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Collapse probability and resistance factor calibration of 2D steel frames under gravity loads

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Abstract

The current advanced analysis techniques for steel frames generally use structural analyses with geometric and material nonlinearities to capture the collapse strength of the steel frame. Unfortunately, the true strength of a steel frame cannot be predicted with accuracy because of the uncertainties of the most significant design variables. Building codes of steel structures apply a resistance factor to account for the uncertainties present in the design variables and thus ensure a target level of structural reliability. This article examines the reliability of planar steel frames subject to gravitational loads by advanced structural analysis (second-order inelastic analysis). To calculate the collapse probability of planar steel frames, we utilized the first-order reliability method (FORM). The advanced analyses were performed using the program MASTAN2 and considered the geometric nonlinearities and inelasticity of the steel. The collapse probabilities of planar steel frames were evaluated and the adequacy of the resistance factor applied was discussed. The current inelastic design procedure of ANSI 360 reduces the yield strength and stiffness of all members by a factor of 0.90. Thus, the present study suggests that the adopted resistance factor must be equal to 0.85 for the target reliability index equal to 3.0, or it must be equal to 0.69 for the target reliability index equal to 3.8.

Keywords: collapse probability, resistance factor, steel frame structures, inelastic behavior, structural reliability, target reliability index.

1. Introduction

The current advanced analysis techniques for steel frames generally use structural analyses with geometric and material nonlinearities to capture the collapse strength of the steel frame. The advanced analysis may result in more efficient designs due to more accurate predictions of the true strength of the structural system. It simultaneously evaluates the strength and stability of the structure without the necessity of individually checking the capacity of the members.

Unfortunately, even with the advanced nonlinear structural analysis method, the true strength of a steel frame cannot be predicted with accuracy because of the uncertainties of the most significant design variables, which are the properties of the material, the applied external loads, and the geometric properties of the cross-sections of the steel shapes.

Current codes have a deterministic format. However, the effect of uncertainties is considered through the application of safety factors (Shayan, 2013). Building codes for steel structures apply a resistance factor to account for the uncertainties present in the design variables. However, this semi-probabilistic method does not allow real knowledge of the collapse probabilities of the structure in service (Agostini *et al.*, 2018). Reliability methods allow the direct evaluation of the structure's failure probability.

In the present study, we utilized the first-order reliability method (FORM) to calculate the failure probability of planar steel frames to collapse; this method uses the probability density function of each uncertain variable to determine the failure probability.

In this article, through the advanced analysis (second-order inelastic analysis), the reliability of the system of planar steel frames subject to gravitational loads was evaluated. Such analysis was performed using the program MASTAN2 (McGuire, Gallagher, and Ziemian, 2000) and considered the geometric nonlinearities and inelasticity of the steel. The failure probabilities of numerical examples of planar steel frames were compared to other authors, after which the reliability implications of this methodology and the adequacy of the resistance factor were then evaluated and discussed.

2. Structural reliability

In the structural reliability analysis, random variables model the maximum demand (load effect), S, and the available resistance (structural capacity), R. The aim of the reliability analysis is to ensure the event $R>S$ throughout the structure's lifespan in terms of probability.

Failure occurs if R is less than S, and this event can be represented in terms of probability as $P(R<S)$. If both the R and S random variables have normal distributions and are statistically independent, then the Z random variable can be entered as $Z=R-S$. The failure probability can be defined as:

$$P_f = P\left(Z < 0\right) = \int_{-\infty}^{0} f_Z\left(z\right).dz = \Phi\left(\frac{0-\mu_Z}{\sigma_Z}\right) = \Phi\left(-\beta\right), \quad (1)$$

where $\mu_Z=\mu_R-\mu_S$, $\sigma_Z=\sqrt{\sigma_R^2+\sigma_S^2}$, Φ is the CDF of the standard normal distribution and β is the reliability index (Cornell, 1969), defined below:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_R-\mu_S}{\sqrt{\sigma_R^2+\sigma_S^2}}. \quad (2)$$

Initially, the reliability index was evaluated in the FOSM method simply as a function of the means and standard deviations of the available resistance, R, and the maximum demand, S, as indicated in Eq. (2). Subsequently, the reliability index is initially obtained by analytical methods based on approximations in the first-order Taylor series (FORM method).

