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Research Paper

The effect of metal corrosion on the structural reliability of the Pre-Engineered steel frame

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A B S T R A C T

Nowadays, Pre-Engineered steel buildings are widely used in the field of the industrial construction. However, design standards often only care about the safety (or reliability) at the start time but not concerned about the deterioration of reliability during used under the metal corrosive of environment. Meanwhile, reliability and durability of steel structure depend heavily on metal corrosion of environmental, this is uncertainty parameters. In this research presents the effect of the safety of Pre-Engineered steel frames considering metal corrosion. The metal corrosion modeling used to propose by M.E. Komp. Reliability of the structure is evaluated using Monte Carlo simulation method and Finite Element Method (FEM). This computer program is written by using the MATLAB programming language. The results numbers are reliability and durability behaviors under corrosion are determined for exposure about from 10 - 50 years. Effects of input parameters are also investigated.

1 Introduction

The phenomenon of metal corrosion has a great impact on social, economic and buildings. According to statistics [1] the phenomenon of metal corrosion causing damage to the economy of various countries accounted for up to 3,0% for maintenance and replacement of structures damaged. Moreover, the phenomenon of metal corrosion also impact the deterioration of appearance produces the reduced value of constructions.

The effect of metal corrosion on the structural reliability this topic is also interesting to the researchers all over the world. In 2005, E.M. Robert has announced the effect of corrosion on the structural reliability of steel offshore structures this studied considers essential theoretical concepts and data requirements for engineering structural reliability assessment suitable for the estimation of the safety and reliability of corroding ships, offshore structures and pipelines [2]. Meanwhile, studied the corrosion behaviour of one-fifth scale lid models of transport cask submerged in sea bottom by K. Akio, in this studied

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corrosion tests have been performed for one-fifth scale lid models of typical transport cask for radioactive materials and for other crevice corrosion test specimens at a sea bottom for maximum 6 years [3]. Also, in 2005 K. Zen has announced corrosion and life cycle management of port structures[4]and K. Hiroshi et al. presented studied a corrosion prediction method for weathering steels [5].The application corrosion modelling proposed by M.E. Komp [6], in 2010, R. Landolfo has announced modelling of Metal Structure Corrosion Damage: A State of the Art Report. In this paper, the modelling approaches of atmospheric corrosion damage of metal structures, which are available in both ISO standards and the literature, are presented. A comparison among selected degradation models is shown in order to evaluate the possibility of developing a general approach to the evaluation of thickness loss due to corrosion [1]. Meanwhile, M. Seccer et al. presented studied corrosion damage analysis of steel frame considering lateral torsional bucking in [7].

Impact assessment of corrosion on the structural mentioned in the design standards of some countries and EN ISO: 9223, these recommendations depend on each standard. The European structural design codes [8-10]. The standards and code references there have been recommendations for design structures. However, there is not a specific process with quantitative goals aimed at determining the reliability and durability during used.

Concerning the reliability assessment of the structure this topic is also interesting to the researchers. In 2009, A. Omishore and Z. Kala assessed reliability analysis of steel structures with imperfections, in this studied the verification of the design reliability of a steel element according to the concepts of standards EUROCODE 3 and EN1990 [11]. The same method was used in [12] S. Afshan et al. to study reliability analysis of structural stainless steel design provisions. In 2015, K. Ivana to study the reliability analysis of steel frame structure with permanent load, variable load, and wind. Reliability calculation is carried out by the FOSM. When studied the structural reliability analysis of steel plane frames with semi-rigid connections B. M. Agostini et al used to First Order Reliability Method (FORM) to analyze reliability. Meanwhile, T.H Nguyen assessed reliability assessment of frame steel considering semi-rigid connections used to Monte-Carlo simulation for reliability assessment of frame steel under the internal force of semi-rigid joints when random input variable [13-15]. However, in our knowledge, reliability assessment of steel frame under metal corrosion environmental is still less studied.

This paper intends to study the effect of the safety of Pre-Engineered steel frames considering metal corrosion. In this research, the metal corrosion modelling used to propose by M.E. Komp [6]. Reliability of the structure is evaluated using Monte Carlo simulation method. This program computer is written by using the MATLAB language. The results numbers are reliability and durability behaviours under corrosion are determined for exposure about from 10 - 50 years. Effects of input parameters are also investigated.

