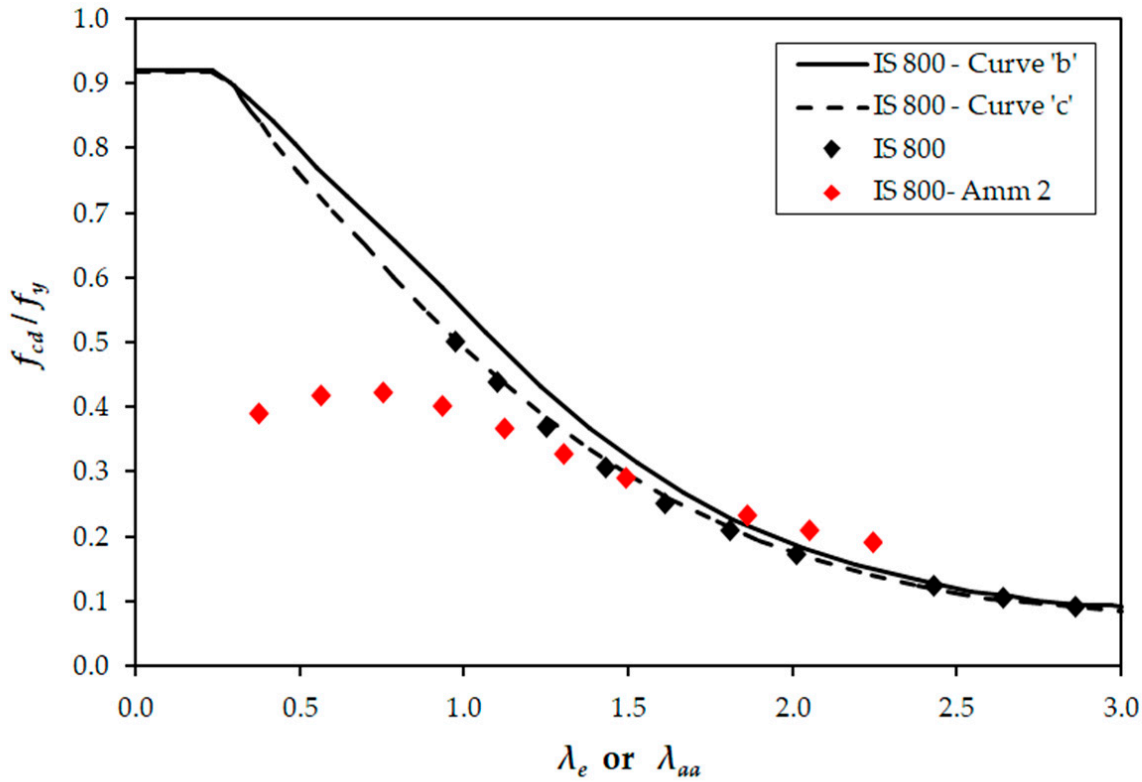


Figure 8. f_{cd}/f_y vs. λ plot for two bolt/welded—hinged case.

