

Article

Design Methodology and Mechanical Behavior of Prestressed Modular Box-Type Foundation

Han Yao ¹, Chen Chen ¹, Jingbo Song ¹, Fei Guo ¹, Liang Zhang ¹, Xin Yang ² and Yang Yang ^{2,*} 

¹ Economic Research Institute, State Grid Henan Electric Power Company, Zhengzhou 450052, China; yaohan29@163.com (H.Y.); sunshin0920@sina.com (C.C.); guofei-fly@163.com (F.G.); zhangliang12@ha.sgcc.com.cn (L.Z.)

² School of Civil Engineering, Chongqing University, Chongqing 400045, China; 202516131193@stu.cqu.edu.cn

* Correspondence: 20121601009@cqu.edu.cn

Abstract

To address the limitations of traditional cast-in-place foundations, such as long construction cycles, high material consumption, and extensive on-site wet operations, this study develops a prestressed modular box-type foundation suitable for heavy load-bearing structures. A refined three-dimensional finite element model was established using Midas FEA NX 2025 software to systematically investigate the mechanical response of the structure under self-weight, concentrated live loads, pavement loads, and prestressing forces. The model comprehensively accounts for material nonlinearity, prestress effects, and foundation–soil contact nonlinearity. The results demonstrate a uniform stress distribution within the proposed foundation. Under combined load cases, the maximum tensile stress of the concrete is 0.97 MPa, which is significantly lower than the design tensile strength of C35 concrete. The application of prestressing forces effectively inhibits the initiation and propagation of cracks, with only negligible localized micro-cracks observed on the top surface of the foundation. Furthermore, the maximum vertical displacement is 1.17 mm, corresponding to a deflection ratio of 1/1709.4. These values are well within standard limit requirements, demonstrating adequate structural stiffness under the simulated conditions. This study provides a crucial numerical analysis basis and engineering design reference for the broader application of modular prestressed foundations in civil engineering and prefabricated industrial structures.

Keywords: prefabricated foundation; prestressed structure; finite element analysis; modular design; mechanical performance



Academic Editor: Humberto Varum

Received: 23 April 2026

Revised: 11 June 2026

Accepted: 22 June 2026

Published: 29 June 2026

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1. Introduction

Amid rapid global urbanization, green building practices and environmental sustainability have garnered significant attention. Traditional cast-in-place construction models face numerous challenges, including significant resource waste, prolonged construction cycles, environmental pollution, and difficulties in quality control. These conventional methods fail to meet the demands of modern development, necessitating urgent industrial transformation and upgrading. As a new form of industrialization, prefabricated construction enables a substantial increase in efficiency, reduction in environmental impact, and minimization of resource consumption by pre-manufacturing components in factories and assembling them on site. By shortening construction periods while ensuring structural quality, it offers significant economic and social benefits.

The concept of prefabricated foundations was introduced in the mid-to-late 20th century. However, due to the limitations of manufacturing processes and assembly techniques at that time, there was insufficient in-depth exploration of the complex mechanical mechanisms and joint performance of these systems. With recent advancements in materials science and computational mechanics, theories regarding precast components and connection nodes have evolved significantly, facilitating the widespread application of prefabricated foundations across various engineering sectors.

In recent years, many scholars have conducted extensive research on the structural forms and mechanical behaviors of prefabricated foundations for different application scenarios:

In the field of lightweight and modular housing, Pujadas et al. [1] investigated lightweight prefabricated foundations, exploring the application of materials such as EPS. They noted that these systems offer excellent installation efficiency and thermal performance, making them highly suitable for lightweight modular buildings. Regarding the concept of circularity in industrialized construction, researchers [2] systematically evaluated precast foundation systems such as block-type and micropile foundations, pointing out that prefabricated foundations fully align with the principles of “Design for Manufacturing and Assembly (DfMA)” in terms of material optimization, assembly efficiency, and rapid post-disaster deployment. Moreover, innovative foundation systems have demonstrated crucial adaptability in historical preservation and structural reconstruction contexts. For instance, Zucca et al. [3] successfully implemented a novel foundation system for the reconstruction of the Basilica di Collemaggio, underscoring the critical importance of modular and optimized design strategies in complex engineering scenarios.

In power transmission and substation engineering, considering the construction constraints of mountainous and complex terrains, scholars [4] conducted experimental research on assembly schemes for plate foundations used in transmission towers. They proposed various assembly techniques, including grout sleeve connections, post-tensioned prestressed connections, and post-poured strip overlaps, which effectively solve the transportation and hoisting challenges of large-scale foundations. Furthermore, to address the prefabrication needs of substation buildings, researchers [5] designed and analyzed a novel prefabricated foundation. Static load tests and finite element analysis (FEA) results indicated that the structure exhibits excellent ductility and fails in a flexural mode. It was also found that the thickness ratio of the foundation slab significantly enhances the ultimate bearing capacity. Other studies [6] performed FEA on a new precast foundation for cable termination supports, showing that the structure remains in the elastic stage under service loads, reduces concrete consumption by approximately 40%, and demonstrates superior anti-settlement performance.

In large-scale wind power engineering, to meet the challenges of increasing project scales, recent studies [7] have developed analytical models and optimized joint systems for modular precast foundations used in onshore wind turbines. By evaluating key mechanical performance indicators under dynamic wind loads, researchers demonstrated that the modular joint configurations substantially surpass design load requirements, offering high structural stability and overturning resistance. Furthermore, related research [8] introduced a cost-effective novel precast foundation for onshore wind power. Theoretical calculations and analyses showed that compared to traditional spread foundations, this new prefabricated foundation reduces concrete consumption by 30.00% and reinforcement by 34.69%, while satisfying all regulatory standards for bearing capacity and stability. Regarding the experimental validation of precast components, a recent study [9] investigated the mechanical characteristics of a prefabricated dense ribs foundation for onshore wind turbines. By conducting stacking load tests on a full-scale specimen composed of multiple prefabricated units connected via grouted plug-in joints, the researchers

verified that the ultimate bending moment of the assembled test specimen reached 83% of its theoretical section capacity, demonstrating excellent load transfer efficiency and structural reliability.

