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Original Research Article

Reliability-Based Analysis of Steel Portal Frame Eurocode Design Criteria Subjected to Flexural and Lateral Torsional Instability

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Abstract

Steel portal frames, used in factories, workshops, shopping complexes, and warehouses, provide large clear spans. Eurocode 3, a semi-probabilistic design code based on limit state design and adopted in Nigeria, does not fully address uncertainties in load and resistance variables, affecting frame performance in service. This study evaluates the reliability of three-hinged steel portal frames by analyzing three primary failure modes: flexural instability of frame stanchions (failure mode 1), flexural instability of frame rafters (failure mode 2), and lateral torsional instability of stanchions and rafters (failure mode 3). Limit state functions for these failure modes were derived from Eurocode 3 specifications and structural analysis load effects. Stochastic models of uncertain parameters are obtained from the literature. The First-Order Reliability Method (FORM), using a MATLAB program, evaluates the limit state functions and determines failure probability. Results show that for failure mode 1, Eurocode's target reliability of 3.8 is met if Xc is at least 0.85 for stanchions and 0.6 for rafters. At flexural buckling (X_{LT} = 1.0), lateral-torsional stability yields a safety index of 5.8. For failure modes 1 and 2, the safety index decreases with a higher dead load to variable load ratio. For all failure modes, the safety index increases with higher steel grades and coefficient of variation (CoV). To ensure safety, the study recommends fully accounting for uncertainties in design to prevent up to a 60% compromise in portal frame safety.

Keywords: Flexural instability; lateral torsional instability; reliability analysis; safety index; steel portal frame.

1.0 INTRODUCTION

1.1 Background to the Study

Nigeria, a developing country, has a number of factories spread out over the nation. In order to provide ample workspace in workshops, steel portal frames are one of the main industrial materials utilized in the construction of these facilities. Bracing the columns and rafters that make up the portal frames of the steelwork. The frame is composed of cold-formed thin-walled steel, hot-rolled H-shaped steel, or welded H-shaped steel with equal or varying cross-sections (Fieber *et al.*, 2020).

Uncertainties in load and resistance variables are not well addressed by Eurocode 3, a semi-probabilistic design code based on limit state design which was implemented in Nigeria, hence it affects frame performance in service (Ebenuwa and Tee, 2019). For a long time, statisticians, scientists, engineers, and other professionals have debated the nature of uncertainty and how to handle them in designs (Paté-Cornell, 1996; Lindley, 2000; Kiureghian and Ditlevsen, 2009; Benjamin and Cornell, 2014). Because steel portal frames are essential structures that provide the highest level of safety, it is necessary to look into how uncertainties in design variables that have not been sufficiently taken into account in the Eurocode 3 design criteria affect their performance.

The structure's reliability or failure probability is one of the most important ways to provide a suitable condition for preserving the structure's safety (Jebur and Al-Zaidee, 2019). A specific technique for estimating the likelihood of failure in intricate structural systems while taking into account the uncertainties influencing their behavior is reliability analysis. It can be seen to offer the framework for logically resolving building safety and serviceability challenges since it helps engineers make better-informed judgments (Ellingwood, 2000; Chabridon *et al.*, 2017). Furthermore, structural reliability provides a unified strategy for handling uncertainties that affect a structure's performance. Additionally, it creates a quantifiable link between structural engineering methods and the impact they have on society (Ellingwood *et al.*, 2025).

There are various methods for reliability analysis, including Monte Carlo Simulation (MCS), Support Vector Machine (SVM), Second Order Reliability Method (SORM), and First Order Reliability Method (FORM) (Chabridon *et al.*, 2017; Roy and Chakraborty, 2023). This study employs the First Order Reliability Method (FORM) via a well-written program in MatLab to perform a reliability-based analysis of a three-hinged steel portal structure.

To the best of the authors' knowledge, there are very scarce studies related to the reliability-based analysis of the Eurocode design criteria of steel portal frames subjected to member flexural and lateral-torsional instability. Thus the need for the study.

The output of this study is particularly cardinal as it can provide valuable insights into the adequacy of Eurocode 3 design criteria, thereby, leading to contributions that can help in improving design codes to better account for uncertainties and enhance the safety of steel portal frames. Also by studying various modes of failure and performing sensitivity analysis on different design parameters, the study provides a holistic assessment of the structural integrity of steel portal frames in service conditions and this can assist engineers and designers produce a more robust and reliable design of the steel portal frame, thus, enhancing the safety of the portal frame structure.

1.2 Theoretical Framework of Structural Reliability

As earlier presented, the concept of structural reliability has been presented by different authors. For example; Melchers (2007) believed that calculating and forecasting the likelihood of limit state violation for constructed structures is the focus of structural reliability. Also, Ditlevsen and Madsen (1996) regarded structural reliability as a technique that aims to address the many causes of uncertainty in a logical manner. Tsompanakis and Papadrakakis (2004) considered structural reliability analysis as a tool that helps the design engineer determine a structure's chance of failure by accounting for all potential uncertainties throughout the design, construction, and lifetime phases of the structure.

The loads and strengths of materials and sections are represented by their known or assumed distributions, which are described in terms of distribution type, mean, and standard deviation, in reliability-based design, a probabilistic design method. For a given design instance, the probability that the greatest total load effect will be greater than the resistance to failure is the probability of failure (Pf). This contrasts with the semi-statistical design approach known as the limit state design (LSD) method, which treats probabilistic elements during the code creation phase to specify characteristic values and partial safety factors for resistance and load that are used to guarantee, on average, a sufficiently low failure probability throughout the entire range of design cases (Kadry and Smaili, 2007).

LSD is employed as a workable way to integrate dependability techniques into the standard design process in the current European design codes. The statistical aspect of the fundamental design variable was not well addressed by this method. Partial safety factors are a cautious way to group uncertainty. To guarantee that specific, suitable dependability levels—referred to as target values—are reached in design, a limit state code is calibrated (Zimmerman *et al.*, 2002).

