

Research on decision and risk analysis for portal frame

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Abstract. As lightweight and high-strength structures, steel structures are gradually widely used in various buildings, such as high-rise buildings and industrial plants. After comparing various probability analysis methods, Monte Carlo simulation is chosen to analyze the moment of inertia design value for a non-sway steel frame. This paper mainly introduces the engineering load and reliability design principles of gantry steel frames. By consulting specifications and using relevant calculation formulas and basic parameters, different engineering load effects are discussed and calculated. After completing the load condition calculations, the resistance design value is calculated by using the limit state design principles. Finally, the sectional dimension design is conducted based on Monte Carlo simulation. By means of limit state design and Monte Carlo simulation, the ideal design value for calculating the moment of inertia can be easily calculated. Through these analyses, the superiority of Monte Carlo simulation in analyzing the failure probability of complex buildings is demonstrated. Unlike other methods, it does not need to compute functions of various factors in advance.

Keywords: Monte Carlo simulation; reliability analysis; limit state design; analysis of forces.

1. Introduction

In recent years, an increasing number of factories have started to use steel structures, which are recyclable, light-weight, and easy to assemble, as a form of large-span structures. The prefabricated steel structure building has the advantages of resource recycling, environmental friendliness, short construction period, good seismic performance and so on, which is more in line with the use requirements of green buildings [1]. This structure has the characteristics of large span and high floor height. Its design and construction differ from conventional frame structures. And currently, domestic researchers have conducted a significant amount of theoretical analysis and experimental research on lightweight portal frame structural systems. The research achievements have been widely applied in practical engineering. During the design and selection stage, it is necessary to discuss in advance the construction measures and various issues that may arise during construction and use. In reality, structures are subjected to various loads, and the magnitudes of these loads also fluctuate.

Structural reliability theory first appeared in the early 20th century, when probability theory was applied to study structural reliability [2]. Using the total probability method, Freudenthal [3] put forth fundamental structural safety issues under random load, which laid the groundwork for structural reliability theory. Zhao et al [4]. suggested that the first-order second-moment approach might be used

to study the structural safety coefficient. He then suggested numerical simulation methods for the structural reliability, including the hybrid simulation method, the response surface method, and Monte Carlo simulation. Based on these, more appropriate specifications of I-beams can be used to improve economic efficiency.

Also as an alternative, when designing (or checking) with direct techniques, sophisticated simulations may be used to directly estimate the strength of a part or a structure without the need for further tests. When applied to systems, the load redistribution capacity, redundancy, and robustness of the structures can be fully exploited, ensuring a more uniform reliability across different structural systems and potentially leading to lighter and more affordable designs [5]. Therefore, this paper needs to identify potential load conditions and combine them, and then perform reliability analysis using Monte Carlo simulation.

2. Engineering background

The chosen structure in figure 1 is a typical single-story residential house, often found in factory building. Details of the house's layout and construction materials are included in the report. The section data and parameters for I-beam sections are provided in figure 2.



Figure 1. Structure Diagram.

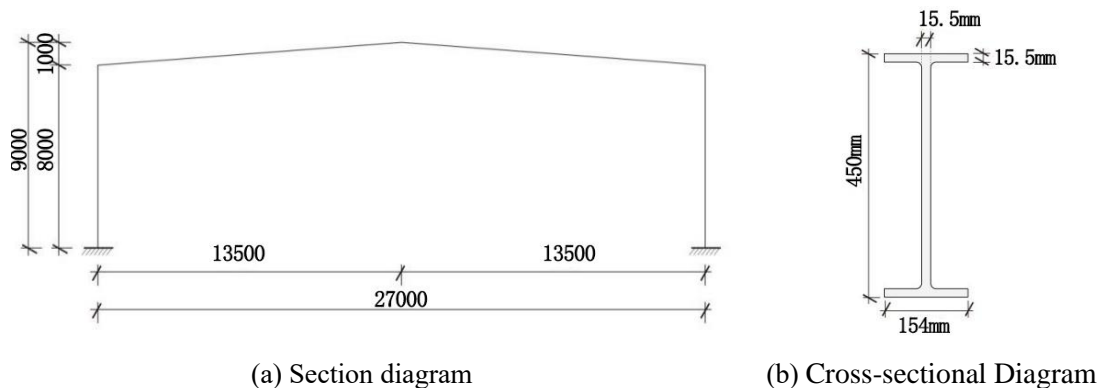


Figure 2. Parameters of the members.