2 Monte Carlo simulation method and corrosion modeling

2.1 Monte Carlo simulation

Monte Carlo simulation method is based on the use of pseudo-random numbers and the law of large number to assess the reliability of any system. If the safe domain is defined by the condition $f(\boldsymbol{X}) > 0$, where $\boldsymbol{X}$ is a random vector containing all the input random variables, the unsafe probability of the system is determined by:

$$P_f = \int I_{f(\mathrm{X})<0} f_{\mathrm{X}}(x)\, dx = E\left[I_{f(\mathrm{X})<0} \right] \tag{1}$$

Where, $I_{f(\mathrm{X})<0}$ is the indicator function and is defined by.

$$I_{f(\mathrm{X})<0} = \begin{cases} 1 & \text{if} \quad f(\mathrm{X}) \le 0 \\ 0 & \text{if} \quad f(\mathrm{X}) > 0 \end{cases} \tag{2}$$

According to the theory of statistics, if we have N realizations of the random vector $\boldsymbol{X}$ by propagating the randomness, we obtain a sample of N realizations of the indicator function. The expected value of the indicator function can be approximately determined by taking the mean of the sample.

$$\widehat{P}_f = E\left[I_{f(\mathrm{X})<0} \right] = \frac{1}{N} \sum_{i=1}^{N} I^i_{f(\mathrm{X})<0} \tag{3}$$

A 95% confidence interval of the estimation is defined by [16].

$$\widehat{P}_f\left(1-1,96\sqrt{\frac{1-\widehat{P}_f}{N\widehat{P}_f}}\right)\le P_f\le \widehat{P}_f\left(1+1,96\sqrt{\frac{1-\widehat{P}_f}{N\widehat{P}_f}}\right) \tag{4}$$

The reliability assessment by Monte Carlo simulation is established on MATLAB language and shown in Fig. 1

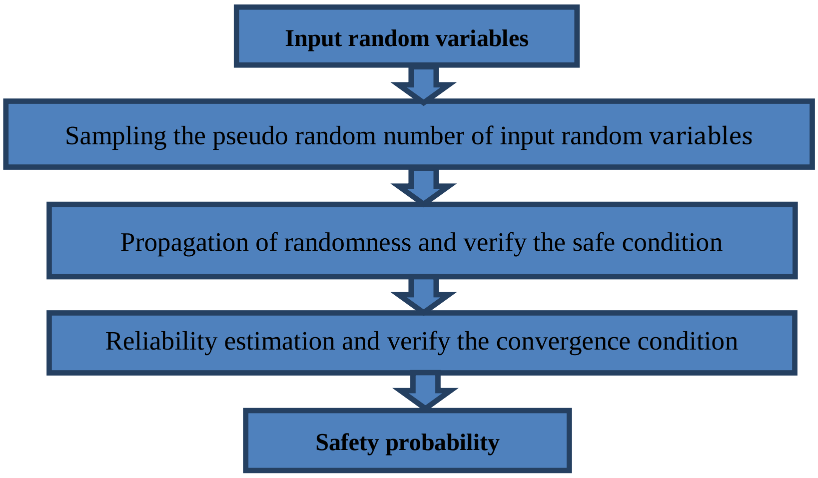

Fig. 1 - Flowchart of the reliability assessment by Monte Carlo simulation

2.2 Corrosion modeling

The corrosion modeling of steel structures in various environments intensively studied and proposed by M.E. Komp in [6]. Corrosion models usually describe the corrosion depth as a function of time in the form of a power model and are written as follows.

$$d(t)=At^{B} \tag{5}$$

Where, $d(t)$ is corrosion depth $\left(\mu m,\ {}^{g}\!/\!{}_{m^2}\right)$, t is exposure time (*years*), A is corrosion rate in the first year of exposure, B is corrosion rate long-term decrease. Obtains from [6] A and B are parameters depend on the environment in which the structure is located and they presented in Table 1. The modeling in (5) and average values for corrosion parameters in Table 1 have reliable. It is used for many studies in the literature [1,7].