Despite these advancements, a critical research gap remains: the synergistic load-bearing mechanism of combining prestressed technology with multi-cavity modular box units for industrial equipment has not been systematically investigated. Previous studies largely focused on simple assembly joints without addressing the global stress redistribution offered by prestressing.

2. Structural Design

2.1. Design Basis

To address the engineering challenges associated with traditional cast-in-place foundations—such as prolonged construction cycles, extensive on-site wet operations, and inadequate crack resistance—this study develops a novel modular structural system. Based on design codes for concrete structures and the theory of prestressed prefabricated structures [10–13], and comprehensively considering the load characteristics of industrial equipment alongside on-site construction conditions, the proposed system establishes a synergistic load-bearing mechanism among the main bearing module, the auxiliary connection module, and the prestressed anchoring module. This approach realizes a pier-less foundation, effectively optimizing both mechanical performance and construction efficiency.

Compared to existing prefabricated foundations, which primarily rely on passive connection nodes such as grouted sleeves, post-poured strips, or bolted flanges—all inherently susceptible to localized stress concentrations—the mechanical innovation of this system lies in its active prestressing mechanism. While conventional systems transfer loads through discrete interfaces, the prestressing tendons in the proposed system actively compress the multi-cavity box units together, transforming isolated modules into a highly integrated, synergistic continuum. This active pre-compression dynamically reconstructs the internal stress field, effectively offsetting the tensile stresses induced by localized equipment loads. Although direct quantitative numerical comparisons with existing literature models, such as those for large-scale wind turbine foundations, are constrained by fundamentally different load scales and geometric boundaries, qualitatively, this prestressed box-type configuration offers superior global stress redistribution and structural integrity for industrial equipment applications. The design of this foundation comprehensively accounts for the requirements of diverse engineering scenarios. Its core advantage lies in its minimal reliance on underlying soil conditions. Consequently, this modular design mitigates typical engineering difficulties related to extensive on-site cast-in-place operations, thereby optimizing construction efficiency.

Furthermore, the structural design rigorously considers the long-term service performance and the dispersion of effective prestress. In actual engineering, effective prestress is affected by multi-source uncertainties during both the construction phase and the long-term service phase (e.g., steel relaxation and concrete shrinkage and creep), exhibiting significant randomness and non-uniformity. To scientifically evaluate and account for this service dispersion, the design basis incorporates a probabilistic analysis model based on Gaussian mixture theory. Assuming the effective prestress of a single tendon x is a normally distributed random variable, the probability density function $P(x)$ of the structural effective prestress system can be calculated using a Gaussian mixture model weighted by finite normal sub-distributions:

$$P(x) = \sum_{i=1}^n \omega_i \frac{1}{\sqrt{2\pi}s_i} \exp\left[-\frac{(x - \mu_i)^2}{2s_i^2}\right] = \sum_{i=1}^n \omega_i p(x|\mu_i, s_i^2) \quad (1)$$

where n represents the total number of prestressing tendons, ω_i is the weight coefficient of the i -th tendon ($\omega_i = 1/n$), and μ_i and s_i are the mean and standard deviation of the effective prestress for the i -th tendon, respectively. By applying this model and utilizing a 95% guarantee rate as a constraint parameter for performance evaluation, the foundation design scientifically quantifies the service dispersion of the prestressing force. This advanced theoretical evaluation ensures robust long-term reliability, deformation control, and structural safety for the industrial equipment foundation throughout its lifecycle. To interface the theoretical probabilistic model with the deterministic finite element analysis (FEA), the GMM is utilized as a statistical pre-processing tool. Instead of performing a computationally expensive stochastic finite element analysis with varying prestress values assigned to individual tendons, the GMM results are imported into the FEA model by extracting a deterministic lower-bound value that satisfies the 95% guarantee rate. The primary engineering contribution of this GMM framework is not to perform a stochastic parametric sweep, but to establish a statistically rigorous, highly reliable design baseline. It successfully bridges the complex multi-source uncertainties of actual service environments with practical, safety-oriented structural evaluation. This probabilistic evaluation mathematically justifies the uniform effective prestress value (842 MPa) applied to all tendons in the subsequent FEA model, ensuring the global numerical simulation is efficiently grounded in a conservative, 95% reliable baseline.

2.2. Modular Decomposition Design

The footprint of the equipment base covers two localized regions, each measuring 1 m $\times$ 0.5 m. To accommodate the layout dimensions of industrial equipment and the highway transportation width limit (≤ 2.5 m), the foundation is decomposed into three distinct components: the main load-bearing module, the auxiliary connection module, and the prestressed anchoring module.

As illustrated in Figure 1, each foundation module adopts a rectangular configuration, measuring 2 m in length, 1 m in width, and 0.5 m in height. In the numerical model, the foundation material properties are defined based on helical-ribbed stress-relieved steel wires, HRB400-grade steel reinforcement, and C35 concrete, in accordance with Chinese national design codes. Structurally, it comprises a base box and a cover plate.

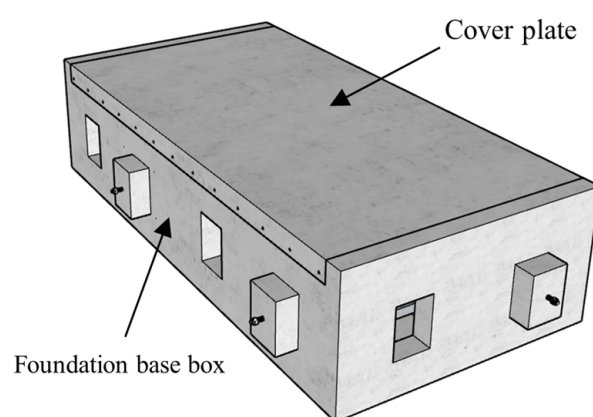


Figure 1. Prefabricated unit.

Figure 2 presents the decomposition diagram (exploded view) of the foundation. The side walls of the base box have a uniform thickness of 80 mm, which is consistent with the thickness of the internal longitudinal and transverse webs. Rectangular openings are incorporated into both the side walls and internal webs to reduce the overall weight of the foundation. The side walls and internal webs are reinforced with HRB400-grade stirrups (nominal yield strength of 400 MPa), featuring a diameter of 6 mm and a spacing of 200 mm.