3. Load conditions

In terms of mechanical analysis, the essay primarily use the method of moment distribution, and for reliability analysis, and also employ MATLAB for Monte Carlo simulations. The parameters found in the building code specifications are listed in table 1 [6].

Table 1. Basic Parameters.

Type of Load	Dead Load	Snow Pressure	Wind Pressure
Standard Value	0.5kN/m ²	0.5kN/m ²	0.35kN/m ²

3.1. Dead load and snow load

Snow load calculation formula is as below.

$$s_k = u_r s_0 \quad (1)$$

where u_r is Roof Snow Load Distribution Coefficient, and s_0 is basic snow pressure. Since the angle formed between the roof and the horizontal plane is significantly less than 25 degrees, the parameter u_r is taken as 1 in this case [6]. The snow load can be calculated by equation (1), and its value is equal to 0.5kN/m^2 . However, to simplify the issue, the paper only focus on one section in figure 2. Since the spacing of the portal frame columns is 7.5m , the uniformly distributed load should be converted into a uniformly distributed line load along the length of the inclined beam.

$$0.5\text{kN/m}^2 \times 7.5\text{m} = 3.75\text{kN/m} \text{ (Dead Load)} \quad (2)$$

$$0.5\text{kN/m}^2 \times 7.5\text{m} = 3.75\text{kN/m} \text{ (Snow Load)} \quad (3)$$

After they are superimposed, the force action situation is shown in figure 3. Regardless of the displacement in the horizontal direction, the bending moment distribution method was used. And after transmitting for two times, the bending moment diagram was drawn below in figure 4.

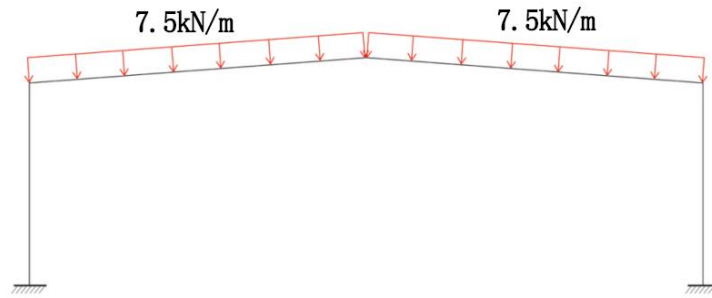


Figure 3. Diagram of snow load combined with dead load.

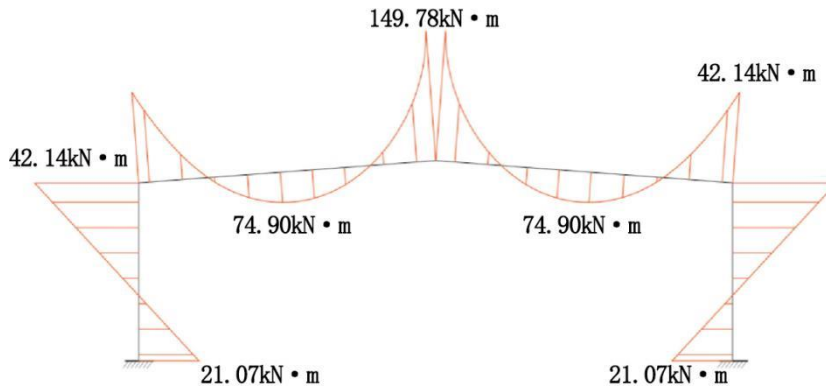


Figure 4. Bending moment diagram.

3.2. Wind load

In this paper, the wind is coming from the left of the structure. According to the code specifications, the wind can be calculated by equation (4), and the result is 0.3675kN/m^2 [7, 8].

$$w_k = u_s u_z w_0 \quad (4)$$

where u_z is the Wind Load Height Variation Coefficient and $u_z = 1$, and w_0 is the basic wind pressure, and $w_0 = 0.35\text{kN/m}^2 \times 1.05 = 0.3675\text{kN/m}^2$. The loads on members are calculated in table 2. The simplified diagram of wind load can be drawn in figure 5. In addition, using the bending moment distribution method, the bending moment diagram can be easily drawn in figure 6.

Table 2. Wind load^a.

Parameter	Left column	Right column	Left beam	Right beam
u_s	+0.25	-0.55	-1.00	-0.65
Wind load (kN/m)	+0.685	-1.516	-2.756	-1.792

^a Notes: Positive sign (pressure) indicates wind force acting from the outside towards the surface; negative sign indicates wind force moving away from the surface, indicating a load direction that is away from the component surface, i.e., generating suction.