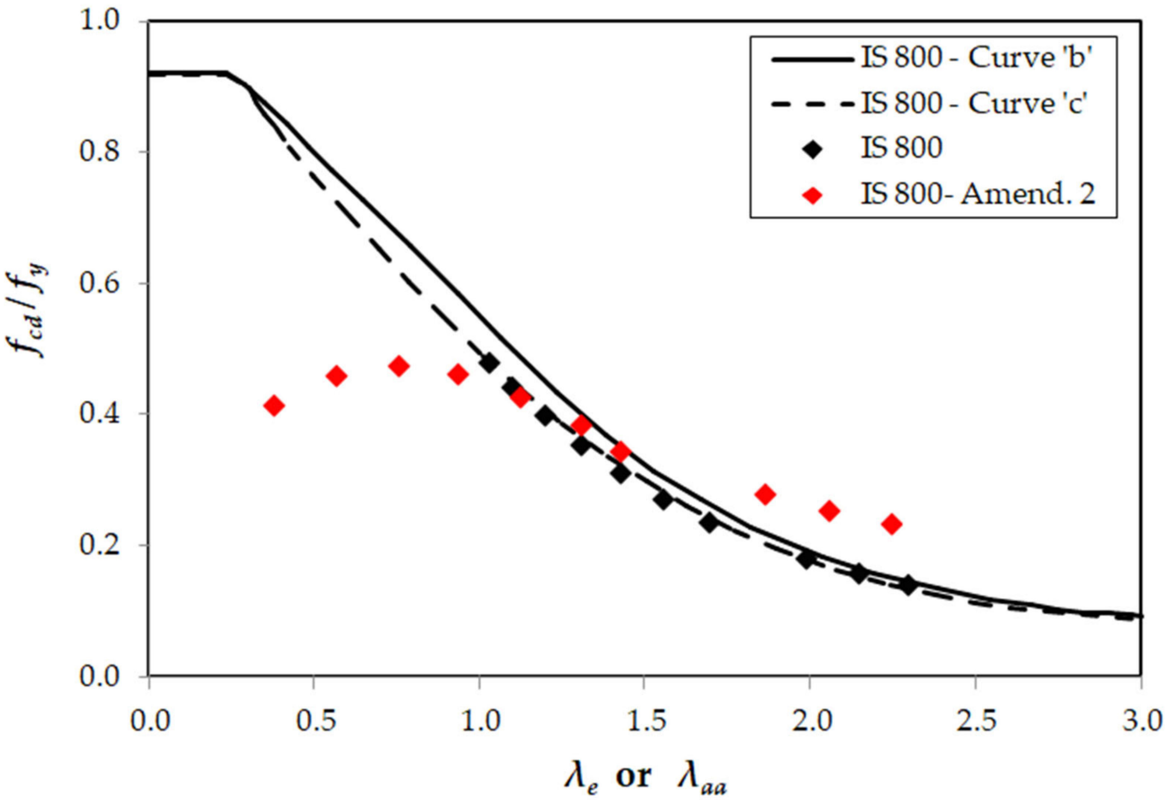


Figure 9. f_{cd}/f_y vs. λ plot for one bolt-fixed case.

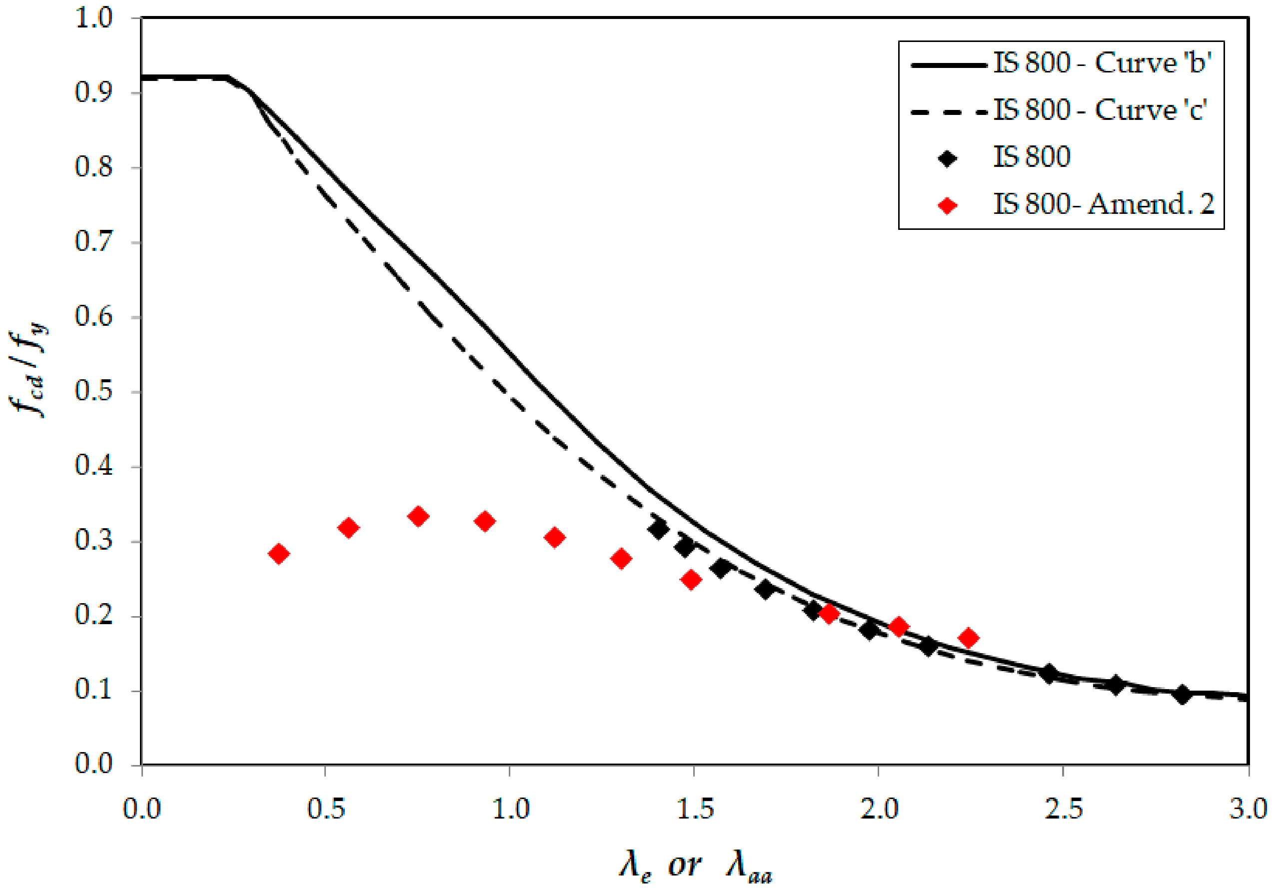


Figure 10. f_{cd}/f_y vs. λ plot for one bolt-hinged case.