The upper cover plate has a thickness of 80 mm. Its longitudinal reinforcement (oriented parallel to the long side) consists of HRB400-grade bars with a 6 mm diameter and 150 mm spacing. For the transverse reinforcement (parallel to the short side), which functions as the primary load-bearing reinforcement, HRB400-grade bars with an 8 mm diameter and 150 mm spacing are utilized. These transverse bars are positioned 30 mm from the bottom edge of the cover plate and are tied to the longitudinal bars. To ensure precise alignment and relative stability between the components, the cover plate is equipped with locating tenons, while the base box features corresponding locating mortises.

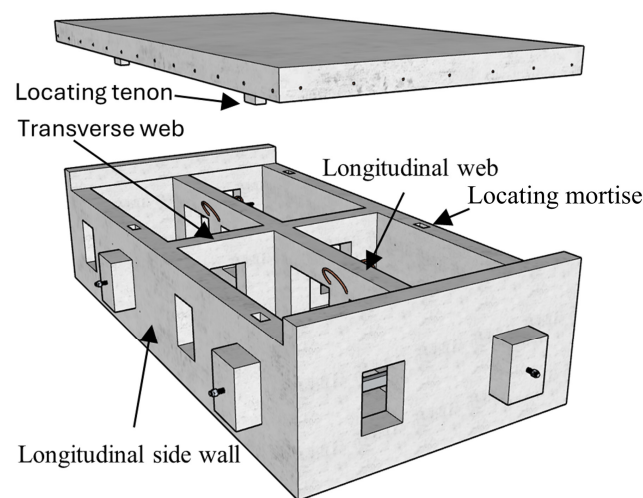


Figure 2. Foundation decomposition diagram.

Figure 3 illustrates the reinforcement cage within the base box. The horizontal reinforcement of the base box consists of HRB400-grade steel bars (nominal yield strength of 400 MPa) with a diameter of 8 mm. For the longitudinal direction, helical-ribbed prestressing steel wires are employed, featuring a characteristic tensile strength (f_{ptk}) of 1570 MPa and a diameter of 5 mm. Regarding the layout, two wires are positioned at the bottom of each longitudinal side wall, two at the bottom of each longitudinal web, and two are uniformly distributed between the side walls and the webs, resulting in a total of 10 prestressing wires at the bottom. The performance of these stress-relieved helical-ribbed steel wires complies with the requirements of “Steel wire for prestressed concrete” (GB/T 5223) [10]. The tensioning control stress for the prestressing tendons is set at $0.6f_{ptk}$.

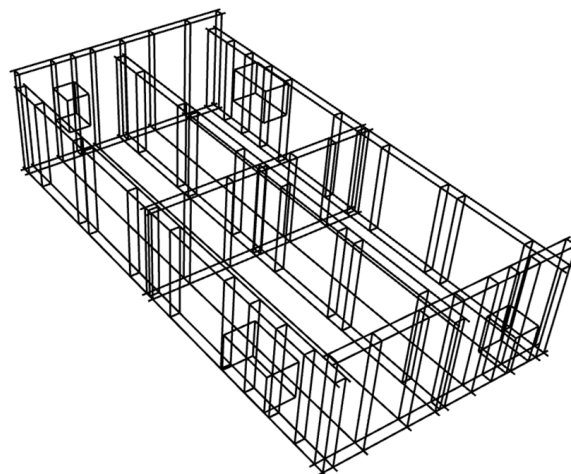


Figure 3. Reinforcement cage of the base box.

Here, f_{ptk} represents the characteristic ultimate tensile strength of the stress-relieved steel wire. During production utilizing the long-line pretensioning method, the prestress loss attributed to anchorage deformation and tendon draw-in at the tensioning end is calculated as 10.25 MPa. This calculation assumes a slip value of 5 mm for the prestressing tendons within the threaded rod anchorages at both ends, alongside a pedestal length of 100 m. During the thermal curing of the concrete, given that the temperature difference between the tensioned tendons and the pedestal is controlled within 5 °C, the prestress loss induced by this temperature variation is taken as 10 N/mm². Furthermore, the prestress loss caused by the stress relaxation of the tendons is calculated assuming low-relaxation properties, in accordance with the “Code for Design of Concrete Structures” (GB 50010) [11]. The prestress loss resulting from the shrinkage and creep of the concrete is also calculated according to this same standard [11]. If the sum of the calculated prestress losses is less than 100 N/mm², a minimum total prestress loss of 100 N/mm² is adopted. Finally, at the time of prestress transfer, the cubic compressive strength of the concrete must reach at least 75% of its specified design strength grade. It should be noted that while this code-based macroscopic approach conservatively accounts for the total prestress loss for engineering design purposes, it represents a simplified treatment of the complex, fully coupled time-dependent phenomena of creep, shrinkage, and relaxation.

Figure 4 illustrates the assembly schematic of the base box. The lifting hooks are fabricated from HPB300-grade plain round bars (nominal yield strength of 300 MPa) with a diameter of 12 mm. To effectively prevent water accumulation within the structure, drainage holes are incorporated to promptly discharge any rainwater seeping into the base box directly into the underlying foundation. Once the base box is assembled, the two adjacent units can be connected using angle steels and bolts.

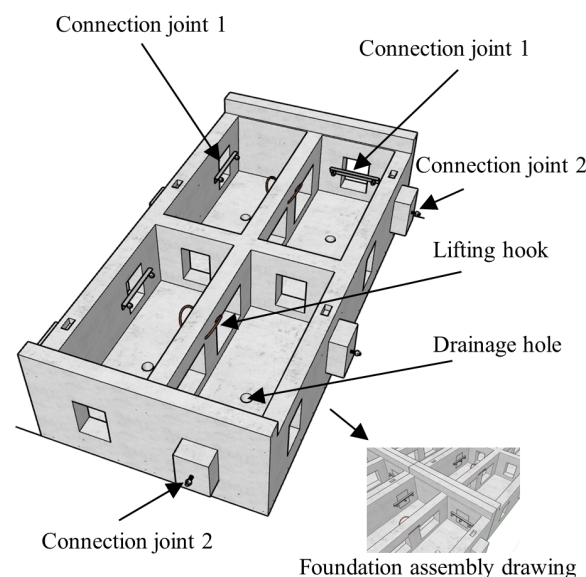


Figure 4. Assembly components of the base box.