In the reliability-based concept, the performance function of a structural system according to a specified mission is given by:

M = performance criterion – given criterion limit = $g(X_1,X_2, ... ,X_n)$ ……………… (1)

In which the X_i ($i = 1, ...,n$) are the n basic random variables (input parameters), with g( ) being the functional relationship between the random variables and the failure of the system. The performance function can be defined such that the limit state of failure surface, is given by $M = 0$. The failure event is defined as the space where $M > 0$. Thus a probability of failure can be evaluated by the following integral.

$P_f = \iint . . . \int f_x(x_1, ... ,x_n)dx_i . . . dx_n$ …………….….. (2)

Where f_x is the joint density function of $x_1, x_2, . . .,x_n$ and the integration is performed over the region where $M < 0$. Because each of the basic random variables has a unique distribution and they interact, the integral cannot be easily evaluated. Use is made of the approximate method.

Traditionally, the concern of researchers was on the evaluation of the structural reliability of steel and concrete structures and/or components (Holicky and Retief, 2005; Afolayan and Opeyemi, 2010). The performance of timber structures during recent extreme occurrences, such Hurricane Andrew and the Northridge earthquake, has drawn a lot of attention to the dependability of wood frame assemblies and components under both normal and unintentional loads (Folz and Filiatrault, 2001; Christopoulos *et al.*, 2002; Rosowsky, 2002; Rosowsky and Ellingwood, 2002; Afolayan, 2003; Paevere *et al.*, 2003; Van de Lindt, 2005).

Studies on the adequacy of the current design codes, that is based on limit state design philosophy is being assessed, as many attempts have been made to calibrate these new generation codes (Ranta-Maunus *et al.*, 2001; Ranta-Maunus and Toratti, 2002; Ranta-Maunus, 2004; Thomos and Trezos, 2006).

2.0 METHODOLOGY

The typical steel portal frame considered in this study is presented in Table 1 below.

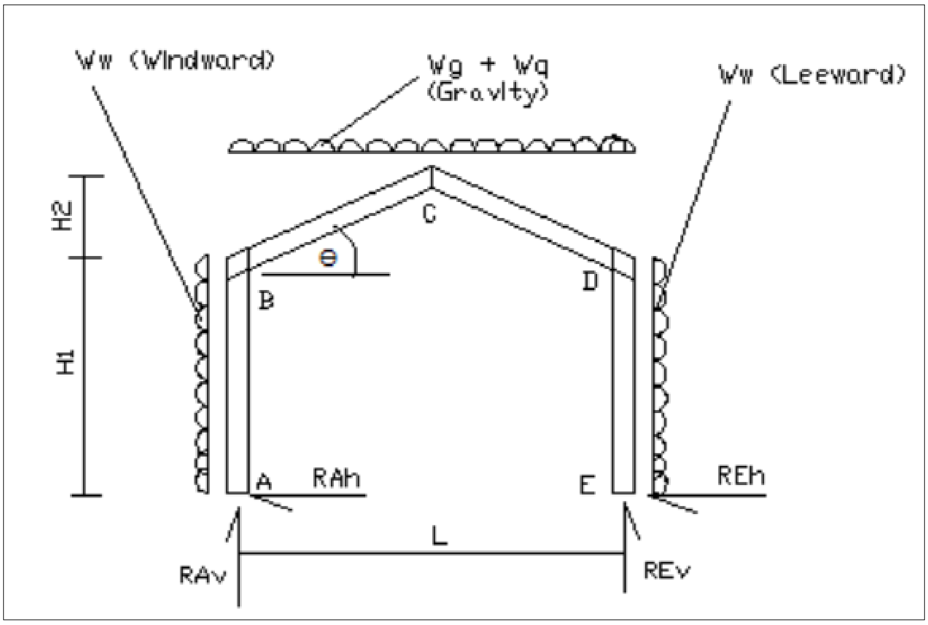

Figure 1: Typical Steel Portal Frame (Sýkora, 2002)

For reliability analysis, Three modes of failure are to be considered for the three-hinged portal frame, which includes:

i. Failure due to flexural instability of the frame columns (failure mode 1).
ii. Failure due to flexural instability of the frame rafters (failure mode 2).
iii. Failure due to lateral torsional instability of the frame columns/rafters (failure mode 3).

2.2 Limit State Functions of the Failure Modes

Each mode of failure must have its limit state function developed in order to conduct structural reliability analysis. The resistance component and the load effect component are the two parts of the limit state functions. While the load impact models are developed from structural analysis, the resistance models are developed from design code specifications. Because of the three possibilities of failure, the limit state function has the following general form.

2.2.1 Limit State Function for the Flexural Instability of the Column

The limit state function for the axial compression of the column according to Eurocode 3 BS:EN:1993-1-1 (2005) is given by equation 3.

G(X)-I = $\frac{f_y}{\gamma_m} X_c A_C - N_C$ …………………… (3)

Where f_y is the yield strength of the steel section which depends on the steel grade. In this research, steel grade S275 is used, therefore the yield strength is 275N/mm^2, A_C is the cross-sectional area of the steel column section which is obtained from the relevant table for dimensions and properties of steel sections, γ_m is the partial safety factor the steel material property. X_c is the reduction factor due to column flexural buckling. N_C is the applied axial load in the frame column which is defined as the critical vertical support reaction.

2.2.2 Limit State Function for the Flexural Instability of the Rafter

The limit state function for the axial compression of the rafter according to Eurocode 3 BS:EN:1993-1-1 (2005) is given by equation 4.

G(X)-II = $\frac{f_y}{\gamma_m} X_c A_R - N_R$ …………….. (4)

Where f_y is the yield strength of the steel section which depends on the steel grade. In this research, steel grade S275 is used, therefore the yield strength is 275N/mm^2, A_R is the cross-sectional area of the steel rafter section which is obtained from the relevant table for dimensions and properties of steel sections, γ_m is the partial safety factor the steel material property. X_c is the reduction factor due to column flexural buckling. N_R is the applied axial load in the frame rafter which is defined as the critical rafter axial force.

