Single-Story Steel Frame Structure in 3D: Sensitivity Study Utilizing FEM Simulations

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Abstract

This paper presents a sensitivity study of a single-story steel frame structure. A basic sensitivity study of linear (Pearson) correlations was conducted to examine the relationships between input parameters and the frame's ultimate resistance. This study provides a basic insight into a problematic of how variations in key input parameters influence the ultimate resistance of the structure across different slenderness levels (various frame heights, keeping the same cross-sections). The input parameters considered in the analysis are treated stochastically and include two material properties (elastic modulus and yield stress) along with four parameters defining the initial geometrical imperfections of the frame: global sway and local bow imperfections in two orthogonal directions. These imperfections affect the structural behavior and stability of steel frames, making their probabilistic assessment crucial. To determine the ultimate resistance based on stochastic data, a combination of the First Order Reliability Method (FORM), the finite element method (FEM), and geometrically and materially nonlinear imperfect analyses (GMNIA) is employed. The FORM-based resistance values are then compared with two deterministic estimates of ultimate resistance obtained following the European standard for steel structure design (EC3). The first deterministic estimate directly utilizes FEM, while the second is based on EC3 buckling provisions, where the critical length is determined using prior eigenvalue buckling analysis of the frame structure. The findings of this study provide valuable insights into the sensitivity of the selected steel frame and demonstrate how probabilistic methods can complement conventional deterministic design approaches. By incorporating both stochastic and deterministic analyses, the research highlights the influence of uncertain parameters on the structural response and enhances the understanding of steel frame behavior under varying conditions. The comparison between probabilistic and deterministic resistance estimates also reinforces the accuracy and reliability of stochastic approaches, which may offer a more refined alternative to traditional design methods. The basic sensitivity study of linear correlations identifies key relationships between input variables and structural resistance, helping to better interpret the influence of different parameters on the overall stability of the frame. These results contribute to improving structural safety assessments and refining design methodologies for steel frames, particularly in cases where uncertainties play a significant role in performance evaluation.

Keywords: *Single-Story Steel Frame; Initial Geometric Imperfection; Geometrically Nonlinear Analysis; First order reliability method*

1. Introduction

Taking initial imperfections into account during the global analysis of steel frame structures is crucial, as they can significantly affect the load-bearing capacity [1]. These imperfections can be categorized into three main types: material, structural, and geometrical [2] [3]. Geometrical imperfections arise due to manufacturing and erection tolerances and can be classified into two forms: global (such as floor sway or "out-of-plumb" deviations in the structure) and local (such as bowing of individual members, like columns). This study applies stochastic methods to model initial geometrical imperfections in a three-dimensional numerical model of a steel frame, emphasizing both global and local imperfections as random stochastic variables. However, cross-sectional geometrical imperfections and material imperfections are not considered.

Several methods can be used to account for initial geometrical imperfections:

- The Notional Horizontal Forces (NHF) method, in which the effects of initial sway imperfections are substituted by a system of equivalent horizontal forces acting in opposite directions at the top and bottom of each column. Local bow imperfections, on the other hand, are represented by a system of equivalent horizontal forces (aligned in the same direction) at the column's base and top, along with a line load of equal magnitude but acting in the opposite direction to these nodal forces. This approach is permitted in various steel design codes, such as the European standard 1993-1-1, specifically in section 5.3.2(7) [4], and has been utilized in research conducted by Liew et al. [5].
- The Elastic Buckling Mode (EBM) scaling method involves performing a linear elastic buckling analysis (eigenvalue buckling) on a structure with an idealized, perfect geometry. The selected buckling mode—typically the first—is then scaled to approximate the shape of the initial imperfection. This method was applied, for example, by Gu et al. [6]. However, a potential issue arises if the first and second buckling modes coincide [7]. Ongoing refinements to this method have been made, such as by Aguero et al., who extended its application to non-uniform cross-sections [8]. Since plastic deformations can alter the final failure shape compared to what is predicted by the elastic buckling mode, an alternative approach involves conducting a plastic second-order analysis to determine the actual failure shape, which is then scaled to generate the initial imperfection geometry [9].