Table 1 - Average values for corrosion parameters *A* and *B* for Carbon steel and Weathering steel

Environment	Carbon steel		Weathering steel	
	A	*B*	*A*	*B*
Rural	34,0	0,65	33,3	0,50
Urban	80,2	0,59	50,7	0,57
Marine	70,6	0,79	40,2	0,56

3 Geometric and effects of actions

3.1 Geometric data of the Pre-Engineered steel frame

The Pre-Engineered steel frames analyses in this studied as indicated in Fig. 2. The height of the column of frame is (*H*), the span of the frame is (*L*), the bay spacingis (B_1), the symbol of cross-section beam and column as shown in Table 2 andthe slope of the roof is approximately $\alpha = 5,71^0$.

Table 2. Geometric data of deterministic variables for Pre-Engineered steel frames

Properties	Variables	Name of basic Variables	Units	Values	Distribution
Geometric data	L	Girder span	m	24,0	D*
	H	Eave height	m	10,0	D
	B_1	Bay spacing	m	6,0	D

D* = Deterministic values

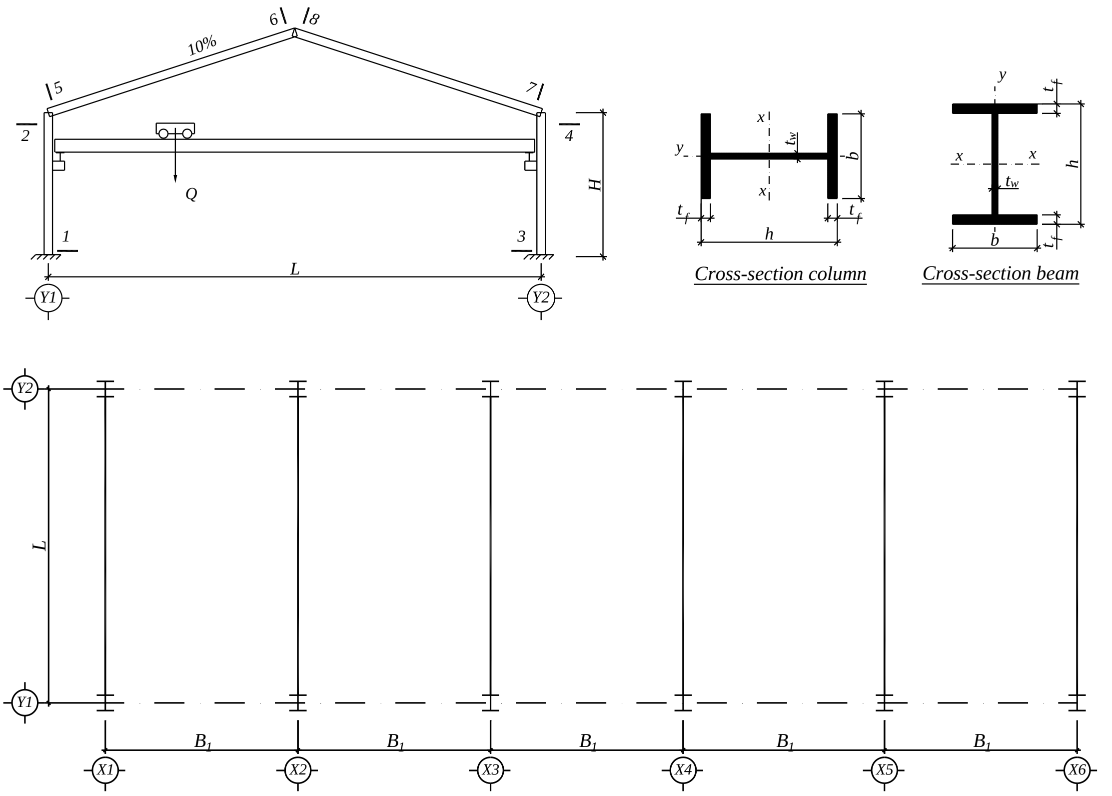

Fig. 2 – Geometry of the Pre-Engineered steel frames

3.2 Effects of actions and load combinations

The Pre-Engineered steel frame is exposed to the self-weight of the load-bearing girders and the roof (*DL*), the roof live loads (*LL*), wind action (*WL*) and overhead crane load (*CR*). The effect of other load actions is negligible. The load combinations in this study given in Section 9 of [17], the following four load combinations are considered for Pre-Engineered steel frames in this study and then it follows that.

$$\begin{aligned} &1.\ DL + LL \\ &2.\ DL + WL \\ &3.\ DL + CR \\ &4.\ DL + CR + 0.5\ WL \\ &5.\ DL + CR + 0.5\ LL \end{aligned} \tag{6}$$