2.2.3 Limit State Function for the Lateral Torsional Instability for the Columns/Rafters

The limit state function for the flexural buckling of the column is given by equation 5 as follows:

G(X)-III = $\frac{f_y}{\gamma_m} X_{LT} W_C - M_C$ ………………… (5)

Where f_y is the yield strength of the steel section which depends on the steel grade. In this research, steel grade S275 is used, therefore the yield strength is 275N/mm^2, W_C is the section modulus of the column section, γ_m is the partial safety factor of the steel material property. X_{LT} is the reduction factor due to column lateral torsional buckling. M_C is the applied bending moment in the column which is defined as the critical rafter-column joint moment for a three-hinge steel portal frame. The columns and rafters are assumed to be of the safe section, subjected to the same critical moment. Hence failure mode 3 applies to both the columns and the rafters.

2.3 Determination of the Stochastic Models of Uncertain Parameters

Each of the column and rafter failure modes is a function of many deterministic and uncertain variables. To carry out structural reliability analysis, the stochastic models of the uncertain variables have to be determined. The stochastic models include the following.

i. The mean value of each uncertain variable
ii. Standard deviation
iii. Coefficient of variation
iv. Probability distribution model

The stochastic models in this study are based on data reported in the literature. The mean value of the loading and the material properties are taken as the default design values of the portal frame. The coefficient of variations as well as the probability distribution models were obtained from the literature. With the coefficient of variation, the standard deviation for each variable is obtained as the product of the coefficient of variation and the mean. With this, Table 1 below presents stochastic models as assembled for each basic variable.

Table 1: Variables of Steel Portal Frame

Basic Variable (V)	Coefficient of Variation	Unit	Distribution Model	Reference
Area of section (A)	0.05	mm^2	Normal	Steinmetz *et al.*, (2023)
Axial Load Action (N)	0.05	N	Normal	Sýkora (2002)
Bending Moment (M)	0.1	N	Normal	Sýkora (2002)
Section Modulus (W)	0.05	mm^3	Normal	JSCC (2001)
Coefficient of Lateral Flexural Buckling (X_c)	0.06	-	Normal	Sýkora (2002)
Coefficient of Lateral Torsional Buckling (X_{LT})	0.05	-	Normal	Sýkora (2002)
Steel Yield Strength (f_y)	0.07	N/mm^2	Lognormal	JSCC (2001)

2.4 Evaluation of the Limit State Function

The limit state functions for the various modes of failure could be evaluated to determine the probability of failure mode and the corresponding safety indices. The probability of failure for each limit state function can be evaluated by considering the solution of generalised convolution integral as follows:

$P_f = P(g(x_1, x_2, \ldots x_n) = \iint f_x(x)dx$ ….. (6)

Where $x_1, x_2, \ldots x_n$ are the uncertain variables, f(x)dx is the joint probability density function of the basic design variables.

The closed-form solution of equation (6) to determine the probability of failure is hardly possible because of its complexity as it contains the joint probability density function of the uncertain variables which makes the equation complex. Alternatively, an approximate solution such as First order reliability method is available and used in this study.

First first-order reliability Method (FORM) was adopted in this research based on its effectiveness in incorporating uncertainties in material properties, loads, and geometrical parameters. It also provides insights into the sensitivity of failure probability to various parameters and is computationally less intensive than other methods.

Generally, a structural reliability problem can be formulated as an optimization problem with the compact notation as shown in equations 7 and 8:

Minimize $\beta = ||\mu||^2 = \mu^T - \mu$(Objective function) …. (7)

$g(\mu) = 0$ ………………………… (8)

Where μ is the vector of standard normal variates; g(μ) is the limit state function; β is the reliability index.

The problem in Equation 6, is a constrained nonlinear optimization problem. In this research, the First Order Reliability Method (FORM) will be utilized in the evaluation of the limit state function.

The flow chart for the reliability analysis using the First Order Reliability Method is presented in Figure 2.