2.1 First-order reliability method (FORM)

In the FORM method, the random variables U, whose distributions can be normal and non-normal and may or may not be dependent on each other, are transformed into standard normal V variables that are statistically independent, with the failure function $G(U)$ written in the space of the reduced variables (space V) as $g(V)$. Hence, the failure surface defined by $g(V) = 0$ is approximated by a linear surface (or hyperplane) at the point with the shortest distance to the origin, identified as V^* (design point in the space of the reduced variables). One of the steps of the FORM method is the transformation of U variables with any distributions into statistically independent standard V variables using the Nataf transformation (Melchers & Beck, 2017):

$$V = \Gamma\sigma^{-1}\left(U - m\right) \quad (3)$$

where m is the vector with the means of the variables U, σ is the diagonal matrix containing the standard deviations of the variables U and $\Gamma= L^{-1}$, where L is the lower triangular matrix obtained from the Choleski decomposition of the matrix of the correlation coefficients of U. Another important step of the FORM method is the search for the point on the failure surface closest to the origin of the reduced system (design point).

To find the design point, the algorithm called HLRF, developed by Hasofer & Lind (1974) and improved by Rackwitz & Fiessler (1978), is commonly used. The iterative process generated by the HLRF algorithm searches for the design point by solving the following equation:

$$V^{i+1} = \frac{1}{\left|\nabla g\left(V^i\right)\right|^2}.\left[\nabla g\left(V^i\right)^T V^i - g\left(V^i\right)\right].\nabla g\left(V^i\right). \quad (4)$$

During the iterative process, the reliability index β is determined by calculating $|V^{i+1}|$, and the process stops when the β value converges. The failure probability can then be obtained using Eq. (1).

3. Methodology

In this study, two-dimensional advanced analyses were performed using the software MASTAN2 (McGuire, Gallagher, and Ziemian, 2000). Version 3.5.5 of MASTAN2 was used for second-order inelastic analyses and to obtain the ultimate load factor (λ_u), which is necessary to assess the performance function in structural reliability analyses. In the nonlinear structural analysis of the frames, the plastic hinge formulation presented in MASTAN2 and the strategy of a constant increase of the load parameter with a predictor-corrector solution scheme were used. Also, the modified tangent elastic modulus (E_{tm}) and the incremental load factor fixed at 1% of the applied load were considered. Each structural element (beams and columns) was discretized into 4 finite elements. The yield surface used in MASTAN2 is a function of a member's axial force and bending moment. The yield surface, developed by McGuire, Gallagher, and Ziemian (2000), is expressed by the polynomial equation:

$$p^2 + m_x^2 + 3.5p^2m_x^2 = 1, \quad (5)$$

where $p=P/P_y$ is the ratio of the axial force to the squash load and $m_x=M_x/M_{px}$ is the ratio of the strong axis bending moment to the corresponding plastic moment. The load, P_y, and the plastic moment, M_{px}, are, respectively, the section's area and plastic section modulus times σ_y.

The performance function (limit state equation) is usually an implicit function of random variables in the reliability of complex structures analysis. The reliability analyses performed were a combination of the FORM method and the deterministic finite element method implemented in MASTAN2. The performance function was formulated as a function of the available resistance (R) of the structural system and as a function of the maximum load (S) in the structural system. The performance function was formulated according to the equation:

$$G(U) = 1 - \frac{S}{R} = 1 - \frac{1}{\lambda} \quad (6)$$

In Eq. (6), the overall resistance of the structure was expressed as a function of a load factor $\lambda=R/S$, which provides how many times the resistance to the collapse of the structure is greater than the acting load, based on the structure's advanced analysis.