Table 7. Design strength of various angles—two bolts and fixed case.

Angle	b/t	L/r_{vv}	L/r_{aa}	λ_e	λ_{aa}	P_{d1} [26]	P_{d2} [27]	P_{d2}/P_{d1}
45 × 45 × 6	7.50	172.41	111.11	1.29	1.25	45.37	69.05	1.52
55 × 55 × 6	9.17	141.51	90.36	1.14	1.02	65.90	96.50	1.46
55 × 55 × 8	6.88	141.51	91.46	1.10	1.03	90.14	130.91	1.45
60 × 60 × 6	10.00	130.43	82.42	1.10	0.93	75.37	108.73	1.44
60 × 60 × 8	7.50	130.43	83.33	1.05	0.94	104.47	149.28	1.43
65 × 65 × 6	10.83	119.05	75.76	1.06	0.85	85.47	119.90	1.40
65 × 65 × 8	8.13	120.00	76.53	1.00	0.86	119.41	166.38	1.39
70 × 70 × 6	11.67	110.29	70.09	1.04	0.79	94.60	129.55	1.37
70 × 70 × 8	8.75	111.11	70.75	0.97	0.80	134.29	182.48	1.36
75 × 75 × 6	12.50	102.74	65.22	1.03	0.73	102.73	138.34	1.35
75 × 75 × 8	9.38	103.45	65.79	0.95	0.74	148.00	196.27	1.33
75 × 75 × 10	7.50	103.45	66.37	0.90	0.75	190.11	251.23	1.32
80 × 80 × 8	10.00	96.77	61.48	0.93	0.69	160.92	208.59	1.30
80 × 80 × 10	10.00	103.45	66.37	0.88	0.75	188.01	248.42	1.32
80 × 80 × 12	6.67	97.40	62.76	0.86	0.71	253.62	328.29	1.29
90 × 90 × 8	11.25	85.71	54.55	0.92	0.61	184.30	229.42	1.24
90 × 90 × 10	9.00	86.21	54.95	0.86	0.62	242.02	299.43	1.24
90 × 90 × 12	8.33	86.21	55.35	0.82	0.62	298.31	369.01	1.24

Table 8. Design strength of various angles—one bolt and hinged case.

Angle	b/t	L/r_{vv}	L/r_{aa}	λ_e	λ_{aa}	P_{d1}	P_{d2}	P_{d1}/P_{d2}
45 × 45 × 6	7.50	172.41	111.11	1.89	1.25	24.97	37.36	1.50
55 × 55 × 6	9.17	141.51	90.36	1.78	1.02	34.1	48.57	1.42
55 × 55 × 8	6.88	141.51	91.46	1.70	1.03	48.09	68.93	1.43
60 × 60 × 6	10.00	130.43	82.42	1.76	0.93	37.95	52.59	1.39
60 × 60 × 8	7.50	130.43	83.33	1.66	0.94	54.57	76.29	1.40
65 × 65 × 6	10.83	119.05	75.76	1.74	0.85	41.81	55.56	1.33
65 × 65 × 8	8.13	120	76.53	1.63	0.86	61.08	82.47	1.35
70 × 70 × 6	11.67	110.29	70.09	1.75	0.79	45.11	57.51	1.27
70 × 70 × 8	8.75	111.11	70.75	1.62	0.80	67.35	87.61	1.30
75 × 75 × 6	12.50	102.74	65.22	1.76	0.73	47.81	58.22	1.22
75 × 75 × 8	9.38	103.45	65.79	1.61	0.74	72.83	91.15	1.25
75 × 75 × 10	7.50	103.45	66.37	1.53	0.75	96.59	122.55	1.27
80 × 80 × 8	10.00	96.77	61.48	1.61	0.69	77.77	93.53	1.20
80 × 80 × 10	10.00	103.45	66.37	1.53	0.75	94.74	119.74	1.26
80 × 80 × 12	6.67	97.40	62.76	1.48	0.71	130.00	161.48	1.24
90 × 90 × 8	11.25	85.71	54.55	1.64	0.61	86.01	95.39	1.11
90 × 90 × 10	9.00	86.21	54.95	1.53	0.62	118.03	134.56	1.14
90 × 90 × 12	8.33	86.21	55.35	1.47	0.62	149.63	173.41	1.16

