

COMPRESSION AND BENDING RESISTANCE OF STEEL MEMBERS: A COMPARATIVE STUDY ON EN 1993-1-1 AND AISC 360-22 PROVISIONS

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Abstract: This paper presents a comparative study of provisions in two widely used design standards for structural steel members: the European standard EN 1993-1-1 (Eurocode 3) [9] and the American standard AISC 360-22[10]. The research concentrates on the verification of steel members subjected to combined axial compression and bending -frequently encountered in buildings, bridges, and industrial structures. In the beginning, this paper briefly compares the relations in both standards to identify key similarities and differences in terms of verification formulae, safety factors, and underlying design assumptions. A focus is placed on the influence of member slenderness, as the study considers columns across a range of slenderness ratios to assess how each standard responds to varying levels of slenderness. Furthermore, the main body of the research investigates in detail the resistance predictions of both standards through a structured parametric analysis. The study considers three representative cases of moment diagrams along the bar: uniform bending, linearly varying moment, and opposite end moments, each examined across a range of member slenderness ratios. Results are presented graphically in a structured and sequential manner, to facilitate interpretation and to highlight cases where the two standards yield possible significantly different resistance capacities. The comparative study shows that differences between EN 1993-1-1[9] and AISC 360-22[10] can be noticed in the member and the combination of applied loads.

Keywords: beam-columns, interaction formulae, member slenderness, design standards, parametric analysis

1. Introduction

A steel beam-column is a structural member designed to resist compression force and bending moment. The most frequent failure mode is by buckling (flexural, torsional or flexural-torsional), but there are also situations when resistance failure of the cross-section can be noticed. The nature of buckling failure depends on member slenderness, cross-sectional geometry, flexural and torsional rigidity, and boundary conditions, all of which influence the interaction between axial compression and bending response. As a result, the design of beam-columns is governed by stability considerations rather than cross-sectional strength. [1,2,3]

In steel design practice, beam-column resistance is determined through codified rules that account for member stability, cross-section classification, and the interaction between compression and bending [4]. EN 1993-1-1[9] and AISC 360-22[10] adopt different formulations for these checks, which can lead to different predicted resistances even for the same member and load case. Such interaction ($N - M_y - M_z$) relations are completely demonstrated at the limits (where two of the three efforts are zero) and need to be “built up” on the intervals among limits. This interaction can be managed in many ways, trying to find a good compromise between accuracy and ease of use. EN 1993-1-1[9] uses two interaction relations on the entire range of values,

while AISC 360-22[10] and other codes, like the old Romanian one, STAS 10108/0-78 [16], use a single relation having two different forms, depending on the level of compression loading. The differences between the two approaches can be pronounced for slender members with welded I-sections. [2,3,4]

The two standards differ not only in their verification formulae but also in their underlying design policy. EN 1993-1-1[9] adopts a limit state design approach based on partial safety factors (γ_{M0} and γ_{M1} for material, coupled with the ones for actions, defined in EN 1990 [17]), using χ reduction factors taken from buckling curves depending on several factors (shape of the cross-section, axis of the cross section, steel grade, etc.). AISC 360-22[10] supports two design approaches: Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD), with buckling characterized by a slenderness-dependent critical stress formulation.

A full critical analysis of these differences would imply a vast program of numerical testing using high-performance finite element programs, doubled by extensive laboratory testing for proper validation, which is obviously beyond the aims of this article. This article intends to enrich the experience in this field, of steel members in compression and bending, and is addressed mainly to practitioners both in the United States and Europe, including Romania. It focuses on the $N - M_y$ interaction to better calibrate wider future $N - M_y - M_z$ analyses.

Shankar et al. [5] showed that the slenderness limits for elements under axial and flexural compression differ between Eurocode 3 [9], AISC 341-05 [11], and AISC 360-10[12], directly affecting the available resistance of steel cross-sections. Pinarbasi et al. [6] demonstrated that the buckling resistance of hot-rolled I-shaped compression members, designed according to AISC 360-16 [13] and EC3 [9], is strongly influenced by both member length and steel grade.

Section classification represents another key source of differences between the two codes. For members subjected to combined compression and bending, both the shape of the interaction curve and the treatment of second-order effects have been identified as critical design aspects. Fang et al. [1] investigated octagonal high-strength steel tubular stub columns under combined loading and found that both European and American design approaches yield conservative, yet safe, strength predictions for the member types considered. Ma et al. [7] similarly observed that, for cold-formed high-strength steel circular hollow section beam-columns, where code predictions were shown to be strongly dependent on section slenderness and the specific load combination applied. Liu et al. [8] further demonstrated that EN 1993-1-12[14] and ANSI/AISC 360-16 [13]-based predictions for irregular hexagonal hollow section stub columns under combined loading are consistently conservative, though not identical in magnitude, with improved cross-sectional resistance models leading to enhanced prediction accuracy.

Although previous studies compared EN 1993-1-1[9] with earlier versions of the AISC specification, notably Yong et al. [4], the present study employs the most recent edition, AISC 360-22[10].

2. Resistance of members subjected to combined axial compression and bending

The following section briefly outlines the interaction formulae provided by EN 1993-1-1[9] and AISC 360-22[10] for the verification of steel members subjected to combined axial compression and bending.

2.1. EN 1993-1-1[9] design formulae for combined axial compression and bending

EN 1993-1-1[9] addresses the verification of steel members subjected to combined compression and bending through a set of two interaction formulae defined in Section 6.3.3 of the standard. These formulae apply to members susceptible to instability phenomena and account for the simultaneous effects of axial compression and bending about one or both main axes of the cross-section. The standard proposes two alternative methods - given in Annex A and Annex B - to calculate the interaction factors. The reduction factors (χ_y , χ_z and χ_{LT}) represent the effects of geometrical imperfections and residual stresses about the main axes of the cross-section.

The general form of the interaction equations for members susceptible to buckling under combined axial compression and uniaxial bending is as follows:

$$\frac{N_{Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (1)$$

$$\frac{N_{Ed}}{\chi_z \cdot \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (2)$$

In these equations, N_{Ed} is the design value of the axial compressive force; χ_y and χ_z are the reduction factors for flexural buckling about the major and minor axis respectively; N_{Rk} is the characteristic value of the cross-sectional resistance to axial force; and γ_{M1} is the partial safety factor for member instability, generally taken as 1.0. Regarding the bending terms about the major axis, $M_{y,Ed}$ is the design value of the bending moment, $\Delta M_{y,Ed}$ is an additional moment accounting for the shift of the centroidal axis applicable to Class 4 cross-sections, χ_{LT} is the reduction factor for lateral-torsional buckling, and $M_{y,Rk}$ is the characteristic moment resistance about the major axis. Similarly, $M_{z,Ed}$ and $\Delta M_{z,Ed}$ are the design bending moment and the additional moment about the minor axis respectively, and $M_{z,Rk}$ is the corresponding characteristic moment resistance. The interaction factors k_{yy} and k_{zy} appear in Eq.1 (Eq.6.61 in EN 1993-1-1[9]) and Eq.2 (Eq.6.62 in EN 1993-1-1[9]) respectively and account for the combined effect of axial compression and bending about the major axis, while k_{yz} and k_{zz} account for the combined effect of axial compression and bending about the minor axis. Since the present study is limited to uniaxial bending about the major axis only, the terms involving $M_{z,Ed}$ are equal to zero, and the influence of interaction factors k_{yz} and k_{zz} was not considered, resulting in a simplified form of the interaction formulae.

2.2. AISC 360-22[10] design formulae for combined axial compression and bending

AISC 360-22[10] addresses the verification of steel members subjected to combined compression and bending through the provisions of Chapter H of the specification. The standard provides design relationships for both Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD); however, the present paper deals with the LRFD approach, in which the required strengths are compared against the available design strengths, each multiplied by the appropriate resistance factor ϕ . For members subjected to combined axial compression and bending, AISC 360-22[10] uses a set of linear interaction equations that relate the ratio of the required axial

compressive strength to the available axial compressive strength, and the ratios of the required flexural strengths to the available flexural strengths about each main axis.

The buckling check in AISC 360-22[10] uses one main equation that changes depending on the axial load ratio. When the required axial strength ratio is 0.2 or higher, the full interaction equation applies. If the ratio is less than 0.2, a reduced interaction equation is used, which puts more emphasis on the bending terms. Compressive strength is found using Chapter E of the specification, considering flexural, torsional, and flexural-torsional buckling with column curves that depend on slenderness. The available flexural strength is determined using Chapter F, which considers lateral-torsional buckling, local flange buckling, and local web buckling based on the cross-sectional classification. Slenderness effects are included either by the effective length method or by direct analysis, which accounts considers geometric imperfections and reduced stiffness in the structural analysis.

AISC 360-22[10] addresses the interaction between axial compression and bending through a set of linear interaction equations defined in Chapter H of the specification. For members subjected to combined axial compression and bending, the standard prescribes two interaction equations depending on the magnitude of the axial load ratio. When the ratio of the required compressive strength to the available compressive strength is equal to or greater than 0.2, the following interaction equation applies:

$$\frac{P_r}{P_c} + \frac{8}{9} \cdot \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1 \quad (3)$$

When this ratio falls below 0.2, a second interaction equation is used, which places greater relative weight on the bending terms:

$$\frac{P_r}{2 \cdot P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1 \quad (4)$$

In both equations, P_r represents the required axial compressive strength, P_c is the available axial compressive strength, M_{rx} and M_{ry} are the required flexural strengths about the x and y axes respectively, and M_{cx} and M_{cy} are the corresponding available flexural strengths. The available compressive strength P_c is determined in accordance with Chapter E of the specification, and is defined as:

$$P_c = \phi_c \cdot P_n = \phi_c \cdot F_{cr} \cdot A \quad (5)$$

where $\phi_c = 0.9$ is the resistance factor for compression, and P_n is the nominal compressive strength, computed as a function of the critical buckling stress F_{cr} and the gross cross-sectional area A . The available flexural strength M_c is determined in accordance with Chapter F, and is defined as:

$$M_c = \phi_c \cdot M_n = \phi_c \cdot Z_x \cdot F_y \quad (6)$$

where $\phi_c = 0.9$ is the resistance factor for flexure, and M_n is the nominal flexural strength, which accounts for lateral-torsional buckling, local flange buckling, and local web buckling depending on the compactness classification of the cross-section.

3. Numerical analyses

This paper compares results obtained with the design formulae proposed by EN 1993-1-1[9] and AISC 360-22[10] for the verification of structural steel members subjected to combined axial compression and bending. For the sake of simplicity, Class 4 cross-sections as defined by EN 1993-1-1[9] were excluded from this study, for which

additional provisions, given in EN 1993-1-5[15], for considering local buckling effects would be required. Furthermore, the study is limited to uniaxial bending, considering bending about the major axis only.

To compare the two standards, we construct interaction curves that show the relationship between bending moment and axial force, using the verification formulae from each standard. These diagrams illustrate how each standard defines the resistance range of the member as a function of the combined loading demands. The points on the interaction curves were determined by incrementally varying the axial force N_{Ed} from zero to the maximum compressive capacity of the section, using a constant step size to ensure consistency and comparability of the results.

The material used in the numerical analyses is structural steel S235, with a yield strength of $f_y = 235 \text{ MPa}$ and an elastic modulus of $E = 210000 \text{ MPa}$, as specified by EN 1993-1-1[9]. In accordance with AISC 360-22[10], an elastic modulus of $E = 200000 \text{ MPa}$ was adopted for the American standard provisions. The cross-section considered is a doubly symmetric welded I-section (Figure 1), comprising a web with thickness $t_w = 10 \text{ mm}$ and height $h_w = 420 \text{ mm}$, and flanges with thickness $t_f = 20 \text{ mm}$ and width $b_f = 300 \text{ mm}$.