4. Results and discussion

In this section, we present the results of the structural reliability analysis of steel structures. Reliability analyses made it possible to assess the collapse probabilities of the structures designed by ANSI 360 (AISC, 2010). By analyzing the obtained results and comparing them with those found by other authors, it was possible to validate the computational implementation, attesting its accuracy and efficiency in the structural reliability analysis of steel frames.

In the first example, a continuous beam subjected to a concentrated vertical load is presented and the failure probability regarding the plastic collapse is investigated. In the second example, the failure probability was obtained for an unsymmetrical two-story steel frame with two bays that was under gravity loads. The two structures have significant load redistribution capacity following initial yielding. In the third example, we obtained the failure probability of a beam-column frame (inverted "L" frame). In this structure, the strength is governed by a single critical member.

All beams and columns for the structures were compact and laterally braced, so the plastic capacity of each section could be achieved without local buckling. Connections were assumed to be fully rigid. The steel material property is modeled as elastic-perfectly plastic.

4.1 Example 1: Three-span continuous beam

As for the first example, we considered a continuous beam subjected to a concentrated vertical load in the middle span. The geometric dimensions, load, and support conditions of the structure are shown in Fig. 1. The following load combination suggested in ASCE 7-10 (ASCE, 2010) is used to select the size of the members: $1.2D_n + 1.6L_n$, where D_n and L_n are nominal dead load and nominal live load, respectively. The cross-sections of the beam are laminated steel shapes: W690 × 125 in the first span, W410 × 46.1 in the second span, and W460 × 52 in the third span. All members are made of the same grade of steel: the nominal yield stress (F_{yn}) is 345 MPa with a nominal Young's modulus (E_n) of 200 GPa.

Performing the inelastic analysis of the continuous beam, and reducing nominal values of yield stress ($0.9F_{yn}$) and modulus of elasticity ($0.9E_n$) for all members according to ANSI 360 (AISC, 2010), it was found that the first plastic hinge is formed in section B, with a load factor $\lambda_1 = 0.981$; the second plastic hinge is formed in section C with a load factor $\lambda_2 = 1.20$; and the third plastic hinge is formed in section D with a load factor $\lambda_u = 1.29$. Zhang *et al.* (2018) also performed the advanced analysis of this beam and came to the same conclusion: the continuous beam supports approximately 129% of the total load $P = 349.19$ kN applied in Fig. 1, and the first hinge is formed with a load factor $\lambda_1 = 1.00$. The beam had a significant capability for redistributing forces after the first yield.

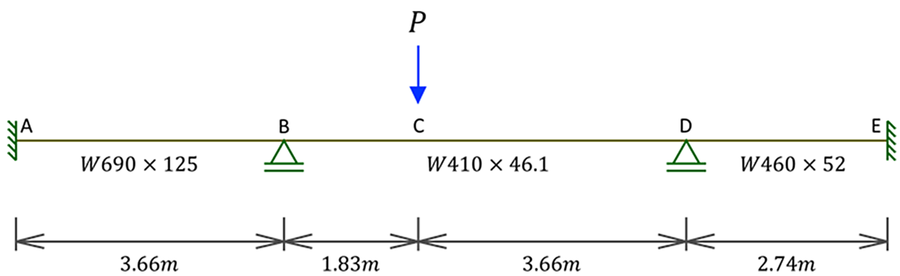

Figure 1 - Three-span continuous beam.

In order to investigate the collapse probability of the continuous beam, reliability analyses were performed considering the basic random variables: live load (L), dead load (D), cross-sectional area (A), moment of inertia (I), yield strength (F_y) and Young's modulus (E). Table 1 summarizes the statistical information for these basic random variables. The structural load shown in Fig. 1 represents the gravity load combination $P = \lambda * (1.2D_n + 1.6L_n)$, with the nominal live-to-dead load ratio assumed to be $L_n = 1.5D_n = 145.5$ kN.