However, this method may be overly conservative [10] because it imposes the collapse shape at the very beginning of the structural analysis. To address this, Shayan et al. [10] provide guidelines for combining multiple eigenmodes using recommended scale factors to define geometrical imperfections in 2D frame structures. Another strategy determines scale factors by estimating the entropy of the buckling mode [11].

- A reduction in model stiffness by applying 85% of the material's elastic modulus was proposed by Kim [12]. While this method is simple to implement, it has not yet been fully validated through a probabilistic approach [10].
- The most direct way to model initial geometrical imperfections is by offsetting nodal coordinates from their original positions. This method explicitly modifies the geometry but can be complex, especially for large structures, as it requires extensive manual adjustments or advanced computational algorithms. In global frame analysis, the imperfection pattern is typically selected to represent the most unfavorable case, maximizing destabilizing effects. For instance, a multi-story structure may be assigned a global sway in a single direction, following the recommendations of section 5.3.2(3) in EN 1993-1-1 [4]. However, in certain scenarios, such assumptions can be overly conservative, potentially leading to inefficient and uneconomical designs [10].

To model initial geometrical imperfections in the most rational manner, employing probabilistic methods (reliability analysis) and treating these imperfections as stochastic (random) variables is the most effective approach [13]. The most accurate representation of geometrical imperfections is achieved when probabilistic methods are combined with direct geometric modeling [10]. However, this method is highly complex and computationally demanding. Therefore, statistical techniques are primarily used to validate other, more practical (typically deterministic) methods, as highlighted by Machowski et al. [14] and Ding et al. [15]. The European standard EN 1990 [16] identifies two main categories of reliability methods:

- Fully probabilistic methods, which require statistical data not only for resistances (such as material and geometric properties) but also for actions (loads). Due to their complexity, they are less commonly applied.
- First-order reliability method (FORM), which is one of the most significant techniques for assessing structural reliability, particularly when combined with advanced numerical analysis methods like the finite element method (FEM). FORM is frequently employed in such cases, as noted by Faber [17] and Zhao [18]. When using FORM to evaluate structural reliability, stochastic parameters related to loads may be disregarded under standard loading conditions.

The objective of this study is to perform a sensitivity study of a selected steel frame structure with a single-story configuration. The significance of the initial input parameters (particularly those defining the initial geometrical imperfections) on the ultimate resistance of the frame is investigated for different frame heights. A simple sensitivity study is conducted, examining the linear Pearson correlations between random input parameters and the ultimate resistance N_u.

The initial geometrical imperfections are modeled through direct geometric modifications, which is considered one of the most rational methods for incorporating such imperfections [10]. Statistical parameter values for these imperfections are derived from the erection tolerance criteria specified in EN 1090-2:2018 [19], with their implementation validated in previous research [20], [21].

To determine the ultimate resistances of the steel frame, advanced numerical geometrically and materially nonlinear analyses with initial imperfections (GMNIA) are carried out using ANSYS software [22]. The stochastic dataset of ultimate resistances obtained from these simulations is further used to determine the design ultimate resistance via the FORM method. Additionally, this resistance is compared with design resistances estimated using two deterministic approaches, following the assumptions outlined in EN 1993-1-1 [4].

This study offers valuable insight into the impact of initial geometrical imperfections concerning variations in structural height. While the findings are based on a specific single-story frame configuration, the methodology presented here can be adapted for future research applications.

