Comparative study of the AISC-LRFD, Eurocode 3 & SP 16.13330.2017 steel members design

T H Gebre1,2*, E V Lebed3 and V V Galishnikova1

1 Peoples’ Friendship University of Russia (RUDN University), Moscow, Russia
2 College of Engineering and Technology, Asmara, Eritrea
3 Moscow State University of Civil Engineering (National Research University), Moscow, Russia

Email: tesfaldethg@gmail.com

Abstract: This paper presents the comparative design procedures for steel frame elements using three different International design codes, viz. the American Institute of Steel Construction (AISC), the European Code (EC3) and the Russian Design Code (SP). It focuses on the resistance capacity of steel members subjected to different loads, such as tension, compression and flexure. It compares the formulation of all codes that identify their similarities and differences. The comparative study is performed for a compact rolled I sections with different slenderness and lateral bracing lengths. The results are presented graphically in different ways such as strength curves, Moment capacity versus lateral bracing length and normalized graphs. The comparative study shows that the resistance capacity specified by all specifications can be different for some considered cases. For compression member, SP specifications have higher capacity and economical approach. For flexural member, AISC gives higher capacity for inelastic region and EC3 gives higher capacities for elastic region but SP approach is lower for flexural capacity.

Keywords: tension member, compression member, design curve, flexural members, lateral-torsional buckling

1. Introduction

A very important task of any designer is to size the steel frame elements to be safe for the entire life of the structure. Owners may require the use of widely accepted steel design codes regardless of the location where the structure is going to be built. Nowadays, designers are exposed to the challenge of being competent with several design specifications for a particular material type. This paper, presents a comparison of different design specifications for the design of Steel frames elements and those are AISC, EC3 and SP as per the studies by different authors [1-8]. In the United States, specification for structural steel buildings was developed by the AISC which utilizes both load and resistance factor design (LRFD) and allowable strength design (ASD) formats [1, 6]. In Europe, “Design of Steel Structures, EN 1993 (EC3)” was developed by the European Committee for Standardization [3, 5]. In Russia, “Steel Structures Code SNIP II-232-81” is continuously being used in neighboring countries, after this referred to as the SP 16.13330.2011/17 specification [4, 7, 8]. The AISC specification is a compiled document whereas the EC3 and SP specifications contain parts and subparts in addition, each part is concentrated on a particular structure type. The comparative study is performed for a rolled I sections with different slenderness and lateral bracing lengths. The results are presented graphically by different means such as strength curve, non-dimensionless analysis and moment capacity versus lateral bracing curves to simplify the understanding and to detect the difference in the resistance.

2. Methods of research

2.1 Design of Members for Tension
For tension members, all specifications consider tensile yielding with gross section and tensile rupture with net section as the two primary limit states for tension members [5, 7]. The main difference between all specifications is the calculation of shear lag factor U. An elaborate treatment is tabulated in AISC. However, a less elaborate treatment is given in EC3 and SP.

2.2 Design of Members for Compression

The strength of compression member is governed by the yielding of the material for short ones, by elastic buckling for long ones and by inelastic buckling for ones of intermediate lengths. For extremely stocky members, failure may occur by compressive yielding rather than buckling. The design of columns makes an extensive use of the column strength curves. For a compression member in the absence of imperfections and assuming a linear-elastic constitutive law (Euler column), a value of the axial force can be found to trigger element instability, called elastic critical load [9-12]. The complete AISC specification for compressive strength curve is given below.

\[
\frac{KL}{r} \leq 4.71 \sqrt{\frac{F_y}{E}}, \quad F_{cr} = \left(0.658\lambda^2\right) \times \frac{F_y}{F_e} \leq 2.25
\]

\[
\frac{KL}{r} > 4.71 \sqrt{\frac{F_y}{E}}, \quad F_{cr} = \left(0.658\lambda^2\right) \times \frac{F_y}{F_e} > 2.25
\]

Using EC3 specification, the reduction factor (\(\chi\)) has the following forms:

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1.0
\]

\[
\Phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2],
\]

where \(\bar{\lambda}\) is the non-dimensional slenderness; \(\alpha\) is an imperfection factor.