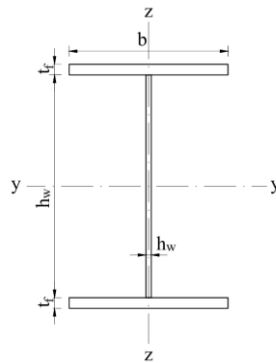


Fig. 1 - Geometry of the welded I-section

Since EN 1993-1-1[9] provides two distinct calculation methods for the interaction factors through Annexes A and B, together with a cross-section resistance check, while AISC 360-22[10] defines a single interaction relationship with two distinct forms, the results are organized in the interaction diagrams as follows:

A Curve: the governing envelope between the cross-section resistance curve and the interaction curve obtained by applying Eq. (1) and Eq. (2) using Annex A of EN 1993-1-1[9];

B Curve: the governing envelope between the cross-section resistance curve and the interaction curve obtained by applying Eq. (1) and Eq. (2) using Annex B of EN 1993-1-1[9];

Curve AISC: the interaction curve derived from the provisions of AISC 360-22[10].

Parametric analysis begins with an examination of the variations in the shape of the bending moment diagram. For this purpose, three representative bending moment diagrams were considered in the analysis:

a. Member subjected to a uniform bending moment diagram;

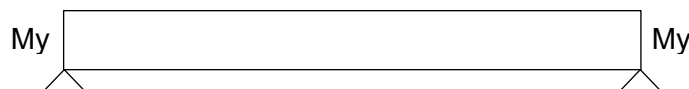


Fig. 2 - Constant bending moment diagram

b. Member subjected to a linearly varying bending moment, with zero moment at one end and maximum moment at the opposite end;

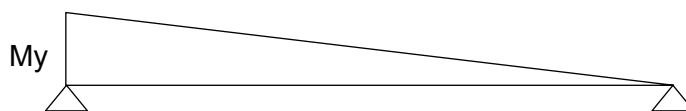


Fig. 2 - Linearly varying bending moment diagram

c. Member subjected to opposite end moments, with an equal maximum positive moment at one end and a maximum negative moment at the other end.

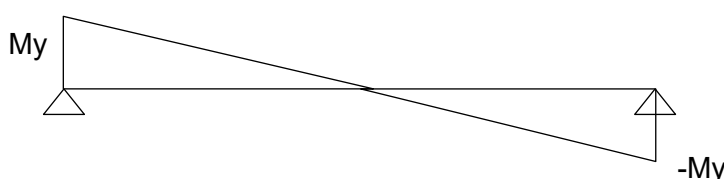


Fig. 4 - Opposite end moment diagram

In addition to the variation of the bending moment diagram shape, the slenderness ratio of the member was systematically varied for each of the three diagrams considered. This approach allowed for a thorough investigation of the interaction between effects of moment diagram and member slenderness - two parameters that are known to jointly govern the lateral-torsional buckling behaviour of structural elements.

The slenderness ratio of the member was varied by adjusting the member length from 5 m to 15 m. Figures 5 to 37 present the results corresponding to each bending moment diagram. Uniform bending moments are illustrated in Figures 5 to 15, linearly varying bending moments in Figures 16 to 26, and opposite end moment diagrams in Figures 27 to 37. The buckling lengths considered range from 5 m to 15 m, with slenderness ratios about the strong axis from 25.10 to 75.29 and about the weak axis from 67.07 to 201.21.

3.1 Case 1: Influence of slenderness ratio under constant bending moment diagram

The results indicate that, at low slenderness values ($\lambda_z \leq 80.48$), A Curve and B Curve yield higher resistances than the AISC curve, with differences in maximum axial capacity reaching approximately 35% (Fig.5-Fig.6). As the slenderness increases, a transition zone is observed around $\lambda_z = 93.90$, in which the curves intersect (Fig.7). For $\lambda_z \geq 107.31$, the AISC curve is systematically above both Eurocode curves over the entire interaction domain (Fig.8-Fig.10). At high slenderness values $\lambda_z = 147.55$, the maximum between the two standards is in the range of 20–25% (Fig.11). For slenderness values exceeding 148, the AISC curve remains above A Curve and B Curve for axial forces up to approximately 0.5 of the maximum cross-sectional axial resistance, converging to values approximately equal to those of A Curve and B Curve in the interaction zone (Fig.11-Fig.15). The difference between A Curve and B Curve

remains small and relatively constant throughout the entire range of lengths considered.