3. Constitutive Model Analysis and Parameter Determination

In this study, the general-purpose finite element analysis software Midas FEA NX 2025 (<https://product.midasit.cn/index/products-FEANX-Building.asp>, accessed on 12 June 2026) is employed to conduct the structural analysis of the foundation unit. Both concrete and steel reinforcement (including prestressing tendons) are defined as isotropic materials within the software environment [12], assuming uniform physical properties in all directions. Specifically, the C35 concrete is modeled using solid elements, while the HRB400-grade steel reinforcement is simulated using embedded truss elements. The

mechanism of the embedded truss involves embedding the truss elements directly into the concrete solid elements. The software facilitates the transfer of forces and deformations between these components through automatically established nodal constraint relationships. Specifically, under tension, the tensile force is transmitted to the concrete via bonding stress; under compression, the stress is transferred through contact constraints. Consequently, the stiffness of the embedded truss elements works synergistically with that of the concrete solid elements to collectively resist external loads. Although bond-slip between steel and concrete [13] is an inherent phenomenon in reinforced concrete structures, it is neglected in this analysis. This assumption is justified by the clear load-transfer paths, the relatively small magnitudes of the applied vertical static loads, and the absence of cyclic loading effects. Under such service conditions, the internal stresses remain well within the elastic range, rendering the localized slip effect macroscopically negligible and ensuring computational efficiency without compromising the accuracy of the global structural response.

3.1. Concrete Constitutive Model

Finite element analysis for concrete structures is now a well-established methodology [14,15]. Selecting an appropriate constitutive model tailored to specific project requirements is fundamental to ensuring the accuracy of numerical results. Experimental studies [16–18] have demonstrated that the primary source of error in FEA typically stems from the inability of the selected constitutive model to accurately characterize the mechanical behavior of the concrete material. Therefore, a rigorous investigation into concrete constitutive relationships—which serve as the mathematical expressions of a material's mechanical properties—is essential. Due to its low tensile strength, concrete is susceptible to cracking under minimal tensile stress and frequently operates in a post-cracked regime. Consequently, the actual stress–strain relationship deviates significantly from a linear elastic constitutive model. Non-linear constitutive equations, derived from extensive experimental data while accounting for various influencing factors, provide a more realistic representation of structural performance. As a result, they are now widely utilized in the non-linear FEA of reinforced concrete structures. The typical uniaxial stress–strain curve for concrete is illustrated in Figure 5.

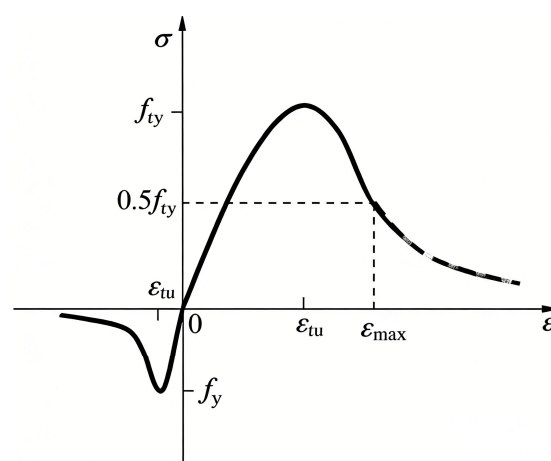


Figure 5. Uniaxial stress–strain curve of concrete.

Approximate formulas are adopted in practical engineering [11]:

$$\text{Ascending branch: } \sigma_c = \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_0} \right)^2 \right], \quad 0 \leq \varepsilon \leq \varepsilon_0$$

$$\text{Descending branch: } \sigma_c = f_c, \quad \varepsilon_0 \leq \varepsilon \leq \varepsilon_u$$

3.2. Constitutive Models for Ordinary Reinforcement and Prestressing Strands

Steel reinforcement is a structural component primarily subjected to tension. Its tensile stress–strain relationship curves are generally classified into two categories: those with a distinct yield plateau and those without a distinct yield plateau. Ordinary reinforcement typically exhibits a clear yield plateau, which can be characterized by three distinct stages: the elastic stage, the yield stage, and the strain-hardening stage. Conversely, for prestressing steel (such as high-strength carbon steel wires and strands), the stress–strain relationship generally lacks a distinct yield plateau. The schematic constitutive relationships for both types of steel are illustrated in Figure 6.

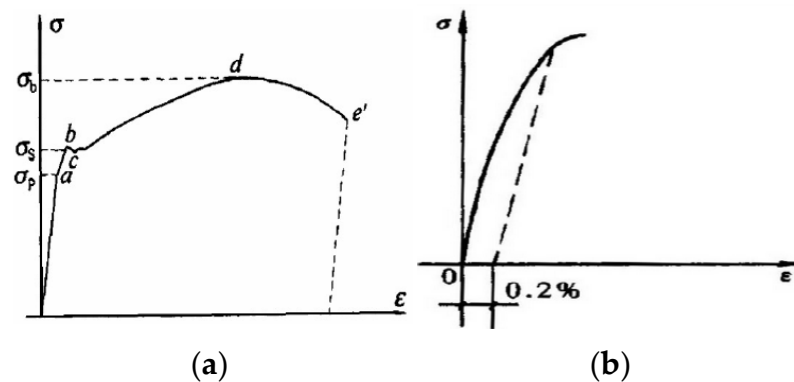


Figure 6. Constitutive relationship curves of steel reinforcement: (a) Constitutive relationship of ordinary reinforcement; (b) constitutive relationship of prestressing steel.

According to the provisions of Clause 6.1.8 in the “Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts” (JTG 3362-2018) [19], for the design of pre-tensioned prestressed concrete components, the actual stress of the prestressing tendons within the transfer length (l_{tr}) is assumed to be zero at the component end and reaches the effective prestress value (σ_{pe}) at the end of the transfer length. The stress distribution between these two points is assumed to vary linearly. According to Table 6.1.8 of the specification, the transfer length l_{tr} is taken as $60d$. Specifically, when the construction process involves the sudden release of prestressing tendons, the calculation should initiate from a distance of $0.25l_{tr} = 75$ mm from the member end.