As stated by EC3, there are five different buckling curves for columns analysis and the imperfection factor corresponding to the appropriate buckling curve is given in EN 1993-1-1 [10]. EC3 utilizes an imperfection coefficient (\(\alpha\)) to distinguish between different column strength curves [13-15].

Similarly, as stated by SP the reduction factor (\(\phi\)) has the following forms,

\[
\phi = \frac{\delta - \sqrt{\delta^2 - 39.48\bar{\lambda}^2}}{\bar{\lambda}^2},
\]

where \(\delta\) is the coefficient of stability under central compression, whose value at \(\bar{\lambda} \geq 0.4\); the value of the coefficient \(\delta\) in the above formula should be calculated as given below:

\[
\delta = 9.87(1 - \alpha + \beta \bar{\lambda}) + \bar{\lambda}^2,
\]

where \(\bar{\lambda}^2 = \lambda \sqrt{\frac{F_y}{E}}, \lambda = \sqrt{\frac{KL}{r}}\); \(\delta\) and \(\beta\) are the conditional flexibility coefficients of the member and determined from the code depending on the type of the section [7, 8].

2.3. Design of Flexure Members for compact section

In all specifications, yielding and lateral torsional buckling are the two limit states for flexural members and they are treated separately for clarity of comparisons. For compact section (class 1 or 2 in EC3 and class 3 or 2 in SP). All specifications allow the member to reach its plastic moment capacity and the nominal moment capacity for these types of sections is determined as follows:

\[
M_n = M_{pl} = Z \times F_y \quad \text{(AISC: compact)}
\]

\[
M_{c,Rd} = M_{pl,Rd} = W_{pl} \frac{f_y}{\gamma_{MO}} \quad \text{(EC3: class 1 and 2)}
\]

\[
M_x = \phi_b W_{cx} R_y g_c \quad \text{(SP: class 3 and 2)}
\]
where $M_n$ is the nominal moment capacity; $W_{pl}$ is the plastic section modulus; $f_y$ is the yield strength; $\gamma_{Mo}$ is the partial safety factor; $W_{cx}$ is the section modulus of cross section about the principal axis $x$; $\phi_b$ is the stability factor in bending; and $R_{yx}$ design yield strength.

For non-compact sections in AISC, the nominal moment capacity reduces linearly with an increase in the flange slenderness and varies between the plastic moment capacity ($M_p$) and the yield moment considering residual stresses ($0.7 \cdot M_y$). On the contrary, the nominal moment capacity is directly equal to the yield moment in the EC3 specification for class 3 cross-sections and SP specification uses reduction factor to compute the nominal moment capacity for non-compact section as well. The nominal moment capacity for members with non-compact flanges is determined as follows:

$$M_n = \frac{M_p}{M_y} - \left( \frac{M_p}{M_y} - 1 \right) \left( \frac{\lambda - \lambda_{plf}}{\lambda_{rw} - \lambda_{plf}} \right) M_y \quad \text{(AISC)}$$

$$M_{c,Rd} = M_{el,Rd} = \frac{f_y}{\gamma_{MO}} \quad \text{(EC3: if flange is a class 3)}$$

$$M_x = \phi_b W_{cx} R_y I_c \quad \text{(SP: if flange is a class 2)}$$

### 2.4. Lateral Torsional Buckling (LTB) of Compact I-shaped Members

The problem of lateral torsional buckling (LTB) of steel beams has been studied extensively by many authors, including Trahair and others [16-21]. All specifications have difference with the treatment of lateral torsional buckling. In the case of AISC specification, LTB is classified in to three different parts of buckling depending on the unbraced length of the member ($L_o$) and it has two threshold values for unbraced length namely $L_p$ and $L_r$ are defined in AISC [22-25]. The $C_b$ factor given in design specifications for non-uniform moment diagrams can be used to estimate the increased brace requirements for other loading cases [21]. The following equations summarize the nominal moment capacity for lateral torsional buckling as per the AISC specification:

$$M_n = M_p = Z \cdot F_y \quad \text{when } L_b \leq L_p$$

$$M_n = C_b \left[ M_p - (M_p - 0.7S_x F_y) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad \text{when } L_p < L_b \leq L_r$$

