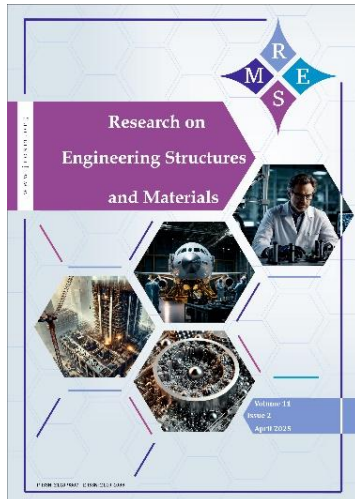




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Wind-resistant optimization design of super high-rise buildings based on an improved genetic algorithm

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Article Info	Abstract
<p>Article History:</p> <p>Received 16 Jul 2026</p> <p>Accepted 08 Sep 2026</p> <p>Keywords:</p> <p>Genetic algorithm; Penalty function method; Super high-rise buildings; SAP2000 FEA; Wind-resistant optimization design</p>	<p>To solve the problems of traditional structural optimization methods being prone to local optima, long computation time, and high memory consumption when dealing with high-dimensional nonlinear problems, a wind-resistant optimization design method for super high-rise intelligent buildings grounded on an improved genetic algorithm is proposed by studying the improvement of genetic algorithm through penalty function method. The findings indicate that this method can enhance the natural frequencies of the 1st and 2nd order structures, with amplification rates of 6.90% and 8.64%, respectively. After optimizing the design, the total values of transverse and downwind base shear forces are reduced by 3.60% and 1.21%, respectively. The total weight of the structure decreases from 52500 tons to 49830 tons, a decrease of 5.09%. Moreover, the research method has been demonstrated to outperform alternative approaches in terms of fitness convergence, solution stability, and constraint handling ability. The average optimization time, average objective function value, and fitness standard deviation of the research method are 1232s, 9.23×10^6, and 0.016×10^6, respectively. From this, the research method can enhance the lateral stiffness and response balance of the structure, verifying the engineering applicability and practical value of the proposed method in wind-induced response control of the structure.</p>

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1. Introduction

With the continuous growth of urban population and the increasing scarcity of urban land resources, the utilization of building space has gradually shifted from horizontal expansion to vertical development. Super high-rise buildings (SHRBs), which can provide a large building usable area under limited land resources, have been widely used in urban buildings such as commercial complexes, offices and mixed residences. However, with the continuous increase in building height, the safety and comfort issues caused by wind-induced response have become increasingly prominent [1]. Compared with general multi-story or high-rise buildings, super high-rise buildings have the characteristics of long structural period, low damping ratio and relatively small lateral stiffness. Under long-term wind load, they are prone to large top lateral displacement, inter-story displacement and wind-induced vibration, which may not only affect structural safety, but also reduce the comfort of people inside the building. Therefore, on the basis of meeting the requirements of structural safety and normal use, reducing the self-weight of the structure and improving wind resistance by reasonably configuring the size of structural components is a key issue that needs to be addressed in the structural design of super high-rise buildings [2-3].

Existing wind-resistant design methods for super high-rise buildings primarily reduce wind-induced response through structural system optimization, component size optimization, and building form optimization. Among these, structural optimization design can achieve coordination between structural stiffness and material usage by adjusting the size and arrangement of major

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lateral resistance components such as beams, columns, and shear walls, without altering the overall function and usage requirements of the building. In accordance with the type of design variables, the optimization of a structure may be categorized into the following: topology optimization, size optimization, and shape optimization. For instance, Lu et al. advanced a dynamic characteristic optimization design approach for wind resistance evaluation of large steel frames founded on topology optimization. The findings demonstrated that by reasonably arranging shear walls, supports, and tube structures, the lateral displacement, inter-story drift, and overturning moment of high-rise buildings could be effectively reduced [4]. He Z et al. examined the structure of spiral SHRBs and put forward a strategy for aerodynamic shape optimization that integrated a multi-objective vibration acceleration approach. The findings showed that this strategy could effectively reduce wind-induced vibration response and optimize the aerodynamic performance of SHRBs while meeting the requirements of structural strength and comfort [5].

Furthermore, with the development of data-driven methods and intelligent optimization technologies, machine learning and heuristic optimization algorithms are increasingly being applied to performance prediction and parameter optimization of complex building structures [6]. For instance, Li Y et al. utilized ridge regression, decision tree, random forest, and gradient-enhanced regression tree to predict the wind pressure of SHRBs. The findings suggested that the last model exhibited satisfactory predictive capabilities for both average and fluctuating wind pressures [7]. Shan W et al. designed a Hooke-Jeeves algorithm based on structural constraints with the goal of minimizing the cost of structural materials, combined with finite element analysis (FEA). The findings indicated that the algorithm had good convergence and optimization effects, which can effectively promote the structural design's safety and economy [8]. However, there are still problems in the optimization design of SHRB structures, such as discrete design variables, difficulty in explicitly representing some constraint conditions through design variables, and non-differentiable objective functions (OFs) [9]. This type of feature makes traditional structural optimization design methods prone to getting stuck in local optima during the solving process, making it difficult to effectively cope with complex practical engineering conditions.

Genetic Algorithm (GA), as a biomimetic intelligent optimization algorithm based on natural selection and genetic mechanisms, has good global optimization ability and robustness. The optimization process does not rely on derivative information of the OF, nor does it require explicit modeling of constraint conditions [10]. Therefore, it can demonstrate stronger adaptability and flexibility in dealing with nonlinear, multimodal, and uncertain problems, effectively compensating for the shortcomings of traditional structural optimization design methods. For example, Yang J et al. conducted a comprehensive analysis and exploration of wind-resistant optimization design (WROD) for SHRBs based on hybrid GA, and proposed a design method that considers the joint probability distribution of wind speed and direction, reliability, and structural performance. The results indicated that this method not only improved the reliability and safety of building structural design, but also achieved optimization of material utilization and engineering costs [11]. Le Gia T T et al. proposed a comprehensive optimization design strategy using NSGA-II. The findings demonstrated that this strategy could effectively improve the comprehensive performance of early architectural design schemes in Vietnam in terms of energy efficiency, comfort, and adaptability [12].

In summary, certain achievements have been made in optimizing the structure of SHRBs under wind loads. However, existing research still has certain limitations: on the one hand, many structural optimization methods fail to fully consider the randomness of wind loads and their nonlinear coupling relationship with structural response in the modeling process, resulting in deviations between optimization results and real engineering environments. On the other hand, the application of GA in structural optimization mostly focuses on simple structural forms, and there are still problems such as long computation time and high memory consumption when facing high-dimensional design space. To this end, an innovative GA-based WROD method for super high-rise buildings is proposed. Through this method, the research aims to provide an efficient and feasible intelligent design strategy for the structural optimization of SHRBs under wind loads, thereby providing theoretical support and engineering practice reference for the fields of wind engineering and structural optimization.