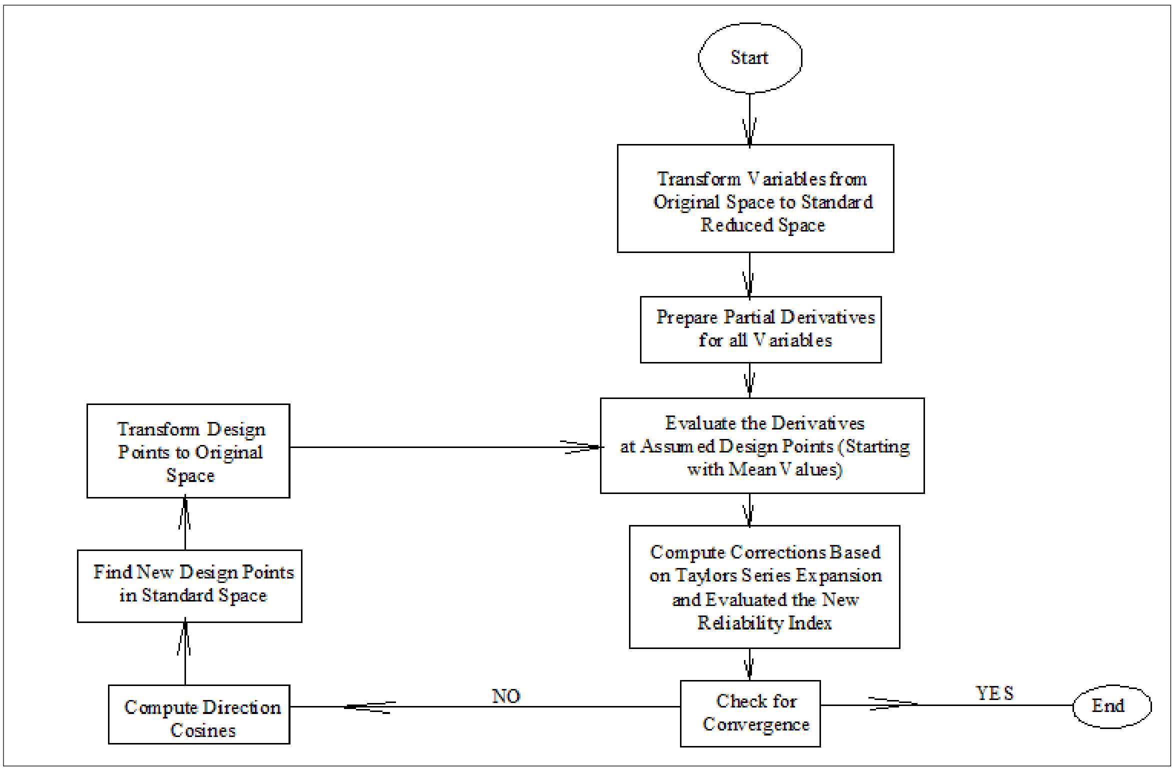

Figure 2: Structural Reliability using First Order Reliability Method

The computer program was developed based on the FORM algorithm in Figure 2, using MATLAB for the reliability analysis of the steel portal frame.

3.0 RESULTS AND DISCUSSIONS

3.1 Results of the Sensitivity Analysis

Sensitivity analysis was carried out using the developed MATLAB program with the view to investigate the response of the safety of the portal frame to changes in the values of the design variables. The results are presented as follows;

3.1.1 Response of the Frame Columns and Rafters to Flexural Buckling

In Figure 3, the relationship between the safety index for flexural instability of the frame members is displayed. Failure case 1 is for the flexural instability of the frame column, while failure case 2 is for the flexural instability of the frame rafter. The variation of the safety indices is compared with a pre-defined target reliability index of 3.8 (β_{target} = 3.8) with reference to Eurocode 1990 (2004).

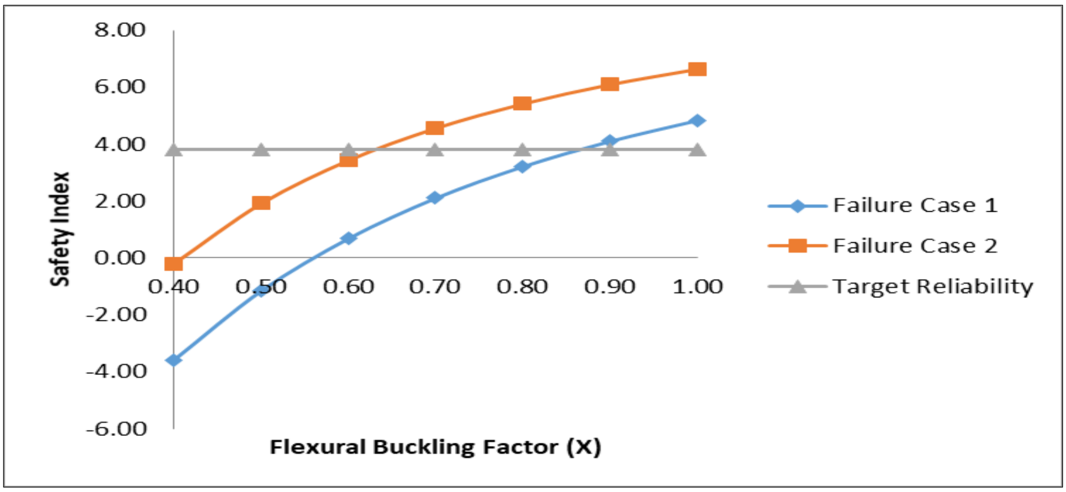

Figure 3: Safety Index for Failure Mode 1 and 2 against Flexural Buckling Coefficient

From the plot in Figure 3, at no flexural instability (no flexural buckling), that is when the flexural buckling factor X_c is 1.0, the safety indices for the column and the rafter are respectively equal to 4.5 and 6.8, which are all above the pre-defined target reliability index 3.8. The safety index then decreases exponentially with a decreasing value of X_c. The safety index of the frame column equalizes with the target safety index at $X_c = 0.85$ at which point the safety index of the frame rafter instability is 6.0. At X_c equal to 0.6, the safety index for the rafter flexural buckling is equal to the target safety index of 3.8 and at that point, the safety index of the column flexural buckling is 1.0. At the lower limit of $X_{c,}$ the safety indices for the column and the rafter modes of failure are respectively equal to -3.8 and 0. In a nutshell, so long as X_c is not below 0.85 for the column and 0.6 for the rafter, the Eurocode target reliability of 3.8 would be achieved.

3.1.2 Response of the Frame Columns and Rafters to Lateral Torsional Buckling

The variation of the safety index with the lateral buckling coefficient is presented in Figure 4. The plot was made alongside the Eurocode specified target reliability of 3.8.

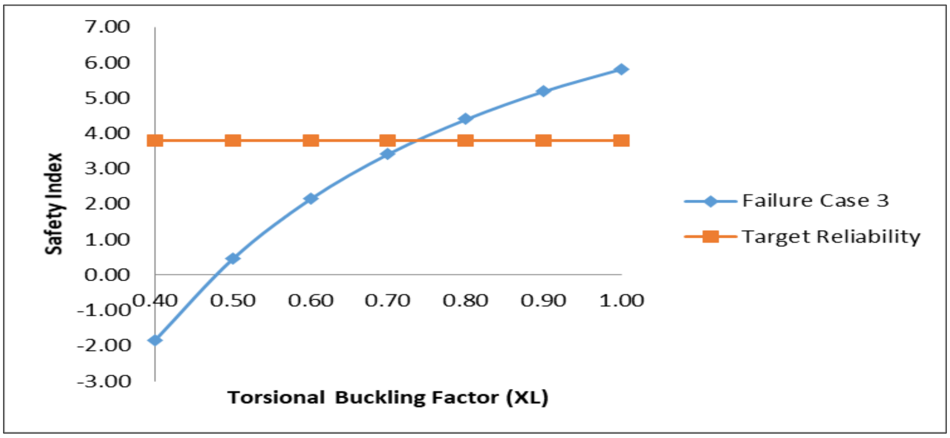

Figure 4: Safety Index for Failure Mode 3 against Torsional Buckling Coefficient

It is clear from Figure 4 that when the frame members are perfectly stable, implying zero buckling ($X_{LT} = 1.0$), the safety index for the lateral-torsional stability is 5.8 which is far above the target reliability index. However, as the value of X_{LT} reduced, the safety index kept decreasing but remained above the target safety index until X_{LT} was 7.4 when the implied safety index for the frame members' lateral torsional instability became equal to the target reliability index.

