

## Article

# Design and Analysis of a Novel Prefabricated Foundation for Substation Buildings

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**Abstract:** In recent years, prefabricated components have been widely used in the construction of substation superstructures, while cast-in-place foundations remain the primary method for substation foundations. This paper presents and evaluates a novel prefabricated foundation design aimed at enhancing construction efficiency and load-bearing performance. The foundation features a modular design, with each module weighing only half that of a cast-in-place foundation of the same size, significantly improving construction convenience and transportation efficiency. The load-bearing performance of the foundation was validated through static load tests and finite element modeling. The results indicate that the foundation demonstrates excellent ductility, with flexural failure as the primary mode, characterized by multiple cracks across the mid-span and complete yielding of longitudinal reinforcing steels. Further parametric analysis shows that increasing the plate thickness ratio ( $\lambda$ ) improves the ultimate bearing capacity of the foundation significantly. Additionally, enlarging the cross-sectional size of the shear key or increasing the strength of the wet joint material enhances overall structural synergy, reduces local deformation, and improves load distribution efficiency. Overall, the sensitivity order of factors influencing the bearing capacity of the new prefabricated foundation is plate thickness ratio ( $\lambda$ ) > wet joint strength > shear key cross-sectional size.

**Keywords:** substation construction; prefabricated extended foundation; static load test; finite element analysis; load-bearing performance



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## 1. Introduction

Currently, the world is confronting dual challenges related to energy and technology. To address these issues and promote sustainable development, the global energy sector is shifting towards diversification, cleanliness, and low carbonization, with the ultimate goal of achieving carbon neutrality [1,2]. Strategic priorities for this transformation encompass the integration and expansion of renewable energy sources, enhanced energy efficiency, and the use of environmentally friendly materials. The rapid deployment of clean energy technologies is increasingly recognized as a vital driver of this transition [3]. However, clean energy generation facilities are often located in remote, resource-rich areas, such as deserts and offshore wind farms [4–8], which presents unique challenges for energy integration. Substations, as crucial components of power systems, play a key role in accessing and incorporating renewable energy, thereby supporting energy diversification and transformation. According to recent industry analyses by Fortune Business Insights, the global substation market size is projected to reach USD 139.23 billion in 2023, with an anticipated increase to USD 145.28 billion in 2024, ultimately reaching USD 204.82 billion by 2032, at a compound annual growth rate of 4.4%.

Furthermore, the EU Action Plan of the International Energy Agency estimates that approximately USD 633 billion will be invested in the power grid by 2030, with about 68.6% allocated to distribution networks. In line with the rise of distributed generation, substations are expected to facilitate the integration of local renewable energy sources,

promoting decentralized, resilient, and self-sufficient energy systems. Future research could explore advanced technologies within substations to optimize decentralized energy integration further and enhance system resilience.

Environmental pressures and resource constraints stemming from global climate change and energy shortages have rendered energy demand and its impacts critical drivers of the worldwide energy transition and sustainable development [9–12]. Traditional substation construction, which encompasses civil engineering, equipment installation, and structural design, frequently relies on cast-in-place methods. This approach prolongs construction timelines and escalates energy consumption. Between 2010 and 2020, energy consumption increased by 8.9%, with the industrial and construction sectors accounting for 66% and 26% of this total, respectively [13]. This trend has significant implications for surrounding environments and raises pressing sustainability concerns [14]. Prefabricated substations offer solutions to these challenges by reducing construction waste by 50% [15], adapting to complex environments, decreasing resource use by 35.82%, and mitigating damage to health and ecosystems [16]. These substations provide flexibility, scalability, energy efficiency, and sustainability. This paper investigates the potential of prefabricated substations to enhance resource efficiency and align with decarbonization objectives while recommending strategies for their integration. Industry forecasts indicate a growing demand for prefabricated substations to facilitate renewable energy integration in remote areas. Future research should focus on advanced materials and modular designs to optimize prefabricated substations for diverse environments and energy requirements.

Currently, substations, both domestically and internationally, have increasingly adopted prefabricated or fully prefabricated construction methods. However, these prefabricated components are primarily utilized for the superstructure of substations [17–20]. For smaller buildings or power transmission equipment, integral prefabricated foundations are employed [21]. Typically, the mass and volume of such foundations are limited, which restricts their bearing capacity and results in a lack of bending resistance. In contrast, larger structures, such as main control buildings and distribution buildings, continue to rely on cast-in situ foundations, as large monolithic prefabricated foundations incur significant transportation and hoisting costs. To further advance the development of construction industrialization and the establishment of fully prefabricated substations, it is essential to enhance the design and development of prefabricated foundations.

The system developed by Yu et al. incorporates modular rib panels, making it suitable for applications such as precast concrete wall panels, garage floors, and sidewalks while also offering effective thermal insulation [22]. The new onshore wind turbine prefabricated foundation designed by Li et al. utilized theoretical calculations and finite element analysis to reduce concrete and steel bar consumption by 30.00% and 34.69%, respectively [23]. Wang et al. designed a prefabricated foundation for a wind turbine that employs key tooth joints and prestressed dry connections, with analysis indicating that the shear structure is a critical factor in its performance [24]. Additionally, Li et al. conducted static load tests and numerical analyses on prestressed high-strength concrete (PHC) pipe piles, revealing the main influencing factors on their ultimate bearing capacity in multi-layer soil, thus providing a foundation for optimizing pipe pile performance [25]. The current literature primarily focuses on the construction efficiency of modular prefabricated foundations, with limited attention to connection forms and their impact on structural synergy. Moreover, most studies focus on small-scale equipment foundations, which are unsuitable for large buildings.

This article integrates the functional requirements of the main substation building with the benefits of an extended foundation while considering the convenience of transportation and hoisting. A design scheme for a new type of prefabricated extended foundation is proposed. The solution aims to optimize the existing form of the prefabricated extended foundation by effectively distributing the load and reducing the lifting costs associated with the structure. The contents of this article are organized as follows: Section 2 presents the test procedure; Section 3 analyzes the test results; Section 4 establishes the finite element

model and compares the outcomes of the model with the test results; Section 5 details the parametric analysis conducted to identify significant variables influencing the load-bearing performance of the new assembly; and Section 6 concludes the paper.

## 2. Experimental Program

### 2.1. Specimen Design

Existing modular prefabricated foundations utilize prestressed steel reinforcement for connections between modules; however, the seams between these modules compromise the initial stiffness of the foundation. Over time, the prestress in the steel reinforcement diminishes, and the exposed steel becomes susceptible to rust, thereby reducing the service life of the foundation. To tackle the challenges of low initial stiffness and inadequate surface strength in block foundations, we propose an innovative design for an assembled extended foundation that integrates the structural form of the extended foundation. Compared to traditional foundation designs, the proposed foundation demonstrates superior performance in enhancing initial stiffness, extending service life, and meeting the demands of large buildings. It also optimizes the synergistic effect of wet joints and shear keys, providing a practical solution for both the industrialized construction of substations and the improvement of foundation performance.

Figure 1 illustrates the design concept of the novel prefabricated foundation, composed of components 1, 2, and 3. Under typical conditions, stress concentration is most pronounced at the mid-span of the foundation. To improve the initial stiffness and strength, wet joints are incorporated between components 1 and 2, providing excellent bending and shear resistance. These are commonly used in multi-sectional structural connections [26,27]. Components 1 and 2 together form the foundation base, while component 3 serves as the pedestal. The pedestal and base are connected via embedded reinforcing steels and high-strength bolts. To further enhance initial stiffness and overall structural integrity [28], four rectangular shear keys are positioned between the base and the pedestal.

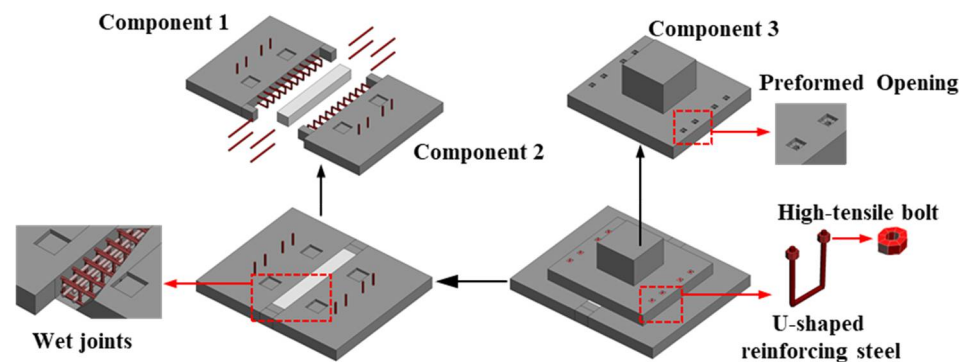


Figure 1. Design concept.

The base dimensions and thickness of the novel prefabricated foundation were determined using the design value from the standard load combination and the characteristic foundation bearing capacity. The base reinforcement of the novel prefabricated foundation follows that of a cast-in-place model of equivalent dimensions. After calculating the reinforcement area, the longitudinal bars were cut at the midpoint and designed with hooks to facilitate later assembly. The pedestal section was reinforced according to the minimum reinforcement ratio.