1.1 Research Significance and Contribution

From an engineering application perspective, super high-rise buildings are characterized by their large structural height, numerous design variables, and significant wind-induced response. Traditional experience-based cross-section design methods struggle to balance structural safety, user comfort, and material economy. This study aims to minimize the total structural mass while incorporating performance constraints such as natural frequency, inter-story drift angle, top lateral displacement, and wind-induced peak ground acceleration. This approach enables the rational allocation of component materials while meeting wind resistance requirements, providing a reference for the coordinated design of lightweight and wind-resistant super high-rise buildings.

From a methodological perspective, this study addresses the problems of numerous infeasible solutions, insufficient convergence stability, and low computational efficiency that traditional GA algorithms may encounter in high-dimensional, multi-constraint structural optimization. It introduces a penalty function mechanism based on the degree of constraint deviation into the GA algorithm, enabling differentiated penalties based on the degree of constraint violation for different design schemes, thereby enhancing the algorithm's search capability in the feasible solution space. Simultaneously, an automated co-simulation process is established through the MATLAB and SAP2000 application programming interface (API), realizing automatic iteration between design variable generation, finite element modeling, wind load calculation, structural response analysis, and fitness evaluation. This provides a feasible technical path for the collaborative application of intelligent optimization algorithms and finite element analysis software.

The main contributions of this study are reflected in the following three aspects: (1) A multi-constraint optimization model for wind-resistant design of super high-rise buildings was established, which unified the structural quality target with the performance requirements such as natural frequency, inter-story drift angle, top lateral displacement and wind-induced peak acceleration, thereby improving the adaptability of the optimization design to actual engineering constraints; (2) An improved GA algorithm with a differentiated penalty function mechanism was proposed. This paper does not take the conventional multiplicative penalty function form as the main innovation, but instead applies continuous and differentiated penalties to infeasible solutions of different degrees by quantifying the degree of constraint violation of candidate solutions, and directly uses the modified fitness to the individual selection and evolution process of GA, so as to improve the problem of high infeasible solution ratio and decreased search efficiency in traditional GA; (3) A MATLAB-SAP2000 co-simulation optimization framework was constructed to realize automatic iteration of design variable update, finite element analysis, structural response extraction and fitness evaluation, providing a computational implementation platform for the engineering application of the proposed improved algorithm in wind-resistant optimization of super high-rise buildings. Furthermore, by comparing the proposed method with traditional GA, Particle Swarm Optimization (PSO), Ant Colony Optimization (ACO), and Differential Evolution (DE), the overall performance of the proposed method in terms of optimization efficiency, solution stability, and constraint satisfaction is verified.

2. Methods and Materials

2.1 Mathematical Model for Wind Load Calculation and Wind-Resistance Optimization of SHRBs

In the structural design of SHRBs, wind load calculations usually require consideration of a number of influencing factors, including basic wind pressure, the wind pressure height change coefficient, the wind carrier type coefficient, and the wind vibration coefficient. The basic wind pressure is primarily determined using meteorological observation data from the building's location over many years, reflecting the region's fundamental wind conditions. The wind pressure height variation coefficient considers the effect of wind speed on changes in height, and is usually based on boundary layer theory and the empirical formula recommended by the specification for correction. The wind carrier type coefficient relates closely to the building's geometric shape, windward surface arrangement and windward angle, and is used to characterize the distribution of wind pressure on different building surfaces. The wind vibration coefficient primarily reflects

the building's additional dynamic response under the influence of pulsating wind. According to the different directions of wind action, wind load can be subdivided into downwind load (pressure generated along the direction of wind flow), crosswind load (lateral force perpendicular to the direction of wind flow), and torsional load (torsional moment around the vertical axis caused by asymmetric wind pressure) [13-14]. This paper determines the basic wind pressure, wind pressure height variation coefficient, and wind load shape coefficient according to the "Load Code for Design of Building Structures" (GB 50009-2012), and obtains the wind-induced response of the structure using the wind vibration response calculation method in the code. Based on this, the average response, background response, and resonance response generated by wind load are equivalent to static loads, and the equivalent static wind loads in the along wind and crosswind directions are calculated separately. For each structural optimization scheme, the corresponding wind vibration response and equivalent static wind load are recalculated based on the current structure's mass, stiffness, and natural frequency. The formula for calculating the equivalent static wind load (ESWL) in the downwind direction is given in equation (1).

$$\hat{F}_D(z) = \bar{F}_D(z) + \sqrt{F_{DB}(z)^2 + F_{DR}(z)^2} \quad (1)$$

In formula (1), $\hat{F}_D(z)$ represents the ESWL of the total downwind direction at height z ; $\bar{F}_D(z)$ represents the average wind load component, which is the steady-state load generated by the average wind speed; $F_{DB}(z)$ represents the background component, which is the quasi-static response caused by low-frequency wind speed fluctuations; $F_{DR}(z)$ represents the resonance component, which is the dynamic response generated by the resonance between the structure and the wind. The calculation formula for ESWL in crosswind direction is shown in formula (2).

$$\hat{F}_L(z) = \sqrt{F_{LB}(z)^2 + F_{LR}(z)^2} \quad (2)$$

In formula (2), $\hat{F}_L(z)$ represents the total ESWL in the crosswind direction at height z ; $F_{LB}(z)$ represents the background wind load component, which is the quasi-static crosswind response caused by low-frequency pulsating wind; $F_{LR}(z)$ represents the resonant wind load component, which is the dynamic response caused by the resonance between crosswind and structural vibration. Single high-rise buildings with symmetrical shapes usually have weak torsional effects, so research will not explore them. In the process of structural optimization, constraints are often difficult to express explicitly through design variables [15]. To improve modeling efficiency and analysis accuracy, finite element software SAP2000 is used for solving. By calling the API interface of SAP2000, it is easy to define and adjust the structural parameters and geometric dimensions of the model, and efficiently obtain analysis results. At the same time, considering that a single software platform is difficult to meet multiple requirements, the study adopts MATLAB and SAP2000 software for joint simulation. The schematic diagram of the joint simulation process structure of MATLAB and SAP2000 is illustrated in Figure 1.