$$M_n = M_{cr} = S_x \left( \frac{C_b \pi^2 E}{(L_b)^2 r_{ts}^2} \right) \sqrt{1 + 0.078 \frac{J}{S_a h_o} \left( \frac{L_b}{r_{ts}} \right)^2} \quad \text{when } L_b > L_r$$

$$L_b = 1.7 \frac{r_{ts}}{F_y} \frac{E}{F_y} \sqrt{\frac{J}{S_x h_o}}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \frac{J}{S_x h_o} \sqrt{1 + \frac{1}{1 + 6.76 \left( \frac{0.7F_y S_x h_o}{E J} \right)^2}}$$

$$r_{ts}^2 = \sqrt{\frac{C_w}{S_x}}$$

where $S_x$ is the section modulus of the compression flange about the x-axis; $r_{ts}$ is the radius of gyration of cross-section; $h_o$ is the distance from the centroid of the top flange to the centroid of the bottom flange; $L_b$ is the unbraced length; and $L_r$ and $L_p$ are the two threshold values for unbraced length for the inelastic range and $C_b$ is the moment gradient factor.

In EC3, a reduction factor ($\chi$) is used to compute the capacity of a member with respect to the buckling and instability [2]. This factor is strongly dependent on the member slenderness parameter $\lambda$ [26]. According to EC3, the beam should be verified against lateral-torsional buckling resistance as
follows: The elastic critical moment \( M_{cr} \) is used as the basis for the methods given in design codes for determining the slenderness of a section [27]. The elastic critical moment \( M_{cr} \) is similar to the Euler (flexural) buckling of a strut in that it defines a buckling load [28]. According to Clause 6.3.2.1(1) of EN 1993-1-1, the beam should be verified against lateral-torsional buckling resistance as follows [6]:

\[
M_{c,Rd} = M_{pl,Rd} = \chi_{LT} W_y f_y \frac{f_y}{\gamma_{MO}}
\]

\[
\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \beta \lambda_{LT}^2}}, \quad \text{with } \chi_{LT} \leq 1 \text{ and } \chi_{LT} \leq \left( \frac{1}{\lambda_{LT}} \right)^2.
\]

And \( \Phi_{LT} \) is defined,

\[
\Phi_{LT} = 0.5\left[1 + \alpha_{LT} (\lambda_{LT} - \lambda_{LT,0}) + \beta \lambda_{LT}^2 \right],
\]

\[
\lambda_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}},
\]

where \( \alpha_{LT} \) is the imperfection factor corresponding to the appropriate buckling curve; \( \gamma_{MO} \) is the partial factor for member instability (the recommended value of 1.0 in EC3); \( W_y f_y \) is the section moment resistance; \( \lambda_{LT} \) is the modified slenderness; and the values of \( \alpha_{LT} \) and \( \beta \) depend on the type of beam section.

SP approach also uses a reduction factor \( \phi \) to treat lateral torsional buckling problem [22, 26]. If the member is loaded with moment in one of the principal plane only, the design buckling resistance moment (nominal moment capacity for LTB) should be calculated as follow:

\[
M_{b,RD} = \chi_{LT} W_y f_y \gamma_c
\]

When \( \chi_{LT} \geq 0.85 \), the section is in the elasto-plastic stage, as the result, the Young modulus declines and the buckling factor has to be modified. The modification of buckling factor is specified in SP code and this is done by finding the coefficients \( \alpha \) (section SP16 G.4) and \( \beta \) (SP16 Tables G.1 and G.2). The buckling factor for For members with doubly-symmetric I-sections is calculated as follows:

\[
\phi_1 = \frac{I_y}{I_y} \left( \frac{h}{L} \right)^2 \frac{E}{f_y}
\]

\[
\chi_{LT} = \begin{cases} 
\phi_1, & \text{if } \phi_1 \leq 0.85 \\
0.68 + 0.21\phi_1, & \text{if } \phi_1 > 0.85
\end{cases}
\]

where \( \phi_1 \) is defined in the SP code (section SP 16 G1).