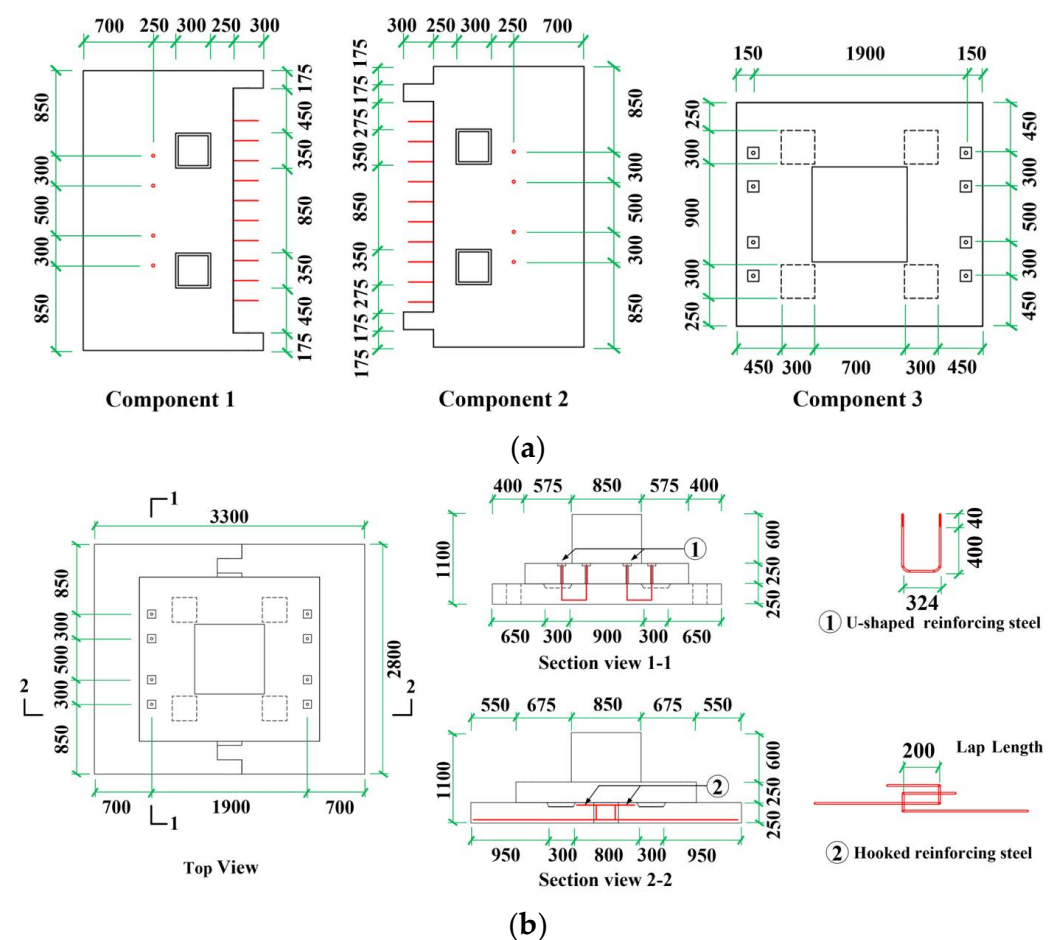
As stipulated in Section 3.0.5 of the literature [29], the crack width calculation for the prefabricated foundation under normal service conditions uses the effect design value derived from the standard load combination, while for strength evaluation at the limit state, the effect design value from the basic load standard combination is applied. Table 1 presents the load combination results for the main control building of the 220 V substation, and Table 2 details the associated foundation soil parameters. The dimensions of the test specimen are presented in Figure 2.

**Table 1.** Physical and mechanical properties of foundation soil.

| Name      | Thickness (m) | Density (kg/m <sup>3</sup> ) | Internal Friction Angle (°) | Cohesion Force (kPa) | Elastic Modulus (MPa) |
|-----------|---------------|------------------------------|-----------------------------|----------------------|-----------------------|
| Soft clay | 6.7           | 1800                         | 25                          | 11                   | 8.58                  |

**Table 2.** The standard combination of superstructure loads.

| Load Combination           | Axial Pressure (kN) | Bending Moment 1 (kN·m) | Bending Moment 2 (kN·m) |
|----------------------------|---------------------|-------------------------|-------------------------|
| Characteristic Combination | 1100.0              | 185.1                   | 86.7                    |
| Fundamental Combination    | 1485.0              | 207.3                   | 97.1                    |
| Test Combination           | 2200.0              | 370.2                   | 173.4                   |

**Figure 2.** The dimensions of the test specimen: (a) plan view of components 1, 2, and 3; (b) overall dimensions.

## 2.2. Test Setup and Instrumentation

The static load test is a commonly used and direct method for evaluating foundation-bearing capacity, particularly suitable for testing the foundation performance of heavy structures such as high-rise buildings and industrial plants. Based on static load test results, the bearing capacity and settlement deformation characteristics of an independent foundation under column loading can be directly assessed, providing a reliable basis for structural design and engineering applications. The overall layout of the static load test for the novel prefabricated foundation is shown in Figure 3a. Figure 3b–d displays photos of the static load test site.

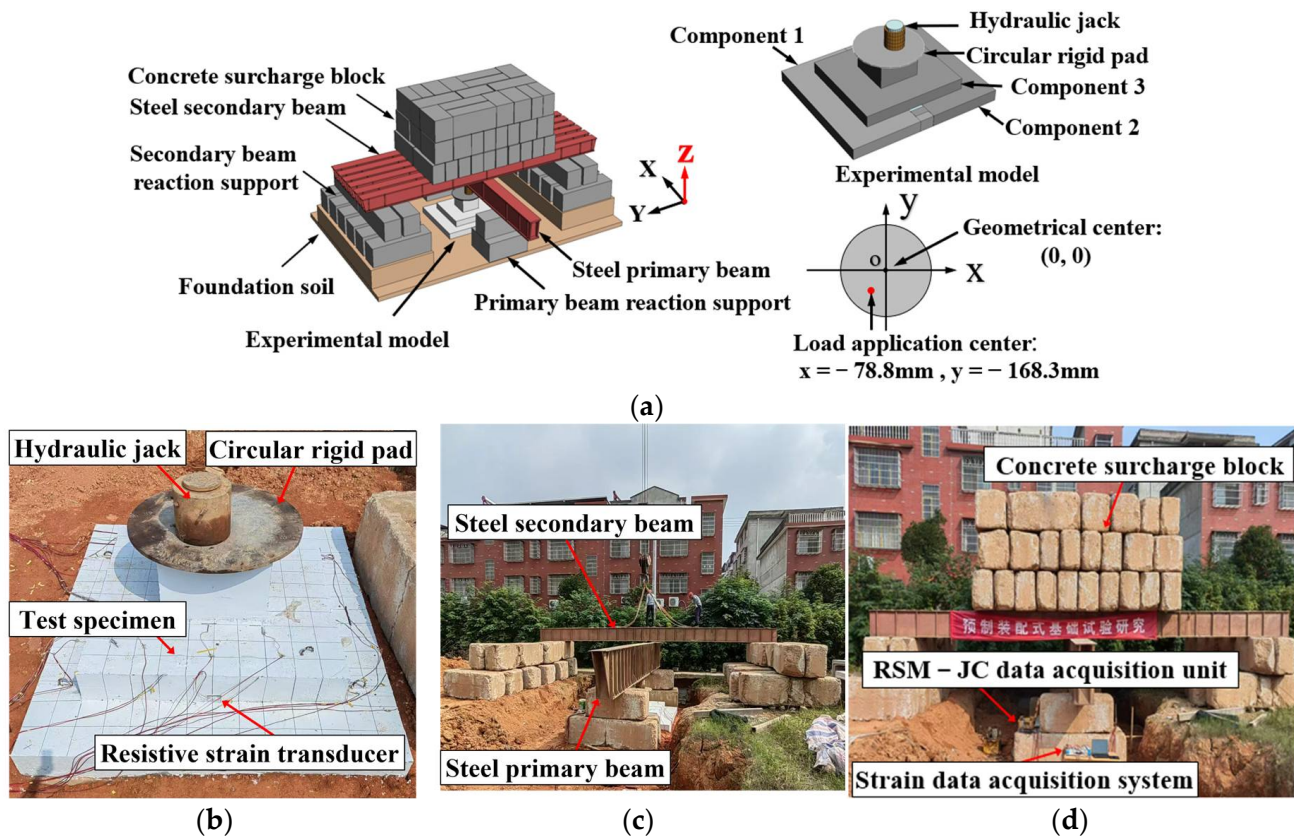


**Figure 3.** Photos of the static load test: (a) overall layout of the static load test; (b) determining the loading center; (c) assembling the reaction system; (d) stacking the counterweights.

The load conditions for each level of the test specimen are presented in Table 3. For levels 1 to 11, the displacement at each foundation measuring point was recorded 15 min after load stabilization. For levels 12 to 17, displacement readings were taken every 30 min, and at level 18, readings were conducted every 45 min. Figure 4 illustrates the locations of the measurement points and data acquisition devices in the test specimen. The TZX3826E static signal testing and analysis system was employed to collect strain data from the reinforcing steels, while the RSM-JC series static load testing system was utilized to monitor the test progress and gather settlement and soil pressure data from each measurement point.