According to [17] dead load (*DL*) includes the self-weight of rigid frames and imposed dead load due to secondary elements like roof sheeting, purlins, insulation, etc. The roof live load depends on the tributary area of rigid frames. Refer to Table 3.1 and Section 3. The wind loads are determined in accordance with Section 5. Wind loads are governed by wind speed, roof slope, eave height and open wall conditions of the building. Wind design pressure *p* depends on importance factor

I_w, velocity pressure q and pressure coefficient GC_p as shown:

$$p = I_w q\left(GC_p\right) \tag{7}$$

where, velocity pressure q is evaluated as:

$$q\left({kN}/{m^2}\right) = 2.456V^2H^{2/7}10^{-5} \tag{8}$$

where, V is wind velocity (km/h), H is eave height, GC_p are given for Rigid Frames for transverse and longitudinal directions in Tables 5.4a and 5.4b or directly obtained from the summarized Tables 5.7a and 5.7b. I_w is importance factor obtains from Table 5.2a in [17].

According to [17] crane loads are determined using the crane data available from the crane manufacturer and in accordance with Section 6. Crane data includes wheel load, crab weight, crane weight, wheel-base, end hook approach and minimum vertical and horizontal clearances. Wheel load for top running crane in this study has been 2 end truck wheels at one end of bridge.

$$CR_{WL} = 0.25BW + 0.5\left(RC + HT\right) \tag{9}$$

In (9) CR_{WL} is maximum wheel load, RC is rated capacity of the crane, HT is weight of hoist with trolley and BW is bridge weight. In Section 6.5 of MBMA a detailed procedure of crane beam analysis has been provided. The crane beam reactions are then used as applied loads on the mainframe.

The effects of actions on the Pre-Engineered steel frame considered internal forces consist of axial force (*N*), shear force (*V*) and bending moment (*M*). In the study, the internal forces were determined using the Finite Element Method (FEM) and program computer is established in the MATLAB language.

4 The safety condition of the Pre-Engineered steel frames

In the study, the safety conditions of the Pre-Engineered steel frames are must satisfy simultaneously: The safety condition of cross-section beams, the safety condition of cross-section columns and displacement at the top of the connection beam and column. The cross-section beams and column in Pre-engineered steel frames are welded section, the design of beam and column on the base Euro codes - Design of steel buildings EC3-1-1, [9]. The safety conditions of the Pre-Engineered are rewritten as follows.

4.1 The safety condition of cross-section beams

The cross section resistance on the uniaxial bending obtained from clause 6.2.5 of EC3-1-1 [9]are rewritten as follows.

$$M_{Ed}/M_{c,Rd} \leq 1,0 \tag{10}$$

Where, $M_{c.Rd} = W_{pl} f_y / \gamma_{M0}$

Bi-axial bending obtained from clause 6.2.9 of EC3.1.1, [9] are rewritten as follows.

$$\left[M_{y,Ed}/M_{pl,y.Rd}\right]^2 \leq 1,0 \tag{11}$$

The cross section resistance on the shear obtained from clause 6.2.6 of EC3-1-1, [9] are rewritten as follows.

$$V_{Ed}/V_{c,Rd} \leq 1,0 \tag{12}$$

Where, plastic resistance $V_{pl.Rd} = A_v\left(f_y/\sqrt{3}\right)/\gamma_{M0}$ or elastic resistance $V_{pl.Rd} = \dfrac{V_{Ed}.S}{I.t.\, f_y / \sqrt{3}\gamma_{M0}} \leq 1,0$; A_v is shear area obtained from clause 6.2.6 (3) of EC3-1-1 or from tables of profiles.

Bending and Shear Interaction obtained from clause 6.2.8 of EC3-1-1. In the study, the cross-section beam is no reduction.

$$V_{Ed} \leq 50\% V_{pl,Rd} \tag{13}$$

For I and H cross-sections of equal flanges, with bending about the major axis, the bending moment resistance $M_{y,V,Rd}$ is given by (clause 6.2.8 of EC3-1-1).

$$M_{y,V,Ed} = \left(W_{pl,y} - \frac{\rho A_w^2}{4t_w}\right)\frac{f_y}{\gamma_{M0}} \leq M_{y,c,Rd} \quad \text{where} \quad A_w = h_w t_w \tag{14}$$

Lateral-torsional buckling resistance (clause 6.3.2 of EC3-1-1)

$$\frac{M_{Ed}}{M_{b.Rd}} \leq 1,0 \quad \text{where} \quad M_{b.Rd} = \chi_{TL} W_y f_y / \gamma_{M1} ; \tag{15}$$

Where, For I and H cross-sections $W_y = W_{pl.y}$

χ_{TL} is the reduction factor for lateral-torsional buckling, which can be calculated by one of two methods, depending of member cross-section.