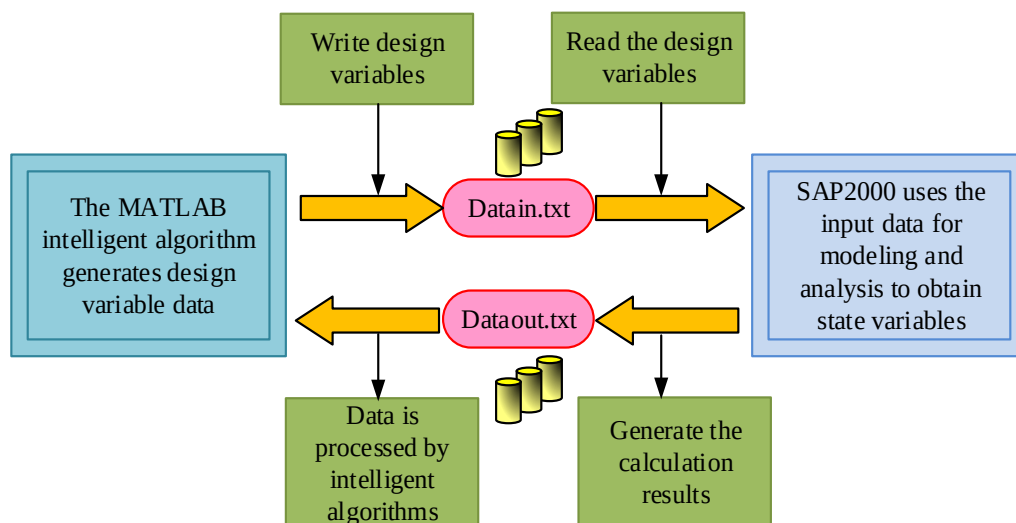


Fig. 1. Schematic diagram of the joint simulation process structure of MATLAB and SAP2000

From Figure 1, the joint simulation of the two is achieved through program control. Firstly, MATLAB executes intelligent optimization algorithms and generates design variables, which are written into input files in the specified format. Subsequently, SAP2000 reads the input data, automatically updates the structural model through API interface, and performs FEA to obtain the structural response results. Next, SAP2000 will provide feedback on the analysis results to MATLAB as a basis for evaluating the degree to which the OF values and constraint conditions of the current design scheme are satisfied. The intelligent optimization algorithm determines whether the convergence criteria are met based on this, and if not, continues to iterate and update until the optimal solution is obtained. Ultimately, the system outputs optimized design results that meet performance requirements, thereby achieving an efficient and automated structural wind resistance optimization process. Due to the large number of uncertain factors in the design of SHRBs, if all factors are fully considered, the model will become too complex to achieve effective control and solution. Therefore, the research will limit the optimization scope to the cross-sectional dimensions of the components. The schematic diagram of the cross-sectional structure of the component is illustrated in Figure 2.

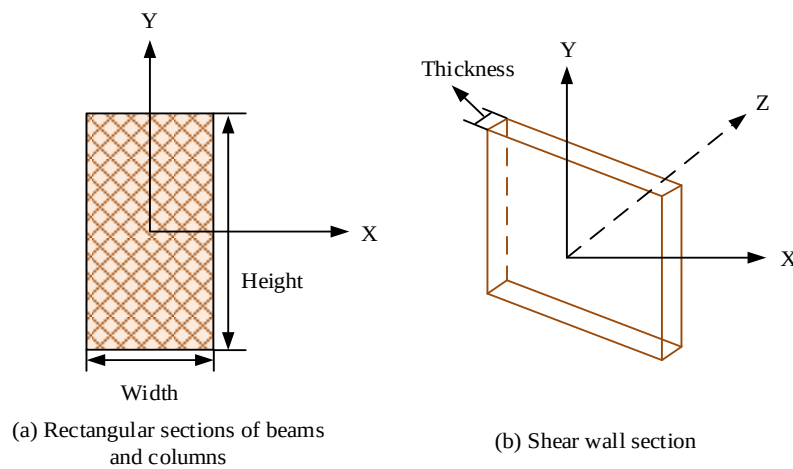


Fig. 2. Schematic diagram of the cross-sectional structure of the component

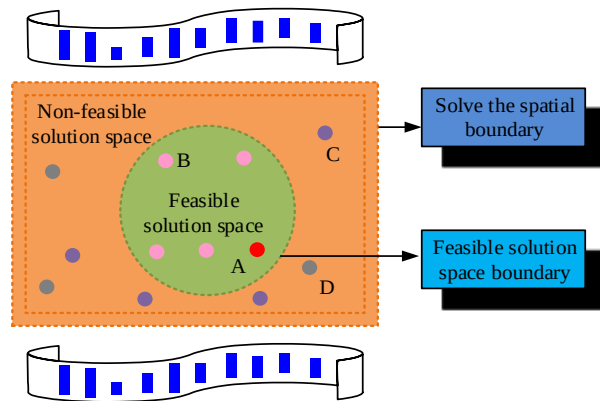


Fig. 3. Punishment mechanism

Figure 2 (a) showcases the rectangular cross-section of beams and columns, while Figure 2 (b) showcases the cross-section of shear walls. As shown in Figure 2, the study selects the height, width, and thickness of shear walls of beams and columns in rectangular cross-sections as design variables. The reasonable handling of constraints during the optimization process has a crucial impact on the feasibility and effectiveness of the results [16]. The penalty function method is a commonly used constraint handling strategy in intelligent optimization algorithms. Its basic idea is to introduce a penalty term for solutions that violate constraints, significantly increasing the OF value and reducing the corresponding fitness [17-18]. A penalty function method is designed to penalize unconstrained optimal solutions that may be located outside the feasible domain boundary based on the degree of constraint deviation. The penalty mechanism of this new penalty

function method is shown in Figure 3. As shown in Figure 3, the orange rectangle represents the possible range of all solutions, divided into feasible solution space and infeasible solution space. The green circle denotes the feasible solution space, the circular boundary represents the feasible solution space boundary, and the area outside the circle but still within the rectangle represents the infeasible solution space. This new penalty function reduces the severity of punishment for infeasible solutions by differentiating them, and can synergistically guide search between feasible and infeasible domains, thereby effectively improving the global optimization ability of intelligent algorithms. The formula for calculating the fitness value $F(x_i)$ of the i th individual in the population is shown in formula (3).

$$F(x_i) = \begin{cases} f_{max}^p - (f(x_i) + P(x_i)) \\ f_{max}^p - f(x_i) \end{cases} \quad (3)$$

In formula (3), $f(x_i)$ represents the target value corresponding to the i th individual, $P(x_i)$ represents the penalty function value corresponding to the i th individual, and f_{max}^p represents the upper limit value of the penalty function. The degree of violation $v_j(x_i)$ of the i th individual x_i against the j th constraint $g_j(x_i)$ is shown in formula (4).

$$v_j(x_i) = \max\{0, |g_j(x_i)|\} \quad (4)$$

If $g_j(x_i) \leq 0$, then $v_j(x_i) = 0$, indicating that the constraint is satisfied. $g_j(x_i) > 0$, then $v_j(x_i) = g_j(x_i)$, the greater the value, the greater the deviation from the constraint. The calculation formula for the OF $\varphi(x)$ after applying the penalty is presented in formula (5).

$$\varphi(x) = F(x) \cdot (1 + K \cdot C) \quad (5)$$

In formula (5), $F(x)$ denotes the original OF; C is the sum of all constraint violations, reflecting overall infeasibility; K represents the penalty coefficient, which is used to control the severity of punishment. The larger the K , the stronger the punishment for constraint violations. By comprehensively comparing the constraint satisfaction, objective function value, and fitness convergence stability under different penalty coefficients, the penalty coefficient was finally determined to be $K = 1.0 \times 10^8$.