Table 2 summarizes the reliability indexes obtained for two load levels: load to form the first plastic hinge ($\lambda = 1.00$) and load to the plastic collapse ($\lambda = 1.29$). Based on the results of the reliability analysis in Tab. 2, some observations can be made: the reliability index $\beta = 4.05$ obtained when the beam was designated to a load level for the formation of the first plastic hinge (λ = 1.00) results in a failure probability of the structural system on the order of 0.00256%. The reliability index β = 2.95 obtained when we design the beam to a load level of plastic collapse (λ = 1.29) results in a failure probability of the structural system on the order of 0.15889%.

Comparing the reliability indexes obtained in the present study with those obtained by other authors, it was observed, that the reliability indexes obtained by Zhang *et al.* (2018) are close to those obtained in the present study, and the probabilities of failure are similar, see Table 2. This table also shows that the reliability indexes obtained by Zhang *et al.* (2018) are slightly lower than the rates obtained in the present study. Such a slight difference between the results can be justified because Zhang *et al.* (2018) used the Monte Carlo direct simulation method to assess the probability of failure and used the plastic zone method (discretization of the cross-section in fibers) with residual stresses in the inelastic analysis and incorporated the strain hardening effect in the steel stress-strain curve.

Table 1 - Description of basic random variables.

Variable	Mean	Coefficient of Variation (COV)	Distribution	Reference
D (*kN*)	1.05 D_n	0.10	Normal	Ellingwood *et al.* (1982)
L (*kN*)	L_n	0.25	Type I Largest	Ellingwood *et al.* (1982)
F_y (*MPa*)	1.10 F_{yn}	0.06	Lognormal	Bartlett *et al.* (2003)
E (*MPa*)	E_n	0.04	Lognormal	Bartlett *et al.* (2003)
A (cm^2)	A_n	0.05	Normal	Ellingwood *et al.* (1982)
I (cm^4)	I_n	0.05	Normal	Ellingwood *et al.* (1982)

If the desired reliability was a failure probability less than 0.13499% (β > 3.0), the total load applied to the structure could not exceed 127% of the total load shown in Figure 1. If the desired reliability was a failure probability less than 0.00723% (β > 3.8), the total load applied to the structure could not exceed 106% of the total load shown in Figure 1.

The target reliability index β = 3.8 corresponds to the minimum value recommended in Table B2 of EN 1990 (European Committee for Standardization, 2002) for CC2 consequence class structures for a 50-year reference period for the ultimate limit state, which is commonly considered in reliability analyses carried out in the Eurocode framework. The target reliability index β = 3.0 corresponds to the minimum value recommended for buildings (steel members) for a 50-year design life in the design code AISC associated with component ultimate limit states (Liu *et al.*, 2021).

The reliability results shown in Table 3 were obtained for an imposed-to-permanent load ratio of $L_n = 3D_n = 174.6$ kN,which corresponds to a typical load ratio in Chapter B – Design Basis of the ANSI 360 (AISC, 2010).

Based on the results of the reliability analysis in Tab. 3, it can be observed that the reliability index β = 3.76 obtained when the beam is designed with a load level for the formation of the first plastic hinge (λ = 1.00) results in a failure probability of the structural system on the order of 0.00850%. The reliability index β = 2.78 obtained when the beam is designed with a load level of plastic collapse (λ = 1.29) results in a probability of failure of the structural system on the order of 0.27179%. If the desired reliability was a failure probability less than 0.13499% (β > 3.0), the total load applied to the structure could not exceed 121% (λ = 1.21) of the total load shown in Figure 1, which would correspond to a system resistance factor equal to 0.85: $0.85F_{yn}$ and $0.85E_n$. If the desired reliability was a failure probability less than 0.00723% (β > 3.8), the total load applied to the structure could not exceed 99% of the total load shown in Figure 1, which would correspond to a resistance factor equal to 0.69: $0.69F_{yn}$ and $0.69E_n$.

Table 2 - Reliability indexes obtained for the continuous beam (L_n=1.5D_n).

Load Level	Reliability index (present study)	Reliability index Zhang *et al.* (2018)
λ = 1.00	*β = 4.05*	*β = 3.90*
λ = 1.29	*β = 2.95*	*β = 2.76*

Table 3 - Reliability indexes obtained for the continuous beam (L_n=3D_n).