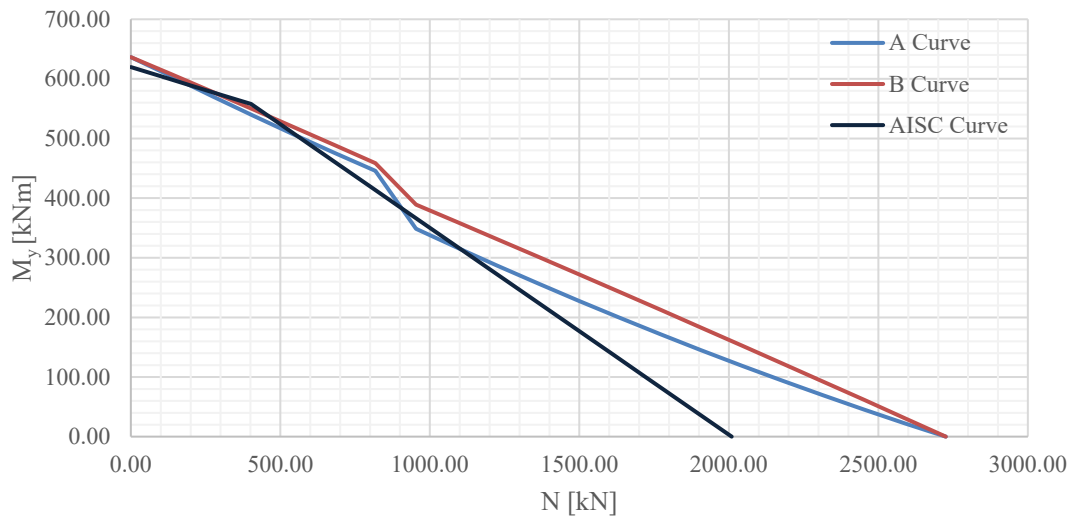


Fig. 3 - $L=5\text{m}$, $\lambda_y = 25.10$, $\lambda_z = 67.07$

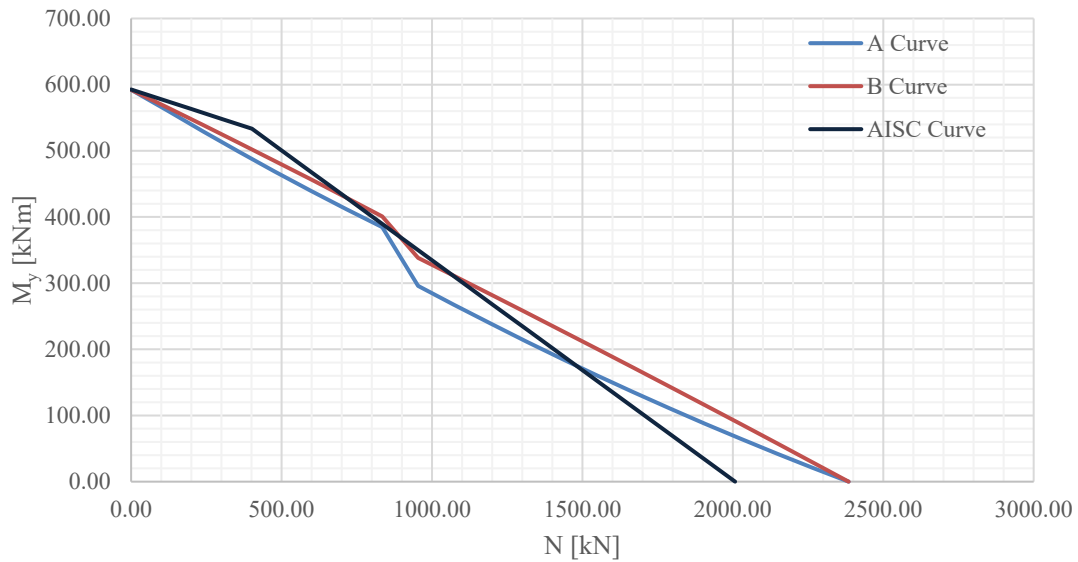


Fig. 4 - $L=6\text{m}$, $\lambda_y = 30.12$, $\lambda_z = 80.48$

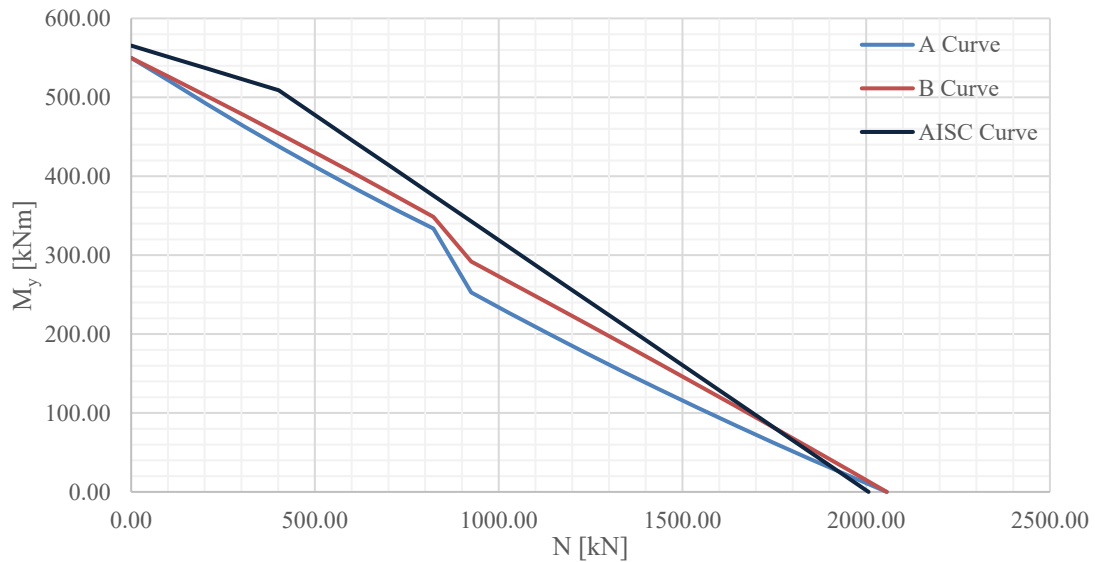


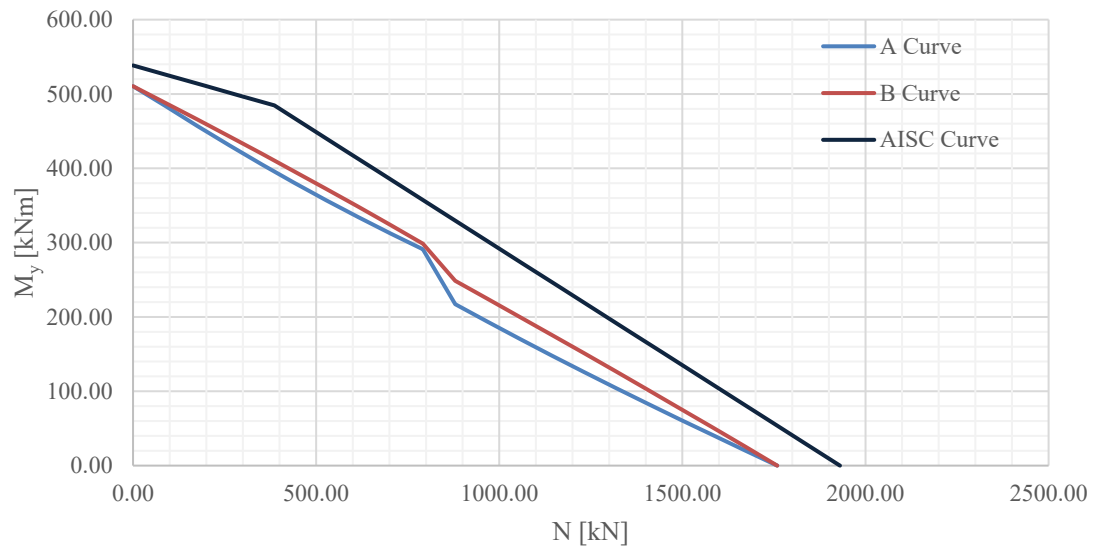
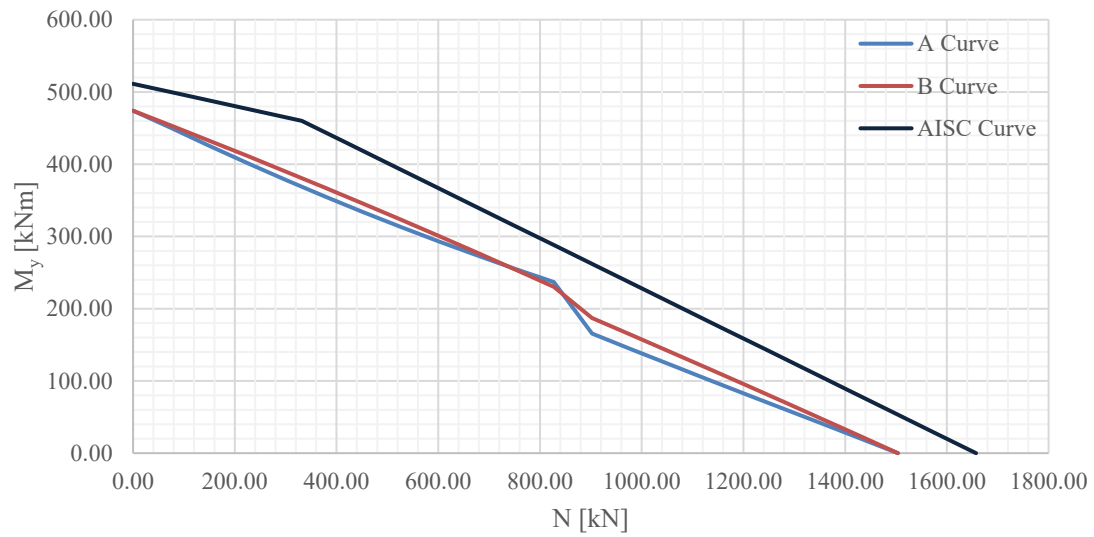
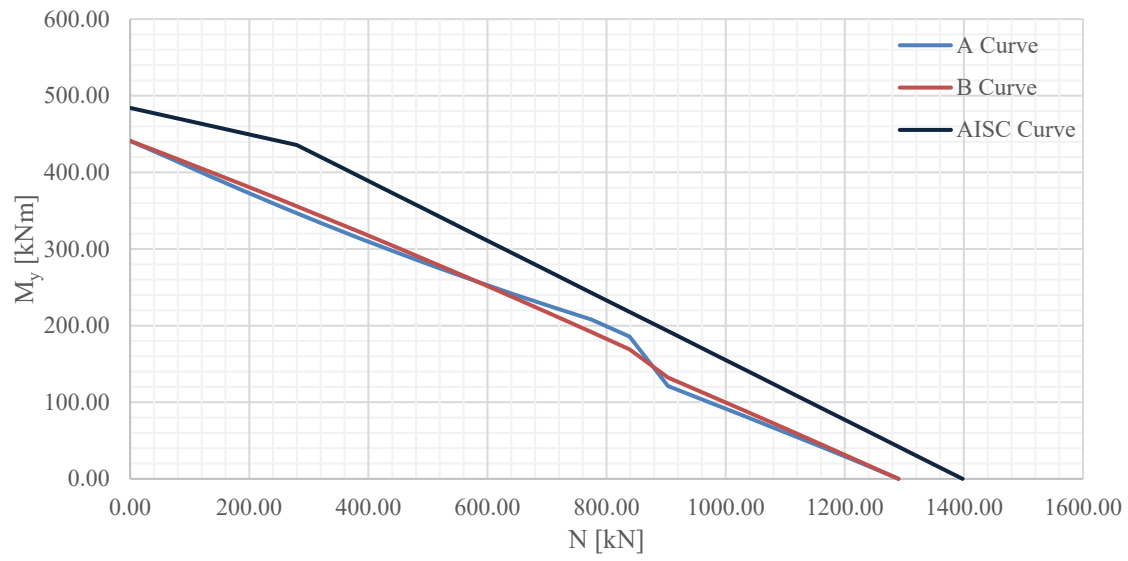
Fig. 5 - $L = 7\text{m}$, $\lambda_y = 35.14$, $\lambda_z = 93.90$ Fig. 6 - $L = 8\text{m}$, $\lambda_y = 40.16$, $\lambda_z = 107.31$ 

Fig. 7 - L= 9m, $\lambda_y = 45.18$, $\lambda_z = 120.72$ Fig.10 - L= 10m, $\lambda_y=50.20$, $\lambda_z=134.14$