In this study, based on the tensioning control stress of $0.6f_{ptk}$ (0.6×1560 MPa = 942 MPa) minus the minimum code-mandated total prestress loss of 100 MPa, the calculated effective prestress of the foundation’s tendons applied in the simulation is deterministically established as $\sigma_{pe} = 842$ MPa ($842 \times \frac{\pi \times 5^2}{4} = 16,524.25$ N).

There are three primary modeling approaches for ordinary reinforced concrete structures: the smeared model, the discrete model, and the embedded model. In this project, given the relatively small amount of reinforcement, the discrete modeling approach was adopted to achieve more ideal analysis results. This method involves treating the steel reinforcement and the concrete as distinct types of elements. Throughout the computational process, no convergence issues were encountered.

Modeling methods for prestressed concrete structures primarily include the equivalent load method and the solid tendon method. While both approaches possess distinct advantages and limitations, the solid tendon method was selected for this study. In this approach, the prestressing tendons and concrete are modeled using separate elements, and the prestress is applied by defining specific attributes for the tendon elements. Compared to the equivalent load method, the solid tendon method enables a more accurate simulation of the stress state within the prestressed structure.

4. Development of Finite Element Model

In the precast foundation model developed in this study, the prestressing tendons are characterized by their limited quantity and simple geometric profiles. The pre-tensioned prestress is applied directly to the embedded truss elements. By utilizing the built-in function feature for prestressing within the software, the prestress magnitudes at both ends of the tendons are modulated to comply with the requirements specified in Clause 6.1.8 of the “Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts” (JTG 3362-2018) [19]. The finite element models of the foundation unit and the reinforcement framework are illustrated in Figure 7 and Figure 8, respectively.

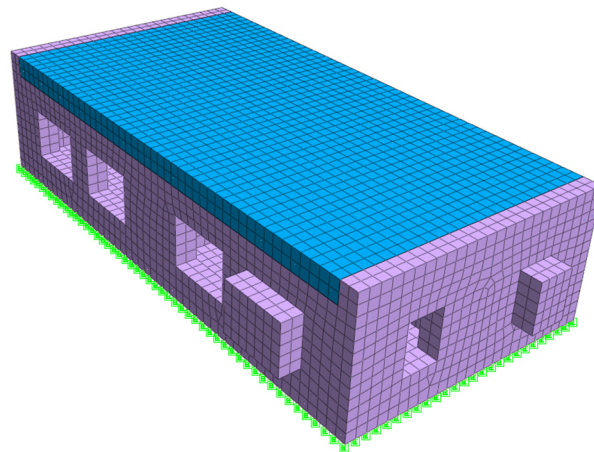


Figure 7. Finite element model of the foundation unit.

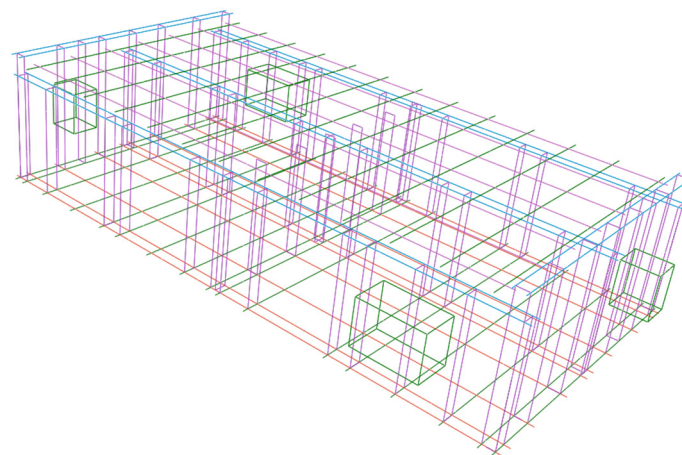


Figure 8. Finite element model of the reinforcement cage.

The C35 concrete is simulated using solid elements. In accordance with relevant design codes, the elastic modulus is set to 3.15×10^4 MPa, with a Poisson's ratio of 0.2, a unit weight of 2.5×10^{-5} N/mm³, a design tensile strength of 1.57 MPa, and a design compressive strength of 16.7 MPa. A uniform mesh size of 40 mm is initially adopted for the concrete solid elements, which is judged based on engineering experience to satisfy the requirements for global performance analysis. For the HRB400-grade steel reinforcement, the Poisson's ratio and unit weight are 0.3 and 7.85×10^{-5} N/mm³, respectively, with a design tensile strength of 360 MPa and a mesh size of 40 mm. The prestressing system employs helical-ribbed steel wires with a diameter of 5 mm. The characteristic ultimate tensile strength (f_{ptk}) is taken as 1570 MPa, and the effective prestress (σ_{pe}) is 842 MPa. These prestressing wires share the same Poisson's ratio (0.3), unit weight (7.85×10^{-5} N/mm³), and mesh size (40 mm) as the ordinary reinforcement.

The foundation in this project is positioned directly on the subgrade. The modulus of the subgrade reaction is determined with reference to the empirical values provided in Appendix H.0.1 of the “Code for Geotechnical Investigation of Urban Rail Transit” (GB 50307-2012) [20]. Following unit conversion, a conservative value of 0.04 N/mm^3 is adopted. While this constant modulus approach is a simplified representation of the inherently nonlinear soil behavior, it is adopted as a conservative baseline suitable for the preliminary design phase, particularly given the relatively small magnitude of the applied static equipment loads. Due to the current study’s focus on the internal structural mechanics of the prestressed module, a systematic sensitivity analysis across different soil types was not conducted. Instead, this relatively low modulus (0.04 N/mm^3) was deliberately selected to simulate unfavorable support conditions, thereby conservatively verifying the structural strength of the concrete box under worse-case bending scenarios. This soil–structure interaction is simulated by applying surface spring supports to the bottom of the foundation within the finite element environment.

It should be noted that a systematic grid convergence analysis was not performed in this preliminary study. The selection of the 40 mm mesh size is based on technical recommendations from Midas FEA and established literature on similar prestressed concrete structures. For a foundation with dimensions of $2000 \text{ mm} \times 1000 \text{ mm} \times 500 \text{ mm}$, this mesh density is generally considered sufficient to capture global stress distributions and deformation characteristics accurately during the conceptual design stage.