2. Numerical FEM Model

2.1 Geometry of the numerical model, boundary conditions and loading

A numerical model of a single-story 3D frame structure is developed using 1D structural beam finite elements. The cross-sections of the members are illustrated in Figure 1, with detailed geometries provided in an online table [23] (excluding the filler radius r [23], which has been omitted). The columns have a span of 5 meters in both directions, while their height h varies from 2 to 10 meters in increments of 0.5 meters. At the base level (z = 0), all six degrees of freedom (DoF) are fully constrained. The columns are positioned so that the minor cross-section axis is aligned with the x-axis of the global coordinate system (GCS). Rigid connections are assumed between the columns and horizontal beams. Each column is discretized into 10 finite elements (FE), while the horizontal beams are divided into 5 FE each. The numerical model is implemented using ANSYS software [22].

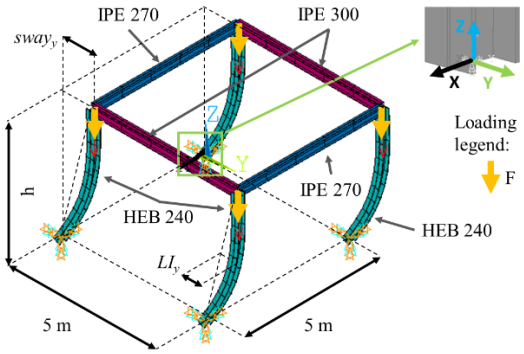

Fig. 1. Frame geometry, and example of the initial geometrical imperfections in y direction (exaggerated).

The load is applied through 4 corner nodes of the frame structure (see orange arrows in the Figure 1). The Ultimate resistance of the frame structure N_u is equal to the sum of vertical reactions R_z of the last converged increment of the analysis. An automatic algorithm to handle the sizes of force increments was used. Geometrically and materially nonlinear imperfect analyses (GMNIA) are conducted using ANSYS software [22]. Full Newton-Raphson equation formulation has been used.

2.2 Numerical analyses using the deterministic approach based on EC3 – material and initial imperfections

All structural elements, including columns and horizontal beams, are assumed to be made of steel with an S355 grade classification [4]. The material behavior is represented using a bilinear model, where the elastic modulus is set to E = 210 GPa, and after reaching the yield stress of 355 MPa, the tangent modulus is defined as 5% of E (kinematic hardening is considered).

Initial geometrical imperfections are incorporated following the guidelines in section 5.3.2 of EC3 [4]. Accordingly, the initial sway imperfections are taken as 1/244.9 in both x and y directions of the global coordinate system (GCS), derived from a base sway ratio of 1/200, with a height reduction factor α_h = 0.94 and a modification factor α_m = 0.866 [4]. Local imperfections of the columns are determined based on Table 5.1 of EC3 [4], where the bow imperfection amplitude is 1/200 of the column height along the x-axis of the GCS (LI_x) and 1/150 along the y-axis (LI_y). The absolute values of these imperfections are computed by multiplying the given relative ratios by the height h of the respective column.

One numerical analysis is conducted for each height of the structure (from 2 to 10 m) utilizing this approach. The obtained ultimate resistances are further noted as $N_{u,EC3,FEM}$.

2.3 Numerical analyses using stochastic methods from the EC0 – FORM – material and initial imperfections

In the stochastic analysis, the material model remains unchanged; however, the elastic modulus E is defined with a mean value of 210 GPa and a standard deviation of 10.5 GPa. The yield strength f_y of S355 steel is considered with a mean of 393.8 MPa and a standard deviation of 22 MPa [3].

The local bow imperfections of the columns, LI_x and LI_y, follow a normal distribution with a mean of 0 and a standard deviation of 1/2000, based on the manufacturing tolerance standard [19] and the "2-sigma rule". To simplify the stochastic input parameters, local imperfections in a given direction are assumed to be identical for all columns, meaning that only two parameters, LI_x and LI_y, are considered.

The standard deviation of the frame's global sway imperfections in both the x and y directions of the global coordinate system (GCS) is also derived from the manufacturing tolerance standard [19] and is taken as 1/600. This value aligns with those used in studies by Shayan et al. [10] and measurements reported by Lindner and Gietzel [24]. The mean value of the sway imperfections in both directions is assumed to be zero.