In addition to the buckling factor, SP also provides equations for stable length limits of the beam (SP 16.13330.2017, Chapter 8.4.4, Table 11 [4]). For a beam with unsupported length, less than the stable length, the buckling factor’s calculation gives the result \( \chi_{LT} = 1 \). For beams with uniform moment distribution, the stable length is calculated as follows:

\[
\frac{l_{ef}}{b_t} < 0.35 + 0.0032 \frac{b}{t} + \left( 0.76 - 0.02 \frac{b}{h} \right) \frac{R_y}{E} \frac{b}{h}, \quad \text{but } \left\{ \begin{array}{l}
1 \leq \frac{b}{h} \leq 6 \\
15 \leq b/t \leq 35
\end{array} \right.
\]

where \( b \) and \( t \) are width and thickness of the compression flange, \( h \), distance (height) between the axes of the flanges.

If the limit slenderness is more than the limit value, it is necessary to provide the intermediate stiffeners for reduction of the effective length \( l_{ef} \) [8]. The nominal moment capacity is suddenly drops from plastic moment capacity for compact section and it is limited to a small lateral bracing length.

### 3 Result and discussion

A single column strength curve is given in the AISC also five separate curves are presented in the EC3 and three curves for SP. In general, all specifications use a non-dimensional slenderness for flexural buckling to define the reduction factor. For comparison, the combined graph of strength curves for Compression member is presented in figure 1.
Accordingly, all strength curves (b, c, d) for EC3 give lower capacity values as compared with AISC curve. Only curve (a) for EC3 gives higher capacities than AISC curve but the use of this curve is quite limited. In that case of SP curves (a, b, c) are not similar with AISC and EC3 somewhat different, because they have higher capacity nevertheless they are not heavy in weight, and economical approach.

Curves of nominal flexural strength of W12x30 of the steel beam sections according to AISC, EC3 and SP methods show the moment capacity of the steel sections across a wide range of lateral bracing length (L_b) and they are shown in the figures 2-4.

**Figure 1.** Combined strength curve using AISC, EC and SP.

**Figure 2.** $\Phi M_n Vs L_b$ for W12X30 using AISC.

**Figure 3.** $\Phi M_n Vs L_b$ for W12X30 using EC3.
A combined curve of the nominal flexural strength of the W12x30 sections using all specifications shows the comparison of capacity across a wide range of lateral bracing distances ($L_b$) and they are shown in the figure 5.

Additionally, the same section (W12x30) is considered to compare lateral torsional buckling capacity of the members for different buckling lengths which ranges from 0 m to 20 m and normalized moment capacities curves are formulated for two different yield strengths. The normalized moment capacity for W12x30 is given in figures 6.

4 Conclusion
According to tensile capacity, all specifications are similar but there difference is the method used to calculate the shear lag factor. For compression members, AISC and EC3 are almost similar however only $a_o$ of EC3 curve gives higher capacities than AISC curve and curves (b, c, d) give lower capacity. SP curves (a, b, c) are not similar with AISC and EC3 rather different, because they have higher capacity and economical approach on the other hand SP needs a skilled engineer to follow the approach properly. For flexural members, all specifications can reach to its plastic moment capacity if the members are compact. Treatment of non-compact flanges is similar to the treatment on non-compact webs.
in all specifications. For slender members, the AISC specification utilizes the elastic critical buckling moment approach. In EC and SP, the post buckling reserve strength approach is utilized. According to SP, the nominal moment capacity is suddenly drops from plastic moment capacity for non-compact section and as a result, it is limited to a small lateral bracing length. Accordingly, AISC gives higher capacity in inelastic region and EC3 gives higher capacities for elastic region. SP approach is lower for flexural capacity therefore it will be uneconomical approach comparing with the other two approaches.