Comparing what was achieved in Figure 3 for the frame member flexural instability, it is clear that the target reliability was achieved when the buckling factor is 0.85 for the frame column and 6.2 for the frame rafter. This implied that the frame rafter flexural instability is the critical mode of failure of the frame member.

3.1.3 Effect of Permanent/dead to Variable Load Ratio on the Safety of the Portal Frame

The effects of permanent load to variable load ratio on the safety of the frame members are presented in Figures 5 to 7. The variable load is the combination of the imposed load and the wind load. The default design value of the imposed load is 1.5kN/m^2 and the default value of the wind load is 0.75kN/m^2. Likewise, the default design load ratio is 1.0 (Dead Load = 2.25kN/m^2). In the analysis the load ratio was varied around the default value and a range of 0.5 to 1.5 was considered. Likewise, three levels of buckling coefficient of 0.5, 0.7 and 1.0 were adopted.

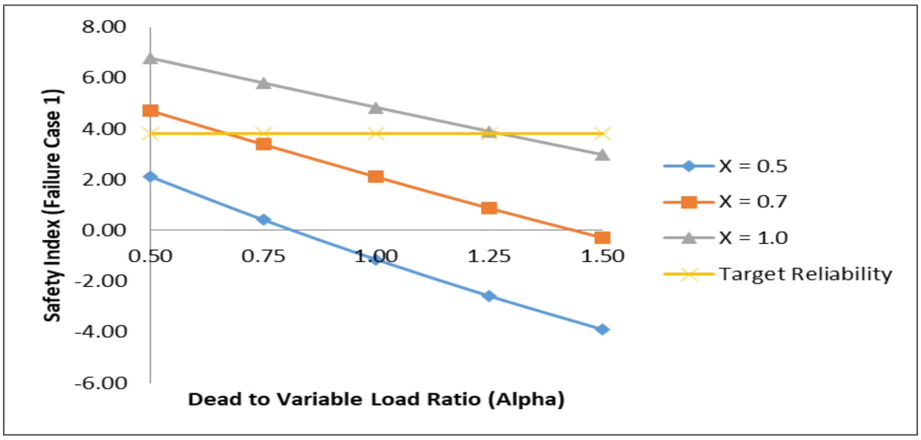

Figure 5: Effect of dead to Variable Load Ratio (Failure Mode 1)

It is clear from Figure 5 that the safety index for the failure mode 1, decreases with increasing dead to variable load ratio. At X_x = 0. 5 the safety indices within the considered range of load ratio are all below the target reliability index, when X_x = 1.0 target reliability index was met at 1.25 and below, at X_x =0.7 X_c = 0.7.

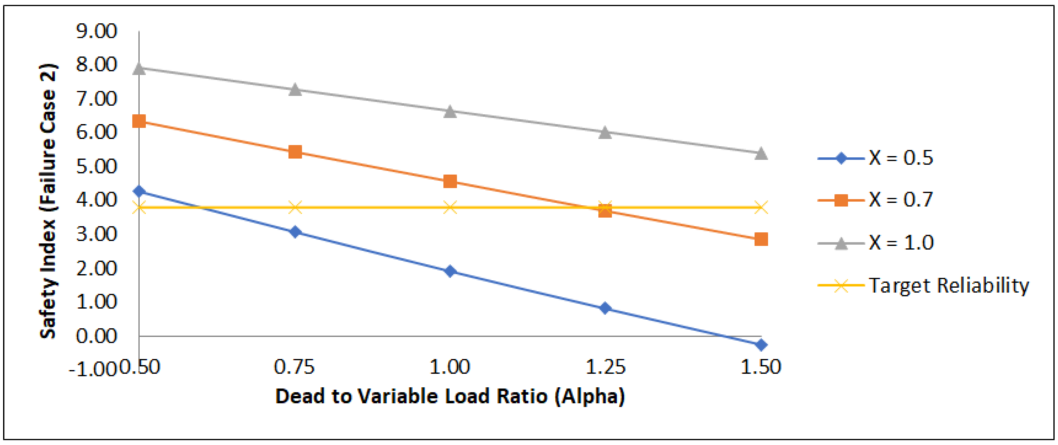

Figure 6: Effect of dead to Variable Load Ratio (Failure Mode 2)

A similar trend is observed for failure mode 2 in Figure 6. The safety index also decreases with increasing dead to variable load ratio. However, in this case, safety indices for all the considered load ratios when X_c is equal to 1.0, are all above the target reliability. However, when X_c is 0.7, target reliability is only achieved when load ratio is at 1.25 and below also, when X_c is equal to 0.5, the target reliability is achieved at 0.6. Comparing Figures 3 and 4 for flexural instability of the frame column and the frame rafter, the failure of the frame column due to flexural instability is critical.