Load Level	Reliability index (present study)
λ = 0.99	*β = 3.80*
λ = 1.00	*β = 3.76*
λ = 1.21	*β = 3.03*
λ = 1.29	*β = 2.78*

4.2 Example 2: Two-story unsymmetrical frame

As for the second example, an unsymmetrical two-story, two-bay rectangular steel frame as shown in Fig. 2 was considered. The geometric dimensions, support conditions, and loads are shown in the same figure. All members are made of the same grade of steel: the nominal yield stress (F_{yn}) is 248 MPa with a nominal Young's modulus (E_n) of 200 GPa. The cross-sections of the frame are laminated steel shapes: W310×28.3 assigned to column C1; W360×237 assigned to column C2; W360×216 assigned to columns C3, C5 and C6; W150×13.5 assigned to column C4; W760×173 assigned to beams B1 and B4; W920×271 assigned to beam B2 and W610×82 assigned to beam B3. The reference load P_0 is 146.95 kN/m.

When performing the inelastic analysis of the steel frame and reducing nominal values of yield stress ($0.9F_{yn}$) and modulus of elasticity ($0.9E_n$) for all members, it was found that the first plastic hinge is formed with a load factor $\lambda_1 = 0.99$, and collapse is reached when a load ratio $\lambda_u = 1.18$ is applied. Zhang *et al.* (2018) also performed an advanced analysis and concluded that the steel frame supports approximately 119% of the total load P_0 applied in Fig. 2, and the first hinge is formed with a load factor $\lambda_1 = 1.00$, which is due to the significant load redistributing ability of the frame.

To investigate the collapse probability of the steel frame, reliability analyses were performed considering the same basic random variables summarized in Table 1. The structural load, shown in Fig. 2, represents the gravity load combination $P_0=\lambda^*(1.2D_n+1.6L_n)$, with the nominal live-to-dead load ratio assumed to be $L_n=1.5D_n=61.23$ kN/m. Table 4 summarizes the reliability indexes obtained for two load levels: load to form the first plastic hinge ($\lambda = 1.00$) and load to collapse ($\lambda = 1.19$).

Based on the reliability analysis results in Table 4, it can be observed that the reliability index $\beta = 3.65$ obtained when the frame designed with a load level for the formation of the first plastic hinge results in a probability of failure of the structural system on the order of 0.01311%. The reliability index $\beta = 2.90$ is obtained when the frame designed for a load level of collapse results in a probability of failure of the structural system on the order of 0.18658%. Comparing the reliability indexes obtained in the present study with those obtained by other authors, it is observed in Table 4 that the reliability indexes obtained by Zhang *et al.* (2018) are close to those obtained in the present study and the probabilities of failure are similar. This slight difference between the results is justified in the previous example. If the desired reliability was a failure probability of less than 0.13499% ($\beta > 3.0$), the total load applied to the structure could not exceed 116% of the total load shown in Figure 2. If the desired reliability was a failure probability less than 0.00723% ($\beta > 3.8$), the total load applied to the structure could not exceed 96% of the total load shown in Figure 2.

The reliability results shown in Table 5 were obtained for an imposed-to-permanent load ratio of $L_n=3D_n=73.475$ kN/m, which corresponds to a typical load ratio in Chapter B – Design Basis of the ANSI 360 (AISC, 2010).