**Table 3.** Loading regimen.

| Load Grade | Current Stage Load (kN) | Cumulative Load (kN) | Load Duration (min) | Load Grade | Current Stage Load (kN) | Cumulative Load (kN) | Load Duration (min) |
|------------|-------------------------|----------------------|---------------------|------------|-------------------------|----------------------|---------------------|
| 1          | 200                     | 200                  | 60                  | 10         | 200                     | 2000                 | 60                  |
| 2          | 200                     | 400                  | 60                  | 11         | 200                     | 2200                 | 60                  |
| 3          | 200                     | 600                  | 60                  | 12         | −200                    | 2000                 | 120                 |
| 4          | 200                     | 800                  | 60                  | 13         | −400                    | 1600                 | 120                 |
| 5          | 200                     | 1000                 | 60                  | 14         | −400                    | 1200                 | 120                 |
| 6          | 200                     | 1200                 | 60                  | 15         | −400                    | 800                  | 120                 |
| 7          | 200                     | 1400                 | 60                  | 16         | −400                    | 400                  | 120                 |
| 8          | 200                     | 1600                 | 60                  | 17         | −400                    | 0                    | 120                 |
| 9          | 200                     | 1800                 | 60                  | 18         | 0                       | 0                    | 180                 |

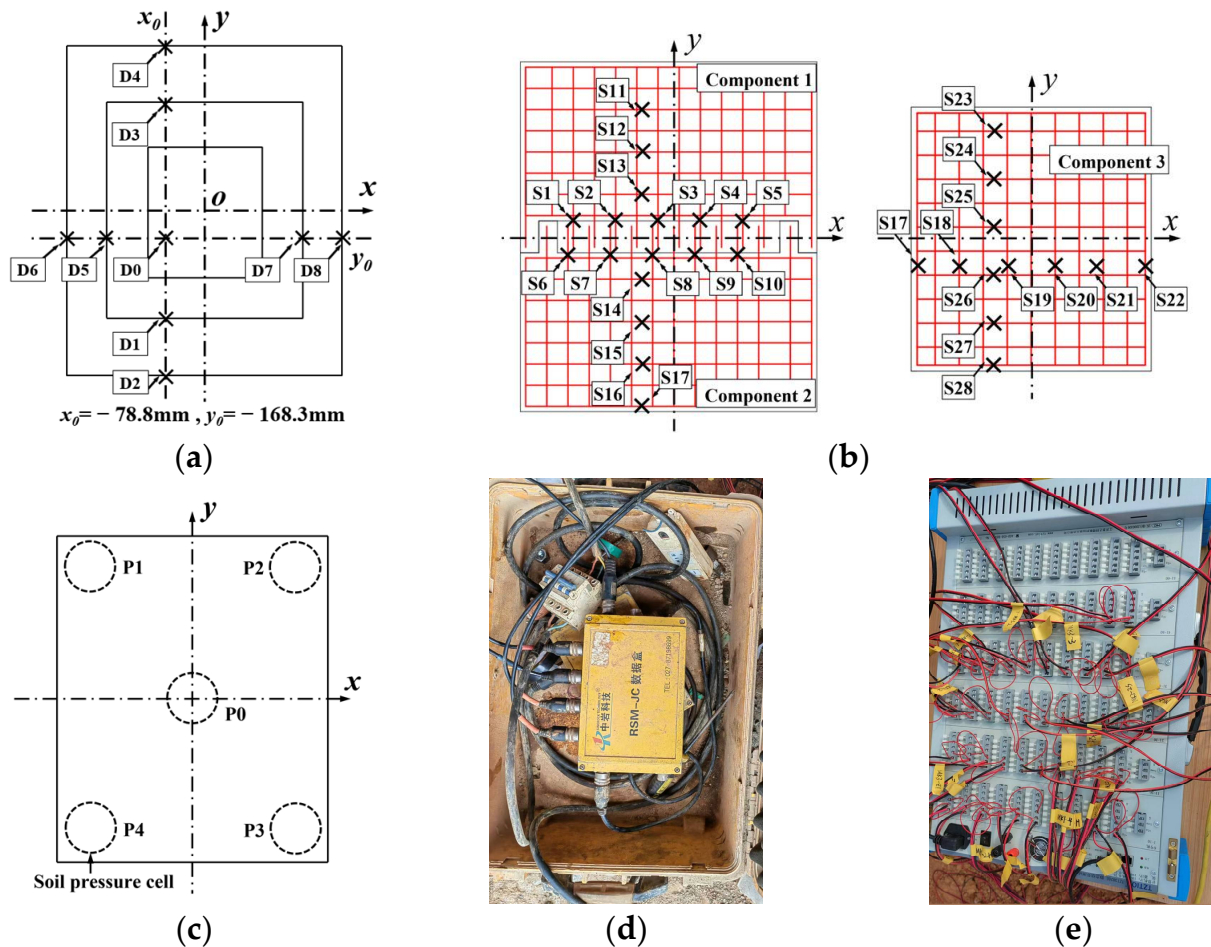


**Figure 4.** Measurement point distribution and data acquisition equipment: (a) displacement sensing points; (b) reinforcement-strain-sensing points; (c) soil-pressure-sensing points; (d) RSM-JC data acquisition unit; (e) TZX3826E static signal data acquisition system.

The tests in this study were limited to soft clay soil conditions and do not account for the effects of other soil types (e.g., sandy soil, clay, gravel) or field conditions (e.g., groundwater level, soil stratigraphy). Additionally, as the tests were conducted under specific conditions, the extrapolation of the results may be limited. Future research should validate these findings across a broader range of soil types and real-world scenarios to improve the applicability and generalizability of the results.

### 3. Experimental Results and Discussion

#### 3.1. Damage Modes

Figure 5 shows the crack distribution on the specimen. At loads between 0 and 800 kN, no noticeable cracks appeared on its outer surface. At 1000 kN, two cracks with a width of 0.2 mm appeared on the outer surface of the foundation at positions 2 and 7. When the load reached 1200 kN, additional cracks emerged at positions 1, 3, 4, 5, 11, and 12. This sudden increase in cracks indicates the onset of yielding, with a corresponding yield load between 1000 kN and 1200 kN. As the load increased from 1200 kN to 2000 kN, additional cracks developed on the surface of the specimen, but no significant crushing was observed. At the end of the test, no shear failure was observed in the base, indicating that the wet joint design of the novel prefabricated foundation was effective. Based on the overall crack distribution, the failure mode of the specimen was bending failure, characterized by numerous cracks beneath the load application point.

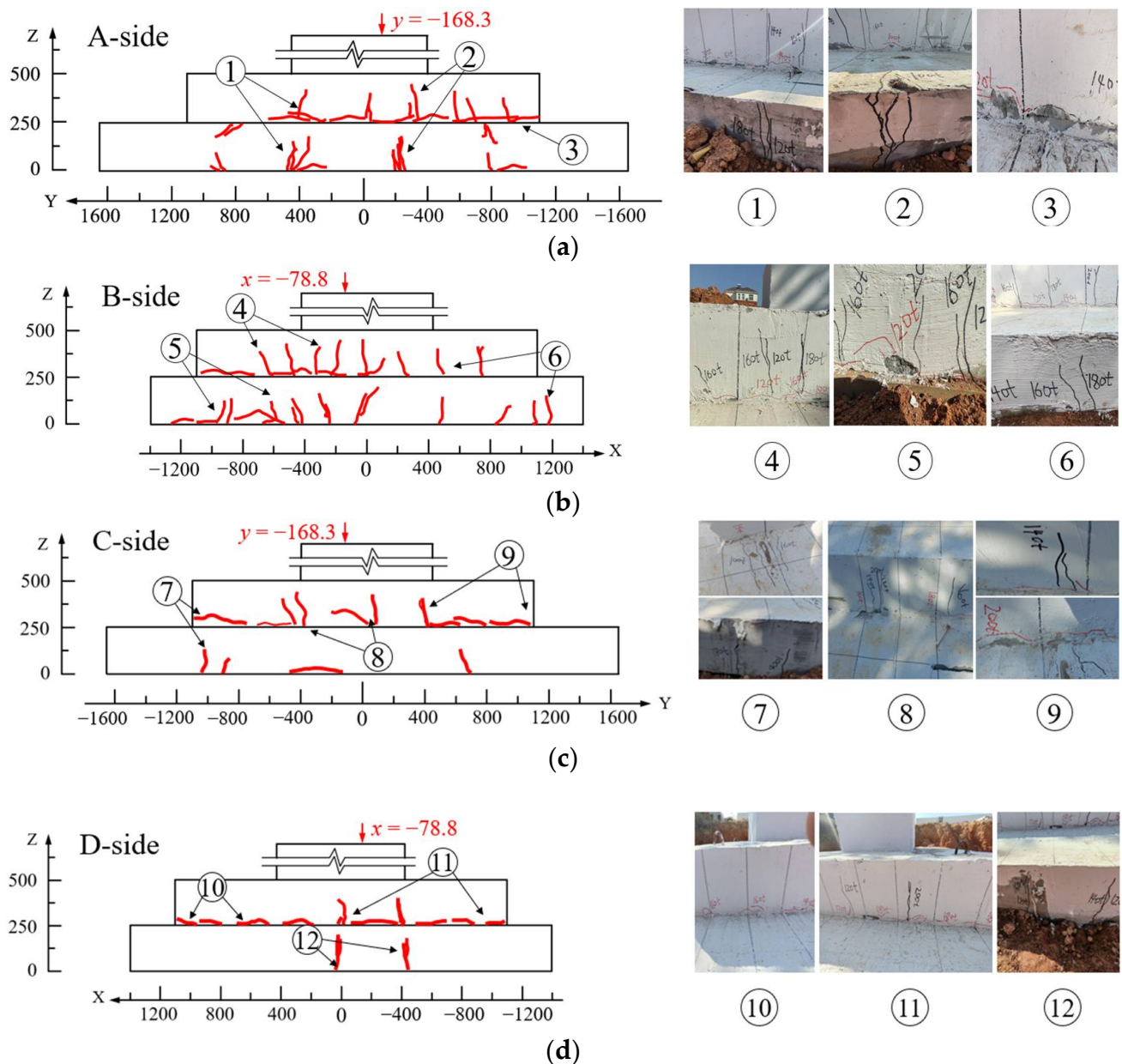


**Figure 5.** Crack distribution: (a) crack distribution on side A; (b) crack distribution on side B; (c) crack distribution on side C; (d) crack distribution on side D.