In summary, this study employs ESWL for wind load modeling of super high-rise buildings. This method transforms the average response, background response, and resonance response generated by actual wind loads into static loads equivalent to the main structural responses. The background component characterizes the quasi-static response caused by fluctuating winds, while the resonance component characterizes the dynamic response generated by the combined effects of wind load frequency components and the structure's natural vibration characteristics. Since changes in structural cross-sectional dimensions cause variations in structural stiffness, mass, and natural frequency, modal analysis is re-performed based on the current structural parameters during optimization, and the corresponding equivalent static wind load is updated accordingly, achieving an iterative correlation between wind loads and structural dynamic characteristics.

2.2 Wind-Resistant Optimization Method for SHRBs Based on Improved GA

Based on the joint simulation of MATLAB and SAP2000, a complete structural response analysis and modeling process is constructed, and an optimization design framework is defined with rectangular section size as the optimization variable and total structural mass as the OF. However, in the face of optimization problems with complex constraints, traditional intelligent optimization algorithms still have certain limitations in terms of solving efficiency and feasibility of results. To this end, the study further introduces a new penalty function into the GA, which improves the convergence ability of the GA on feasible solutions by imposing penalties on design schemes that violate constraints. Finally, a wind-resistant optimization method for SHRBs grounded on the improved GA is proposed. Traditional GA primarily search for optimal solutions through selection, crossover, and mutation operations. However, for multi-constraint problems in wind resistance optimization of super high-rise buildings, this can easily generate a large number of infeasible solutions, increasing the number of finite element analyses and reducing search efficiency. To

address this issue, this study introduces a differentiated penalty function based on the degree of constraint violation into the traditional GA framework to adaptively penalize infeasible solutions. First, GA generates candidate design schemes, and the structural response is calculated using SAP2000 via MATLAB. Then, the objective function and constraint violation degree of each candidate scheme are calculated. For individuals that meet the constraints, their constraint violation is set to 0, incurring no additional penalty. For individuals that violate the constraints, the violation is calculated based on the degree of deviation from the constraint limits, with the penalty intensity increasing with the degree of violation. Therefore, individuals with minor constraint violations are not directly eliminated and can still participate in subsequent genetic searches, while individuals with severe constraint violations are subject to stronger penalties, thus guiding the population to gradually converge towards the feasible region. The fitness, modified by the penalty function, is used for subsequent individual selection, ensuring that the algorithm prioritizes design schemes with better objective functions and higher constraint satisfaction.

The mathematical model calculation formula for WROD is shown in formula (6).

$$W = \sum_{i=1}^N w_i b_i h_i L_i + \sum_{j=1}^{N_w} \rho_{w,j} t_{w,j} A_{w,j} + \rho_f t_f B D M \quad (6)$$

In formula (6), W represents the structure's total weight, w_i represents the density of the i th type of beam column component, b_i and h_i represent the beam and column sections' width and height, L_i represents the total length of the i th type of beam-column member, N represents the number of beam and column section types, $\rho_{w,j}$ represents the material density of the j th type of shear wall, $t_{w,j}$ represents the thickness of the j th type of shear wall; $A_{w,j}$ represents the planar area of the corresponding shear wall; N_w represents the number of shear wall types; ρ_f represents the material density of the floor slab, B and D denote the width and height of the building, ρ_f represents the density of the floor slab, M means the number of floors, and t_f represents the thickness of the floor slab. The mathematical expression for the constraint conditions of component cross-sectional dimensions is shown in formula (7).

$$\begin{cases} b_i^L \leq b_i \leq b_i^U & i = 1, 2, \dots, N \\ h_i^L \leq h_i \leq h_i^U & i = 1, 2, \dots, N \end{cases} \quad (7)$$

In formula (7), L and U represent the lower and upper limits of the rectangular cross-sectional dimensions, respectively. The mathematical expression for the constraint conditions of inter-story drift angle (ISDA) and top layer lateral displacement of the structure is shown in formula (8).

$$\begin{cases} d_H \leq d_H^U \\ d_j \leq d_j^U & j = 1, 2, \dots, M \end{cases} \quad (8)$$

In formula (8), d_j and d_{j-1}^U represent the difference and limit values of inter story lateral displacement, while d_H^U and d_H represent the limit values of building top lateral displacement and building top lateral displacement, respectively. The constraint conditions for wind-induced peak acceleration of the structure are shown in formula (9).

$$\varpi \geq \varpi' \quad (9)$$

In formula (9), ϖ and ϖ' represent the natural frequency of the structure and its lower limit, respectively. The schematic diagram of the wind-resistance optimization method for SHRBs based on improved GA is shown in Figure 4.

In Figure 4, in each iteration, the research method recalculates the structural dynamic characteristics and equivalent static wind load based on the updated structural parameters. Since different cross-sectional dimensions correspond to different structural masses, stiffnesses, and natural frequencies, the resonant response under wind load also adjusts with changes in structural dynamic characteristics. Therefore, the modal analysis results are fed back to the ESWL calculation, and the updated equivalent wind load is reapplied to the structural model to achieve an iterative correlation between structural parameters, dynamic characteristics, and wind-induced response.

By writing interface programs, data exchange and calling between MATLAB and SAP2000 can be achieved, thereby completing the process of updating modal parameters and wind loads. The optimization process includes: first, determining the design objective, defining the design variables, OF and constraints; second, initializing the algorithm parameters and generating the initial population. A finite element model is built grounded on the initial cross-section, and the ESWL is calculated using the three-component method and applied to the centers of mass of each layer to obtain the structural response and thus assess the degree of adaptation. The GA generates a new generation population through selection, crossover, and mutation operations, combined with a novel penalty function to handle constraints and retain excellent individuals. This process continues to loop until the constraints are met, convergence is achieved, or iteration is terminated, ultimately obtaining the optimization result.