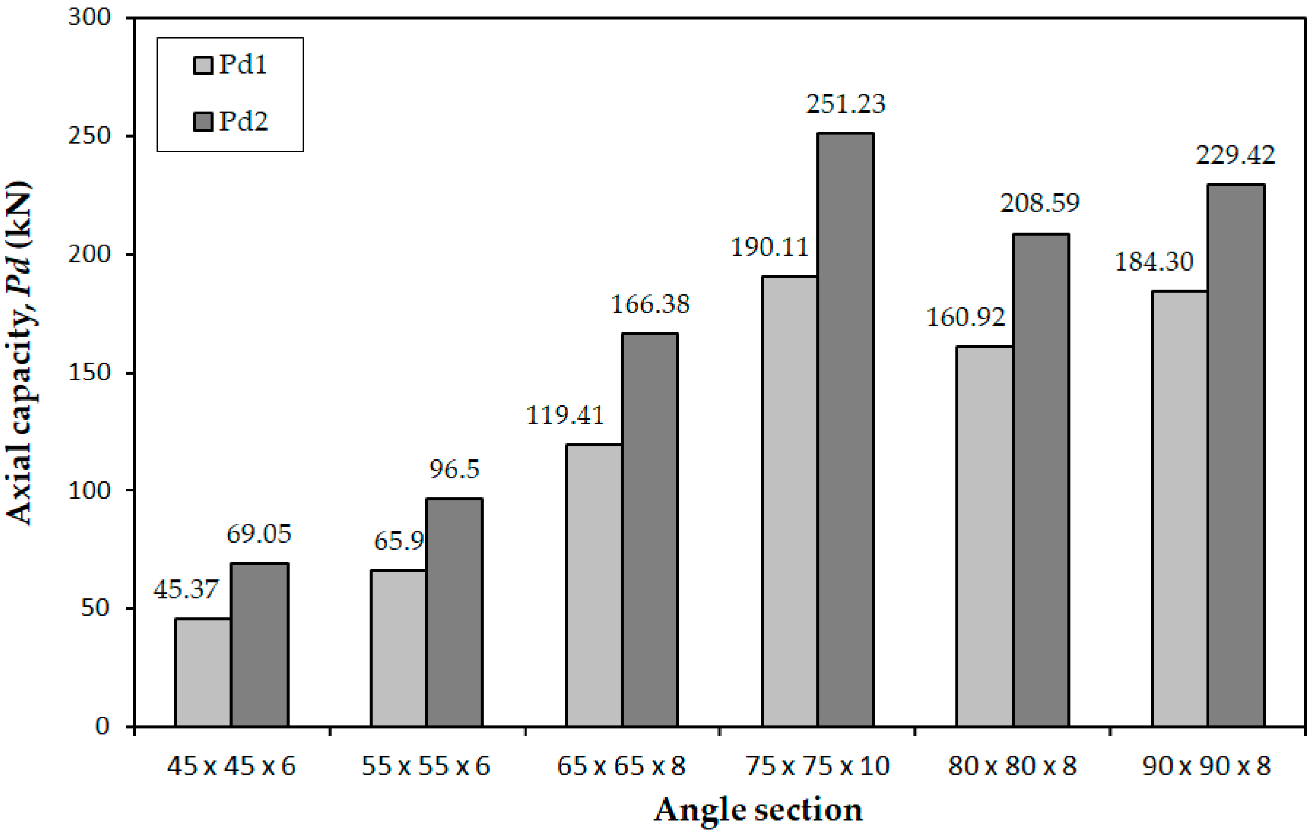


Figure 11. Axial capacity of various angle sections (two bolts—fixed case).

6. Nominal Predicted Strengths vs. Test Strengths

Since a major change in design strength as per [26,27] was noted (Section 5), it is imperative to compare the nominal strength 'P' (i.e., substituting γ_{m0} = 1.0) in Equations (6) and (8) with the available test strengths.

6.1. Comparison with Test Strengths Reported by Bhilawe [21]

Bhilawe [21] carried out tests on single angle eccentrically loaded columns provided with bolted connection and hinged boundary condition. Both single and two bolt cases were considered, though no bolt detailing information was available. This study considers only non-slender sections, to which the design provisions of IS 800 [26,27] are applicable. The test strengths and the nominal strengths as per [26] and [27] were presented in Tables 9 and 10 for single bolt and two bolt cases, respectively. The actual yield strength (f_y) of the angle specimens (Tables 9 and 10) was 351.08 MPa. It is worth mentioning that though the measured eccentricity of the load was stated, it does not influence the design provisions [26,27] and hence was not presented here.

Table 9. Single bolt case.

Specimen	L/r_{vv}	L/r_{aa}	P_{Test} (kN) [21]	P_{FEA} (kN) [21]	P_{IS800}(kN)	$P_{IS800\_Amend.2}$(kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
S2A	62.17	39.53	92.5	106.32	60.95	63.03	1.52	1.47
S2B	114.56	72.83	56.23	61.12	47.77	67.10	1.18	0.84
S2C	156.25	99.34	41.62	43.82	37.80	57.41	1.10	0.72
				Mean			1.27	1.01
				Std Dev			0.22	0.40
				COV			0.18	0.40

Table 10. Two bolts case.