### 3.2. Measured Strains

Figure 6 shows the  $P$ -strain curve of the reinforcing steels. The curve reveals that the reinforcing steels near the wet joint reached yield strain first (S1–S10), as the concrete at the wet joint cannot provide tensile stress, and the reinforcing steels entirely bear the load. At 1000 kN, the foundation reinforcing steels, such as S1 and S6, began to yield. As the load increased to 1200 kN, S19 also entered the yield stage. The more load increase, the more reinforcing steel yield. At 1600 kN, all reinforcing steels near the wet joint yielded. However, the steel reinforcement mesh in the base limited the bending deformation of the foundation, allowing the specimen to continue bearing the load. At 2200 kN, all internal reinforcing steels yielded, and the specimen lost its bearing capacity. A comparison of the four sets of strain curves reveals that the joint reduced the bending stiffness of the foundation. However, the double-layer reinforcement design improves the ductility of the specimen, preventing immediate failure under heavy loads.

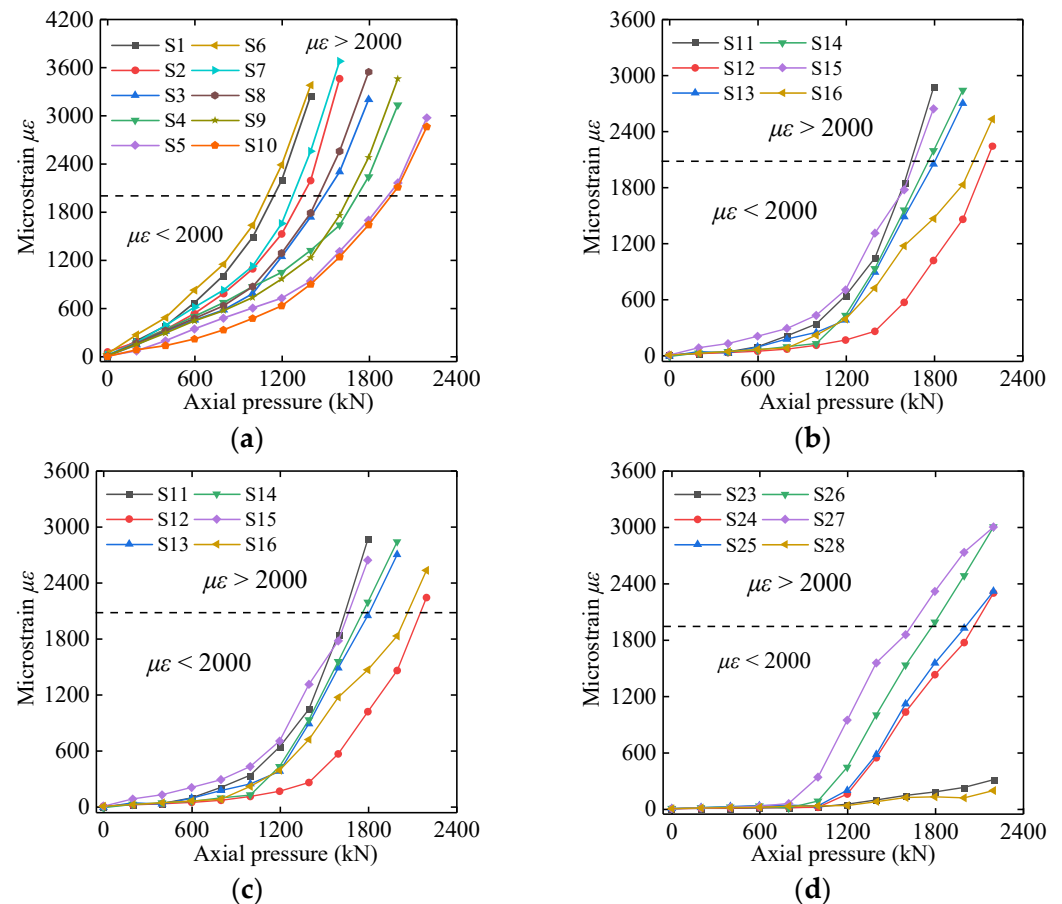


**Figure 6.** *P*–strain curve of reinforcing steels: (a) strain at the wet joint; (b) strain in the short reinforcing steels of the base; (c) strain in the long reinforcing steels of the pedestal; (d) strain in the reinforcing steels of the pedestal.

### 3.3. Measured Displacement

Figure 7a shows the *P*–settlement curve of the specimen. When the load was below 800 kN, settlement increased linearly with the load, indicating that the foundation soil was in the elastic deformation phase. As the load continued to rise, the slope of the curve decreased, marking the transition to elastoplastic deformation. At the maximum load of 2200 kN, the settlement reached 78.42 mm. After unloading, the soil exhibited a rebound of 14.41 mm, with a rebound rate of 21.31%, suggesting that the settlement in this test was primarily due to the plastic deformation of the foundation soil.

Figure 7b illustrates the base stiffness in the Y-axis direction as a deflection function. Initially, the stiffness of the specimen was 2094.2 kN/mm. When deflection reached 1.2 mm (corresponding to a load of 1200 kN), stiffness decreased to 594.3 kN/mm. The curve declined rapidly at first and then gradually leveled off. This behavior is attributed to the hook-shaped and structural reinforcing bars at the wet joints, which jointly restrict base separation, indicating a reasonable design for the reinforcement at the joints.

Figure 7c,d show the *P*–uplift curves of the specimen. The upward warping values at measurement points D1, D2, D5, and D6 transitioned from negative to positive, indicating that, before 1100 kN, the bending deformation of the specimen was minimal, with the specimen deflecting primarily around the load center. At 1100 kN, all the curves displayed inflection points, and the sudden increase in the edge uplift values corresponded to the yielding of the longitudinal reinforcement, marking the transition of the specimen to the overall yielding stage.

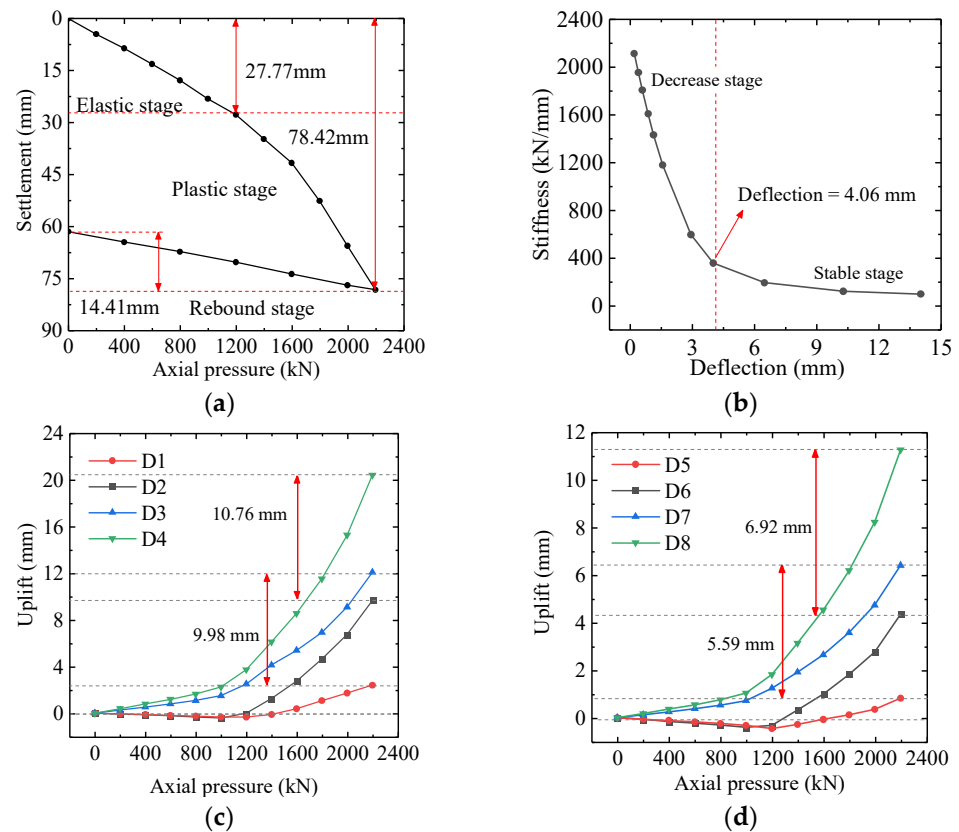


**Figure 7.** Test curves: (a) *P*–settlement curve; (b) stiffness–deflection curve; (c) *P*–uplift curve (y); (d) *P*–uplift curve (x).