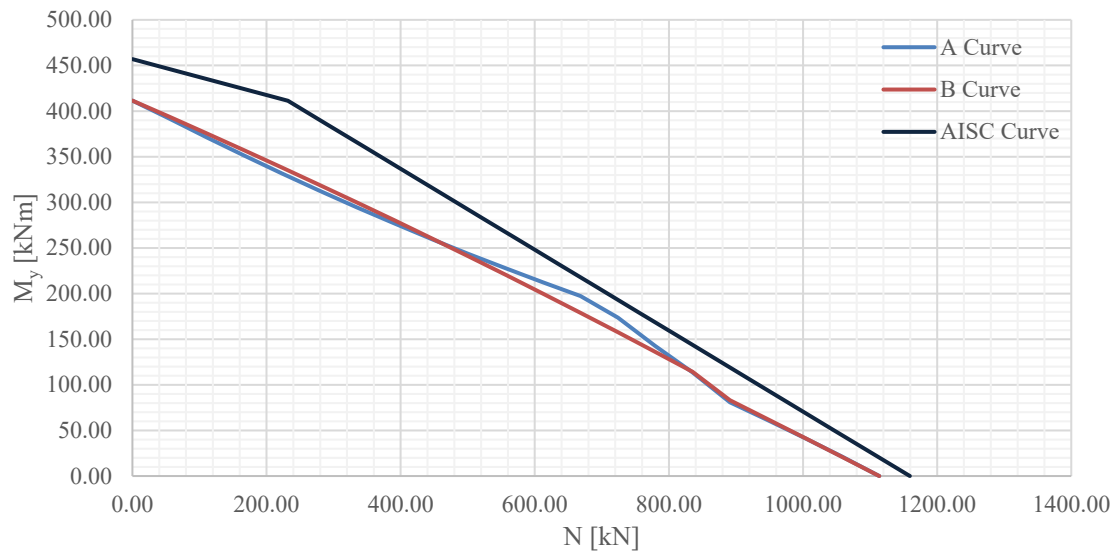


Fig.8 - $L = 11\text{m}$, $\lambda_y = 55.22$, $\lambda_z = 147.55$

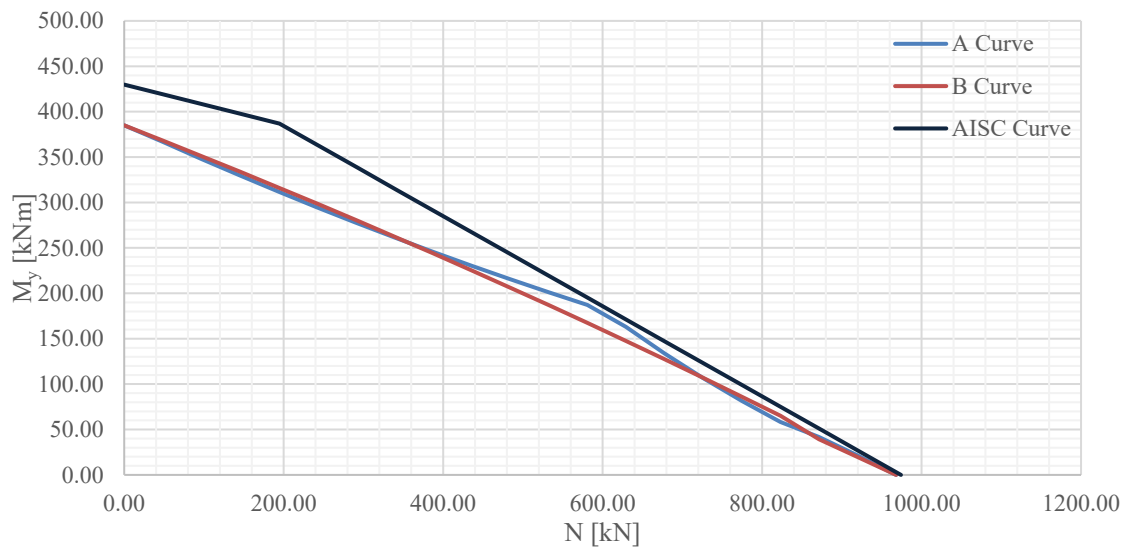


Fig.9 - $L = 12\text{m}$, $\lambda_y = 60.24$, $\lambda_z = 160.97$

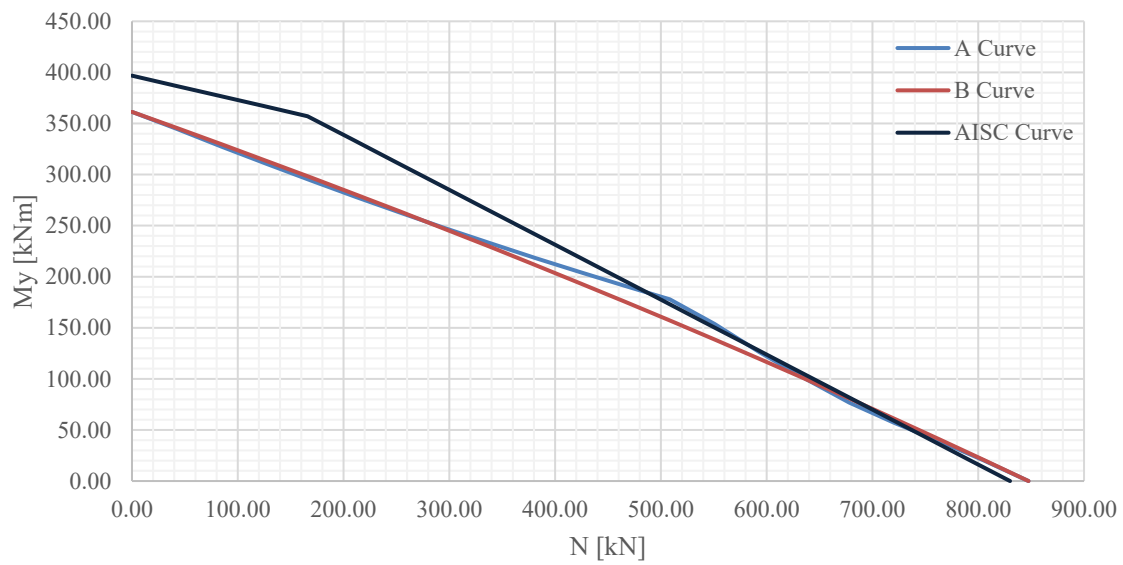


Fig.13 - $L = 13\text{m}$, $\lambda_y = 65.26$, $\lambda_z = 174.38$

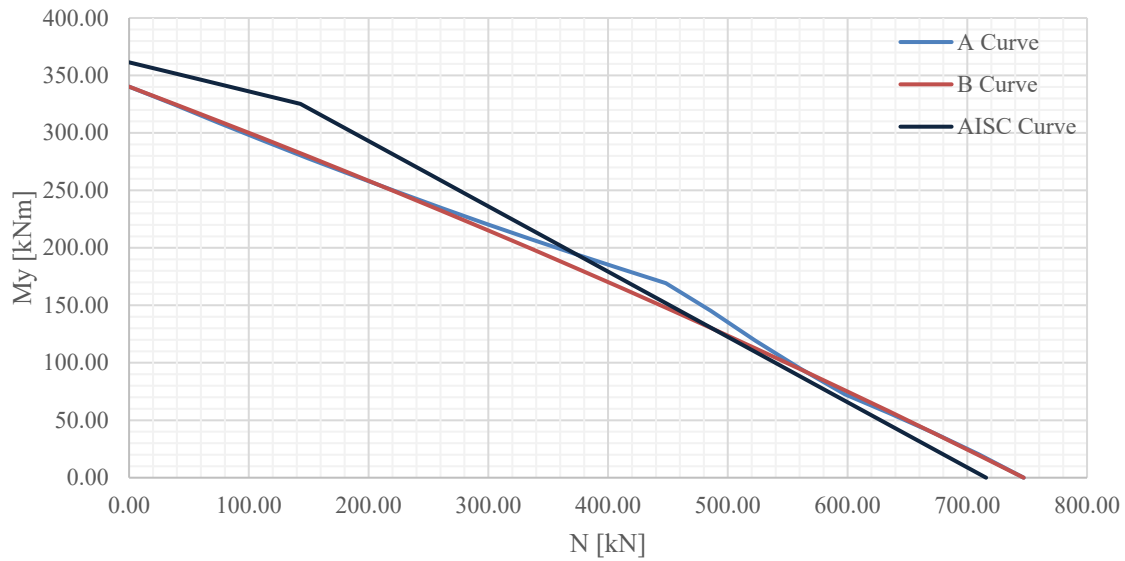


Fig.10 - L=14m, $\lambda_y = 70.27$, $\lambda_z = 187.79$

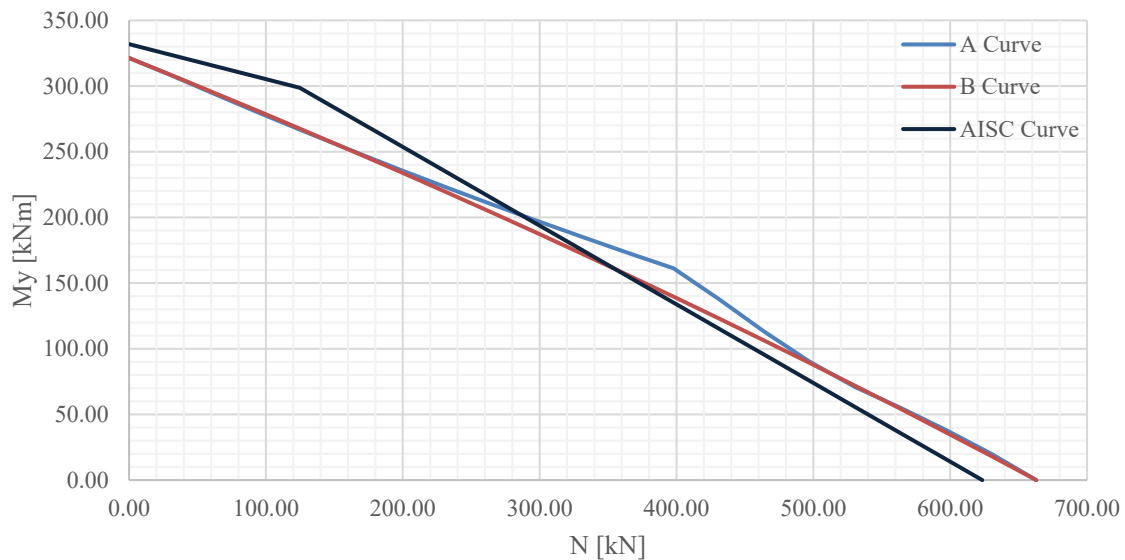


Fig.11 - L= 15m, $\lambda_y = 75.29$, $\lambda_z = 201.21$

3.2 Case 2: Influence of slenderness ratio under linearly varying bending moment diagram

Analysis of Figures 16–26 indicates that at low slenderness values ($\lambda_z = 67.07$), both A Curve and B Curve predict greater moment resistances than the AISC curve, with discrepancies at pure bending reaching approximately 7–8% (Fig.16). At pure bending, EN 1993-1-1[9] yields moment resistances of approximately 700 kNm compared to 650 kNm for AISC [10] (Fig.16). As slenderness increases, the three curves gradually converge, and a crossover point occurs near $\lambda_z = 93.90$, beyond which the AISC curve governs the entire interaction domain (Fig.18). This trend intensifies at higher slenderness values ($\lambda_z = 160.97 - 201.21$) (Fig.23-26), where the AISC curve

exceeds both European curves by about 20–25% in the pure bending region, with AISC [10] reaching $\cong 555 \text{ kNm}$ compared to $\cong 450 \text{ kNm}$ for A Curve (Fig.26).

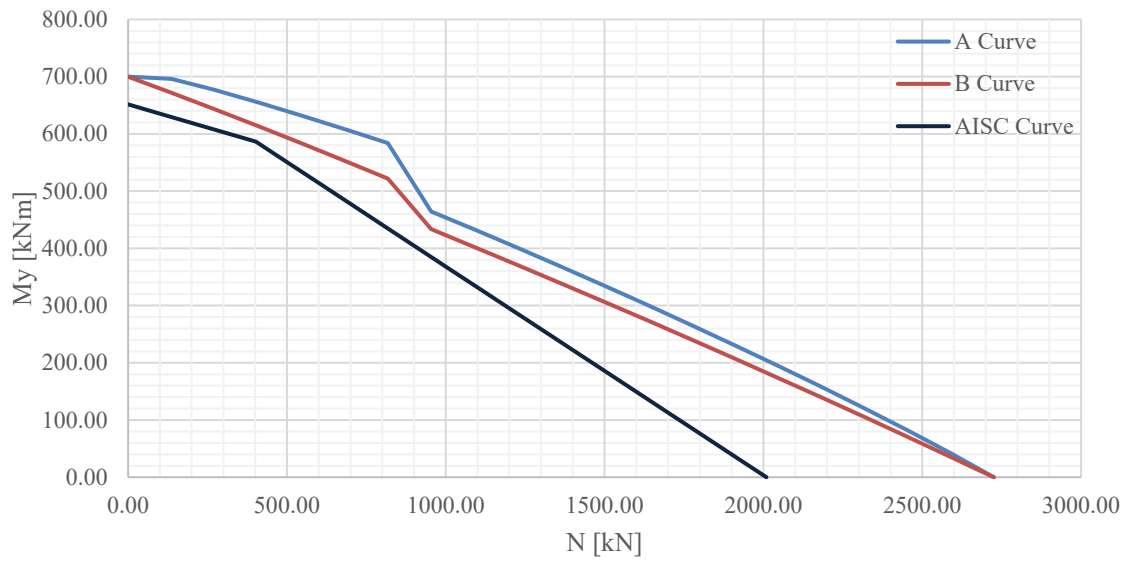


Fig.12 - $L=5\text{m}, \lambda_y = 25.10, \lambda_z = 67.07$

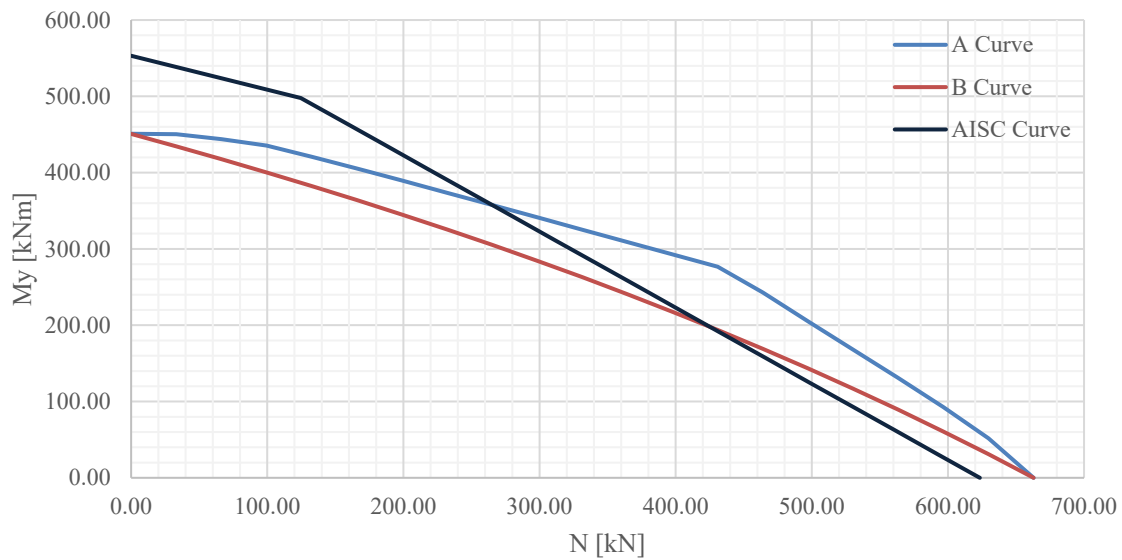
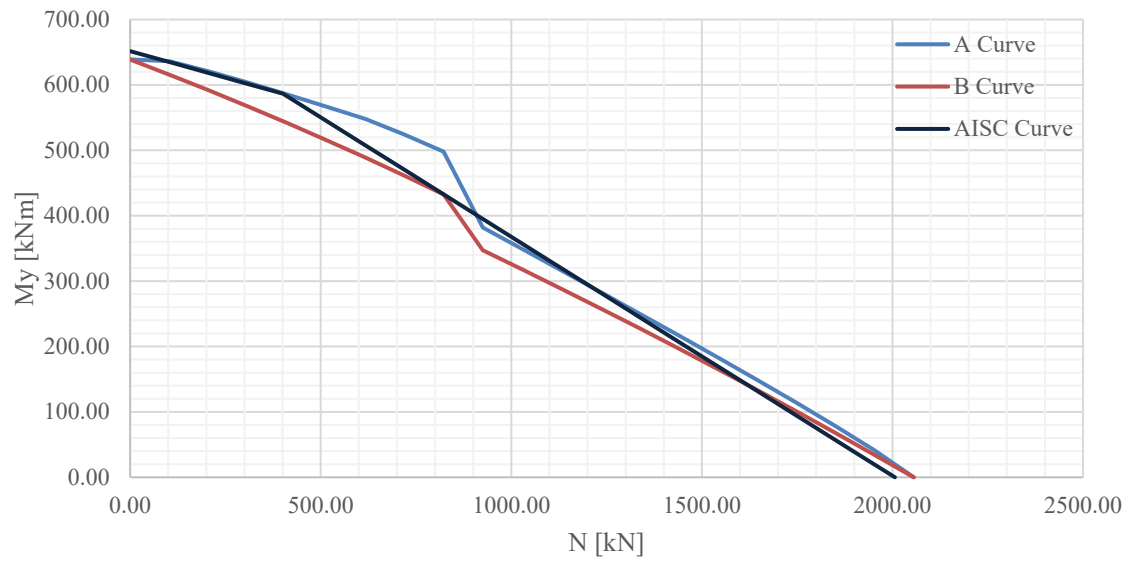