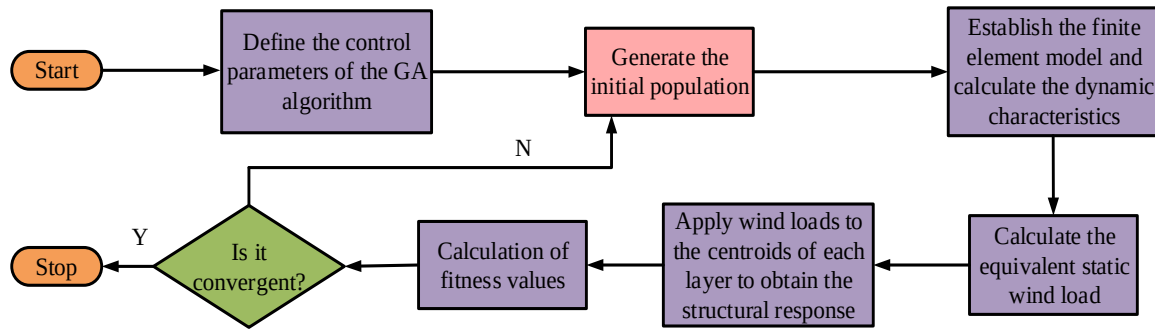


Fig. 4. Schematic diagram of the process structure for WRODs of SHRBs

This study selected a 60-story super high-rise building with a total height of 185.7 m as the research object, employing a reinforced concrete frame-shear wall structural system. Based on the vertical load transfer law and the design characteristic of decreasing component cross-sections with height, the 60 stories were divided into 5 zones, corresponding to columns JZ1-JZ5 and beams JL1-JL5 respectively. The initial cross-sectional dimensions of each component were determined according to the stress requirements of different floors, with upper and lower limits determined by combining initial dimensions, structural stiffness requirements, and the "Technical Specification for Concrete Structures of High-Rise Buildings" (JGJ 3-2010). To reduce design variables, the width-to-height ratio of column cross-sections was fixed at 1.0, the width-to-height ratio of beam cross-sections was fixed at 0.5, and the thickness of shear walls was used as the design variable. The floor slab thickness was uniformly set to 130 mm. All beams, columns, and shear walls were made of reinforced concrete, and the material was assumed to be in a linear elastic state. Specific geometric and physical parameters and the range of design variables are shown in Table 1.

Table 1. Geometric physical parameters of the structural model

Component name	Width/m			Height/m		
	Initial value	Lower limit value	Upper limit value	Initial value	Lower limit value	Upper limit value
JZ1	0.90	0.70	1.20	0.90	0.70	1.20
JZ2	0.85	0.65	1.15	0.85	0.65	1.15
JZ3	0.80	0.60	1.10	0.80	0.60	1.10
JZ4	0.75	0.55	1.05	0.75	0.55	1.05
JZ5	0.70	0.50	1.00	0.70	0.50	1.00
JL1	0.60	0.50	0.80	0.30	0.25	0.40
JL2	0.58	0.48	0.78	0.29	0.24	0.39
JL3	0.56	0.46	0.76	0.28	0.23	0.38
JL4	0.54	0.44	0.74	0.27	0.22	0.37
JL5	0.52	0.42	0.72	0.26	0.21	0.36

According to Table 1, JZ1 to JZ5 respectively represent the frame columns of floors 1-10, 11-20, 21-30, 31-40, and 41-60, while JL1 to JL5 correspond to the frame beams of their respective floors.

The initial structural model is subjected to a modal analysis, which is performed using the block-wise Lanzo method, and the first two natural frequencies are obtained to be 0.145Hz and 0.162Hz, respectively. The constraint limit setting requirements are shown in Figure 5.

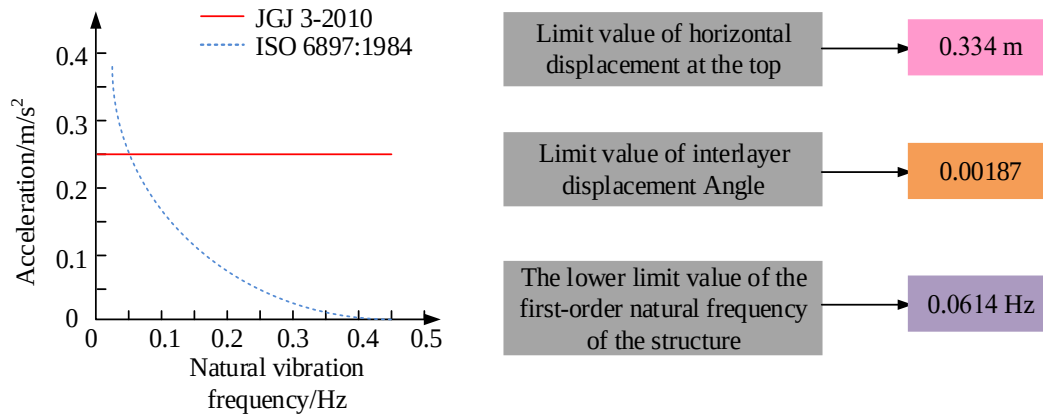


Fig. 5. Requirements for setting the limit values of constraint conditions

From Figure 5, to ensure the optimized structure meets safety and normal use requirements, this study determined normative limit values for structural response based on the "Technical Specification for Concrete Structures of High-Rise Buildings" (JGJ 3-2010) and ISO 6897:1984, and used these as constraints in the wind-resistant optimization process. Considering a 60-story reinforced concrete frame-shear wall structure with a total height of 185.7 m, the structural performance was controlled primarily from three aspects: top horizontal displacement, inter-story drift angle, and wind-induced vibration comfort. The top horizontal displacement limit was determined according to the relevant requirements of JGJ 3-2010, and was controlled within 1/550 of the total building height, corresponding to a top horizontal displacement control value of 0.334 m for this study structure. The inter-story drift angle limit was also determined according to JGJ 3-2010, combined with the structural system and functional use of the study object, and set to a control value of 0.00187 to limit the relative lateral deformation between adjacent floors. Regarding wind-induced vibration comfort, considering that the research subjects are mainly used in long-term use scenarios such as offices and hotels, in accordance with the relevant requirements of ISO 6897:1984, the maximum acceptable acceleration control value for the human body is set to 0.25 m/s², and based on this, the lower limit of the first-order natural frequency of the structure is determined to be 0.0614 Hz. The formula for calculating the peak acceleration $\ddot{x}_a$ within the duration T of the R year return period is shown in formula (10).

$$\ddot{x}_a = \sqrt{2 \ln n T} \left(0.68 + \frac{\ln R}{5} \right) \exp(-0.35 - 0.41 \ln n) \quad (10)$$

In formula (10), n represents the first-order frequency of the structure. Therefore, the main standard limits adopted in this study include: top horizontal displacement not exceeding 0.334 m, inter-story drift angle not exceeding 0.00187°, acceptable wind-induced acceleration not exceeding 0.25 m/s², and the first-order natural frequency of the structure not lower than 0.0614 Hz. These limits are used to control the overall lateral deformation of the structure, local floor deformation, and wind-induced vibration comfort, and are uniformly used as the basis for improving the GA algorithm to evaluate the feasibility of candidate design schemes.

3. Results

3.1 Optimization Result Analysis

To evaluate the efficacy of the proposed methodology, a suitable experimental environment was established. A finite element model of a super high-rise building was built using SAP2000 v24. Beams and columns were simulated using frame elements, while floor slabs were simulated using shell elements, with a floor slab thickness of 130 mm. Frame elements were used to describe the axial, bending, and shear stress behavior of beams and columns, while shell elements were used to characterize the in-plane and out-of-plane stiffness of the floor slabs. The total height of the

structure is 185.7 m, with 60 floors, constructed of reinforced concrete, and the material was assumed to be in a linear elastic state. The software and hardware environments and model parameter settings are listed in Table 2.