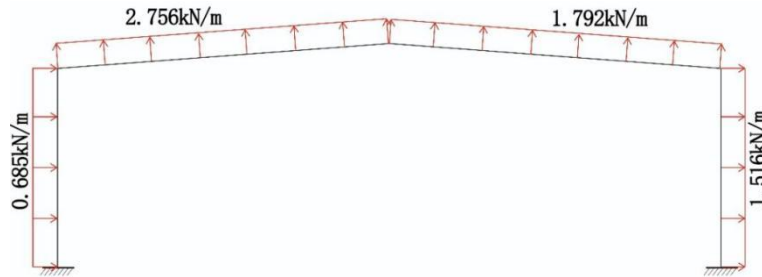


Figure 5. Diagram of wind load.

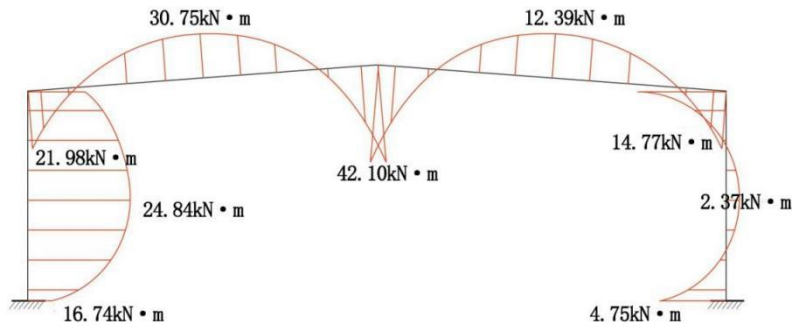


Figure 6. Bending moment diagram.

3.3. Crane load

To simplify the calculation, it is assumed that the maximum (or minimum) wheel pressure of the crane moves 0.5m horizontally along the axis of the rigid frame column, and the corresponding bending moment will be generated [9]. Table 3 shows the crane parameter. The value of the bending moment generated is $310 \text{ kN} \times 0.5\text{m} = 155 \text{ kN}\cdot\text{m}$, and $102\text{kN} \times (-0.5)\text{m} = -51\text{kN}\cdot\text{m}$. And the simplified diagram of crane load can be drawn in figure 7. As the former loads, the bending moment diagram of crane load can be drawn with bending moment distribution method in figure 8.

Table 3. Crane Parameters

Lifting Weight G_n (t)	Span L_k (m)	Weight of the Trolley g (t)	Total Weight G (t)	Maximum Wheel Pressure P_{\max} (kN)	Minimum Wheel Pressure P_{\min} (kN)
32	27	11.1	50	310	102

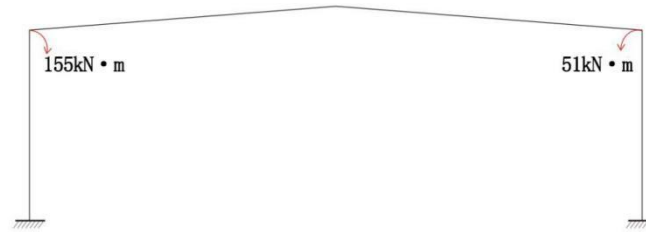


Figure 7. Diagram of crane load.

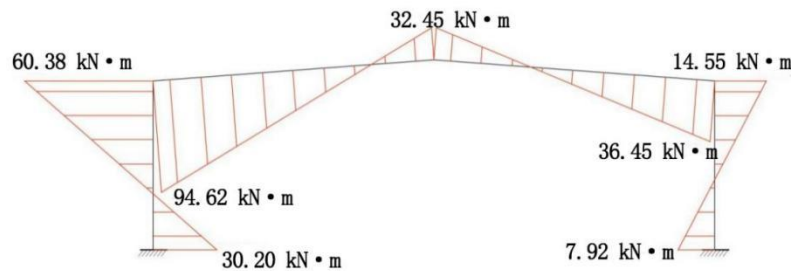
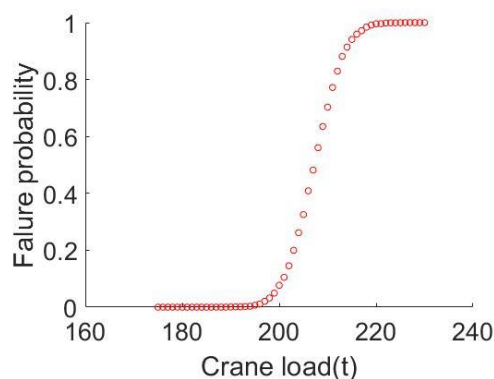