Fig.13 - L= 6m, $\lambda_y = 30.12$, $\lambda_z = 80.48$ Fig.18 - L= 7m, $\lambda_y=35.14$, $\lambda_z=93.90$

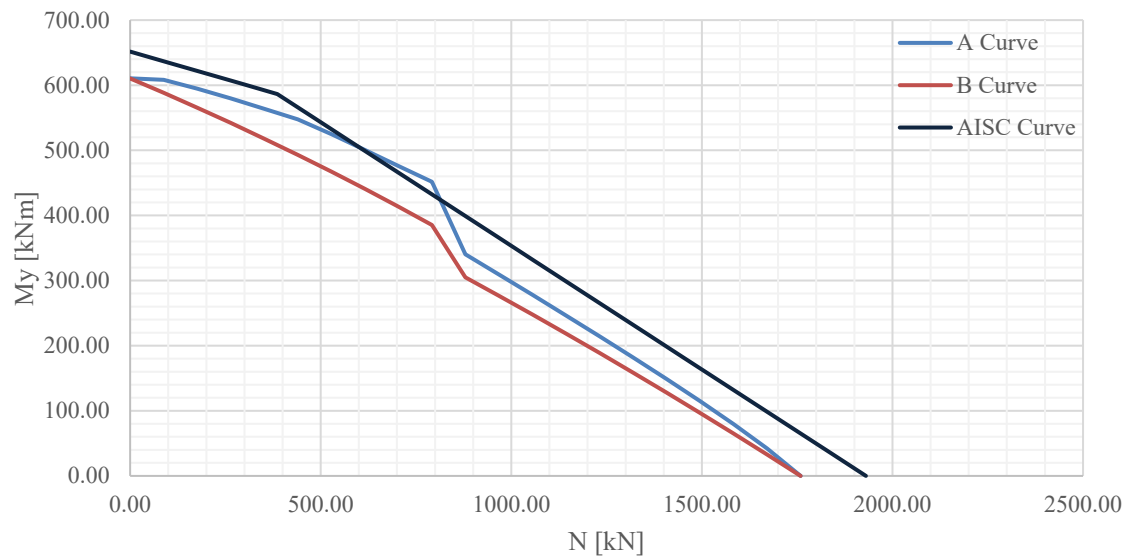


Fig.14 - L= 8m, $\lambda_y = 40.16$, $\lambda_z = 107.31$

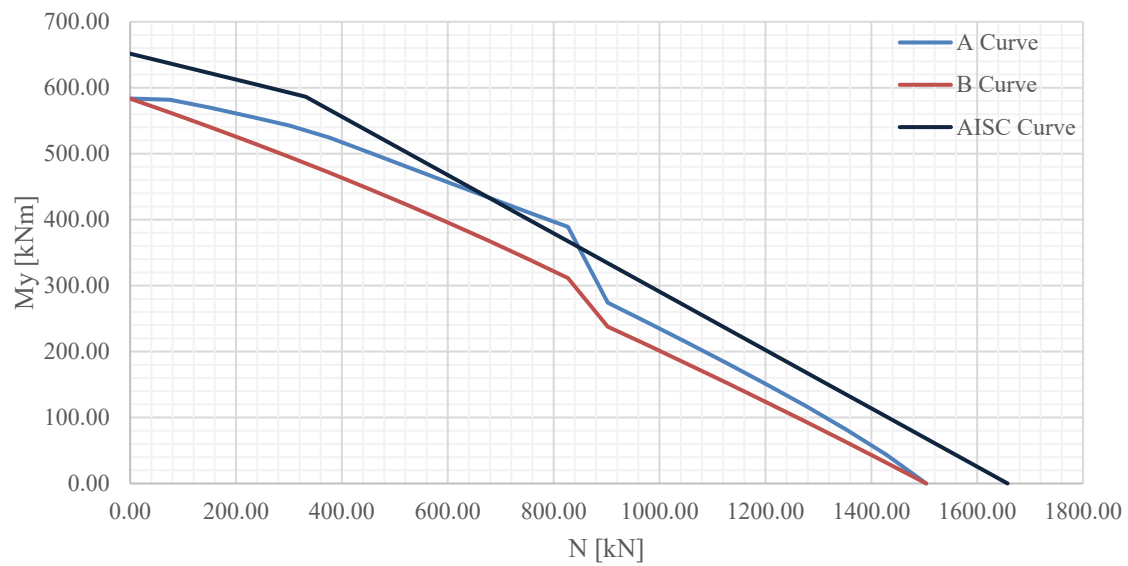


Fig.15 - L= 9m, $\lambda_y = 45.18$, $\lambda_z = 120.72$

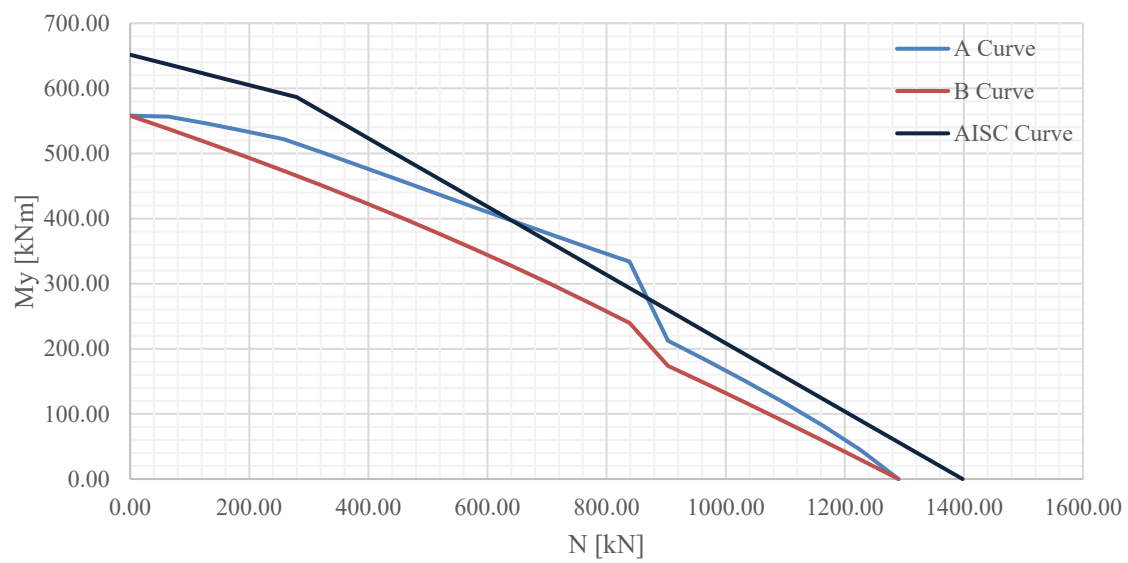


Fig.21 - $L = 10\text{m}$, $\lambda_y = 50.20$, $\lambda_z = 134.14$

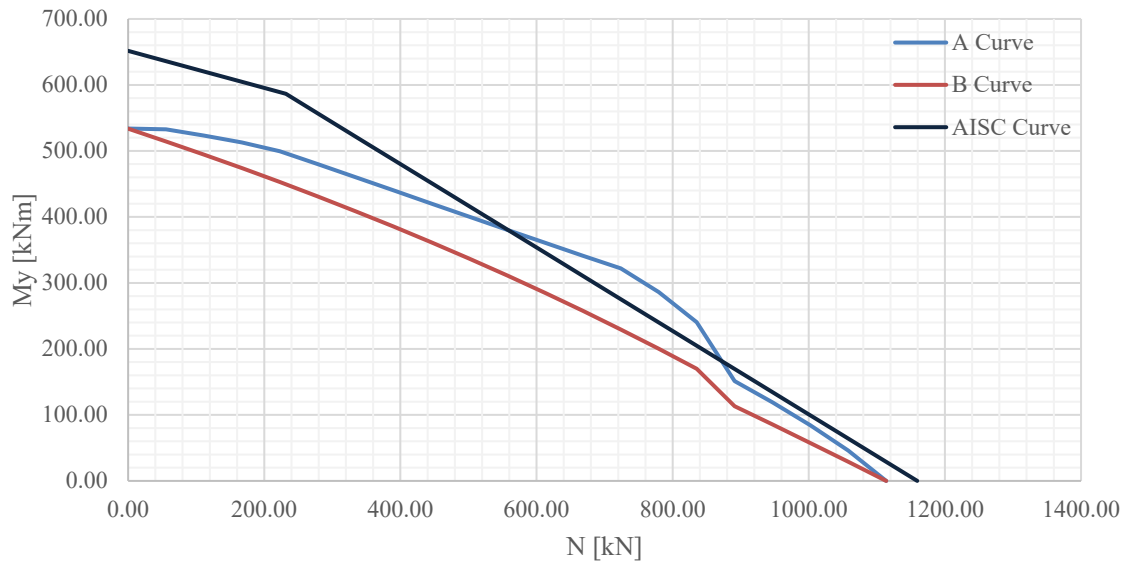


Fig.16 - L= 11m, $\lambda_y = 55.22$, $\lambda_z = 147.55$

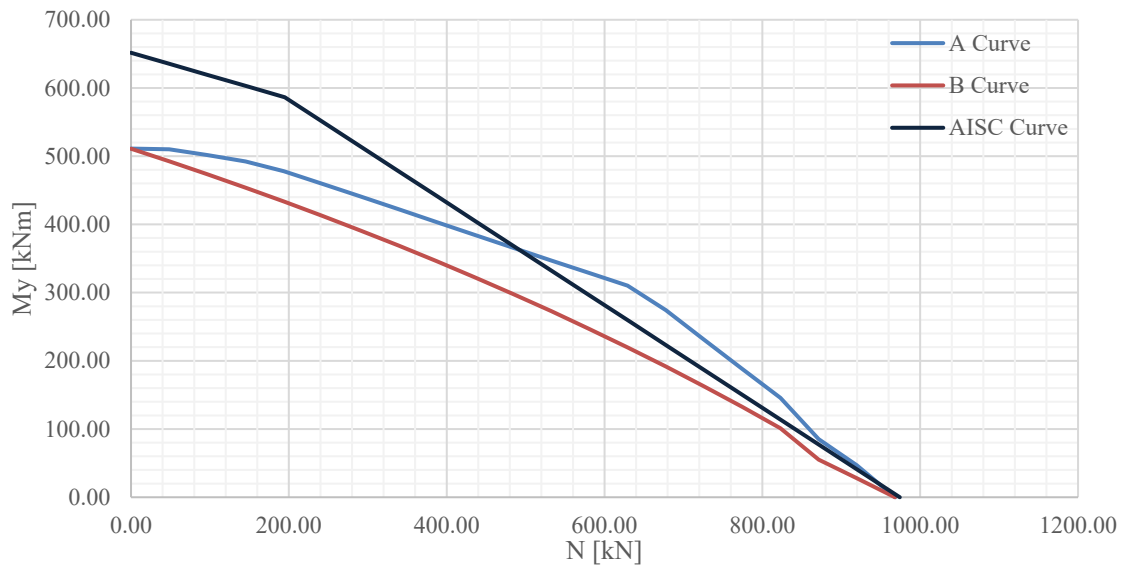


Fig.17 - L= 12m, $\lambda_y = 60.24$, $\lambda_z = 160.97$

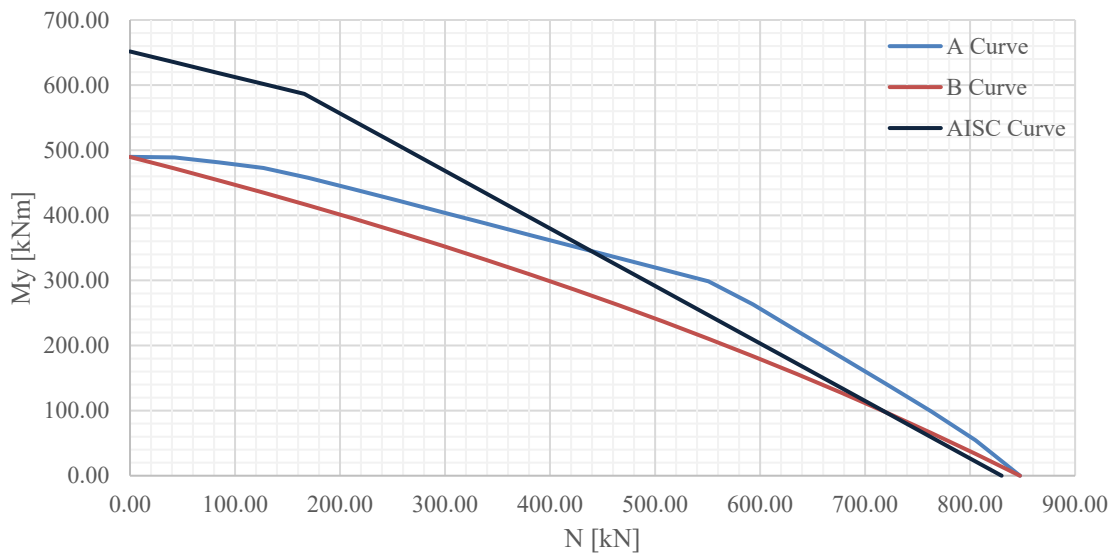


Fig.24 - L= 13m, $\lambda_y=65.26$, $\lambda_z=174.38$

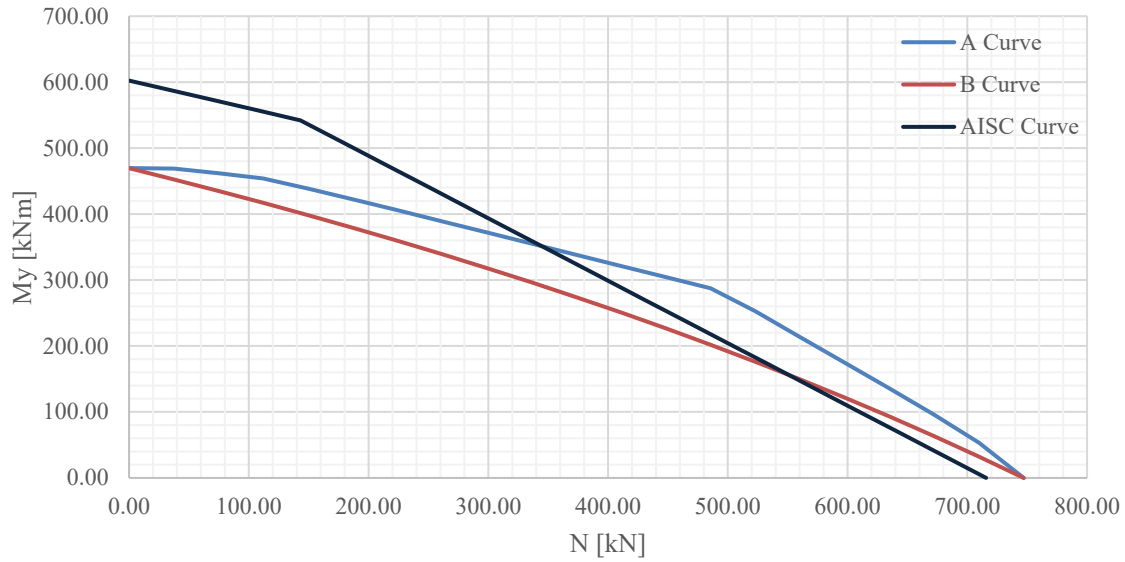


Fig.18 - L= 14m, $\lambda_y = 70.27$, $\lambda_z = 187.79$

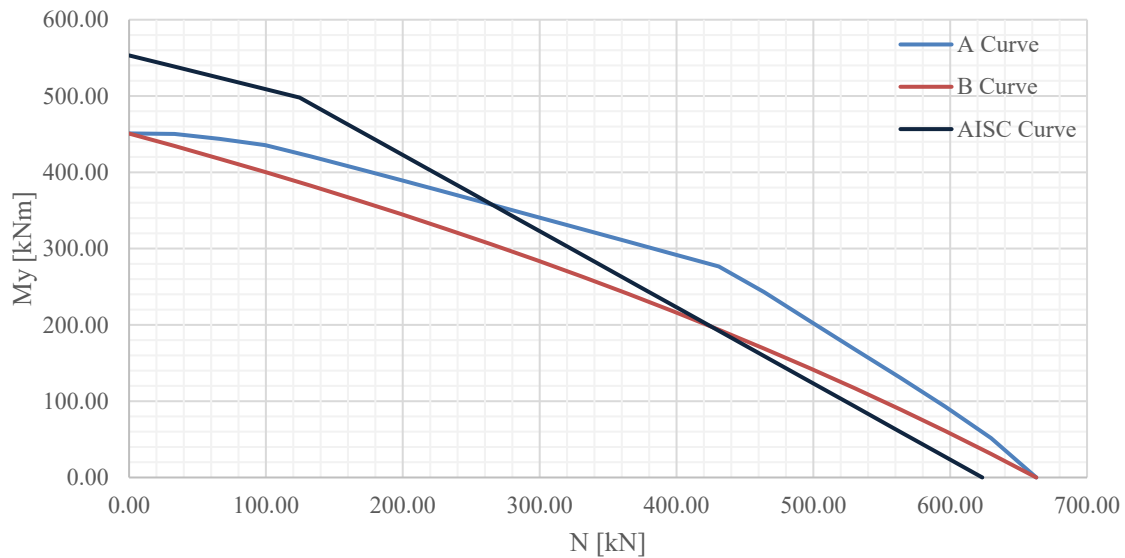


Fig.19 - L= 15m, $\lambda_y = 75.29$, $\lambda_z = 201.21$

3.3 Case 3: Influence of slenderness ratio under opposite end moment diagram

Figures 27–37 indicate that, at low slenderness values ($\lambda_z = 67.07$), the interaction curves derived from EN 1993-1-1 [9] are situated significantly above the AISC curve, with differences at pure bending of approximately 10–11% (Fig.27). At these slenderness values, Eurocode [9] generates the highest moment capacities recorded in the study ($\cong 720 \text{ kNm}$) (Fig.27). As slenderness increases, the three curves converge, with an intersection occurring near $\lambda_z = 93.90$ (Fig.29). Beyond this point, the AISC curve falls below both European curves throughout the interaction domain, and this trend intensifies with further increases in slenderness. At the highest slenderness values considered ($\lambda_z = 147.55 - 201.21$), the divergence between the standards is most pronounced in the pure bending region, where A Curve exceeds the AISC curve by about 20–25% (Fig.33). However, the curves converge again under predominantly axial loading (Fig.37).