### 3.4. Base Pressure

Figure 8 shows the changes in the base pressure of the specimen. In the early stage of the test, the edge pressures (P1–P5) were more significant than the center pressure (P0). As the load increased, the base of the specimen experienced bending deformation, resulting in the increased settlement at the center and plastic deformation of the soil, which caused a redistribution of the base pressure. At 1007.5 kN, the growth rate of the edge pressure slowed down, while the center pressure continued to rise rapidly. At 2107.7 kN, the edge pressure began to stabilize, with the pressure at P3 reaching its peak and eventually stabilizing at 173.7 kPa. The stabilization of the edge pressure indicates that the specimen had reached its design limit state and could no longer bear additional load.

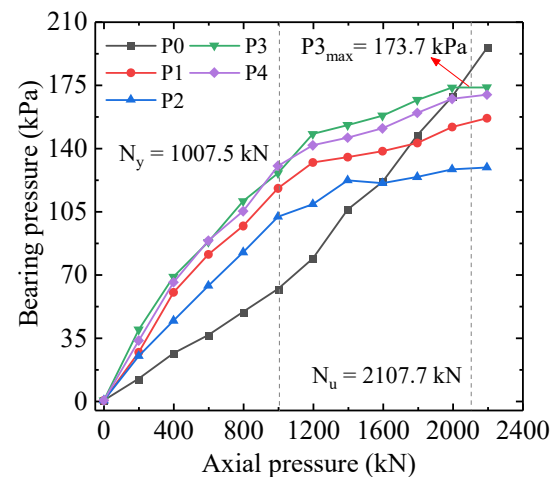


**Figure 8.** *P*–bearing pressure curve.

## 4. Numerical Analysis

### 4.1. Description of 3D FE Models

This study used ANSYS 17 to establish a finite element model of foundation–soil interaction. To verify the accuracy of the model, the numerical simulation results were compared and analyzed with experimental data. After confirming the validity of the model, parameter analysis was conducted to explore the impact of different design parameters on the load-bearing performance of the novel prefabricated foundation.

Figure 9 presents the material's stress–strain curve. As shown in Figure 10a, the finite element model consists of three components: concrete, reinforcing steel, and foundation soil, with the model parameters provided in Table 4. The interaction between each component was modeled using contact elements, and the interaction between the foundation and the soil was achieved through nodal degree-of-freedom coupling. The modeling details of the finite element model are shown in Figure 10b.

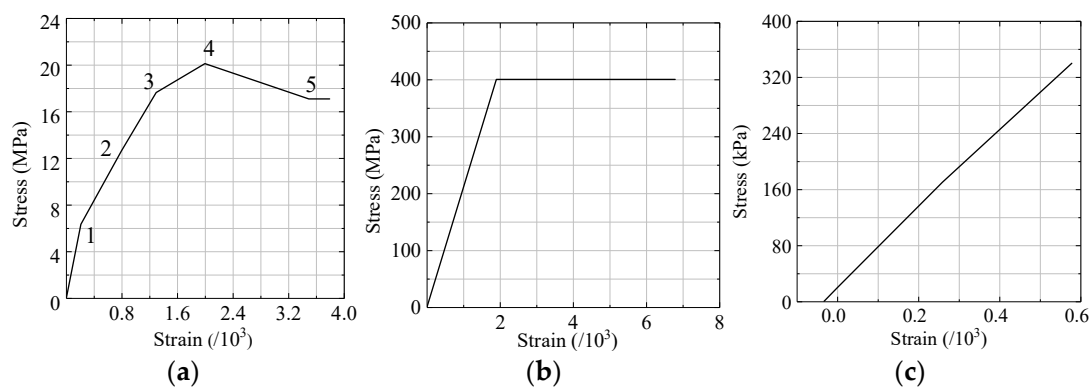


Figure 9. Material constitutive model: (a) concrete model; (b) reinforcing steel model; (c) foundation soil model.

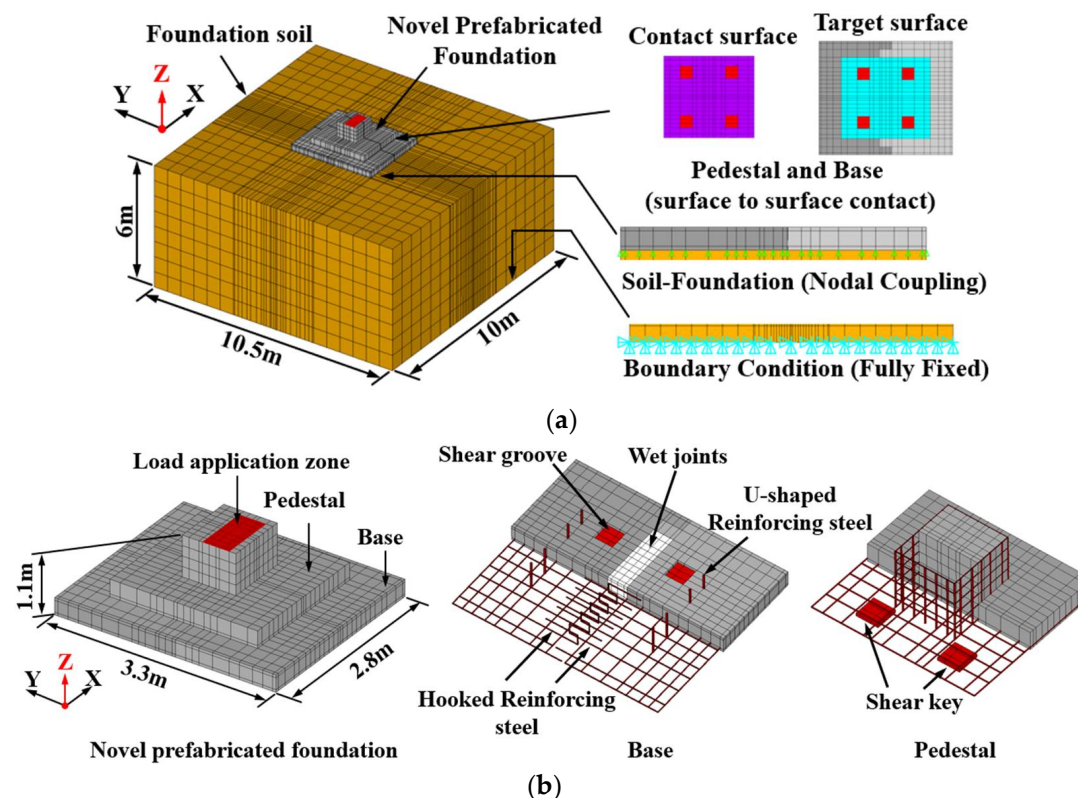


Figure 10. Finite element model of the new prefabricated foundation: (a) overall model; (b) model details.

**Table 4.** Model parameters.

| Element            | Simulation Element | Material Constitutive Model           | Basic Parameters |       |                    |                    |
|--------------------|--------------------|---------------------------------------|------------------|-------|--------------------|--------------------|
|                    |                    |                                       | $E$ (MPa)        | $\nu$ | $\varepsilon_{uc}$ | $\varepsilon_{ut}$ |
| Concrete           | SOLID65            | Multilinear kinematic hardening model | 20,000           | 0.25  | 0.0035             | 0.0005             |
| Reinforcing steels | LINK8              | Bilinear isotropic hardening model    | 200,000          | 0.25  | 0.0020             | 0.0020             |
| Foundation soil    | SOLID45            | Drucker–Prager model                  | 8.58             | 0.40  | /                  | /                  |

Note:  $E$  is the elastic modulus;  $\nu$  is the Poisson's ratio;  $\varepsilon_{uc}$  is the ultimate compressive strain;  $\varepsilon_{ut}$  is the ultimate tensile strain.

#### 4.2. Numerical and Experimental Comparisons

Figure 11 shows the comparison of damage between the numerical results and experimental results when the load reached 2200 kN. The numerical results indicate that severe tensile damage occurred near the base wet joints. Cracks in the base were concentrated at the mid-span, while cracks in the pedestal were primarily distributed around the edges. This is due to the contact elements between the pedestal and the base, which mainly transfer normal pressure, thereby preventing cracks in the base from extending into the pedestal. These phenomena align with the experimental results.