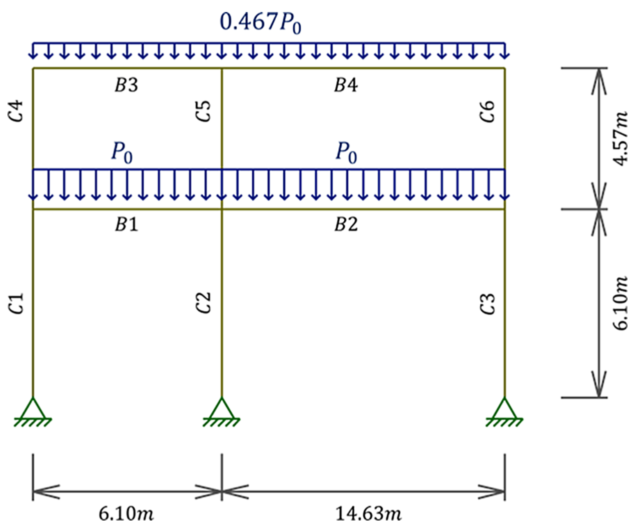

Figure 2 - Two-story unsymmetrical frame.

Table 4 - Reliability indexes obtained for the two-story unsymmetrical frame ($L_n=1.5D_n$).

Load Level	Reliability index (present study)	Reliability index Zhang *et al.* (2018)
$\lambda = 1.00$	$\beta = 3.65$	$\beta = 3.62$
$\lambda = 1.19$	$\beta = 2.90$	$\beta = 2.89$

Table 5 - Reliability indexes obtained for the two-story unsymmetrical frame ($L_n=3D_n$).

Load Level	Reliability index (present study)
$\lambda = 0.90$	$\beta = 3.81$
$\lambda = 1.00$	$\beta = 3.40$
$\lambda = 1.11$	$\beta = 3.00$
$\lambda = 1.19$	$\beta = 2.73$

Based on the results of the reliability analysis in Table 5, some observations can be made: the reliability index β = 3.40 obtained when the frame designed with a load level for the formation of the first plastic hinge (λ = 1.00) results in a failure probability of the structural system on the order of 0.03369%. The reliability index β = 2.73 obtained when we design the frame to a load level of collapse (λ = 1.19) results in a probability of failure of the structural system on the order of 0.31667%. If the desired reliability was a failure probability less than 0.13499% (β > 3.0), the total load applied to the structure could not exceed 111% of the total load shown in Figure 2, which would correspond to a system resistance factor equal to 0.85: $0.85F_{yn}$ and $0.85E_n$. If the desired reliability was a failure probability less than 0.00723% (β > 3.8), the total load applied to the structure could not exceed 90% of the total load shown in Figure 2, which would correspond to a resistance factor equal to 0.69: $0.69F_{yn}$ and $0.69E_n$.

4.3 Example 3: inverted "L" frame

As for the third example, an inverted "L" steel frame as shown in Fig. 3 was considered. The geometric dimensions, support conditions and load are shown in the same figure. All members are made of the same grade of steel: the nominal yield stress (F_{yn}) is 248 MPa with a nominal Young's modulus (E_n) of 200 GPa. The cross-sections of the frame are laminated steel shapes: W460×52 assigned to the beam and W360×101 assigned to the column. The reference load P=2571.07 kN.

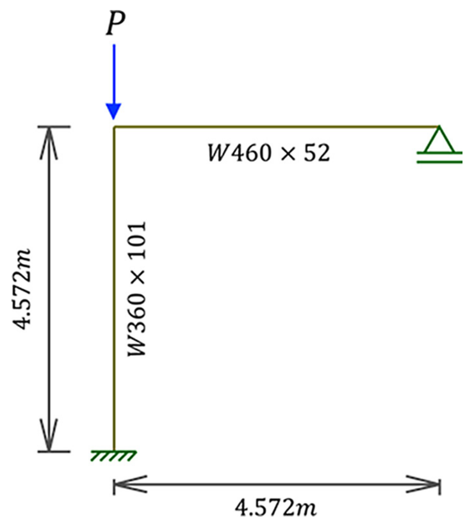

Figure 3 - Inverted "L" steel frame.

Performing the inelastic analysis of the steel frame and reducing nominal values of yield stress ($0.9F_{yn}$) and modulus of elasticity ($0.9E_n$) for all members, we observed that the collapse is reached when a load ratio λ_u = 1.0 is applied. Liu (2019) also performed an advanced analysis of this structure and concluded that the steel frame supports approximately 103% of the total load P applied in Fig. 3.