4.2 The safety condition of cross-section columns

In this study, the design of cross-section columns base on EC3-1-1 [9], cross section resistance clause 6.2.9 of EC3-1-1 as shown.

$$M_{Ed} \leq M_{N,Rd} \tag{16}$$

Where, the section is double-symmetric I as shown: $M_{N,y,Rd} = M_{pl,y,Rd}\dfrac{1-n}{1-0.5a}$

but

$$M_{N,y,Rd} \leq M_{pl,y,Rd}.$$

$$M_{N,y,Rd} = M_{pl,y,Rd} \quad \text{if} \quad n \leq a$$

$$M_{N,y,Rd} = M_{pl,y,Rd}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \text{if} \quad n > a$$

Here,

$$n = N_{Ed} / N_{pl,Rd} \quad \text{and} \quad a = \left(A - 2bt_f\right) / A \leq 0,50$$

Member stabilitywith high slenderness subjected to bending and compression, may fail by flexural buckling or lateral-torsional buckling. Flexural buckling and lateral-torsional buckling (doubly-symmetric cross-section) and according to Eq. 6.61 of EC3-1-1 was rewritten for the Pre-Engineered steel frame.

$$\frac{N_{Ed}}{\chi_y N_{Rk} / \gamma_{M_1}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M_1}} \leq 1,0 \tag{17}$$

k_{yy} is interaction factors obtained from Table B.3 of EC3-1-1; Calculation of the χ_{LT} using the alternative method applicable to rolled or equivalent welded sections (clause 6.3.2.3 of EC3-1-1)

4.3 The safety condition of displacement at the top of the connection beams and columns

The safety condition of displacement at the top of the connection beams and columns on base commentary L4 of AISC 360-10, [18]& Commentary Appendix C. CC.1.2 of ASCE 7-10 and shown as.

$$\Delta_{b-c} \le [\Delta] = H/300 \tag{18}$$

5 Reliability of Pre-engineered steel frames under metal corrosion

5.1 The safety condition

As mentioned above, the safety conditions of the Pre-Engineered steel frame are must satisfy simultaneously: The safety condition of cross-section beam, the safety condition of cross-section columns and displacement at the top of the connection beam and column and is written as follows.

$$G(\boldsymbol{X}) = \begin{cases} \left(M_{Ed}/M_{c,Rd} - 1{,}0 \right) 1/n \le 0 \\ \left(\left[M_{y,Ed}/M_{pl,y.Rd} \right]^2 - 1{,}0 \right) 1/n \le 0 \\ \left(V_{Ed}/V_{c,Rd} - 1{,}0 \right) 1/n \le 0 \\ \left(\left(W_{pl,y} - \dfrac{\rho A_w^2}{4t_w} \right) \dfrac{f_y}{\gamma_{M0}} - M_{y,c,Rd} \right) 1/n \le 0 \\ \left(\dfrac{M_{Ed}}{M_{b.Rd}} - 1{,}0 \right) 1/n \le 0 \\ \left(M_{Ed} - M_{N,Rd} \right) 1/n \le 0 \\ \left(\dfrac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M_1}}} + k_{yy} \dfrac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \dfrac{M_{y,Rk}}{\gamma_{M_1}}} - 1{,}0 \right) 1/n \le 0 \\ \left(\Delta_{b-c} - H/300 \right) 1/n \le 0 \end{cases} \tag{19}$$

Where, *n* is safety factor, this is used to adjust to increase the reliability of the structure at a time design.