For each considered frame height h, a large number of random parameter realizations must be evaluated throughout the analysis. In this study, 200 realizations analyzed numerically were generated for each different frame height h. 6 stochastic input parameters were considered, including two local imperfections, two global imperfections, and two material properties (E and f_y). The Advanced Latin Hypercube Sampling (ALHS) method [25] was employed to produce these random input values within the OptiSLang software [26].

For each individual realization, the ultimate resistance N_u is computed. Following this, the design resistance $N_{u,FORM}$ is determined as the 0.1% quantile of the N_u dataset, which can be estimated using the procedure outlined in EC0 [16] through the following equation:

$$N_{u,FORM} = \mu_{Nu} - \alpha_R \cdot \beta_d \cdot \sigma_{Nu} = \mu_{Nu} - 0.8 \cdot 3.8 \cdot \sigma_{Nu} \tag{1}$$

where α_R is the sensitivity factor for FORM, β_d is reliability index [16], μ_{Nu} and σ_{Nu} are mean value and the standard deviation of the corresponding set of N_u respectively. This assumption results in probability of the ultimate resistance being smaller then $N_{u,FORM}$ of 0.1183% (hence approximately 0.1% quantile) [27].

The stochastic values of the input parameters are summarized in the Table 1. For all the inputs, standard Gauss probability density function was considered.

Tab. 1. Summary of the random input variables.

Variable	Symbol	Mean value	Standard deviation	Unit
Global geom. imperfection – sway in x direction	$sway_x$	0.0	0.0005	-
Global geom. imperfection – sway in y direction	$sway_y$	0.0	0.0005	-
Local geom. imperfection – column box in in x direction	LI_x	0.0	0.001667	-
Local geom. imperfection – column box in in x direction	LI_y	0.0	0.001667	-
Material – Elastic modulus	E	210.0	10.5	GPa
Material – Yield stress	f_y	393.8	22.0	MPa

2.4 Verification by the buckling resistance estimations of the EC3, and Euler critical force.

In order to verify the values of the $N_{u,EC3,FEM}$ and $N_{u,FORM}$, the ultimate resistance $N_{u,EC3}$ is estimated as:

$$N_{u,EC3} = 4 \cdot A \cdot f_y \cdot \chi_1, \tag{2}$$

Where χ_1 is the EC3 buckling factor (based on relative slenderness $\overline{\lambda_1}$) in accordance with the chapter 6.3.1.2 (1), where the imperfection factor α was considered as 0.49 (based on the tables 6.1 and 6.2 from the EC3) [4]. A is the cross-sectional area of one column, hence the multiplication factor of 4 in order to account all 4 columns of the analyzed frame.

The relative slenderness $\overline{\lambda_1}$ is considered as:

$$\overline{\lambda_1} = \sqrt{\frac{A \cdot f_y}{N_{cr,1}}}, \tag{3}$$

Where $N_{cr,1}$ is the Euler critical force (for a single column) determined from the linear stability (eigenvalue buckling) results, considering the 1st critical eigenvalue (eigenshape of global sway in GCS y-axis direction) – factor α_1. N_1 is the normal force of the single column obtained from the initial stress calculation prior to the eigenvalue buckling analysis:

$$N_{cr,1} = \alpha_1 \cdot N_1, \tag{4}$$

The Euler critical force for the whole frame, N_{cr}, is then considered to be 4 times the value of critical force of a single column $N_{cr,1}$.

3. Results

The summary of all the analyzed heights h ranging from 2.0 to 10.0 meters is provided in the Table 2. In the second column of this

table, the relative slenderness (of a single column) is provided (based on the Equation 3). Corresponding buckling length based on this relative slenderness is provided in the third column of the Table 2 and determined as:

$$L_{cr,1} = \sqrt{\frac{\overline{\lambda_1} \cdot \pi^2 \cdot E \cdot I_z}{A \cdot f_y}}, \tag{5}$$

where cross-sectional properties of the column section HEB 240 [23] are used, E is the elastic modulus of 210 GPa, and I_z is the second moment of the area along the minor cross-sectional axis.