To investigate the collapse probability of the steel frame, reliability analyses were performed considering the same basic random variables summarized in Table 1. The structural load shown in Fig. 3 represents the gravity load combination $P=\lambda^*(1.2D_n+1.6L_n)$, with the nominal live-to-dead load ratio assumed to be $L_n=1.5D_n$=1071,28 kN.

Table 6 summarizes the reliability indexes obtained for the two load levels. Based on the results of the reliability analysis in Table 6, some observations can be made: the reliability index β = 2.94 obtained when the frame is designed for a load level λ = 1.0, results in a probability of failure of the structural system on the order of 0.16411%. The reliability index β = 2.81 obtained when the frame is designed for a load level of collapse, results in a probability of failure of the structural system on the order of 0.24771%. Comparing the reliability indexes obtained in the present study with those obtained by other author, we can observe that in Table 6 the reliability indexes obtained by Liu (2019) are the same as those obtained in the present study. If the desired reliability was a failure probability less than 0.13499% (β > 3.0), the total load applied to the structure could not exceed 98% of the total load shown in Figure 3. If the desired reliability was a failure probability less than 0.00723% (β > 3.8), the total load applied to the structure could not exceed 82% of the total load shown in Figure 3.

The reliability results shown in Table 7 were obtained for an imposed-to-permanent load ratio of $L_n=3D_n$=1285.535 kN, which corresponds to a typical load ratio in Chapter B – Design Basis of the ANSI 360 (AISC, 2010). Based on the results of the reliability analysis in Table 7, it can observed that the reliability index β = 2.65 obtained when the frame is designed for a load level of collapse (λ = 1.03) results in a probability of failure of the structural system on the order of 0.40246%.

If the desired reliability was a failure probability less than 0.13499% (β > 3.0), the total load applied to the structure could not exceed 94% of the total load shown in Figure 3, which would correspond to a system resistance factor equal to 0.85: $0.85F_{yn}$ and $0.85E_n$. If the desired reliability was a failure probability less than 0.00723% (β > 3.8), the total load applied to the structure could not exceed 76% of the total load shown in Figure 3, which would correspond to a resistance factor equal to 0.69: $0.69F_{yn}$ and $0.69E_n$.

Table 6 - Reliability indexes obtained for the inverted "L" frame (L_n=1.5D_n).

Load Level	Reliability index (present study)	Reliability index Zhang *et al.* (2018)
$\lambda = 1.00$	$\beta = 2.94$	$\beta = 2.94$
$\lambda = 1.03$	$\beta = 2.81$	$\beta = 2.81$

Table 7 - Reliability indexes obtained for the inverted "L" frame (L_n=3D_n).

Load Level	Reliability index (present study)
$\lambda = 0.76$	$\beta = 3.82$
$\lambda = 0.94$	$\beta = 3.00$
$\lambda = 1.00$	$\beta = 2.76$
$\lambda = 1.03$	$\beta = 2.65$

5. Conclusion

In the present study, reliability analyses of 2D steel structures were carried out through advanced structural analyses considering the effects of geometric nonlinearity and steel inelasticity. The FORM method was used to assess the failure probability of the system in relation to the ultimate limit state of the collapse. The first and the second structures analyzed have significant capacity for redistribution of the inelastic load. Through reliability analysis, it was possible to determine the failure probability for the two design load levels: formation of the first plastic hinge and the plastic collapse. The third structure analyzed fails in an elastic instability mode with limited yielding developed, and through the reliability analysis, it was possible to determine the collapse probability.

The results of the numerical examples showed that it is essential to obtain the collapse probability to account for uncertainties inherent to design variables so that safer structures with target reliability can be obtained. The current inelastic design procedure of ANSI 360 (AISC, 2010) reduces the yield strength and stiffness of all members by a factor of 0.90. The present study suggests that the adopted resistance factor must be equal to 0.85 for the target reliability index equal to 3.0 or it must be equal to 0.69 for the target reliability index equal to 3.8.

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