In the absence of physical laboratory tests or field applications at this conceptual stage, the reliability and accuracy of the finite element model are analytically verified through strict adherence to established structural mechanics and design codes. The constitutive models for the materials (C35 concrete and HRB400 reinforcement) and the empirical formulations for prestress losses are directly adopted from the widely validated “Code for Design of Concrete Structures” (GB 50010), which is fundamentally supported by extensive independent experimental databases. Furthermore, under the specified service loads, the foundation operates strictly within the uncracked, linear-elastic regime. Within this mechanical state, the numerical solutions provided by the established commercial software (Midas FEA NX 2025 (<https://product.midasit.cn/index/products-FEANX-Building.asp> accessed on 12 June 2026)) are mathematically robust and provide highly reliable baseline predictions for preliminary design, although subsequent full-scale physical validation remains essential before field deployment.

Furthermore, the simulation results indicate that the maximum tensile stress (0.97 MPa) is significantly lower than the material strength (1.57 MPa), and the maximum vertical displacement (1.17 mm) is well within the regulatory limits. Even if localized stress peaks were to increase by an estimated 10–20% through further mesh refinement, the structure would remain within a safe operational range. Thus, the current mesh provides a reliable and conservative basis for the design conclusions. However, future studies focusing specifically on the anchorage zones and opening corners are recommended to employ localized mesh refinement to further verify numerical convergence.

5. Mechanical Performance Analysis

5.1. Analysis of Single Prestressing Load Case

To verify the uniformity of the internal prestress distribution and to screen for potential risks of concrete crushing caused by localized stress concentrations, the stress state of the prestressed precast modular industrial equipment foundation was analyzed following the completion of field tensioning. At this stage, the foundation is subjected solely to the combined action of prestress and self-weight (dead load). The first principal stress distribution under this single prestressing load case is illustrated in Figure 9.

As illustrated in Figure 10, the overall stress level of the component remains within a reasonable range. The top surface exhibits a moderate tensile stress with a gradient distribution ranging from 0.2 MPa to 0.4 MPa, while stress peaks emerge in peripheral and localized areas due to load transfer effects. The maximum tensile stress of the foundation is 0.505 MPa, primarily concentrated at the anchorage zones of the prestressing tendons. This concentration is attributed to the tensile forces generated in the concrete by the contraction of the tendons, yet these values remain below the design tensile strength of the C35 concrete. Geometric discontinuities are identified as the primary drivers of stress concentration: at the corners of openings and the connection interfaces between auxiliary side structures and the main body, the tensile stress increases significantly. This creates localized high-stress zones where peak values exceed those in uniform regions by 30% to 50%, indicating critical areas susceptible to cracking. Conversely, the bottom corners and certain side surfaces are dominated by compressive stress or low tensile stress (less than 0.1 MPa), indicating a relatively safe mechanical state. This analysis provides fundamental data for subsequent calculations of prestress loss under standard load combination cases.

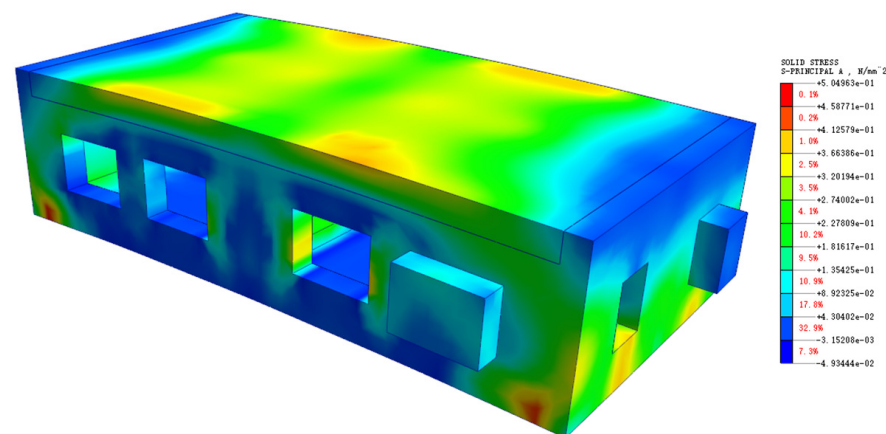


Figure 9. First principal stress under single prestressing load case.

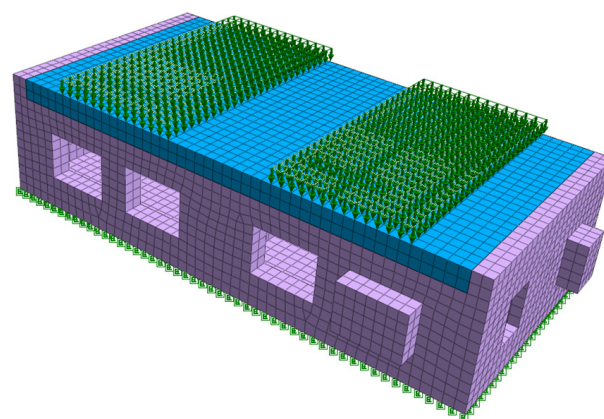


Figure 10. Application of equipment loads.

5.2. Analysis of Equipment Loading Cases

To evaluate the mechanical performance of the equipment foundation under actual service conditions, two categories of loads were applied. First, a uniformly distributed paving load of 1 kN/m^2 was applied to the top surface to simulate the long-term dead load of the surfacing layer. Second, the specific industrial equipment load was incorporated. The proposed foundation is primarily designed to support compact prefabricated power substation equipment. In the numerical simulation, a generic highly integrated HGIS (Hybrid Gas Insulated Switchgear) device is utilized as the theoretical design prototype to define

the applied load. Based on standard miniaturization parameters for rapid deployment, the maximum equipment design weight is set at 7t (approximately 68.67 kN). Following miniaturization optimization to facilitate rapid deployment and modular transportation, the maximum HGIS equipment weight is set at 7t (approximately 68.67 kN). Based on the equipment's geometric dimensions and the field layout scheme, the most unfavorable load case was selected for mechanical analysis. Specifically, the total equipment load was simplified as an equivalent uniform load of 70 kN/m², acting on two localized 1 m × 0.5 m areas on the top surface. This configuration is designed to account for extreme loading scenarios, such as center of gravity offsets and eccentric loading. The equipment loading schematic is illustrated in Figure 10.