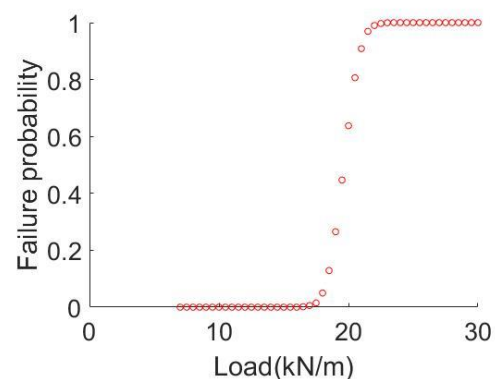
Figure 8. Bending moment diagram.

4. Reliability analysis

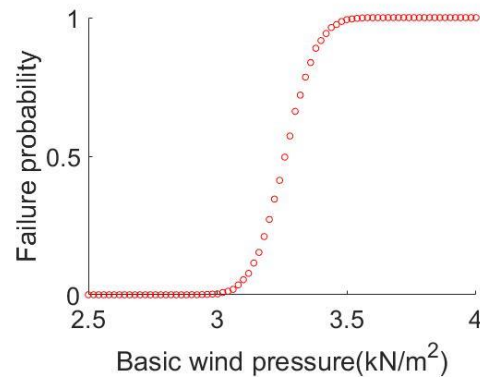
Compared to other probabilistic methods, Monte Carlo simulation avoids complex mathematical computations. By conducting a Monte Carlo simulation in MATLAB with a predetermined number of iterations of 10,000, changing the load magnitude or I-beam cross-sectional properties under three different load conditions, one can obtain a fragile curve for each case. Figure 9 represents the variations of load change with the probability of failure. Figure 10 represents the variations of the moment of inertia of the cross-section with the probability of failure under snow load, wind load, and crane load conditions respectively. From the above set of images, it can be observed that there is always a critical point or a sudden change in the probability of failure with respect to the variables. Looking at individual variables, the crane load should not exceed 190 tons, and the combined snow load and dead load should not exceed 15 kN/m. A basic wind pressure within the range of 2.5 kN/m is considered more suitable. For the moment of inertia, it is believed that it will be better to consider them in a combined manner. Therefore, another round of Monte Carlo simulation is conducted, which only considering the superposition of maximum bending moments. The simulation results are shown in figure 11. Using this method, the appropriate value of moment of inertia should be $2.8 \times 10^{-4} \text{ m}^4$.



(a) Crane load

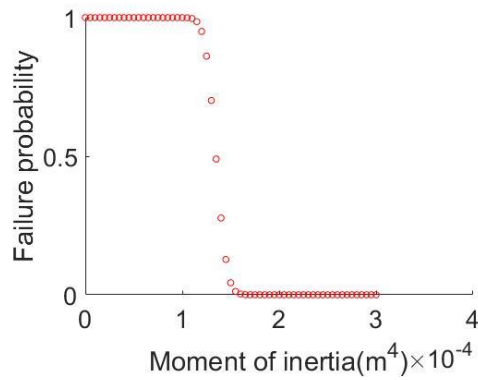


(b) Snow load combined with dead load

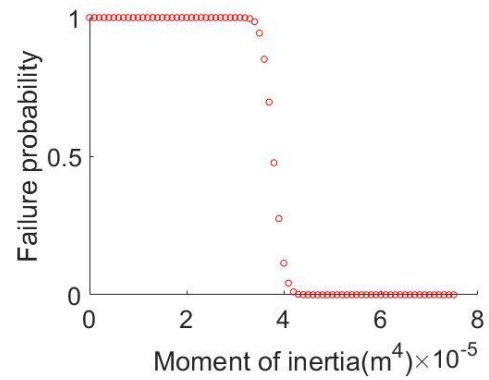


(c) Basic wind pressure

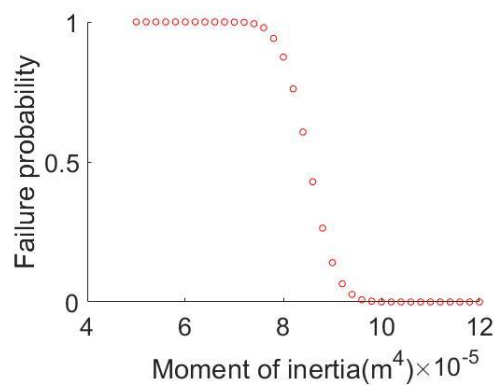
Figure 9. Fragility curve under different load conditions.