Specimen	L/r_{vv}	L/r_{aa}	P_{Test} (kN) [21]	P_{FEA} (kN) [21]	P_{IS800}(kN)	$P_{IS800\_Amend.2}$(kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
S6A	62.17	39.53	97.50	106.32	98.32	88.13	0.99	1.11
S6B	114.56	72.83	62.23	61.12	64.67	85.08	0.96	0.73
S6C	156.25	99.34	50.23	55.18	45.28	69.69	1.11	0.72
				Mean			1.02	0.85
				Std Dev			0.08	0.22
				COV			0.08	0.26

Figures 12 and 13 depict the plots f_{cn}/f_y vs. λ_v corresponding to single and two bolt cases, where 'f_{cn}' is the nominal compressive stress and 'λ_v' is the non-dimensional slenderness ratio with respect to the minor-principal axis. The minor principal axis slenderness parameter 'λ_v' was considered along the x-axis because the radius of gyration is the smallest (and hence, the least slenderness is L/r_{min}, where $r_{min} = r_{vv}$) and is also more convenient for comparison, as 'λ_e' and 'λ_{aa}' result in different coordinates.

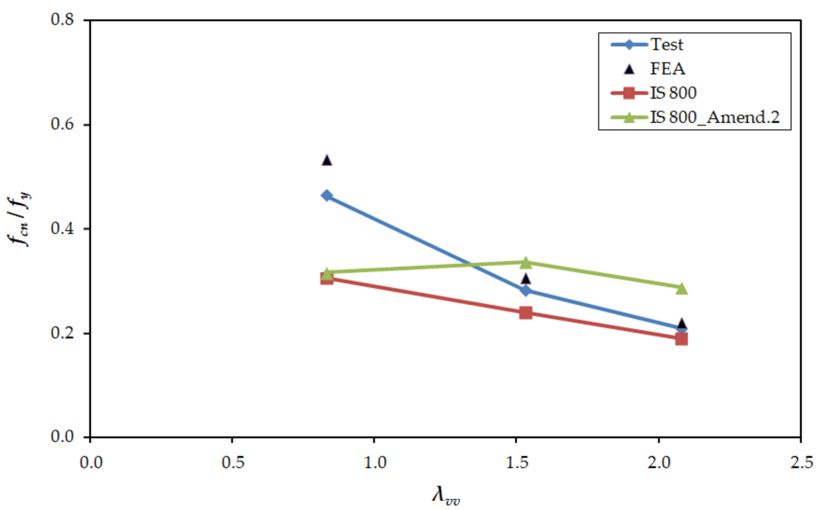


Figure 12. f_{cn}/f_y vs. λ_{vv} plot for single bolt case [21].

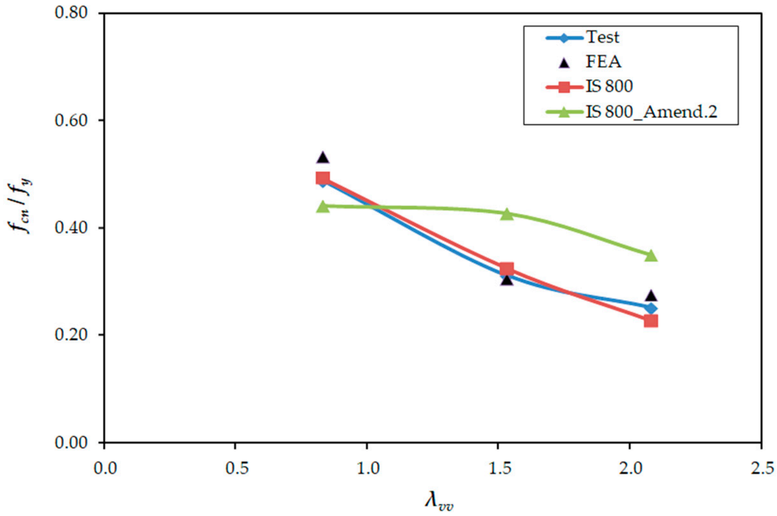


Figure 13. f_{cn}/f_y vs. λ_{vv} plot for two bolt case [21].

6.2. Comparison with Test Strengths Reported by Kettler et al. [22]

Kettler et al. [22] reported an experimental investigation on the behaviour of eccentrically loaded single angles with bolted end connections. The test strengths were validated with the numerical strengths obtained by performing a numerical buckling analysis. The nominal strengths 'P' (γ_{m0} = 1.0) obtained for the considered specimens as per [26,27] were presented in Tables 11–14, which include both non-preloaded and preloaded bolts. The yield strengths 'f_y' of specimens designated with starting letters A, B, C, D, and E were 289.9 MPa, 326.8 MPa, 333.9 MPa, 322.4 MPa, and 299.3 MPa, respectively. Similarly, the modulus of elasticity's 'E' were 212 GPa, 199 GPa, 209 GPa, 195 GPa and 192 GPa, in the same order. The plots f_{cn}/f_y vs. λ_v for a few selected cases were shown in Figures 13–15.