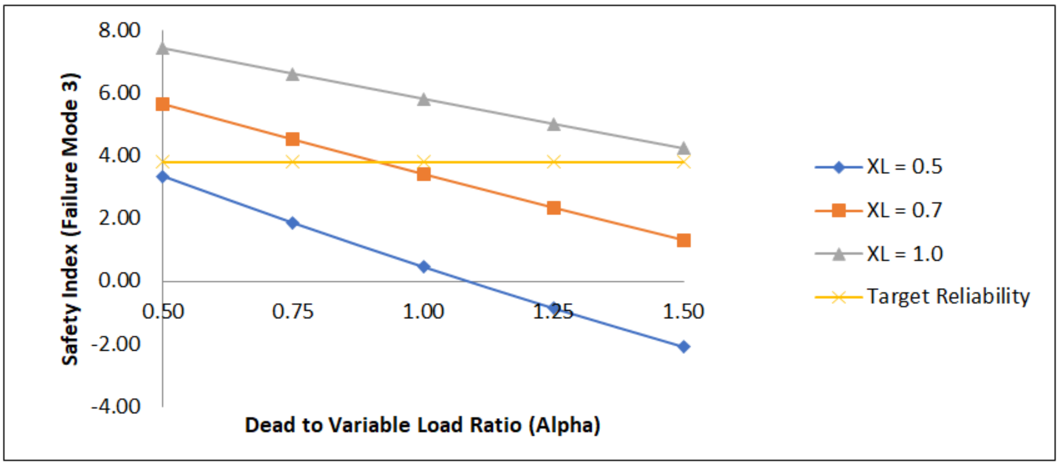

Figure 7: Effect of Dead to Variable Load Ratio (Failure Mode 3)

In Figure 7, it can be observed that the safety index reduces with an increase in dead-to-live ratio, at X_{LT}= 0.5 Safety indices for all the considered load ratios are above the target reliability, at X_{LT} = 0.7 the target reliability was met at 0.9 while X_{LT} = 1.0 safety indices for all the considered load ratio are below the target reliability in all cases.

Now comparing Figures 5, 6 and 7 and considering 1.0 dead to variable load ratio and cases of no buckling, the safety indices for the failure modes one, two and three are 5.0, 6.5 and 5.8. This implies that for frame members are stable against flexural and lateral-torsional instability, the member bending effect (failure mode three) is the critical failure mode.

3.1.4 Effect of Steel Grade on the Safety of the Portal Frame

The effects of steel grades on the flexural and lateral-torsional stability of the portal frame members are presented in Figure 8 for failure mode 1, Figure 9 for failure mode 2 and Figure 10 for failure mode 3.

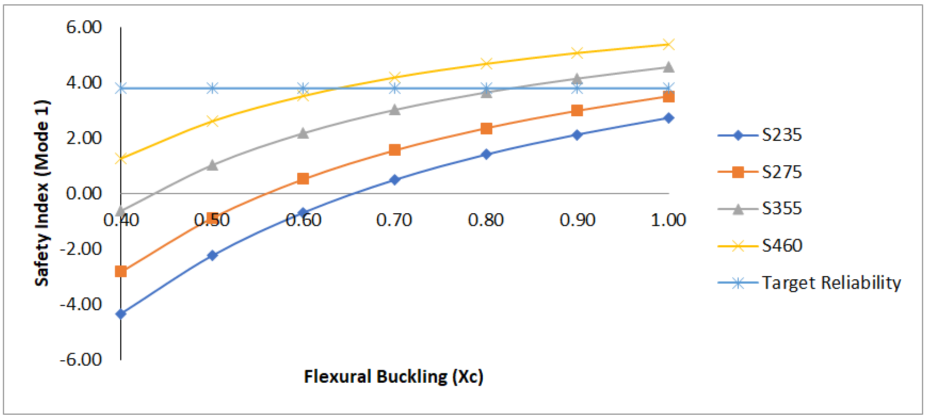

Figure 8: Effect of Permanent to Variable Load Ratio (Failure Mode 1)

Four steel grades were considered namely Grade S235, S275, S355 and S460. In Figure 8, the safety index for failure mode one increases non-linearly with increasing member's flexural buckling coefficient. The target safety index is not achieved for all values of buckling coefficients for steel grades S235 and S275. However, for S275 and the perfectly stable frame column, the implied safety index is very close to the target safety index. However, when higher steel grades are used, the reliability of the frame columns is enhanced. For instance for S355, target reliability is achieved even at the unset of flexural buckling with X_c equal to 0.85 and for S460, the value of X_c is equal to 0.65.

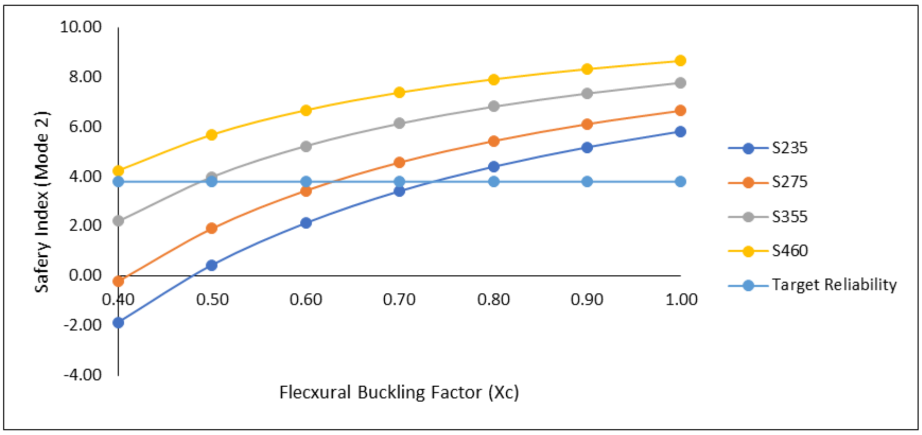

Figure 9: Effect of Permanent to Variable Load Ratio (Failure Mode 2)

In Figure 9, the safety index for failure mode two also increases non-linearly with increasing member's flexural buckling coefficient. The target safety index is achieved for all values of buckling coefficients for steel grades of S235 and S275 for perfectly stable frame rafters. However, the code-specified target reliability is only achieved when Xc is greater than 0.75, 0.62, 0.50 and 0.40 respectively for steel grades S235, S275, S355 and S460.