In the 4th and 5th column of the Table 2 is mean value μ_{Nu} and standard deviation σ_{Nu} of the ultimate resistance N_u dataset from 200 random realizations. Coefficient of variation noted as $N_{u,CoV}$ is then provided in the 6th column (σ_{Nu} / μ_{Nu}). The ultimate resistance based on FORM stochastic method, $N_{u,FORM}$ (chapter 2.3) is provided in the 7th column. Deterministically determined ultimate resistance based on FEM analysis, $N_{u,EC3,FEM}$ (chapter 2.2) is provided in the 8th column. Ultimate resistance $N_{u,EC3}$ (chapter 2.4) based on EC3 buckling factor χ is provided in the 9th column of the Table 2, followed by the Euler critical force N_{cr} in the last column.

Tab. 2. The text of the table caption will be here.

Frame height h [m]	**Relative Slenderness $\overline{\lambda_1}$**	**$L_{cr,1}$ [m]**	**μ_{Nu} [kN]**	**σ_{Nu} [kN]**	**$N_{u,CoV}$ [%]**	**$N_{u,FORM}$ [kN]**	**$N_{u,EC3,FEM}$ [kN]**	**$N_{u,EC3}$ [kN]**	**N_{cr} [kN]**
2.0	0.51	2,42	15 828	869	5.49	13 186	13 634	12 139	55 440
2.5	0.62	2.93	15 405	902	5.86	12 663	12 815	11 237	37 924
3.0	0.73	3.43	14 880	1 028	6.91	11 755	11 931	10 291	27 606
3.5	0.83	3.93	14 054	1 091	7.76	10 737	10 969	9 325	21 005
4.0	0.94	4.43	12 910	1 214	9.40	9 219	9 929	8 376	16 523
4.5	1.04	4.94	11 372	1 061	9.33	8 147	8 919	7 479	13 338
5.0	1.15	5.44	9 792	729	7.44	7 576	7 968	6 659	10 994
5.5	1.25	5.94	8 447	614	7.27	6 580	7 085	5 928	9 218
6.0	1.36	6.44	7 293	485	6.65	5 819	6 309	5 286	7 841
6.5	1.47	6.94	6 356	414	6.51	5 097	5 624	4 727	6 750
7.0	1.57	7.44	5 575	328	5.88	4 578	5 067	4 242	5 873
7.5	1.68	7.94	4 922	287	5.83	4 050	4 528	3 821	5 156
8.0	1.78	8.44	4 386	252	5.75	3 620	4 100	3 456	4 563
8.5	1.89	8.94	3 928	229	5.83	3 232	3 703	3 138	4 066
9.0	1.99	9.44	3 538	195	5.51	2 945	3 356	2 860	3 647
9.5	2.10	9.94	3 201	176	5.50	2 666	3 037	2 615	3 289
10.0	2.21	10.44	2 909	150	5.16	2 453	2 780	2 400	2 981

The ultimate resistances from the 7th – 9th column of the Table 2 and the Euler critical force are also plotted graphically in the Figure 2. For larger slenderness values, the estimations of the ultimate resistances approach the Euler buckling curve. The most conservative value of the resistances might be considered the one determined based on the EC3 approach, $N_{u,EC3}$ using the buckling factor χ. Both FEM methods yielded slightly larger ultimate resistances, but the shape of the curves is overall very similar. Due to randomness, the curve of the $N_{u,FORM}$ is not as smooth as the others.