Table 11. Two bolts—fixed case.

Specimen	L/r_{vv}	L/r_{aa}	Bolt Type	P_{Test} (kN) [22]	P_{FEA} (kN) [22]	P_{IS800} (kN)	$P_{IS800\_Amend.2}$ (kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
A1	75.23	43.07	Preloaded M20 (10.9)	261.1	261.2	225.36	271.03	1.16	0.96
A2	118.69	76.54		238.8	235.8	179.09	253.08	1.33	0.94
A3	170.35	109.88		215.4	202.2	131.89	202.41	1.63	1.06
D1	76.12	48.56	Non-preloaded M20 (10.9)	260.2	273.5	220.38	277.33	1.18	0.94
D2	172.65	109.88		177.5	204.9	120.16	191.49	1.48	0.93
E1	80.42	51.78	Preloaded M27 (10.9)	488.4	512.1	479.4	607.2	1.02	0.80
E2	136.69	87.95		357.2	409.7	341.15	509.71	1.05	0.70
E3	180.26	116.60		267.1	326.8	254.26	401.19	1.05	0.67
				Mean				1.24	0.88
				Std Dev				0.22	0.14
				COV				0.18	0.16

Table 12. Two bolts—hinged case.

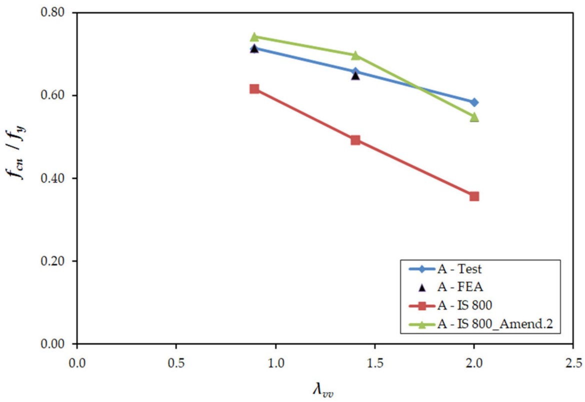
Specimen	L/r_{vv}	L/r_{aa}	Bolt Type	P_{Test} (kN) [22]	P_{FEA} (kN) [22]	P_{IS800} (kN)	$P_{IS800\_Amend.2}$ (kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
B1	96.11	61.32	Preloaded M20 (10.9)	148.0	140.0	150.04	169.34	0.99	0.87
B2	148.10	94.65		86.4	85.0	97.84	141.09	0.88	0.61
B3	183.97	117.28		61.0	61.6	74.33	119.32	0.82	0.51
				Mean				0.90	0.67
				Std Dev				0.08	0.19
				COV				0.09	0.28

Table 13. Single bolt—fixed case.

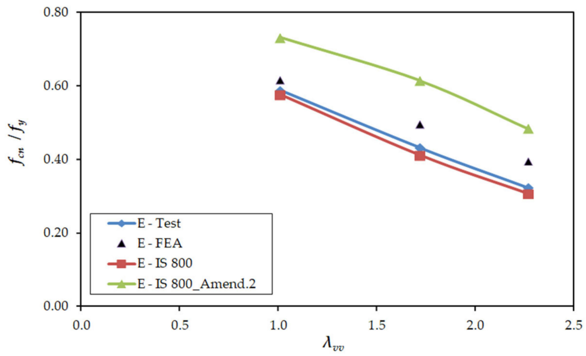
Specimen	L/r_{vv}	L/r_{aa}	Bolt Type	P_{Test} (kN) [22]	P_{FEA} (kN) [22]	P_{IS800} (kN)	$P_{IS800\_Amend.2}$ (kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
B4	76.12	48.56	Preloaded M20 (10.9)	162.9	165.9	169.29	184.74	0.96	0.88
B5	120.27	76.54		132.1	140.6	138.09	183.66	0.96	0.72
C1	207.06	132.1		98.4	106.1	87.04	130.22	1.13	0.76
D3	75.98	48.56	Non-preloaded M20 (10.9)	154.8	150.9	166.68	182.11	0.93	0.85
D4	207.20	132.10		73.1	89.9	82.93	124.76	0.88	0.59
				Mean				0.97	0.76
				Std. Dev.				0.09	0.12
				COV				0.10	0.15

Table 14. Single bolt—hinged case.