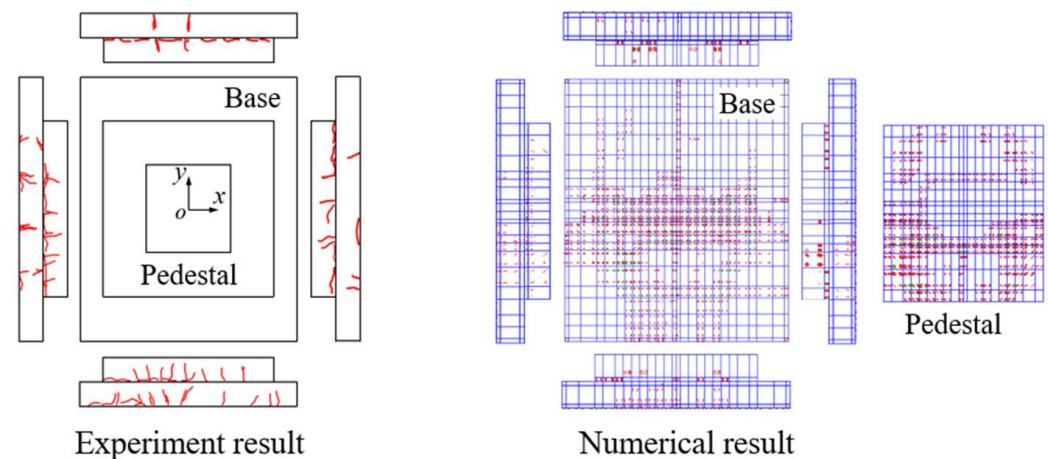
**Figure 11.** Experimental and numerical damage mode of specimen.

Figure 12 presents the cloud diagrams of the finite element model corresponding to the yield load (1029.8 kN), ultimate load (2107.7 kN), and failure load (2379.5 kN). The results indicate that, during the early stages, the foundation experienced deflection, with two groups of longitudinal reinforcement yielding at the wet joints and a relatively uniform base pressure distribution. At the ultimate load, significant bending deformation was observed, with a maximum deflection of 13.43 mm and a span-to-deflection ratio of 1/246. At this stage, all reinforcement had yielded, and pressure concentration occurred at the joint edges, where the pressure value reached 279.25 kPa. When loaded to the failure load, the deflection increased suddenly, and the base exhibited “tearing” along the joints, forming a narrow stress concentration zone. These observations confirm that the final failure mode of the finite element model is bending failure, which is consistent with the experimental conclusions.

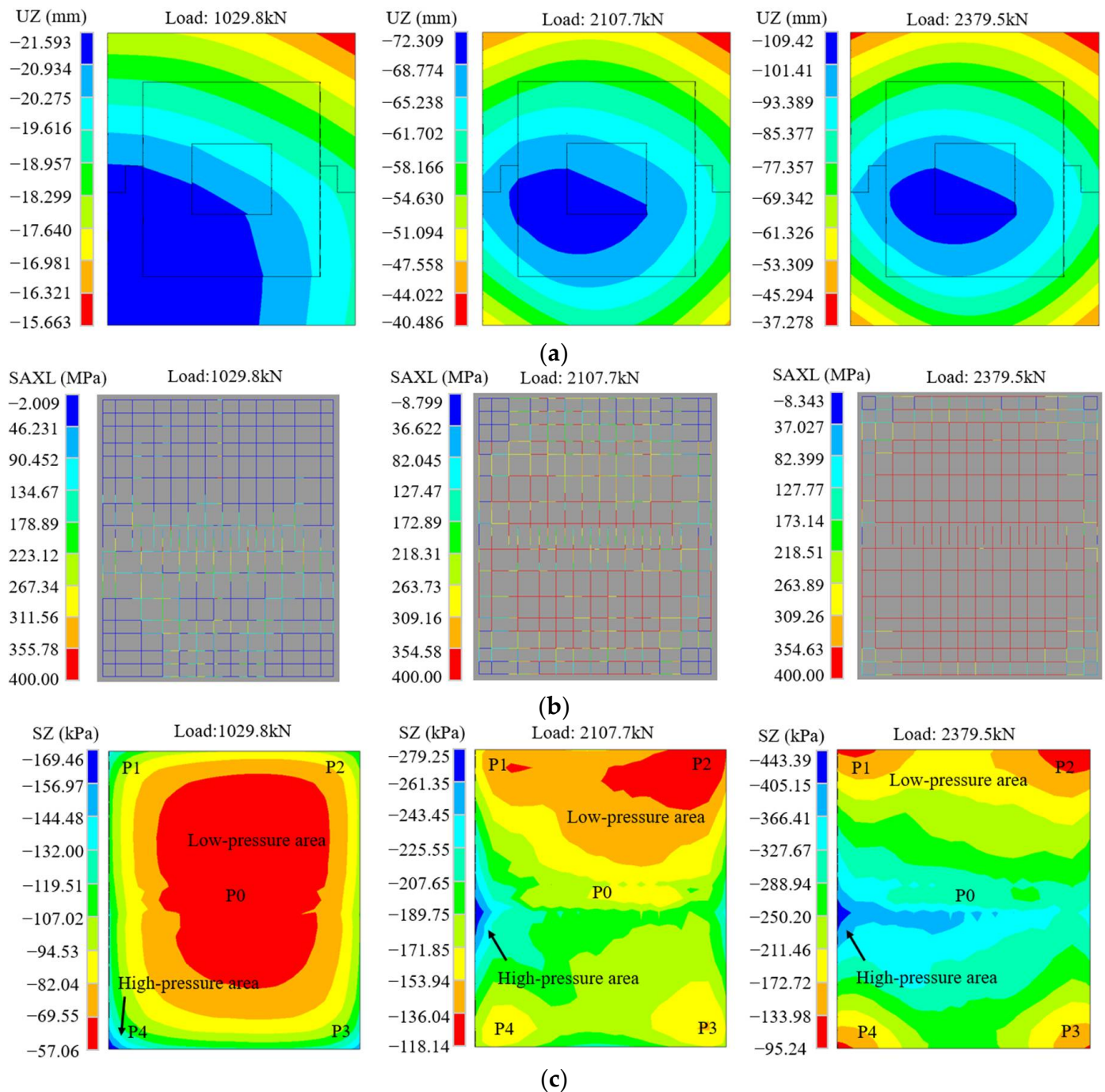


**Figure 12.** Cloud diagram of numerical results: (a) settlement cloud diagram; (b) reinforcement stress cloud diagram; (c) base pressure cloud diagram.

Figure 13a,b compare the numerical and experimental results. The settlement and stiffness degradation curves derived from the finite element model align well with the experimental results. The yield loads obtained from the finite element model and the experimental model were 1029.8 kN and 1007.5 kN, respectively, with a difference of 2.17%. The yield settlements were 21.50 mm and 23.19 mm, with a difference of 7.86%. Figure 11b illustrates the stiffness differences between the numerical and experimental results, showing a stiffness difference of 9.94% at the yield displacement.

Figure 13c,d present the comparison of base pressure between numerical and experimental results. At 1000 kN (near the yield point of the foundation), the base pressure difference at P0 (center measurement point) between the numerical and experimental re-

sults was 3.15%. For P1–P4 (edge measurement points), the largest difference was observed at P2, which is attributed to the steel constitutive model employed in this study. This model does not fully capture the strength degradation of the longitudinal reinforcement. P2, being the furthest measurement point from the load center, was the most affected. When the load reached 2000 kN, the pressures at P1–P4 reached their peak values. The ultimate loads obtained from the finite element model and the experimental model were 2107.7 kN and 1981.5 kN, respectively, with a difference of 6.37%.

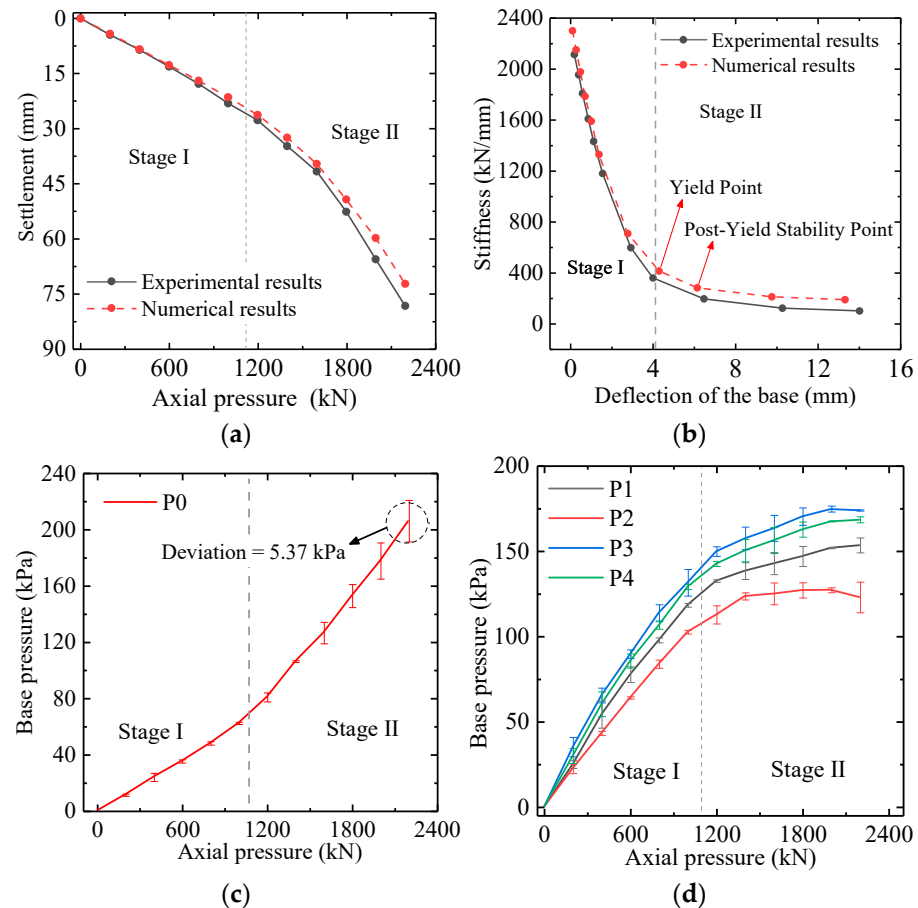


**Figure 13.** Comparison between experimental and numerical results: (a) settlement comparison; (b) stiffness comparison; (c) P0 pressure deviation; (d) P1–P5 pressure deviation.