(a) Snow load combined with dead load



(b) Wind load



(c) Crane load

Figure 10. Curve of Failure Probability Changes with Moment of Inertia under Different Load Conditions.

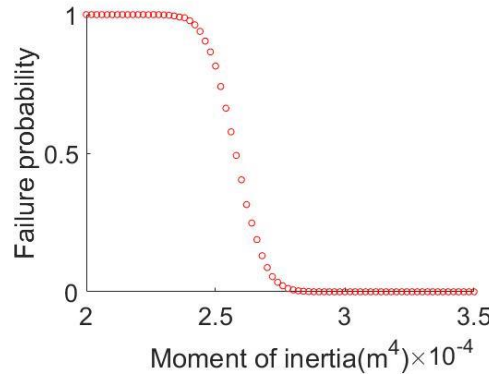


Figure 11. Fragility Curve under Combined Load Effects.

5. Limit state design

When the effects of actions and action effects are considered in a linear relationship, the design value of the basic combination effects should be calculated using the most unfavorable value in the following equation. It is worth mentioning that in the wind load case, the wind load assumed is favorable for the beam. Therefore, the coefficient at the location of the originally planned maximum bending moment is set to 0, and other maximum bending moment values are considered instead. The design service life of this structure is 50 years, therefore the load adjustment factor is taken as 1 [10].

$$S_d = \sum_{i \geq 1} \gamma G_i S G_{ik} + \gamma P S P + \gamma Q_1 \gamma L_1 S Q_{1k} + \sum_{j \geq 1} \gamma Q_j \varphi c_j \gamma L_j S Q_{jk} \quad (5)$$

where γG_i is sub-factor coefficient for the i^{th} permanent load effect, $S G_{ik}$ is effects of the i^{th} permanent load effect's characteristic value, γP is sub-factor coefficient for prestressing effects, $S P$ is effects of representative values related to prestressing, γQ_1 is sub-factor coefficient for the first variable load effect, γL_1 is load adjustment factor for the first consideration of the structural design service life, $S Q_{1k}$ is Effects of the first characteristic value of the variable load effect, γQ_j is sub-factor coefficient for the j^{th} variable load effect, φc_j is Combination value coefficient for the j^{th} variable load effect. In our project, $S p$ is equal to 0 for there is no prestressing. γg is equal to 1.2 and γQ is equal to 1.5 according to unified standard for reliability design of building structures. Besides, due to the design service life of the structure being 50 years, the coefficient γL is set to 1.

With the data above and using equation (5), it can be easily calculated that S_d is equal to 373.904. The limit state design for the ultimate capacity of structural or structural component failure or excessive deformation should meet the following requirement [10].

$$\gamma_0 S_d \leq R_d \quad (6)$$

where γ_0 is importance factor and R_d is design resistance value. In this case, in equation (6), the structural importance factor γ_0 is taken as 1.0, so R_d is equal to 373.904. As shown in figure 12, the moment of inertia should be more than $3.65 \times 10^{-4} \text{ m}^4$.

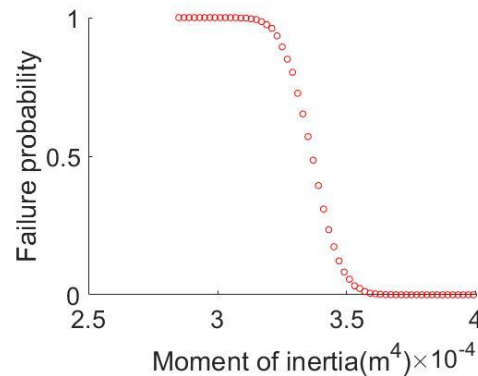


Figure 12. Fragility curve for limit state design.

6. Conclusion

In this paper, Monte Carlo simulation is primarily used to calculate the failure probability in order to determine the appropriate moment of inertia. Additionally, limit state design is employed to calculate the appropriate design value.

The reason why the design value in limit state design is larger than the pure superposition is because limit state design takes into account various factors such as the service life and importance level of the building. Furthermore, it is easy to feel the convenience of Monte Carlo simulation, as it does not involve complex mathematical calculations, yet its accuracy is not inferior to other complex methods. However, it must be acknowledged that there are several factors that have not been taken into account in this paper. For example, the assumption of no lateral displacement in the frame, the assumption of all components using the same material, and the exclusion of seismic loads from consideration.

In future research, there are many other factors that can be taken into consideration. For example, as a portal steel frame, the analysis of the stress on the welding points between components, as well as factors such as the depth of foundation, are worth exploring.

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