5.2 Deterministic model and uncertainty model

Deterministic model of the Pre-Engineered steel frameon base the safety condition (19) , which the input variables are those of geometry properties (*L, H, B₁*), material properties the mechanical characteristics of the material are (f_y, E, G), actions effect (q_{rof}, W, Q), the relevant geometric characteristics cross-section (b, h, t_f, t_w) and corrosion depth coefficients in metal corrosion model (*A*, *B*). This deterministic model can be written in form with $\boldsymbol{X} = \left[L, H, B_1, f_y, E, G, q_{rof}, W, Q, h, b, t_f, t_w, A, B \right]$ is called safety conditions and as shown.

$$G(\boldsymbol{X}) = \Im(\boldsymbol{X}) \tag{20}$$

Uncertainty model is constructed based on the deterministic model by taking into account the randomness of some input variables. In this paper, we distinct two vector of input parameters: The first one of the parameters assumed to be deterministic

$\boldsymbol{X}_1=\left[L,H,B_1,h,b,t_f,t_w\right]$ and the second one of the parameters assumed to be random $\boldsymbol{X}_2(\omega)=\left[f_y(\omega),E(\omega),G(\omega),q_{rof}(\omega),W(\omega),Q(\omega),A(\omega),B(\omega)\right]$ with ω represents the randomness of the parameters. (A, B) are the empirical coefficients that are identified from experimental results because of their empirical origin, these coefficients possessing potential randomness. This model can be written in form.

$$G\left(\boldsymbol{X}(\omega)\right)=\Im\left(\boldsymbol{X}_1,\boldsymbol{X}_2(\omega)\right) \tag{21}$$

5.3 Reliability assessment by the combination of FEM, metal corrosion and Monte Carlo simulation

In this section, we are shown the flowchart reliability assessment of Pre-Engineered steel frames. The program computer developed based on finite element method (FEM), the Komp corrosion model and Monte Carlo simulation. From the flowchart, a programmed computer is established in the MATLAB language. The program computer will be used the reliability assessment of the input random variable for the Pre-Engineered steel frame considering metal corrosion about 0-year, 10-years, 20-years, and 50-years are obtained.

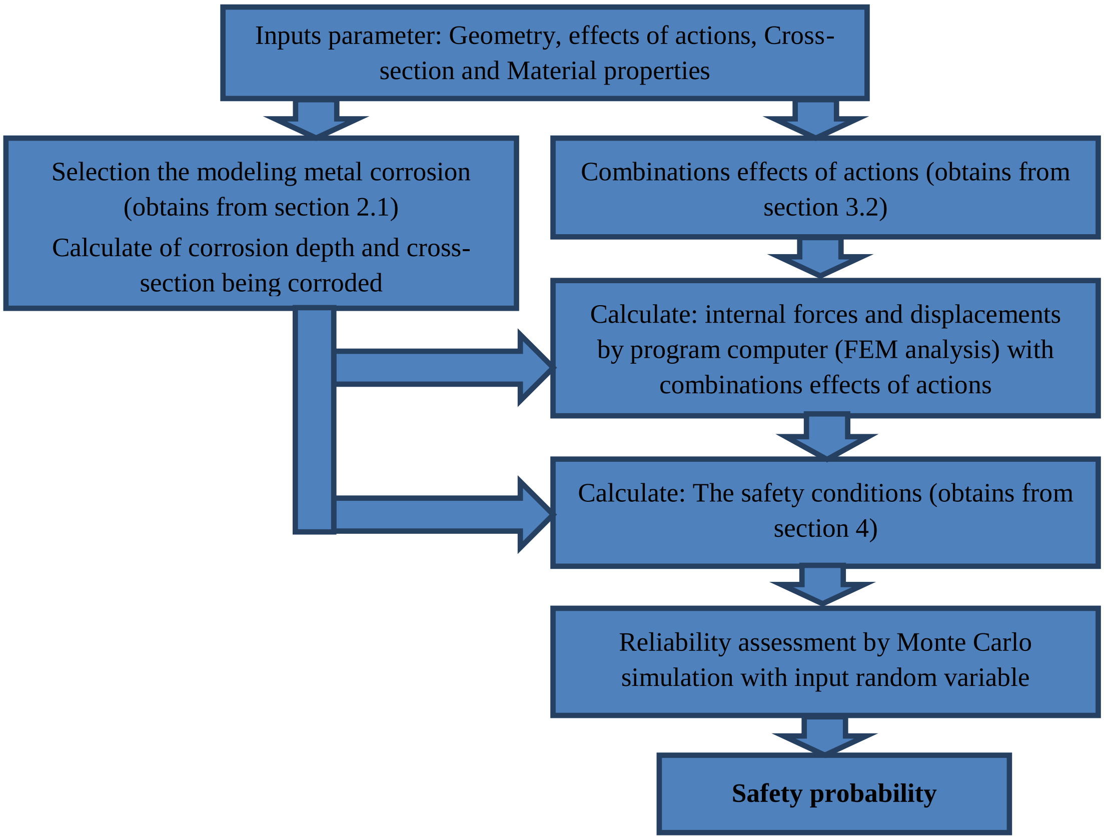

Fig. 3 – Flowchart reliability assessment by the combination of FEM, metal corrosion and Monte Carlo simulation