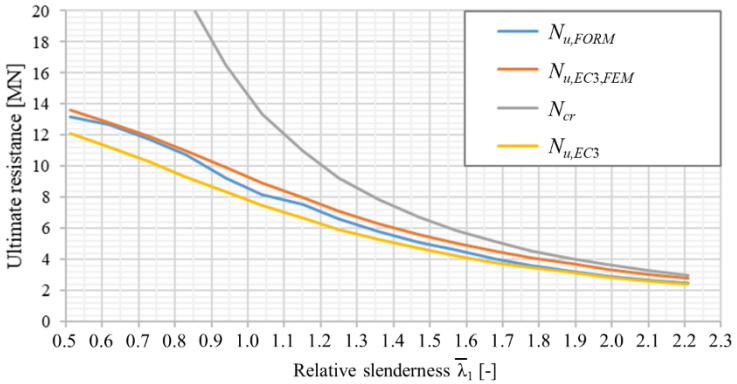

Fig. 2. Ultimate resistance for different relative slenderness of the frame – comparison of several approaches.

For the stochastic analysis, the linear Pearson correlations between the input parameters (initial geometrical imperfections in absolute value) and the output ultimate resistance N_u was monitored through all the analyzed structure heights. These result data are provided in the left part of the Figure 3, in dependence on the relative slenderness. Selected ant-hill plot (N_u vs. $sway_y$) of the 200 random realizations for frame of 4.5 m height is provided in the right part of the Figure 3.

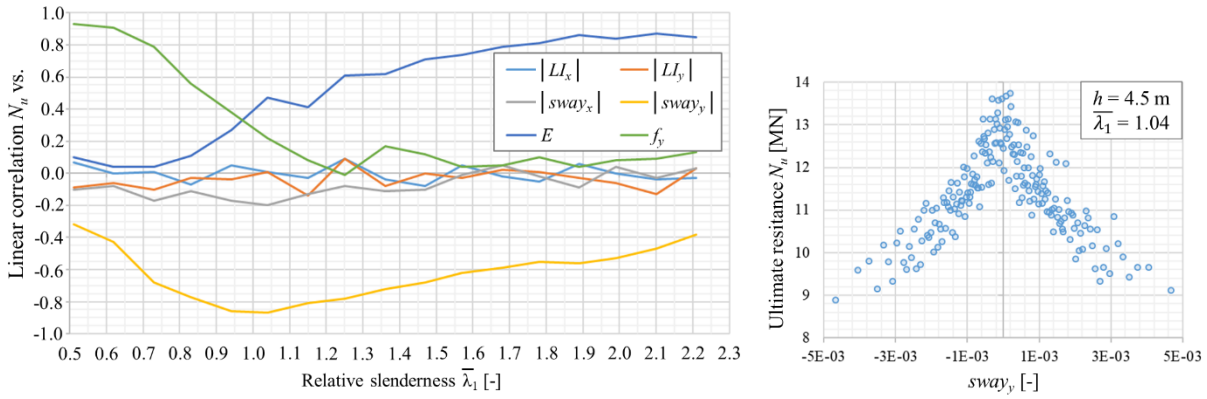

Fig. 3. Left: Linear correlations between the ultimate resistance and the stochastic input parameters for various relative slenderness. Right: Ant-hill plot of the 200 random realizations – ultimate resistance vs. $sway_y$ for frame height of 4.5 m.

4. Discussion

The most conservative approach to determine the ultimate resistance of the analyzed frame structure is the one using buckling factor χ of the EC3, which was based on eigenvalue buckling analysis (using the first mode) – these resistances are noted as $N_{u,EC3}$ (chapter 2.4) and depicted graphically in dependence on the frame relative slenderness by the yellow color in the Figure 2.

On the other hand, the approach using FEM method along with the design values of the input parameters (material and initial imperfections) yielded approximately 12 – 20 % higher values of the ultimate resistances through various frame heights (various relative slenderness) compared to the $N_{u,EC3}$ results – see the curve marked as $N_{u,EC3,FEM}$ in the Figure 2. Otherwise, the global shape of this curve is very similar to the $N_{u,EC3}$ curve, suggesting a consistency in these two approaches.