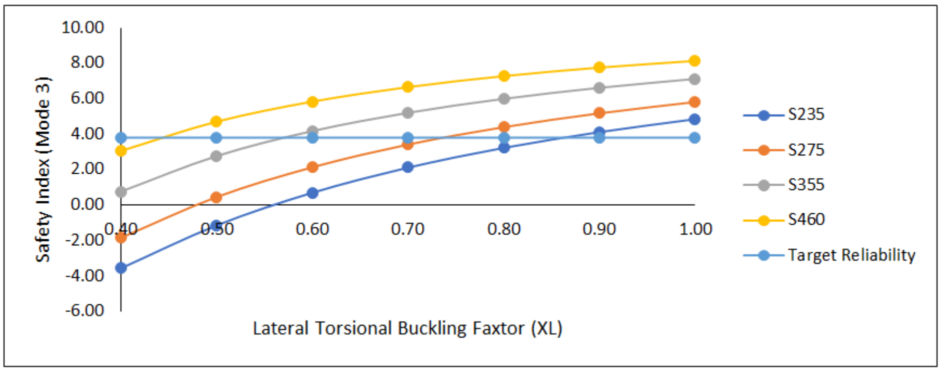

Figure 10: Effect of Permanent to Variable Load Ratio (Failure Mode 3)

The same trend was observed in Figure 9 was seen in 10 for failure mode 3. However, all four steel grades are adequate for both the column and the rafter at the condition of perfect stability (X_{LT} = 1.0). As X_{LT} decreases, the safety indices for all three steel grades decrease exponentially with a decrease in dead to variable load ratio. To achieve the target reliability using any of the four steel grades, X_{LT} must not be lower than 0.85, 0.74, 0.56 and 0.44 respectively for S235, S275, S355 and S460.

3.2 Effect of Axial Load and Moment Uncertainties on the Safety of the Frame Members

Both the axial loading and the bending moments are functions of the dead load, imposed load and the wind loads. The loading especially the imposed and the wind load are highly uncertain, this implied that the axial loads and the moments would also be highly uncertain and subject to random fluctuation. The investigation of the effect of uncertainties in the axial loading and the moments by considering the response of the safety indices to the variation of their coefficients of variations. Coefficient of variation ranging from 0.05 to 0.20 are considered.

The results of the sensitivity analysis of the frame members subject to load uncertainties are presented in Figure 11 to 13.

In Figure 11, the variation of the safety index for failure mode one with flexural buckling coefficient considering four level coefficients of variation of column axial load (Cov-N_{sd} = 0.05, 0.1, 0.15 and 0.20) are presented. For all the considered values of the axial load covariance, the safety index increases with increasing flexural buckling coefficient.

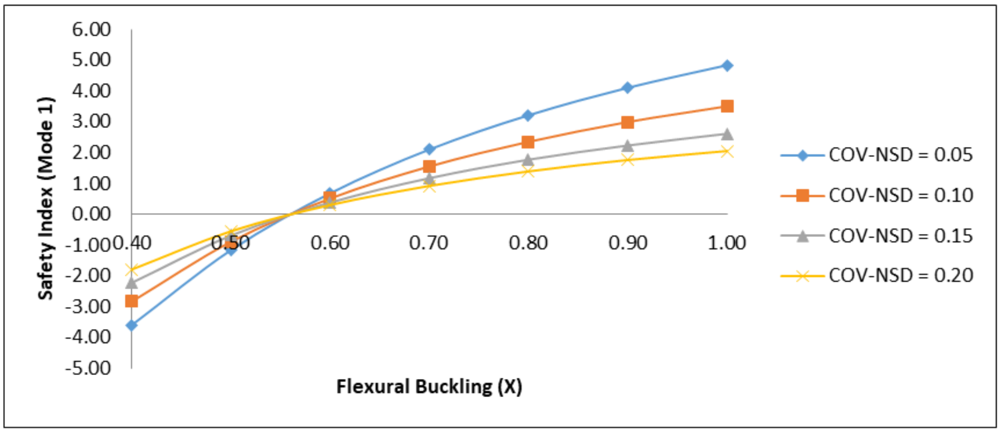

Figure 11: Effect of Uncertainty in Axial Load on the Safety of the Frame Members (Failure Mode 1)

From Figure 11, at the condition of perfect stability, that is when X_c = 1.0, the safety indices for axial load covariance of 0.05, 0.10, 0.15 and 0.20 are respectively equal to 5.0, 3.5, 2.5 and 2.0. This implied that an increase in load uncertainty from Cov-0.05 to Cov-0.10, Cov-0.15 and Cov-0.20 respectively decreases the column safety against flexural instability by 30%, 50% and 60%. It is also observed from the plot that the same level of safety 0.0 is attained irrespective of the axial load covariance when the flexural buckling coefficient is 0.55. Below X_c = 0.55, the column limit state is completely exceeded.

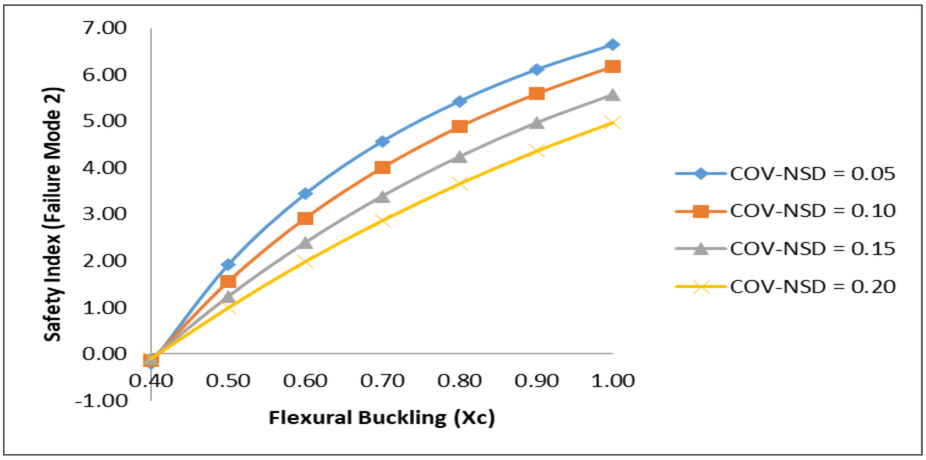

Figure 12: Effect of Uncertainty in Axial Load on the Safety of the Frame Members (Failure Mode 2)

The same trend observed in Figure 11 is seen in Figure 12 for failure mode 2. The safety index increases with increasing value of the frame rafter flexural buckling coefficient. At perfect stability condition (X_c = 1.0), the safety indices for the flexural instability of the rafter for axial loads covariance of 0.05, 0.10, 0.15 and 0.20 are respectively equal to 6.5, 6.2, 5.6 and 5.0. This is equivalent to a decrease in the rafter safety of 4.61%, 13.85% and 23.08% when the axial load coefficient of variation is increased from 0.05 to 0.10, 0.15 and 0.20. At X_l = 0.4, the safety index for the flexural instability of the rafter is 0 irrespective of the level of the axial load uncertainty.