Specimen	L/r_{vv}	L/r_{aa}	Bolt Type	P_{Test} (kN) [22]	P_{FEA} (kN) [22]	P_{IS800} (kN)	$P_{IS800\_Amend.2}$ (kN)	P_{Test}/P_{IS800}	$P_{Test}/P_{IS800\_Amend.2}$
C4	95.54	61.32	Preloaded M20 (10.9)	131.0	139.3	99.34	125.78	1.32	1.04
C5	183.65	117.28		62.6	65.0	64.91	100.29	1.00	0.65
D5	25.83	16.46		145.5	145.7	115.89	71.86	1.26	2.03
D6	35.48	22.60		151.0	151.1	114.97	82.68	1.31	1.83
D7	45.31	28.81		148.8	153.4	112.51	92.03	1.32	1.62
D8	54.83	39.48		145.0	151.1	109.70	100.38	1.32	1.44
				Mean				1.26	1.43
				Std. Dev.				0.13	0.51
				COV				0.10	0.36



(**a**)



(**b**)

Figure 14. f_{cn}/f_y vs. λ_{vv} plot for two bolts—fixed case [22]: (**a**) series—A; (**b**) series—E.

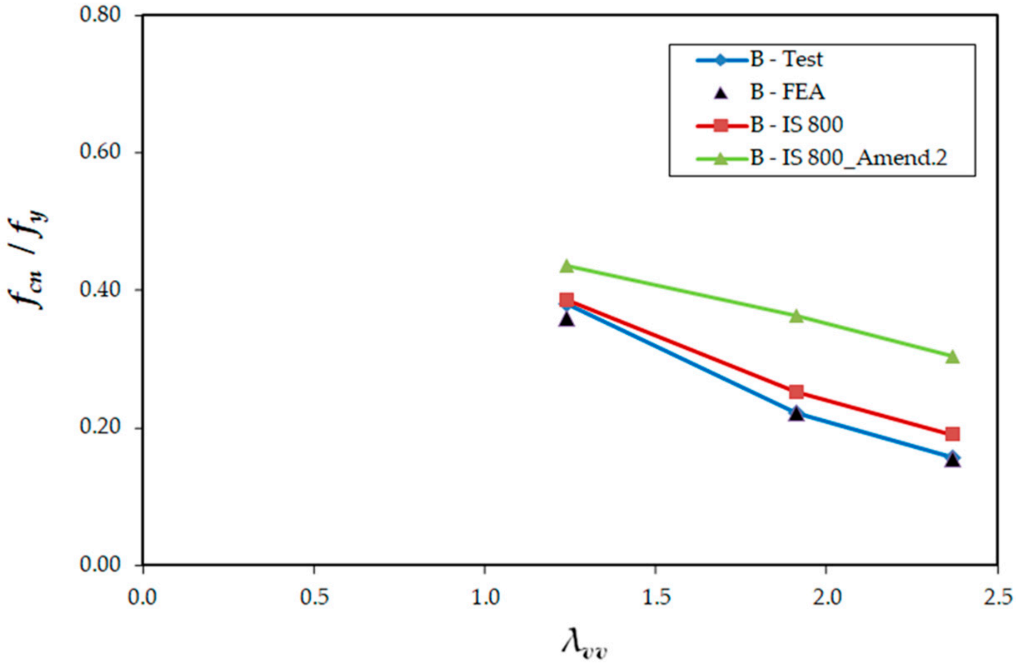


Figure 15. f_{cn}/f_y vs. λ_{vv} plot for two bolts—hinged case [22].

7. Discussion

Based on the results presented in Section 5, the following inferences are drawn:

(a) The design strengths obtained as per the latest provisions [27] were in general much higher in comparison to the corresponding strengths as per earlier provisions [26], except for very few cases of lower column slenderness (Tables 4–6). This could be due to the fact that the imperfection factor reduced from 0.49 to 0.34 as the buckling class was changed from '*c*' to '*b*' as per Amendment 2 [27] and for cases where $K_f > 1$.

(b) A maximum rise in axial capacity of 104.84% was noted for the case of two bolts or equivalent weld—hinged connection case, corresponding to a 3.0 m long angle.

(c) The design strengths P_{d2} do not match with the buckling curve '*b*' (Figures 7–10) [27], as compared to the earlier provisions [26] whereas the design strengths P_{d1} match with the buckling curve '*c*'.

(d) A single curve cannot be adopted for eccentrically loaded angles. This was also verified based on the experimental and numerical work reported by Kettler et al. [34]. This may be due to the applied load being eccentric to the centroid and only one leg being loaded. The design strength of the end connection and its type, upon which the degree of rotational restraint or fixity depends, also influence the design strength of the angle [32,33].

(e) The equivalent slenderness ratio λ_e [26] and slenderness ratio about the axis parallel to the connected leg λ_{aa} [27] were significantly different for angles with lengths varying from 0.5 m to 1.5 m. However, as the length increased beyond 1.5 m, the difference between the two slenderness ratios narrowed.