In this study, a partial factor of 1.3 is adopted for both the paving and dead loads (self-weight), while a partial factor of 1.5 is applied to the equipment loads. Regarding the prestressing force, a partial factor of 1.0 is utilized as it acts as a favorable action in this scenario. Specifically, the peak tensile stress of 0.97 MPa utilizes only approximately 61.7% of the design tensile strength of C35 concrete (1.57 MPa), quantitatively verifying a significant safety margin against initial cracking. Under this specific load combination, the computational results are illustrated in Figure 11.

As demonstrated in Figure 12, the overall stress level of the component ranges from −0.62~0.97 MPa. A core zone of high tensile stress forms on the top surface within the equipment loading area, reaching a peak value of 0.97 MPa. This stress subsequently undergoes gradient attenuation to a moderate tensile stress range of 0.4~0.7 MPa in the surrounding regions. This distribution is a direct consequence of the coupling effect between the concentrated equipment loads and the uniformly distributed paving loads. Geometric discontinuities serve as secondary drivers for stress concentration: at the corners of side openings and the connection interfaces between auxiliary structures and the main body, tensile stresses rise significantly to 0.2~0.5 MPa. This represents a 30%~50% increase compared to uniform areas, further aggravating localized stress risks. Most areas on the side and bottom of the component are dominated by low tensile stresses (0.1~0.3 MPa), while localized corners exhibit a compressive stress state ranging from −0.2~−0.62 MPa, indicating a relatively safe mechanical state. The analysis reveals that the concentrated equipment load is the primary factor inducing high tensile stress on the top surface. When superimposed with the effects of geometric mutations from openings and auxiliary structures, the risk of cracking failure is significantly elevated. Should larger equipment loads be encountered in scenarios without miniaturized units, optimization strategies—such as localized reinforcement, the introduction of fillet transitions, or the addition of stiffeners—could be employed to mitigate stress concentration effects, thereby enhancing the load-bearing capacity and long-term service safety of the equipment foundation.

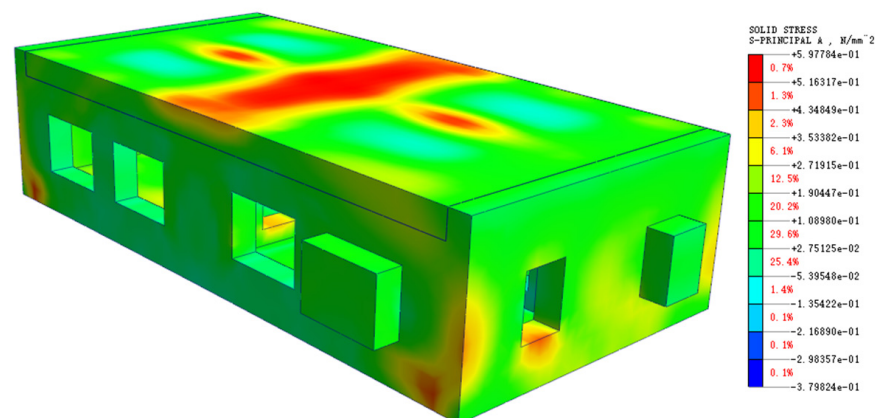


Figure 11. Stress distribution of the foundation under all applied loads.

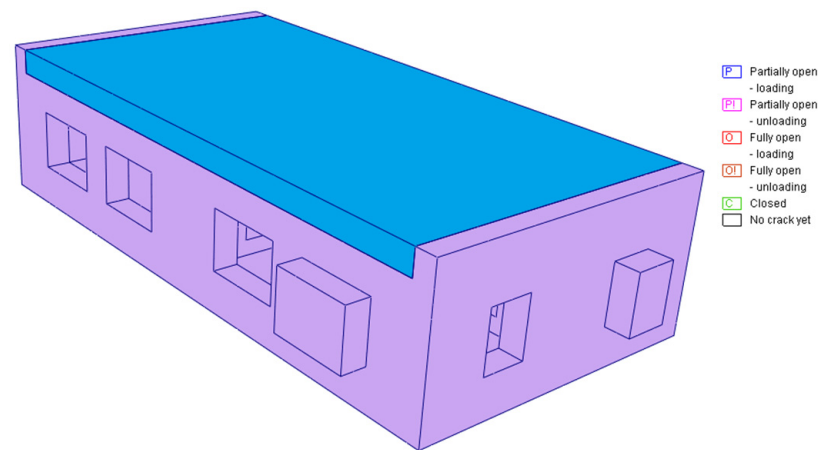


Figure 12. Results of crack analysis.

5.3. Analysis of Cracking Resistance

Figure 12 presents the cracking state contour map of the equipment foundation. The maximum vertical displacement of the component is 1.17 mm, corresponding to a deflection-to-span ratio of 1/1709.4. This value is significantly lower than the allowable limits specified in the design codes, indicating that the foundation possesses an ample stiffness reserve and well-controlled deformation. Regarding the cracking state, only localized micro-cracks are observed on the top surface, while all other regions remain uncracked. No fully developed structural cracks or residual cracks after unloading are present. The crack widths are negligible, thereby verifying the crack resistance and safety of the foundation under the standard load combination. Although localized stress concentrations and minor deformations exist, both the serviceability and crack resistance performance satisfy the regulatory requirements, demonstrating that the foundation possesses excellent load-bearing capacity and long-term service reliability.

6. Limitations of the Study

While this study provides an in-depth investigation into the mechanical behavior of the foundation using refined finite element models, certain limitations must be acknowledged. The present research findings are predominantly based on numerical simulations, and physical model testing has not yet been carried out for cross-validation. At this current stage, the proposed computational model is primarily intended for preliminary performance prediction, scheme comparison, and extensive parametric studies during the initial design phase. However, given the inherent limitations of numerical simulations—such as the simplification of boundary conditions, idealized material constitutive relationships, and complex nonlinear contacts—the theoretical data derived from this model cannot yet serve as a substitute for the physical testing certification required in practical engineering applications.