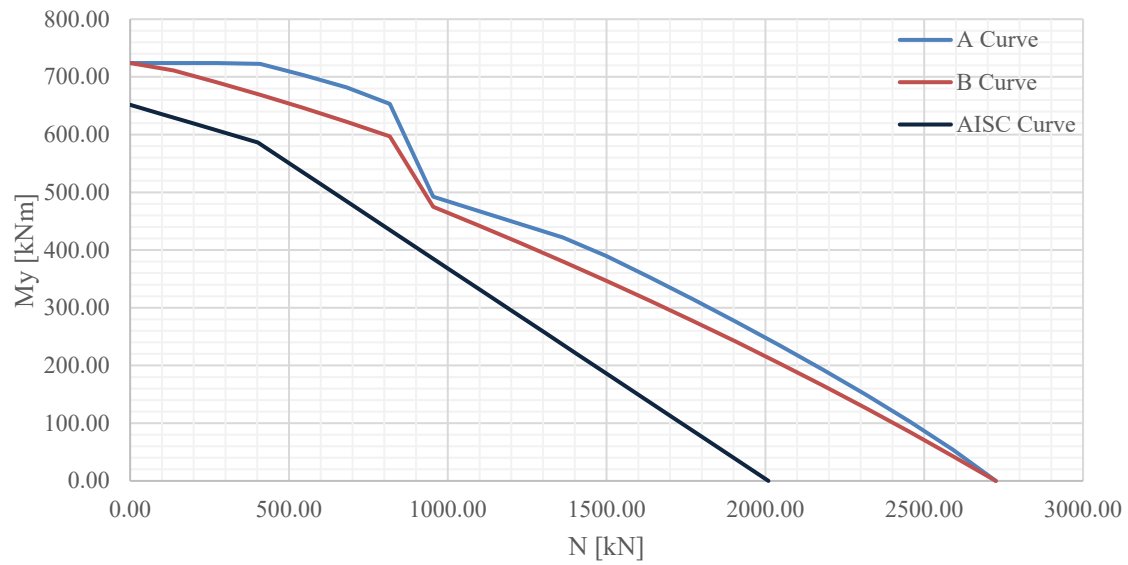


Fig.20 - $L = 5\text{m}$, $\lambda_y = 25.10$, $\lambda_z = 67.07$

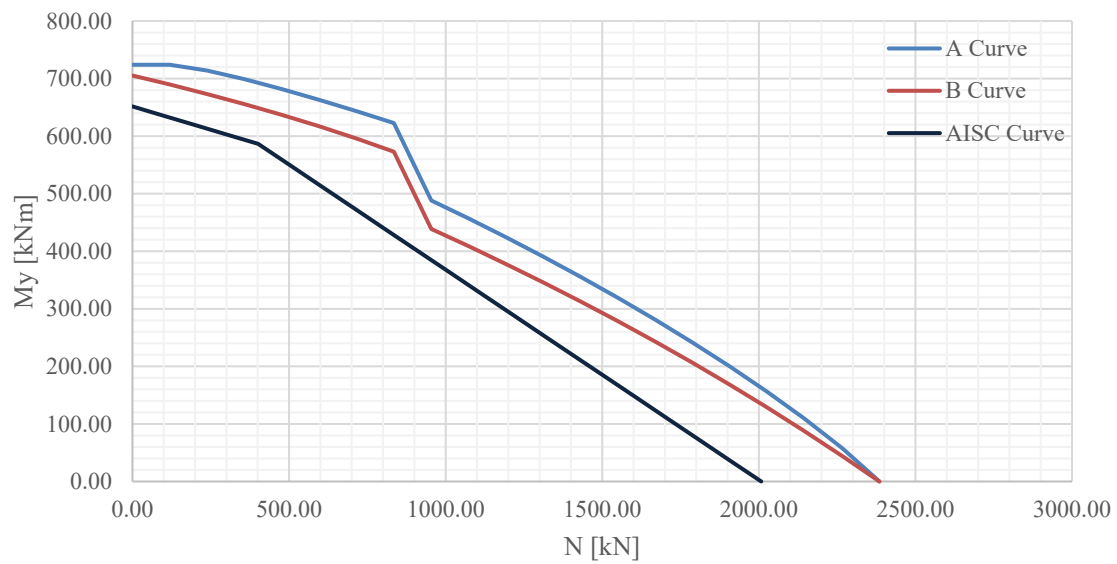


Fig.21 - $L = 6\text{m}$, $\lambda_y = 30.12$, $\lambda_z = 80.48$

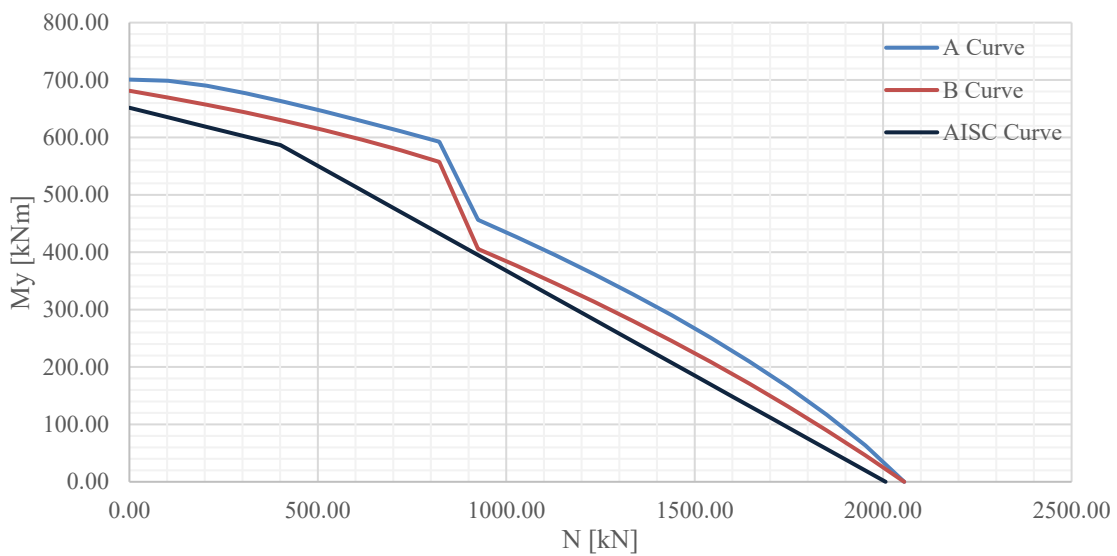


Fig.29 - $L = 7\text{m}$, $\lambda_y = 35.14$, $\lambda_z = 93.90$

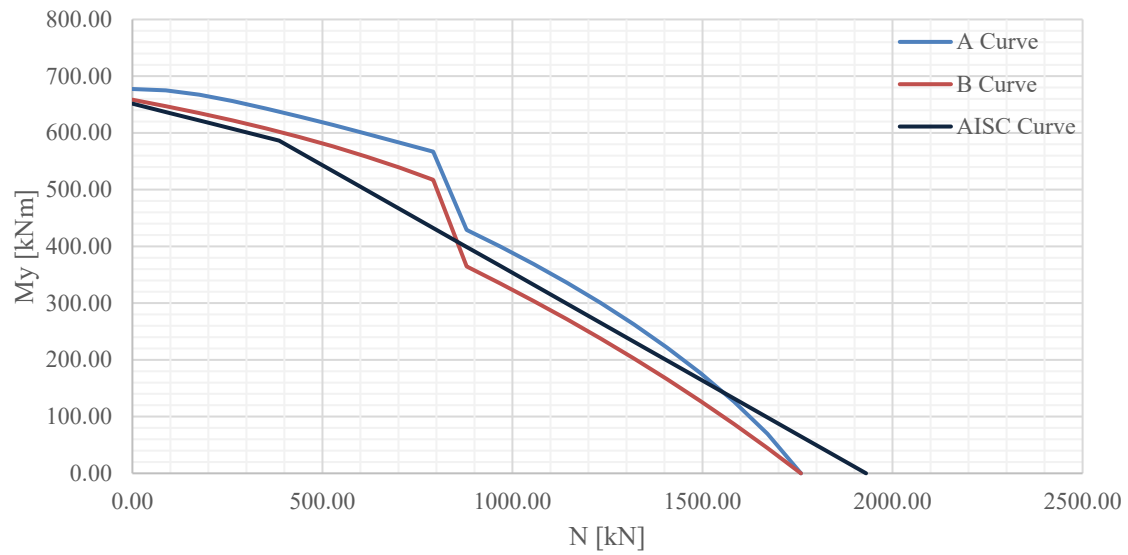


Fig.22 - $L = 8\text{m}$, $\lambda_y = 40.16$, $\lambda_z = 107.31$

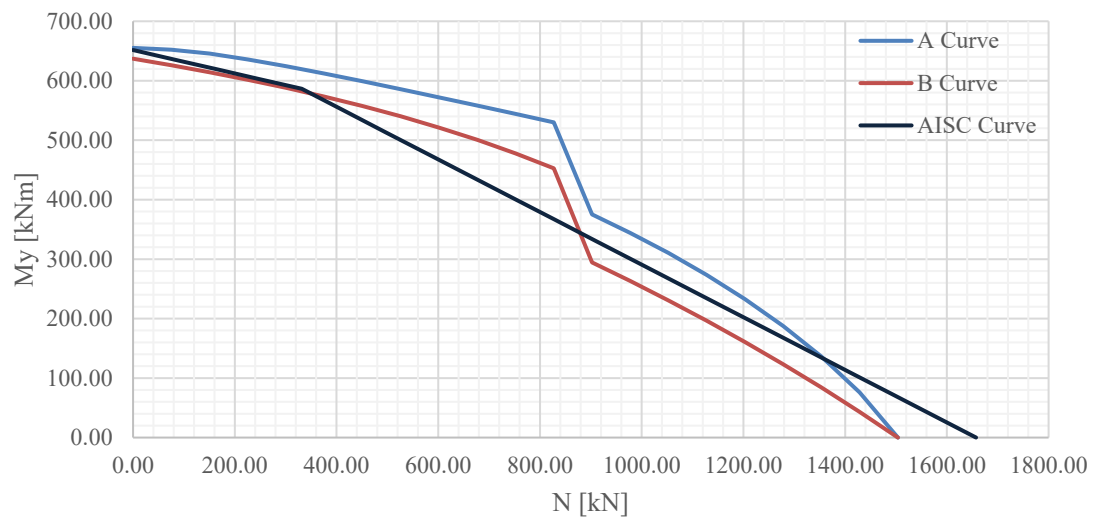


Fig.23 - $L = 9\text{m}$, $\lambda_y = 45.18$, $\lambda_z = 120.72$

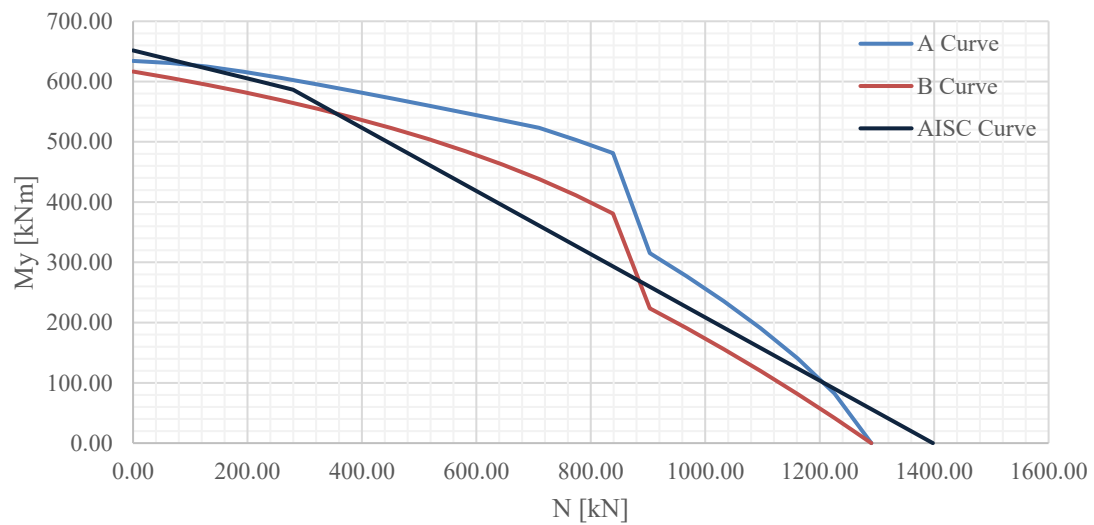


Fig. 32 - $L = 10\text{m}$, $\lambda_y = 50.20$, $\lambda_z = 134.14$

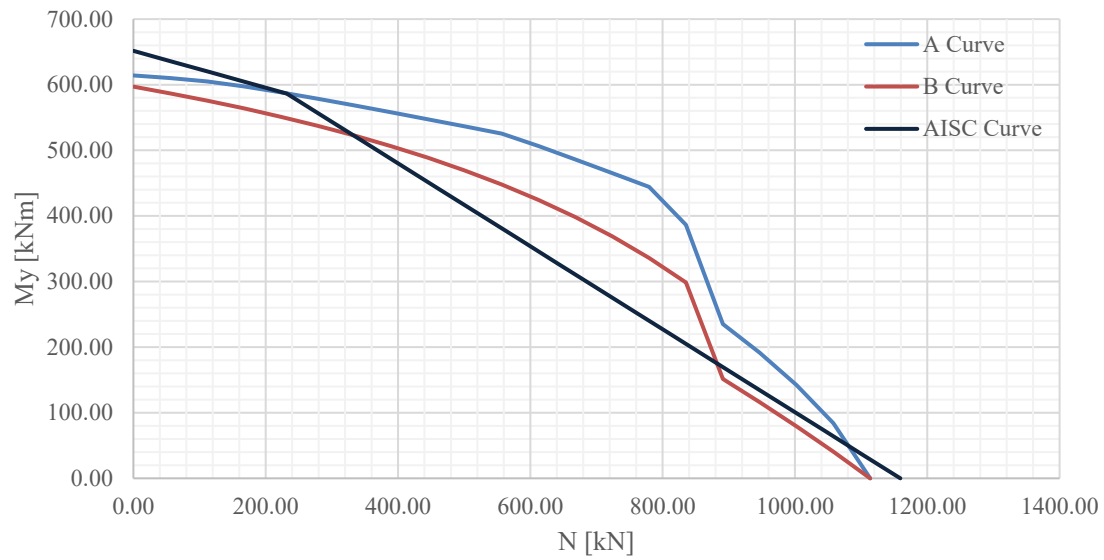


Fig. 24 - $L = 11\text{m}$, $\lambda_y = 55.22$, $\lambda_z = 147.55$

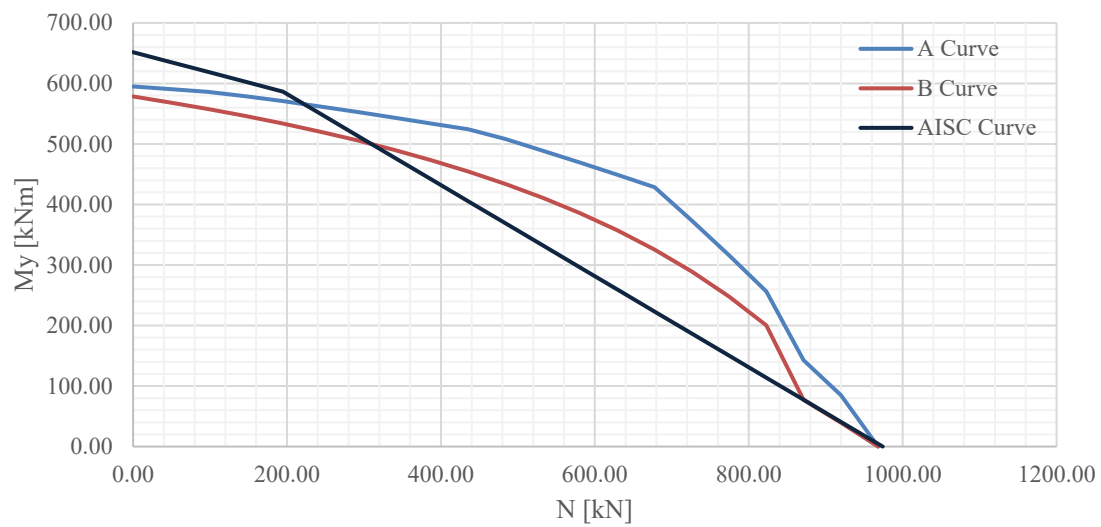


Fig. 25 - $L = 12\text{m}$, $\lambda_y = 60.24$, $\lambda_z = 160.97$

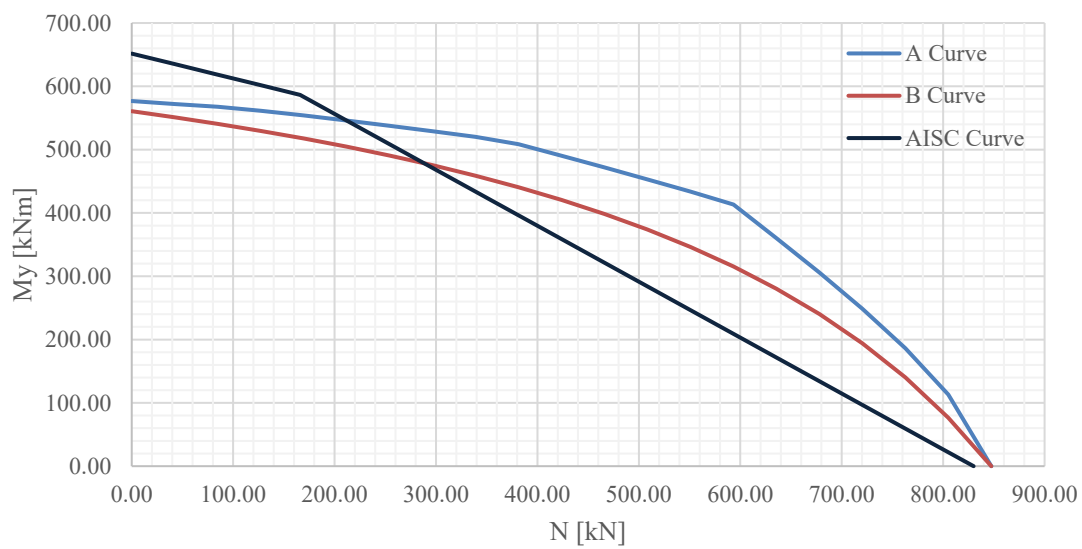


Fig. 35 - $L = 13\text{m}$, $\lambda_y = 65.26$, $\lambda_z = 174.38$

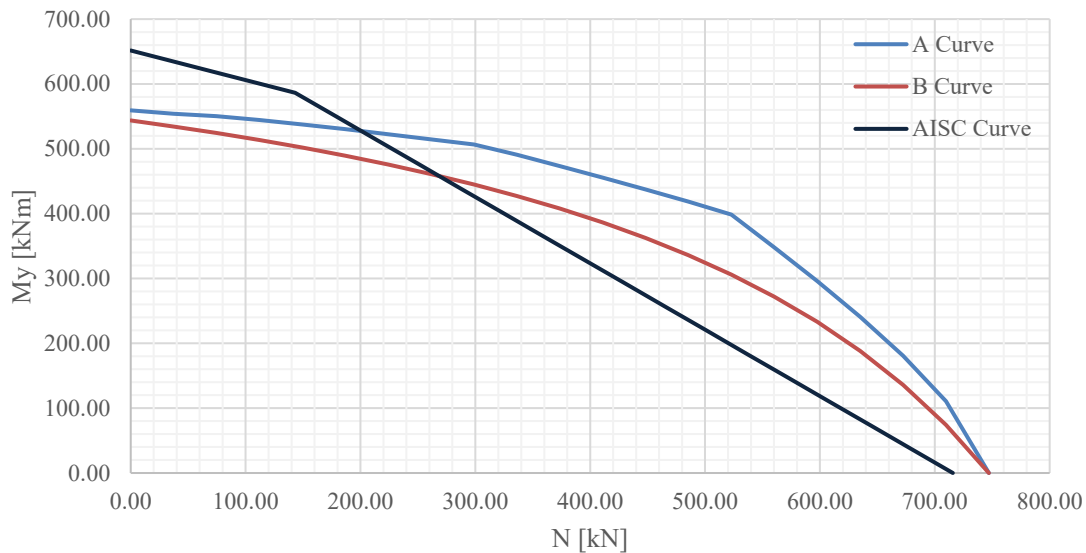


Fig. 26 - $L = 14\text{m}$, $\lambda_y = 70.27$, $\lambda_z = 187.79$

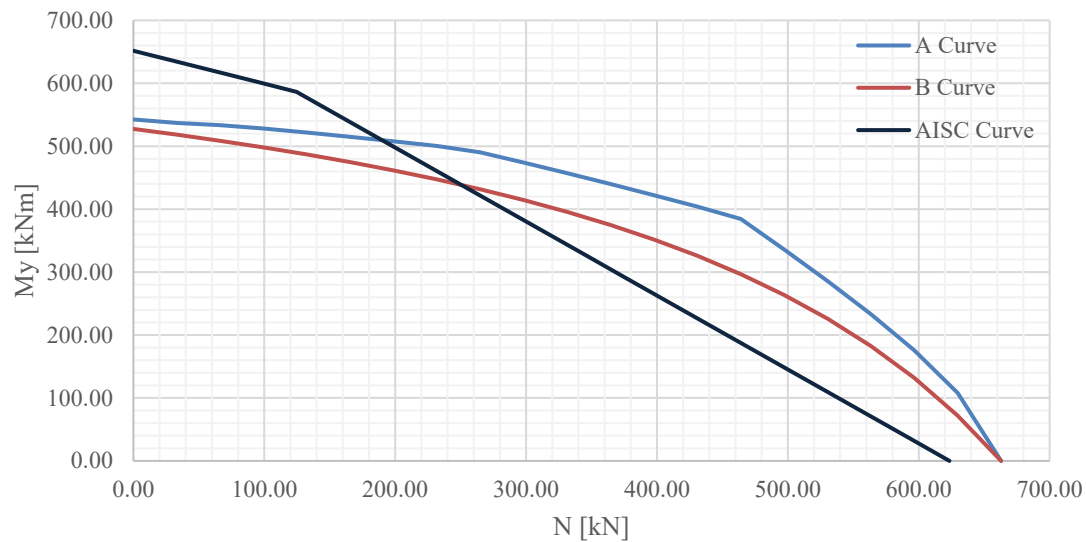


Fig. 27 - $L = 15\text{m}$, $\lambda_y = 75.29$, $\lambda_z = 201.21$

4. General conclusion and future directions

The comparative study carried out in this paper has shown that, although EN 1993-1-1[9] and AISC 360-22[10] pursue the same fundamental objective, the two standards are based on different design policies, leading systematically to distinct results.

From a design policy standpoint, the observed differences cannot be attributed to a single cause but represent the cumulative effect of several factors, among which the following ones could be mentioned.

- The value adopted for the modulus of elasticity: EN 1993-1-1[9] uses the value of 210 000 N/mm², while AISC 360-22[10] adopts 29 000 ksi, equivalent to approximately 200 000 N/mm², a difference that directly influences the calculation of stiffness and critical buckling loads.
- A second important factor is the influence of safety coefficients. In Eurocode, the partial factors γ_{M0} and γ_{M1} are generally taken as 1.0, meaning that the design resistance practically coincides with the characteristic resistance. AISC

360-22[10] uses within the LRFD method a resistance factor $\phi = 0.9$ for members subjected to combined compression and bending, reducing the nominal resistance by 10%-equivalent, from the EN 1993-1-1 [9] perspective to a global safety factor of approximately $\gamma \approx 1.1$.

- A third source of difference lies in dealing with the compression–bending interaction. EN 1993-1-1[9] provides two distinct verification relations. In contrast, AISC 360-22 [10] employs a unified verification equation that is presented in two distinct forms. The selection of the appropriate form is determined by the magnitude of the axial load ratio.