Overall, the finite element model effectively predicted the strength and stiffness of the novel prefabricated foundation, particularly the stiffness variations at wet joints.

## 5. Parametric Study

Based on the verified finite element model, a parametric analysis of the novel prefabricated foundation was conducted. Three parameters were considered in this study: the base-to-pedestal thickness ratio, the cross-sectional dimensions of the shear keys, and the compressive strength of the wet joints. To facilitate the parametric study, the benchmark model was defined with a base-to-pedestal thickness ratio of 1.0, a shear key cross-sectional size of 300 mm × 300 mm, and a wet joint compressive strength of 40 MPa.

### 5.1. Influence of the Pedestal-to-Base Thickness Ratio

Increasing the thickness of the foundation is typically an effective approach to enhancing load-bearing capacity. However, simply increasing the thickness can not only raise the assembly cost of the components but also affect the economic feasibility and practicality of the prefabricated foundation. Considering that the novel prefabricated foundation

consists of two parts—the pedestal and the base—it is of significant practical value to thoroughly investigate the specific impact of the thickness of these two parts on the overall load-bearing capacity.

In this study, a total of five models were constructed, with the total thickness of the pedestal and base fixed at 500 mm. Among these, model C3 served as the benchmark, with a thickness ratio of  $\lambda = 1.0$ . Table 5 presents the effects of varying  $\lambda$ .

**Table 5.** Effects of thickness ratio  $\lambda$ .

| Condition | $\lambda$ | $P$ (kN)        | Settlement (mm) | Deflection (mm) |
|-----------|-----------|-----------------|-----------------|-----------------|
|           |           | Yield/Ultimate  | Yield/Ultimate  | Yield/Ultimate  |
| C1        | 0.8       | 846.40/2001.10  | 25.41/78.90     | 4.82/16.54      |
| C2        | 0.9       | 936.50/2101.60  | 24.69/71.07     | 4.49/15.99      |
| C3        | 1.0       | 1029.80/2107.70 | 24.79/68.45     | 4.06/13.43      |
| C4        | 1.1       | 1132.00/2089.40 | 23.62/62.84     | 3.61/13.98      |
| C5        | 1.2       | 1087.40/1976.80 | 21.90/53.76     | 4.32/14.98      |

Figure 14a shows that the settlement value of the foundation decreased as the thickness ratio  $\lambda$  increased. Figure 14b presents the stiffness degradation curves of each sample. When  $\lambda$  was set to 0.8 or 0.9, both the yield load and the initial stiffness of the corresponding samples were lower than those of the benchmark model. This indicates that when the total height of the foundation is constant, the thickness of the pedestal has a more significant impact on the load-bearing capacity than the base. For  $\lambda$  values of 1.1 and 1.2, the initial stiffness of the relevant samples exceeded that of the benchmark model, and their early stiffness degradation rates were lower.

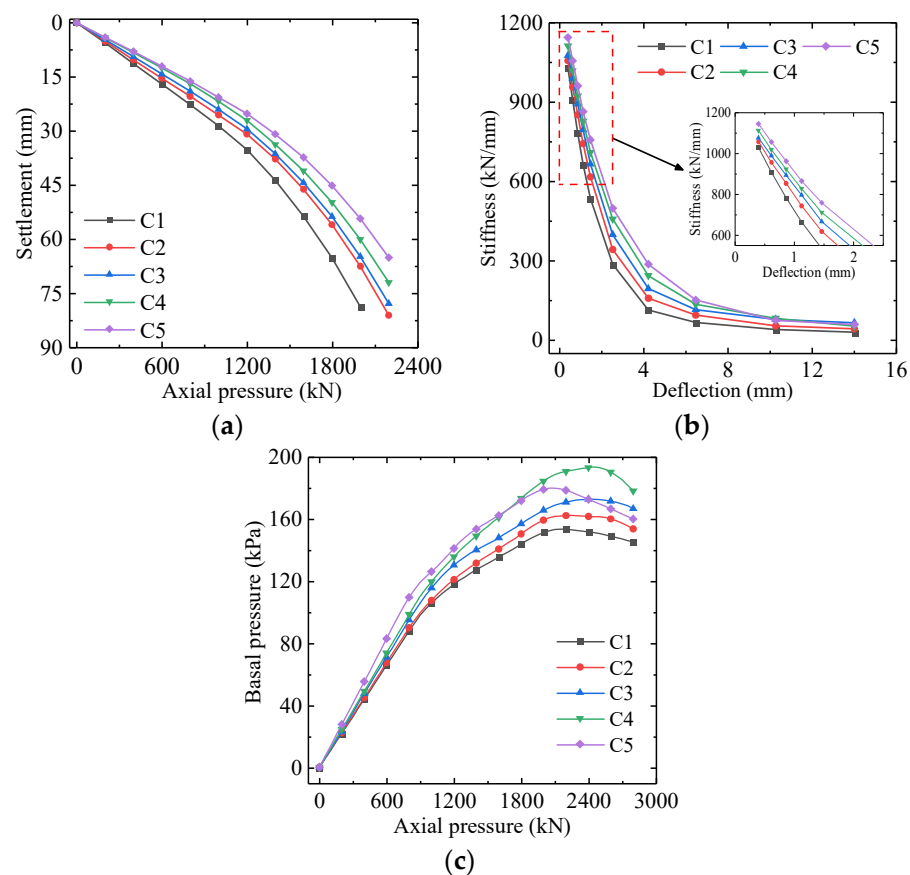


**Figure 14.** Load–displacement curves of models with different thickness ratios: (a)  $P$ –settlement curve; (b) stiffness degradation curve; (c)  $P$ –base pressure curve.

Figure 14c illustrates the base pressure–load curves for each model. The load corresponding to the peak base pressure represents the ultimate load of the base. The yield load increased initially and then decreased as  $\lambda$  increased. The maximum yield load, observed at  $\lambda = 1.1$ , was 1132.0 kN, which is 9.92% higher than that of the benchmark model. The ultimate load of the foundation was less sensitive to  $\lambda$  because the amount of reinforcement primarily influenced it. Therefore, to optimize the initial stiffness and yield load of the foundation, the pedestal-to-base thickness ratio  $\lambda$  should be controlled within the range of 1.0 to 1.1 during the design process.

### 5.2. Influence of Shear Keys

The novel prefabricated foundation incorporates wet joints at its base, where the concrete lacks tensile resistance. Relying solely on the reinforcing steels in the wet joints to resist tensile forces is insufficient to meet stiffness requirements. To limit the horizontal relative displacement between components, shear keys were installed between the pedestal and the base. The load transfer capacity at the interface primarily depends on its shear strength. As the initial stiffness of concrete is higher than that of bolts or embedded reinforcing steels, increasing the cross-sectional size of the concrete shear keys can effectively enhance the initial stiffness of the foundation.

Given the importance of this parameter, it was necessary to conduct an in-depth investigation into the impact of the cross-sectional size of the shear key on the load-bearing capacity of the novel prefabricated foundation. To this end, adjustments were made to the cross-sectional size of the shear key. As illustrated in Figure 15, the shear keys were positioned at the trisection lines of the long and short sides of the base, with the cross-sectional dimensions of the shear key denoted as  $a \times b$ .

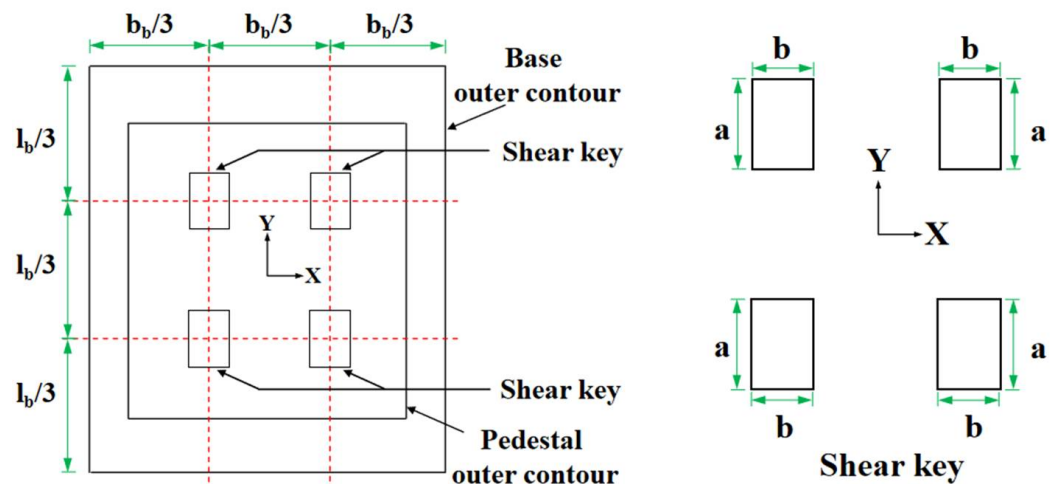


Figure 15. Cross-sectional dimensions of shear keys (unit: mm).