The ultimate resistances using the stochastic method of FORM resulted somewhere in between the resistances based on previous approaches using EC3 method – see the $N_{u,FORM}$ curve in the Figure 2. For smaller slenderness values (0.6 – 0.8), this curve approaches the $N_{u,EC3,FEM}$ curve, and for larger slenderness values (over 1.7), the results of this stochastic approach are practically the same as the $N_{u,EC3}$. This might be influenced by the fact, there is increasing positive linear correlation between the Elastic modulus, E, and the ultimate resistance with increasing slenderness values (Figure 3 left), along with the fact, that the design value of this parameter is actually considered as the mean value in case of the EC3 approach (the $N_{u,EC3,FEM}$ results). Hence, the influence of the standard deviation of this parameter has also significant influence on the results, mainly for larger slenderness values.

For smaller slenderness values (below 0.8), the linear correlation graph suggests larger influence of the yield stress, f_y, on the ultimate resistance. The correlation between E and N_u is rather negligible. The design yield stress of 355 MPa is approximately 3.89 % quantile of the standard Gauss distribution considered in the stochastic method of the FORM. This suggests the stochastic values of the f_y were considered rather concisely.

The largest correlation between the (absolute value of) sway of the structure in GCS y axis direction, $sway_y$, and the ultimate resistance N_u is observed near the relative slenderness of 1.0 (Figure 3, left). Basic sensitivity study of linear correlation suggests, that the other initial imperfection parameters have rather negligible influence on the ultimate resistance N_u. The sway in y axis direction introduces bending of the columns along their minor cross-sectional axes, hence this negative value of the linear correlation is expected. On the other hand, sway in x axis direction would introduce bending along the major cross-sectional axis, where the second moment of area of the HEB profile is much larger, hence any correlation is also expected to be much smaller. An example of ant-hill plot of Nu vs. $sway_y$ is depicted in the Figure 3 right. Due to the symmetry of the structure and its loading, there is also symmetry in this ant-hill plot, as expected.

5. Conclusion

In this study, a basic sensitivity study of a single story steel frame structure has been conducted in order to investigate the linear (Pearson) correlations between the random input parameters (Elastic modulus, yield stress, and 4 initial geometrical imperfections) and the output ultimate resistance of the frame structure. Various slenderness of frame structure are analyzed, keeping the material and cross-sections same, altering the structure height from 2.0 to 10.0 meters. The stochastic values of the initial geometrical imperfections are derived from the tolerance standard EN 1090-2:2018, and also corresponds with real experimental data, making the results obtained from this stochastic approach more realistic. The first order reliability method was used to determine the ultimate resistance from the data set of ultimate resistances based on finite element numerical analyses conducted in ANSYS software.

In order to verify the results of the stochastic FORM method, two deterministic methods to estimate the ultimate resistance of the frame structure are also considered. First method adopts the numerical finite element increment analysis, where design values of the input parameters are based on the corresponding European standard EC3. The other method uses numerical analysis only to obtain Euler critical force for the column of the structure based on the first mode of the eigenvalue buckling analysis. Based on this critical force, the relative slenderness and the buckling factor based on European standard EC3 is determined, which is subsequently used to calculate the ultimate resistance.

It might be concluded, that all 3 approaches resulted in rather similar estimations of the ultimate resistances through the altering structure height (relative slenderness). The most conservative approach is the one using buckling factor of the EC3. The largest ultimate resistances are obtained from the numerical analysis using the design EC3 values (also deterministic approach). Differences in these estimations are consistently between 12 – 20% through analysed frame heights. The results of the stochastic FORM method are between these deterministic estimations, what suggests consistency in these approaches. It might be concluded the stochastic approach has been well verified, and shows that probabilistic methods can complement conventional deterministic design approaches.

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