Table 2. Hardware and software environment and model parameter settings

Serial number	Type	Settings
1	CPU	Intel Core i7-12700K @ 3.6 GHz, 12 cores
2	RAM	32 GB DDR4
3	Operating System	Windows 10 Professional 64-bit
4	MATLAB Version	MATLAB R2022b
5	Finite Element Software	SAP2000 v24
6	API Interface	SAP2000 OAPI (Open API)
7	Population Size	100 individuals
8	Number of Generations	200 generations
9	Crossover Rate	0.85
10	Mutation Rate	0.05

Population size primarily affects the search space coverage and the computational cost per generation. Therefore, while considering the computational complexity of the finite element model for super high-rise buildings, the population size was set to 100, and the number of iterations was set to 200 to ensure the algorithm has a sufficient search process. The crossover rate and mutation rate were set to 0.85 and 0.05, respectively, to maintain population diversity while avoiding excessive random search. All the above parameters were tested in preliminary experiments and determined after comprehensively considering convergence stability, optimization time, and objective function value. To avoid the influence of parameter differences on the algorithm comparison results, traditional GA, PSO, ACO, and DE were all compared and tested using the same computational environment, design variables, and constraints.

During the optimization design process, the trend of the 1st and 2nd natural frequencies of the structure is shown in Figure 6.

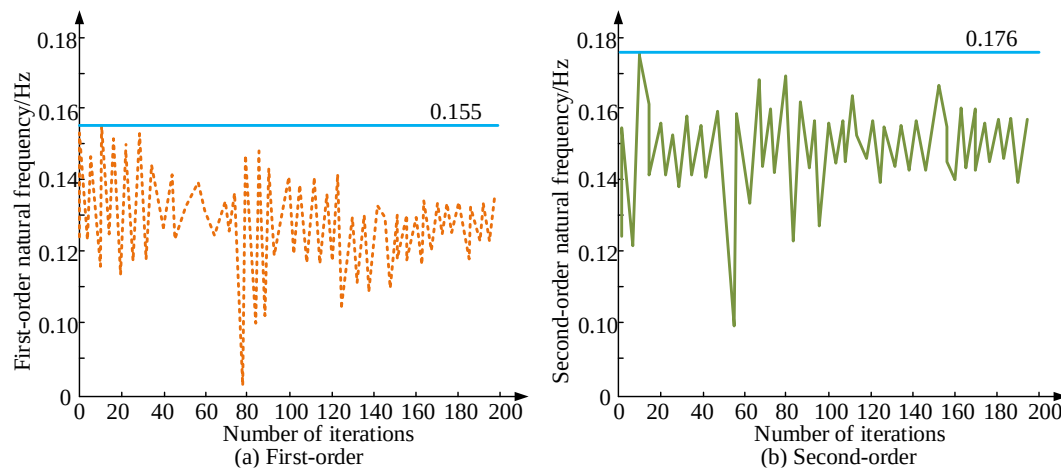


Fig. 6. The variation trends of the first and second order natural frequencies of the structure

In Figures 6(a) and 6(b), the natural frequency variation curves of the first- and second-order structures undergo changes during the optimization process. In Figure 6, the natural frequencies of the first- and second-order structures exhibited a gradual increase in response to the adjustment of design variables during the initial stage of optimization. This increase was followed by a slight decrease after reaching the peak, ultimately leading to stabilization. This observation signifies that the optimization process has attained convergence. The optimized design increased the natural frequencies of the 1st and 2nd order structures from 0.145 Hz and 0.162 Hz to 0.155 Hz and 0.176 Hz, with amplification rates of 6.90% and 8.64%. The increase in natural frequency effectively

improved the dynamic characteristics of the structure, weakened its sensitivity to resonance components in wind loads, and reduced wind-induced response. The ESWL changes in crosswind direction and downwind direction are shown in Table 3.

Table 3. Optimization of the ESWLs of the structure in the downwind and crosswise directions before and after the design

Indicators		Wind load direction	Pre-optimization numerical value
Wind load components/kN	Average component/kN	Downwind direction	19800
	Background component/kN	Downwind direction	6400
	Resonance component/kN	Downwind direction	2000
	Background component/kN	Crosswind direction	5800
	Resonance component/kN	Crosswind direction	18900
Total base shear force/kN		Downwind direction	28,200
Total base shear force/kN		Crosswind direction	24,700
Total base bending moment/kN·m		Downwind direction	9.55×10 ⁶

According to Table 3, after optimization design, the total values of transverse and downwind base shear forces were reduced by 3.60% and 1.21%, respectively. Among them, the reduction in crosswind base shear force was more significant, due to the higher proportion of resonant components in the crosswind response, which are closely related to the structure's natural vibration characteristics. In addition, compared with before optimization design, the crosswind resonance component decreased by 3.65% and the background component decreased by 3.45% after optimization. The background and resonance components in the downwind direction decreased by 2.34% and 9.50%, respectively. The bending moment of the foundation also decreased, decreasing by 1.78% in the downwind direction and 4.28% in the crosswind direction. The total weight of the structure decreased from 52500 tons to 49830 tons, a decrease of 5.09%. The research method improved the utilization efficiency of materials while satisfying constraints such as lateral displacement and acceleration.

3.2 Wind-Resistance Performance Verification

To further verify the reliability of the proposed improved GA algorithm optimization results, the structural response before and after optimization was validated. The structural response validation mainly focused on the top horizontal displacement and inter-story drift angles, and cross-validation was performed using a "SAP2000 finite element calculation result - theoretical calculation result" approach. First, based on the pre-optimization structural model, an SAP2000 finite element model was established according to the actual geometric parameters, material parameters, and wind load calculation results to calculate the top horizontal displacement and inter-story drift angles of each floor. Subsequently, based on the overall lateral stiffness of the structure and the equivalent static response under wind load, the top horizontal displacement was independently calculated theoretically, and the theoretical results were compared with the SAP2000 finite element calculation results. The optimized model was then verified using the same calculation process to check the reliability of the structural response results before and after optimization. The structural response validation results are shown in Table 4.

Table 4. Validation results of structural responses

Structural response	Calculation method	Pre-optimization	Post-optimization	Relative error/%
Top lateral displacement/m	SAP2000	0.295	0.275	—
Top lateral displacement/m	Theoretical calculation	0.291	0.279	1.36/1.45
Maximum inter-story drift ratio	SAP2000	0.00201	0.00176	—
Maximum inter-story drift ratio	Independent calculation	0.00198	0.00173	1.49/1.70

Table 4 shows that before optimization, the SAP2000 calculation result for the horizontal displacement at the top of the structure was 0.295 m, while the independent theoretical calculation result was 0.291 m, with a relative error of 1.36%. After optimization, the SAP2000 calculation result was 0.275 m, while the theoretical calculation result was 0.279 m, with a relative error of 1.45%. The results obtained by the two calculation methods are quite close, with errors controlled within 2%, indicating that the SAP2000 finite element model can accurately reflect the overall lateral deformation characteristics of the structure under wind load. Furthermore, the relative errors of the two calculation results for the maximum inter-story drift angle before and after optimization were 1.49% and 1.70%, respectively, further verifying the reliability of the structural response calculation results.