Furthermore, while the current global FEA model successfully captures the macro-stress distribution, its uniform 40 mm mesh is too coarse to accurately evaluate highly localized micro-mechanical phenomena such as bursting and spalling stresses in the immediate vicinity of the prestressing anchorages, and in this preliminary design phase, these localized structural risks are macroscopically mitigated by assuming strict adherence to code-mandated anti-bursting reinforcement detailing, for instance localized spiral stirrups; however, future investigations must involve the development of highly refined localized sub-models of the anchorage zones to explicitly analyze these complex bursting and spalling stress gradients and accurately optimize the local reinforcement configurations.

Moreover, the current study adopts fixed geometric dimensions ($2 \times 1 \times 0.5$ m) that are strictly dictated by specific equipment footprints and transportation limits, and thus lacks a systematic parametric size optimization. Qualitatively, scaling the foundation modules would significantly alter the mechanical behavior: larger modules would increase flexural spans and shear demands, necessitating enhanced prestressing; conversely, smaller modules might encounter issues regarding global overturning stability and insufficient prestress transfer lengths. Therefore, future parametric investigations must evaluate these scale effects across various module sizes to establish comprehensive, size-independent design guidelines. Additionally, the current FEA relies on a deterministic effective prestress value (842 MPa) derived from standard design codes. Future investigations will incorporate parametric sensitivity analyses across varying pre-tension levels to systematically evaluate their impact on the balance between anchorage zone safety and mid-span crack resistance.

Therefore, to further enhance the reliability and engineering applicability of the proposed numerical model, systematic full-scale (or large-scale) physical model tests are planned for future research endeavors. Subsequent studies will focus on thoroughly comparing the actual mechanical responses obtained from the physical tests (e.g., stress–strain distribution, deformation characteristics, and ultimate failure modes) with the existing FE analytical results. This comparative analysis will be utilized to accurately calibrate and demonstrating the key parameters within the numerical model. Such efforts will not only provide more robust and reliable experimental data support for the practical engineering application of this research, but also further improve and enrich the relevant theoretical calculation framework. Furthermore, future numerical models will incorporate complex nonlinear soil–structure interaction models to replace the current simplified constant subgrade modulus. Additionally, a systematic parametric sensitivity analysis across a wide spectrum of soil profiles, ranging from soft clay to dense sand, must be conducted to fully evaluate the generalized geotechnical applicability and differential settlement behavior of the foundation system under diverse geological conditions. Crucially, because the active prestressing mechanism significantly enhances the global flexural stiffness of the modular foundation, it theoretically offers superior structural resistance against cracking induced by uneven ground movements. However, explicitly simulating and quantifying the structural response under severe differential settlement scenarios—by imposing non-uniform geotechnical stiffness gradients—remains a limitation of the current uniform-spring model and will be a primary focus of our subsequent coupled structural-geotechnical investigations. While the long-term limits of fully coupled time-dependent phenomena, such as concrete creep, shrinkage, and steel relaxation, were macroscopically accounted for through maximum prestress loss calculations based on design codes, explicitly assessing the time-varying evolution of long-term deformations remains a limitation, and to accurately assess these progressive long-term behaviors, future phases will require advanced transient finite element modeling over the service life, incorporating time-dependent material constitutive models, for instance the CEB-FIP model, to simulate the continuous structural degradation.

Finally, while the GMM framework successfully provided a conservative deterministic input for the current model, the full parametric effect of the prestress probability distribution on the structural response was not demonstrated. Future research will employ SFEA (Stochastic Finite Element Analysis) or Monte Carlo simulations to explicitly evaluate the structural reliability and capture how the probabilistic dispersion of prestressing alters the foundation's mechanical behavior.

7. Conclusions

This study developed a prestressed precast modular industrial equipment foundation, and its mechanical performance was predicted through refined finite element analysis. The primary conclusions are summarized as follows:

- (1) The developed refined FEA model—which accounts for material nonlinearity, prestress transfer effects, and foundation–soil contact nonlinearity—accurately captures the structural mechanical response. This provides an effective numerical framework for the design and performance evaluation of similar types of precast industrial equipment foundations.
- (2) Under the standard load combination, the maximum tensile stress in the concrete is 0.97 MPa, which is significantly lower than the design tensile strength of C35 concrete. The maximum vertical displacement is 1.17 mm, yielding a deflection-to-span ratio of 1/1709.4. This indicates sufficient stiffness reserves beyond the regulatory limit (typically 1/500 for industrial equipment foundations), indicating ample stiffness reserves and crack resistance redundancy. Such precise deformation control effectively mitigates installation deviations and ensures the operational stability of the supported equipment.
- (3) The application of prestressing effectively reconstructs the internal stress field of the foundation, substantially reducing tensile stress levels on the top surface and mitigating the large-scale cracking issues prevalent in traditional cast-in-place foundations. Furthermore, the prestressing effect enhances the synergistic load-bearing performance between the precast modules, addressing the common issue of non-uniform stress distribution at connection interfaces and ensuring the structural integrity and reliability of the assembly.

Author Contributions: Methodology, Y.Y.; software, H.Y.; validation, L.Z.; formal analysis, H.Y.; investigation, C.C. and F.G.; resources, C.C.; data curation, H.Y. and Y.Y.; writing—original draft preparation, X.Y. and Y.Y.; writing—review and editing, X.Y. and Y.Y.; visualization, J.S. and L.Z.; supervision, F.G.; project administration, J.S.; funding acquisition, Y.Y. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the Science and Technology Project of State Grid Henan Electric Power Company, grant number 5217L025002P.

Data Availability Statement: The data and models presented in this study are not publicly available due to strict confidentiality agreements. However, the data are available from the corresponding author on reasonable request and subject to appropriate non-disclosure agreements, to verify the authenticity of the research.

Acknowledgments: The authors would like to express their gratitude to the engineers and administrative staff at State Grid Henan Electric Power Company for their technical discussions and support during the data collection phase of this research.

Conflicts of Interest: Authors Han Yao, Chen Chen, Jingbo Song, Fei Guo and Liang Zhang were employed by the company State Grid Henan Electric Power Company. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

Abbreviation

The following abbreviations are used in this manuscript:

FEA finite element analysis

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