6 Numerical examples

6.1 Convergence of the Monte Carlo Simulation at a time design

In this section in the paper, the proposed procedure is applied for the reliability assessment of the Pre-Engineered steel frame at a time design (t = 0-year). Nominal and distribution of input random variable shown in Table 3. The nominal and distribution of material properties on based proposed in [19] and the cross-section and loading on based proposed in [20].

Table 3. Statistical properties of random variables for reliability assessment

Properties	Variables	Nominal	Mean/nominal	COV	Distribution	Reference
Material	f_y	355,0 (MPa)	1,10	0,06	Lognormal	[19]
	E	210,0 (GPa)	1,10	0,06	Lognormal	[19]
	G	81,0 (GPa)	1,10	0,06	Lognormal	[19]
Loading	DL	0.15 (kN/m^2)	1.05	0.10	Normal	[20]
	LL	1,0 (kN/m^2)	1.05	0.10	Normal	[20]
	WL	1,2 (kN/m^2)	0.92	0.37	Gumbel	[20]
	CR	2,5 (kN)	1.05	0.10	Normal	[20]
Cross-section beam	b	250,0 (mm)	1,00	0,05	Normal	[20]
	h	400,0 (mm)	1,00	0,05	Normal	[20]
	t_f	15,0 (mm)	1,00	0,05	Normal	[20]
	t_w	8,0 (mm)	1,00	0,05	Normal	[20]
Cross-section column	b	250,0 (mm)	1,00	0,05	Normal	[20]
	h	380,0 (mm)	1,00	0,05	Normal	[20]
	t_f	15,0 (mm)	1,00	0,05	Normal	[20]
	t_w	8,0 (mm)	1,00	0,05	Normal	[20]

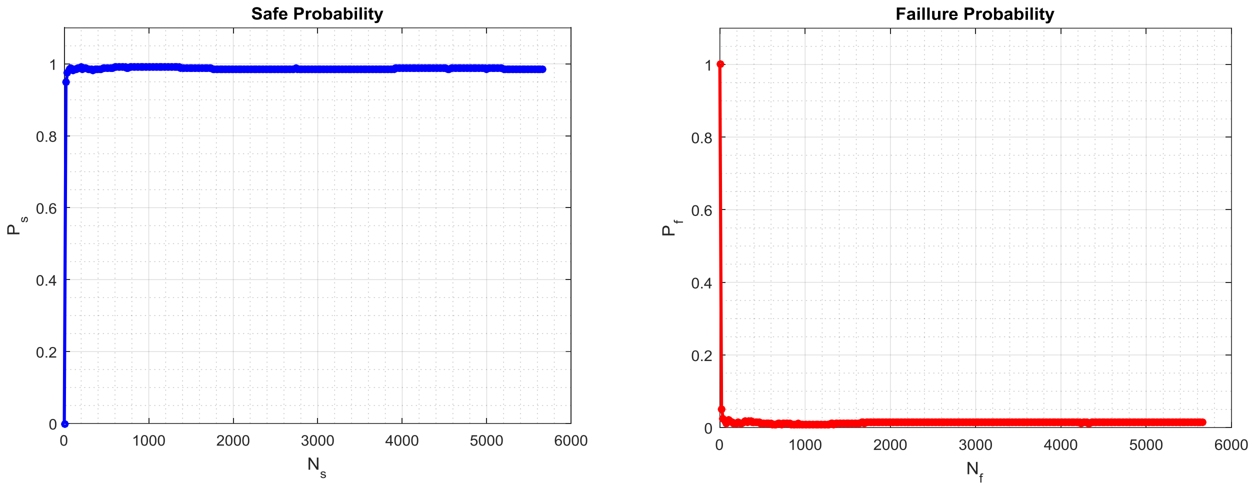

Fig. 4 – Convergence of the safety probability in the Monte Carlo simulation of the Pre-Engineered steel frames at a time design (t = 0-year)