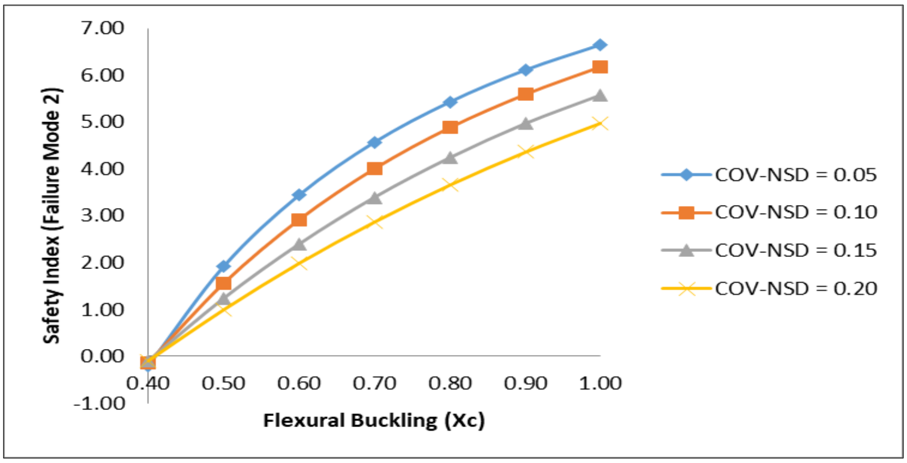

Figure 13: Effect of Uncertainty in Axial Load on the Safety of the Frame Members (Failure Mode 2)

The variation of the safety index with lateral torsional buckling coefficient for failure mode 3 is presented in Figure 13. As implemented in Figures 11 and 12 for the member flexural instability, the plots are for four levels of coefficient of variation of column/rafter moment. The considered coefficient of variation are 0.05, 0.10, 0.15 and 0.20. The trends are also approximately non-linear and the safety indices increase with increasing torsional buckling coefficient.

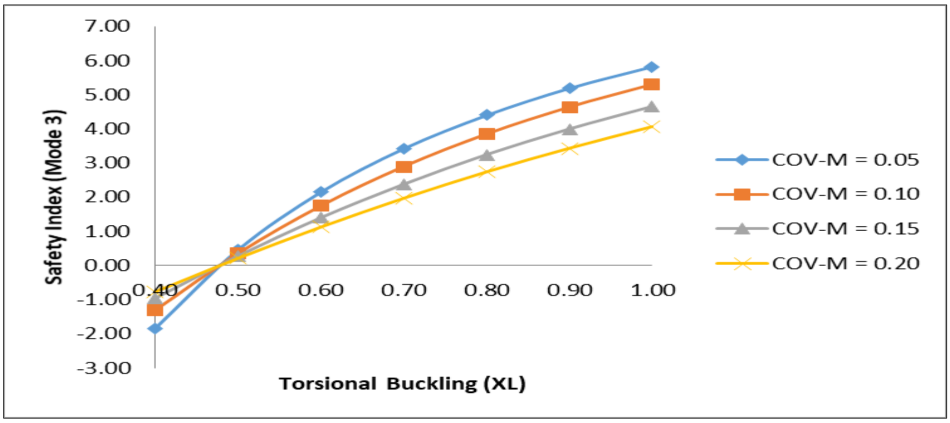

Figure 14: Effect of Uncertainty in Bending Moment on the Safety of the Frame Members (Failure Mode 3)

As observed in Figure 14, at perfect lateral torsional buckling stability (X_{LT} = 1.0), the safety indices corresponding to the coefficient of variation of 0.05, 0.10, 0.15 and 0.20 are respectively equal to 5.8, 5.2, 4.6 and 4.0. Therefore at this level, changes in coefficient of variation from 0.05 to 0.1, 0.15 and 0.20 will result in 0.12%, 0.21% and 31% drops in member structural safety against lateral torsional instability. At X_{LT} = 0.48, the safety index of the frame members irrespective of the load uncertainty is equal to 0.

Comparing Figures 11, 12 and 13 and considering for instance, a column that is stable against both flexural and lateral torsional buckling and considering coefficient of variation of 0.20, the safety indices for failure mode 1, failure mode 2 and failure mode 3 are respectively equal to 5.0, 6.5, 5.8. This implies that failure mode 1 is critical as far as loading uncertainty is concerned.

4.0 CONCLUSION

This study conducted structural reliability on steel portal frame based on Eurocode design criteria subjected to flexural and lateral-torsional instability using the first-order reliability method (FORM). The analysis, performed using the First Order Reliability

Method (FORM), evaluates safety indices for three failure modes and examines the effects of design parameters and uncertainties on structural reliability. The stochastic models of the basic design variables were obtained from the literature and duly acknowledged. The analysis was implemented using a developed computer program written using MATLAB.

The results of the analysis revealed that as long as flexural instability (X_c) is not below 0.85 for the column and 0.6 for the rafter, the Eurocode target reliability of 3.8 would be achieved. Furthermore, the safety index for the failure mode 1, decreases with increasing dead to variable load ratio. The Eurocode target reliability index was only achieved at a load ratio of 1.25 for $X_x = 1.0$, and 0.7 for $X_c = 0.7$, which when X_c is 0.5, the safety indices within the considered range of load ratio are all below the target reliability index.

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