The results of this study lead us to the conclusion that the following aspects represent promising directions for future research.

- A first direction concerns the influence of the classification of cross-sections: no direct equivalence was found between the section classes defined in EN 1993-1-1[9] and those adopted in AISC 360-22[10]; establishing a consistent mapping between the two systems would contribute to a more rigorous comparison. Independent variation of these parameters-including web and flange width-to-thickness ratios and the relative proportions between web and flanges could reveal the sensitivity of each standard relation to local buckling effects.
- A second direction concerns the extension of the analysis to biaxial bending, incorporating simultaneous M_y and M_z moments, which represents a more general loading condition encountered in practice.
- Finally, extending the comparative study to other cross-sectional configurations-including hollow sections, asymmetric profiles, and other cross-sectional types would allow for broader and more generally applicable conclusions.

The topic of compression and bending interaction is continuously evolving. Steps in the evolution can be noticed every day; such an example can be found in the commentary to Chapter H of AISC 360-22[10, p. 404], the restriction based on the ratio I_{yc}/I_y included in previous versions of the specification was found unnecessary and removed starting with the 2022 edition. The American standard is usually revised every five to six years, with each revision generally limited in scope and accompanied by detailed explanations in the commentary section. However, the evolution of the Eurocodes is a more complex and time-consuming process due to the need to harmonize the approval of 34 independent member states. Consequently, the Eurocode revision cycle is longer, and the standard does not include a commentary section.

As a contribution to current research, this study offers an updated and systematic comparison that includes the latest AISC specification (AISC 360-22 [10]) for a welded I-section under three typical loading conditions. The results provide lower and upper bounds for the resistance differences between the two standards, giving practitioners and researchers a useful reference for both European and American design contexts.

In conclusion, this study points out differences between the two standards and provides a background for future research on the design of steel members subjected to combined axial compression and bending moment.

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