Fig. 4shown the convergence of the safety probability of the composite steel-reinforced concrete beams at a time design (t = 0-year) in the Monte Carlo simulation to the value of 0,9860 or P_s = 98,60 (%) after about 5680 samplings in 62 minutes. The used convergence criteria of 2,0 (%) justify the confidence of the estimated reliability. This result also shows that although we have taken the safety factor is 1,10 in the analysis but because of the randomness of some input parameters, the reliability of the structure is only of P_s = 98,60 (%). The assessment of the reliability of the structure thus is necessary, especially considering metal corrosion over time.

6.2 Effect of metal corrosion on the safety probability of the Pre-Engineered steel frames

In this section, the study will be investigated of safety probabilities of the Pre-Engineered steel frames considering metal corrosion about 10- years, 20-years, and 50-years. The cross-section of the Pre-Engineered steel frames is considered due to the corrosion according to the Komp [6] model in the Urban Environment. The nominal and distribution of material properties on based proposed in [19] and the cross-section and loading on based proposed in [20]. The summaries of the safety probability by the Monte Carlo simulation of the Pre-Engineered steel frames considering metal corrosion about 10-years, 20-years, and 50-years are shown in Table 4 and Fig.5.

Table 4. The safety probability of the Pre-Engineered steel framesabout 10-years, 20-years, and 50-years

	0-year	10-years	20-years	50-years
Safety probability (%)	98,60	96,45	94,35	90,79

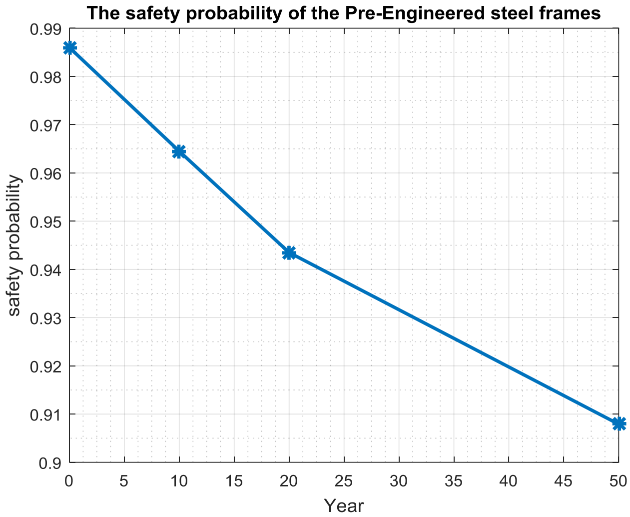

Fig. 5 – The safety probability of the Pre-Engineered steel framesabout 10-years, 20-years, and 50-years

From Table 4 and Fig. 5 shown that the safety probabilities of the Pre-Engineered steel frames considering metal corrosion about 10- years, 20-years, and 50-years by the Monte Carlo simulation to the value from 96,45 to 90,79 (%). The used convergence criteria of 2,0 (%) justify the confidence of the estimated reliability. It can be seen that the safe probability has a decrease compared to the time (t = 0-year) respectively 10-years is 2,15 (%), 20-years is 4.25 (%), 50-years is 7,81 (%).Relatively important numerical investigate results are the basis for adjusting the structure from at time design or forecasting durability of the structure during use.

7 Conclusion

This paper proposed an algorithm to assess the structural reliability of the Pre-Engineered steel frames considering the influence of metal corrosion. The numerical process is developed based on the corrosion model of Komp [6] and Monte Carlo simulation. A wide range of corrosive exposing time from 10 to 50 years is considered in the structural reliability assessment. The flowing conclusions are drawn based on numerical analyses. The proposed algorithm, which is numerically developed based on the Komp corrosion model and Monte Carlo simulation, FEM, the Pre-Engineered steel frames is capable of and corrosion effect. A variation of structural reliability with corrosively exposing timeis quantified. Overall, as time increased the probability of safety is reduced, and an important recommendation for the selection of the input random variables when adjusting the reliability structure at time design.

The developed procedure in this study can be applied for Pre-Engineered steel frames. An extended application for other types of other structures is highly feasible; however additional numerical tests and verifications are required.

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Conflicts of interest

The authors declare that they have no potential conflicts of interest in this paper.

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