References

[1] American Institute of Steel Construction 2011 Steel Construction Manual 13th Edition p 2245
[2] EN 1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General Rules and Rules for Buildings CEN Brussels Belgium 2005
[3] Brown D G et al. Handbook of Structural Steelwork (55) P 440
[4] Design Code SP 16.13330.2017 Steel Structures (Moscow: Standartinform)
[5] Bernuzzi C and Cordova B 2016 Structural Steel Design to Eurocode 3 and AISC Specifications (New Jersey: John Wiley & Sons) p 536
[6] Yong D J, López A and Serna M A 2006 Beam-Column Resistance of Steel Members: A Comparative Study of AISC-LRFD and EC3 Approaches Int. J. Struct. Stab. Dyn. 11(2) pp 345–361
[7] Loorits K and Talvik I 1999 A comparative study of the design basis of the Russian Steel Structures Code and Eurocode 3 J. Constr. Steel Res. 49(2) pp 157–166
[8] Loorits K and Talvik I 2006 Comparative study of the buckling of steel beams in Eurocode 3 and the Russian code J. Constr. Steel Res. 62(12) pp 1290–94.
[9] Lebed E V 2018 Behavior of the frames of large-span metal domes in the process of their installation Structural Mechanics of Engineering Constructions and Buildings 14(6) pp 481–494
[10] Desai P R and Satish C 2018 Effect of Slenderness Ratio on Euler Critical Load for Elastic Columns with ANSYS, Int. J. Eng. Res. Appl. 8(5-I11) pp 40–43
[11] Galishnikova V V and Gebre T H 2018 Al-sabri S A M Saffia-Doe O Second order structural theory for the stability analysis of columns Structural Mechanics of Engineering Constructions and Buildings 14(3) pp 192–197
[12] Topkaya C and Şahin S 2011 A comparative study of AISC-360 and EC3 strength limit states, Int. J. Steel Struct. 11(1) pp. 13–27.
[13] Taras A and Greiner R 2010 New design curves for lateral-torsional buckling-Proposal based on a consistent derivation J. Constr. Steel Res. 66(5) pp 648–663.
[14] Trahair N S 2018. Trends in the code design of steel framed structures Adv. Steel Constr.14 pp 37–56
[15] Carbonell-Márquez J F, Gil-Martín L M and Hernández-Montes E 2013 Strength design optimization of structural steel members Eurocode 3 J. Constr. Steel Res. 80 pp 213–223
[16] Chen Z and Li J Sun L 2018 Calculation of Critical Lateral-Torsional Buckling Loads of Beams Subjected to Arbitrarily Transverse Loads Int. J. Struct. Stab. Dyn. 19 pp 1950-031.
[17] Trahair N S 2009 Buckling analysis design of steel frames J. Constr. Steel Res. 65(7) pp 1459–63
[18] Trahair N S and Hancock G J 2003 Steel Member Strength by Inelastic Lateral Buckling J. Struct. Eng. 130(1) pp 64–69.
[19] Heidari A and Galishnikova V V 2014 Direct Elastic-Plastic Limit Load and Shakedown Analysis of Steel Space Trusses With Large Displacements Structural Mechanics of Engineering Constructions and Buildings 3 pp 51–64
[20] Galishnikova V V and Pahl P J 2018 Analysis of frame buckling without sidesway classification Structural Mechanics of Engineering Constructions and Buildings 14(4) pp 299-312
[21] Park J S and Kang Y J 2008 Flexural-torsional buckling of stepped beams subjected to pure
bending *KSCE J. Civ. Eng.* 8(1) pp 75–82

[22] Gebre T H 2019 The development of chart based method for steel beam designs using the Russian sections *Structural Mechanics of Engineering Constructions and Buildings* 14(1) pp 495–501

[23] Subramanian L and White D W 2016 Reassessment of the Lateral Torsional Buckling Resistance of I-Section Members: Uniform-Moment Studies *J. Struct. Eng.* 14(3) Paper No. 04016194

[24] White D W and Duk Kim Y 2008 Unified Flexural Resistance Equations for Stability Design of Steel I-Section Members: Moment Gradient Tests *J. Struct. Eng.* 134(9) pp 1471–1486

[25] Galishnikova V V and Gebre T H 2019 A comparative study of beam design curves against lateral torsional buckling using AISC, EC and SP *Structural Mechanics of Engineering Constructions and Buildings* 15(1) pp 25–32

[26] Gebre T H and Negash N A 2018 The development of strength curve for compressive members using three different codes: AISC, Euro Code 3 and Russian steel construction *Proc. Engineering Systems 2018* (Moscow, Russia) pp 59–67

[27] Trahair N S, Bradford M A, Nethercot D A and Gardner L 2008 *The Behaviour and Design of Steel Structures to EC3* (Florida: CRC Press) p 513

[28] Ivan B and Jindřich M 2017 Lateral-torsional buckling of beams of mono-symmetrical cross sections loaded perpendicularly to the axis of symmetry *Theor. Anal. Euro St.* 2–3 pp 1086–1095