Based on the structural response verification, the validated SAP2000 finite element model was used to perform response analysis on each generation of structural schemes during the optimization process, and the horizontal displacement of each floor was extracted. The performance of the super high-rise building structure under wind load before and after optimization was compared and analyzed. Based on the standard limits determined in Section 2.2, this paper uses the top horizontal displacement of 0.334 m as the overall lateral deformation control index and the inter-story drift angle of 0.00187 as the floor deformation control index, and uses these to evaluate the wind resistance performance of the structure before and after optimization. The changes in lateral displacement of each floor before and after optimization are shown in Figure 7.

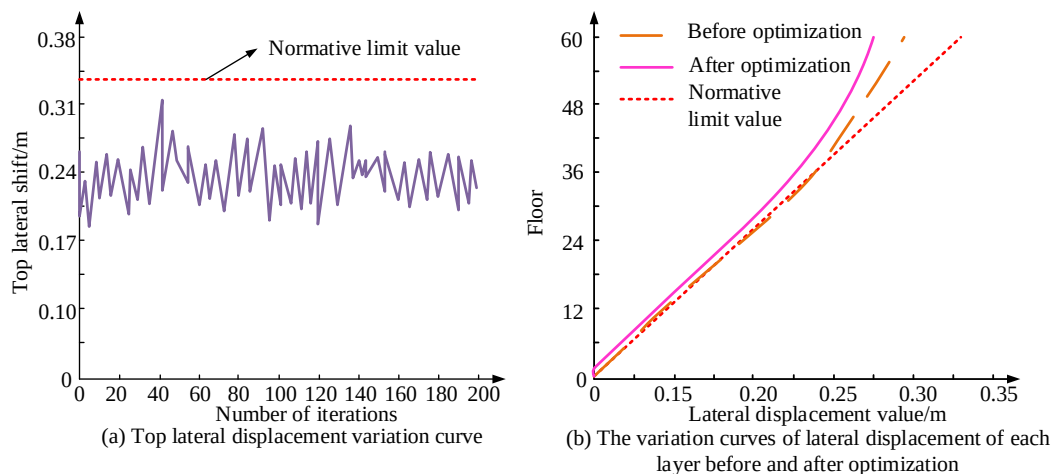


Fig. 7. The variation of lateral displacement of each layer before and after optimization

Figure 7(a) demonstrates the variation curve during the optimization process of top lateral displacement, and Figure 7(b) presents the variation curve of lateral displacement for each layer prior to and post-optimization. In Figure 7, an enhancement in the overall lateral stiffness of the building structure was observed with increased iteration times. Additionally, a reduction in the horizontal displacement at the top from 0.295m to 0.275m, representing a decrease of approximately 6.78%, was evident. Before optimization, the top horizontal displacement already

met the control limit of 0.334 m, and after optimization, it was further reduced, indicating that the overall lateral deformation of the structure was effectively controlled. Meanwhile, the inter-story drift angles of the middle and upper floors exceeded the limit of 0.00187 before optimization, reflecting a certain degree of stiffness deficiency in local areas. The optimized design improved the lateral stiffness of this area by adjusting the column and wall cross-sectional dimensions of the corresponding floors, ensuring that the inter-story drift angles of the optimized structure met the set limit. Therefore, the proposed method can reduce the number of structural materials used while reasonably adjusting the structural stiffness, thereby improving the overall and local wind resistance performance of the building. The changes in the lateral displacement angle between each layer before and after optimization are shown in Figure 8.

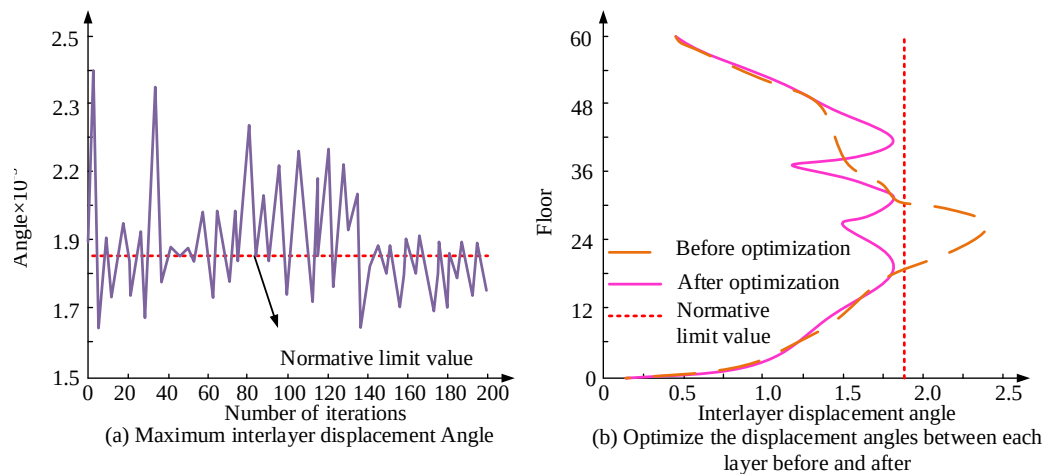


Fig. 8. The variation of the lateral shift angle between each layer before and after optimization

Figure 8 (a) showcases the variation curve of the maximum interlayer displacement angle during the optimization process, while Figure 8 (b) showcases the distribution of interlayer displacement angles before and after optimization. In Figure 8 (a), during the initial stage of optimization, the max ISDA exceeded the limit specified in JGJ 3-2010 due to the unadjusted design variables. As the number of iterations rose, the lateral stiffness of the structure gradually increased, and the max ISDA showed a fluctuating downward trend, ultimately converging within the allowable range of the specifications. As shown in Figure 8 (b), before optimization, the ISDA of the middle floor reached its peak, exceeding the standard limit, indicating insufficient local stiffness in the area. After optimization, by adjusting the cross-sectional dimensions of the columns and beams, the stiffness of the area was significantly improved, the ISDA was significantly reduced, and the overall response was more balanced, meeting the requirements of the specifications. The optimized cross-sectional dimensions of the components decreased with the height of the building, reflecting the principles of reasonable material allocation and center of gravity control. The section widths of the column components on floors 1-10, 11-20, 21-30, 31-40, and 41-60 were 0.65m, 0.60m, 0.55m, 0.50m, and 0.45m, respectively. The section widths of the corresponding floor beams were 0.45 m, 0.45 m, 0.40 m, 0.35 m, and 0.30 m, all within the section range set in Table 1, meeting the dual requirements of structural performance and dimensional constraints. Finally, the study introduced traditional GA, PSO, ACO, and DE as comparative algorithms, each running independently for 30 runs. All algorithms used the same design variables, objective function, constraints, population size, maximum number of iterations, and computational environment. The population was reinitialized for each run using a different random number seed. By statistically analyzing the average optimization time, mean optimal objective function value, worst-case deviation rate, number of constraint violations, and fitness standard deviation across the 30 runs, the computational efficiency, optimization quality, stability, and constraint handling capabilities of the different algorithms were comprehensively compared to verify the effectiveness of the proposed improved GA algorithm. The test findings are presented in Table 5.