(f) An understanding of the implication of updated values for constants k_1, k_2, and k_3 pertaining to end connection fixity and the introduction of new parameter K_f on the design strength is not clear in view of the decreased imperfection factor (refer to (a)).

(g) For a given slenderness ratio, b/t of the connected leg also significantly influences the axial capacity (Tables 7 and 8).

Based on the comparison of nominal strengths obtained as per the code provisions and the available test strengths, as presented in Section 6, the following inferences are drawn:

(a) The nominal strengths obtained as per the latest provisions [27] did not correlate well with the available test strengths (also validated through numerical analysis) [21,22] considered in this study (Tables 9–14), though a good agreement was noticed for few cases (for specimens A1, A2, and A3—see Table 11 and Figure 14a).
(b) For the test data set considered, it is observed that the earlier design provisions [26] agreed very well with the test strengths reported in the literature.
(c) Although with a limited test data set, the nominal strengths as per earlier provisions [26] were lower than the test strengths for the majority of cases resulting in a safe and conservative design, whereas it was otherwise in the case of nominal strengths obtained as per the latest provisions [27] (see Figures 12–16).

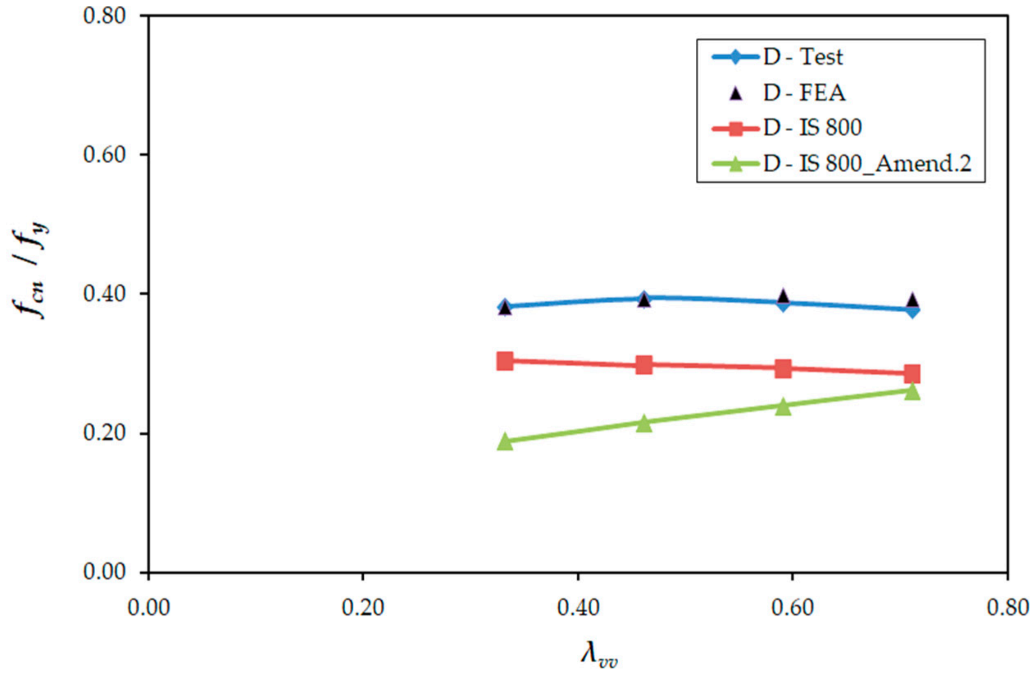


Figure 16. f_{cn}/f_y vs. λ_{vv} plot for single bolt—hinged case [22].

8. Conclusions

Eccentrically loaded single angle columns behave complicatedly and require specific design attention. Different international codes of practice construct such members differently. The literature shows that IS 800:2007 design provisions accurately anticipate axial capacity. Recently, IS 800:2007 was amended to include new design provisions for eccentrically loaded single angle compression members. This study presented a brief theoretical backdrop to the behaviour and evaluated the newest design provisions. From the present study, it is concluded the following:

(1) The latest design provisions resulted in higher axial capacities, which may lead to smaller cross-sectional sizes.
(2) From the reported results, the latest IS design provisions result in the increase of the design capacity by a maximum of 104.84% (as seen in Table 4) in comparison to the earlier design provisions.
(3) But on comparison with the considered data available (Tables 11–13) in the literature, it was observed that, in general, the latest design provisions over predict the nominal strength greatly, potentially indicating unsafe design.
(4) However, the latest IS design provisions resulted in very conservative nominal strengths for the one bolt-hinged case (Table 14).

(5) Hence, it is evident that more extensive experimental, numerical, and theoretical investigations have to be carried out considering all the influencing parameters for the accurate and reliable prediction of the axial capacity of eccentrically loaded single angle columns.

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