According to Table 5, among the five algorithms, the improved GA performed the best with an average optimization time of 1232 seconds, demonstrating high computational efficiency. In terms of fitness convergence performance, the average value of the optimal OF obtained by improving the

GA was 9.23×10^6 , which was greatly better than other algorithms. This indicated that it had stronger global search capability in multi-dimensional high constraint environments. In addition, the worst solution deviation rate of the improved GA was only 2.13%, indicating that it had better solution space stability while maintaining optimization quality. In terms of constraint handling, the improved GA only experienced 2 mild constraint violations, verifying the effective suppression ability of the penalty function on infeasible solutions. The standard deviation of fitness was only 0.016×10^6 , which also reflected the significant improvement of this method in population stability. From this, the improved GA not only has better global search ability and constraint feasibility processing ability, but also demonstrates stronger stability and applicability in engineering practice, providing a more advantageous intelligent design path for high-dimensional and multi constraint structure optimization.

Table 5. Multi-index performance testing

Algorithms	Average optimization time/s	The mean value of the optimal OF	Worst solution deviation rate/%	The number of constraint violations	Standard deviation of fitness
Improved GA	1232	9.23×10^6	2.13	2	0.016×10^6
Traditional GA	1317	9.31×10^6	3.62	4	0.027×10^6
PSO	1389	9.35×10^6	3.04	5	0.034×10^6
ACO	1471	9.44×10^6	4.18	8	0.042×10^6
DE	1295	9.37×10^6	2.89	3	0.031×10^6

4. Discussion

Existing research mainly focuses on reducing the wind-induced response of high-rise buildings through aerodynamic shape optimization, topology optimization, and structural dimension optimization [4-5,8,11]. Aerodynamic shape optimization emphasizes mitigating airflow separation, vortex shedding, and crosswind aerodynamic forces, while structural parameter optimization primarily alters the lateral stiffness and mass distribution of the structure by adjusting component cross-sections [5,10]. This paper's results show that changes in cross-sectional parameters simultaneously alter the structural stiffness matrix, mass matrix, and modal characteristics, resulting in a significant structural dynamic feedback characteristic in the wind-induced response. Therefore, for ultra-high-rise frame-shear wall structures, the key to wind resistance optimization is not simply increasing lateral stiffness, but rather coordinating stiffness distribution, mass participation, and natural vibration characteristics to avoid insufficient local stiffness or excessive stiffness configuration. This aligns with existing research on the synergistic optimization of structural performance and material usage [9-10], but further demonstrates the impact of cross-sectional dimension adjustment on structural dynamic characteristics.

From an optimization solution perspective, ultra-high-rise structural design is a typical high-dimensional, discrete, nonlinear constrained optimization problem. Its fitness evaluation is directly coupled with finite element analysis, leading to high computational costs for single-function evaluation. Existing studies on GA, PSO, and multi-objective genetic algorithms mainly focus on global search and Pareto solution set acquisition [11-12]. However, the examples in this paper further demonstrate that the handling of the feasible region constraints directly affects the population search efficiency. For performance constraints such as inter-story drift angle, top displacement, and wind-induced acceleration, the use of a rigid infeasible solution elimination mechanism may result in the loss of effective search information near the feasible region boundary; continuous penalties based on constraint violations can maintain a certain boundary search capability. It should be noted that this characteristic is closely related to the finite element model size, the degree of dispersion of design variables, and the degree of constraint activity, and cannot be simply generalized to all structural optimization problems. In addition, this paper uses the standard equivalent static wind load and has not yet considered the unsteady aerodynamic forces, vortex-induced response, and building group interference effects caused by complex shapes.

Therefore, the obtained optimization law is mainly applicable to the parametric wind-resistant design of regular super high-rise structures.

5. Conclusions

This study addresses the structural response optimization problem of super high-rise buildings under wind loads. A wind-resistant optimization design method based on an improved GA algorithm is proposed, and iterative optimization of structural parameters, wind loads, and responses is achieved through joint simulation using MATLAB and SAP2000. The main conclusions are as follows:

- The proposed method can effectively improve the dynamic characteristics and wind resistance of super high-rise buildings. During the optimization process, the natural frequencies and equivalent static wind loads are updated according to changes in structural parameters, and the average, background, and resonance responses are comprehensively characterized. After optimization, the first and second natural frequencies of the structure increase from 0.145 Hz and 0.162 Hz to 0.155 Hz and 0.176 Hz, respectively, representing increases of 6.90% and 8.64%; the total base shear forces in the crosswind and along wind directions decrease by 3.60% and 1.21%, respectively.
- After optimization, the total mass of the structure was reduced from 52,500 t to 49,830 t, a reduction of 5.09%; the top horizontal displacement was reduced from 0.295 m to 0.275 m, a reduction of approximately 6.78%; at the same time, the inter-story drift angles of each floor met the set code limits after optimization, effectively improving the problem of insufficient stiffness in some floors.
- The improved GA algorithm exhibits better comprehensive optimization performance. Through 30 independent runs comparing the improved GA, traditional GA, PSO, ACO, and DE, the results show that the average optimization time of the improved GA algorithm is 1232 s, the average value of the optimal objective function is 9.23×10^6 , the worst solution deviation rate is 2.13%, and the fitness standard deviation is 0.016×10^6 . Its overall performance is superior to other comparative algorithms, indicating that the differentiated penalty function can effectively improve the search stability and constraint handling capability in the optimization process of multi-constraint structures.
- The optimized component cross-sectional dimensions generally decrease with the building height. Specifically, the cross-sectional width of column components gradually decreases from 0.65 m to 0.45 m in different height regions, and that of beam components gradually decreases from 0.45 m to 0.30 m. This achieves a reasonable allocation of materials while meeting structural performance constraints.
- Research limitations and future work. Current research primarily focuses on numerical optimization analysis based on typical super high-rise building structural examples. Although 30 independent runs and statistical evaluations have reduced the randomness in comparing stochastic optimization algorithms, existing validations are still mainly concentrated on specific building heights, structural systems, design parameters, and wind environment conditions, and are insufficient to comprehensively reflect the generalization performance of the proposed method under different engineering scenarios. Furthermore, the current wind load is handled using an equivalent static method, which does not fully consider the time-varying stochastic characteristics of actual wind fields. Future research will combine wind tunnel tests, field monitoring data, and more refined wind load time-history simulations to further validate the top horizontal displacement, inter-story drift angle, and wind-induced vibration response, and expand the applicability of the proposed method to different building heights, structural systems, and complex wind environments.

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