Figure 16a illustrates that at lower loads, increasing the dimensions  $a$  or  $b$  of the shear key reduced the settlement of the foundation. However, when the load exceeded 800 kN, an increase in dimension  $b$  led to a more significant settlement. Figure 16b presents the stiffness degradation curves of each sample. When the cross-sectional areas of the shear keys were equal (e.g., C1 and C4, C2 and C5), the initial stiffness of the models was nearly identical. As the load increased and the models entered the yield stage, the stiffness degradation became more pronounced for models C4 and C5.

This behavior is attributed to the effect of increasing the size of the shear key, which enlarges the shear groove in the base. Specifically, increasing dimension  $b$  made the base more susceptible to through-cracks parallel to the wet joints, further compromising the stiffness of the base. Therefore, when designing shear keys, dimension  $a$  can be appropriately increased to enhance performance, while dimension  $b$  should be strictly

controlled. Figure 16c shows the pressure–load curves for each base, further illustrating the impact of shear key dimensions on load distribution.

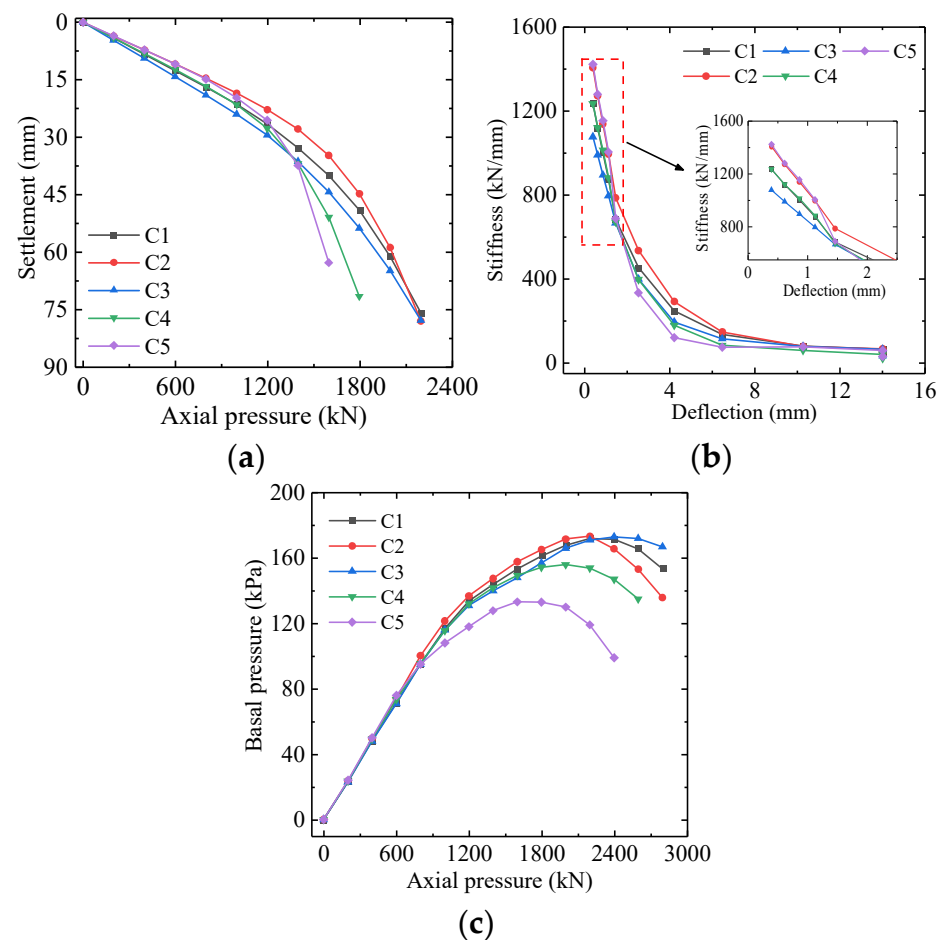


**Figure 16.** Load–displacement curves of models with different shear vital dimensions: (a)  $P$ –settlement curve; (b) stiffness degradation curve; (c)  $P$ –case pressure curve.

Table 6 presents the effects of varying shear vital dimensions. As dimension increased, the yield load gradually rose, indicating that under the given loading conditions, a larger value improves the load-bearing capacity foundation. However, the incremental benefit of increasing a diminished, as evidenced by the reduced yield load increase when a was expanded from 450 mm to 600 mm. This suggests that within a certain range, this increase significantly enhances the yield load, but beyond this range, the improvement becomes less pronounced.

**Table 6.** Influence of shear key cross-sectional dimensions.

| Condition | $a \times b$<br>(mm $\times$ mm) | $P$ (kN)        | Settlement (mm) | Deflection (mm) |
|-----------|----------------------------------|-----------------|-----------------|-----------------|
|           |                                  | Yield/Ultimate  | Yield/Ultimate  | Yield/Ultimate  |
| C1        | 450 $\times$ 300                 | 1057.30/2124.60 | 22.95/70.36     | 4.17/13.54      |
| C2        | 600 $\times$ 300                 | 1072.50/2053.10 | 20.12/64.01     | 4.23/13.08      |
| C3        | 300 $\times$ 300                 | 1029.80/2107.70 | 24.79/68.45     | 4.06/13.43      |
| C4        | 300 $\times$ 450                 | 1045.10/1964.00 | 22.00/71.76     | 4.12/12.52      |
| C5        | 300 $\times$ 600                 | 980.40/1867.30  | 19.16/62.88     | 3.87/11.90      |

When  $a = 300$  mm was fixed, a moderate increase in dimension  $b$  (e.g., from C3 to C4) resulted in a slight increase in the yield load. However, excessive increases in  $b$  (e.g., from C4 to C5) led to a greater likelihood of through-cracks forming parallel to the wet joints, which significantly reduces the yield load. Thus, in the design process, it is crucial to balance the positive effect of increasing  $b$  on the yield load with the risk of through-cracks to ensure the overall stability of foundations and load-bearing capacity.

### 5.3. Influence of the Strength of Wet Joints

The base of the novel prefabricated foundation consists of components 1 and 2, which are connected by reinforcing steel and concrete. Due to the presence of secondary seams, this area becomes a weak link in the overall foundation structure. In the test specimen, 14 HRB400 reinforcing steels with a diameter of 12 mm were embedded in the wet joints and poured with C40 concrete. Since the base was spliced, the tensile forces between the wet joints and components 1 and 2 were primarily borne by the reinforcing steels. Although the shear key partially shared the load, the relative displacement of the base in the Y-axis direction remained significant.

Increasing the initial stiffness of the wet joints can enhance the overall synergy of the foundation and improve its load-bearing capacity. Previous studies [30–37] have demonstrated the advantages of ultra-high-performance concrete (UHPC) as a wet joint filler. In this study, UHPC80 and UHPC100 were used as wet joint fillers and compared with the benchmark model. The results are summarized in Table 7.

**Table 7.** Influence of compressive strength of wet joints.

| Condition | Material | $P$ (kN)        | Settlement (mm) | Deflection (mm) |
|-----------|----------|-----------------|-----------------|-----------------|
|           |          | Yield/Ultimate  | Yield/Ultimate  | Yield/Ultimate  |
| C1        | C40      | 1029.80/2107.70 | 24.79/68.45     | 4.06/13.43      |
| C2        | C50      | 1105.40/2136.10 | 25.11/67.72     | 4.35/14.16      |
| C3        | UHPC80   | 1242.70/2056.20 | 21.86/67.10     | 4.89/15.77      |
| C5        | UHPC100  | 1290.20/2060.50 | 21.32/66.29     | 5.08/15.98      |

Figure 17a,b demonstrate that using ultra-high-performance concrete (UHPC) as a wet joint filler (models C3 and C4) significantly enhanced the settlement resistance and deformation resistance of the novel prefabricated foundation. This improvement is particularly advantageous for mitigating small deflection deformations and reducing foundation deformation under initial loading. Figure 17c shows that UHPC fillers (C3 and C4) provided the foundation with higher initial stiffness due to their superior strength and compactness. The base pressure of C3 and C4 increased more gradually during the initial loading stage, as evidenced by the curve shape. Compared with ordinary concrete fillers (C1 and C2), UHPC fillers demonstrated better control over base pressure during early loading, with a slower rate of pressure increase.

For projects requiring precise control of initial base pressure and enhanced initial stiffness, UHPC emerges as the optimal filler material. It ensures a more uniform distribution of foundation pressure during the initial stage and provides higher compression resistance throughout the loading process, thereby extending the service life of the foundation. Conversely, in scenarios where large deflections are permissible, ordinary concrete fillers (C1 and C2) can provide comparable stiffness and are suitable for applications with moderate load and deformation requirements.

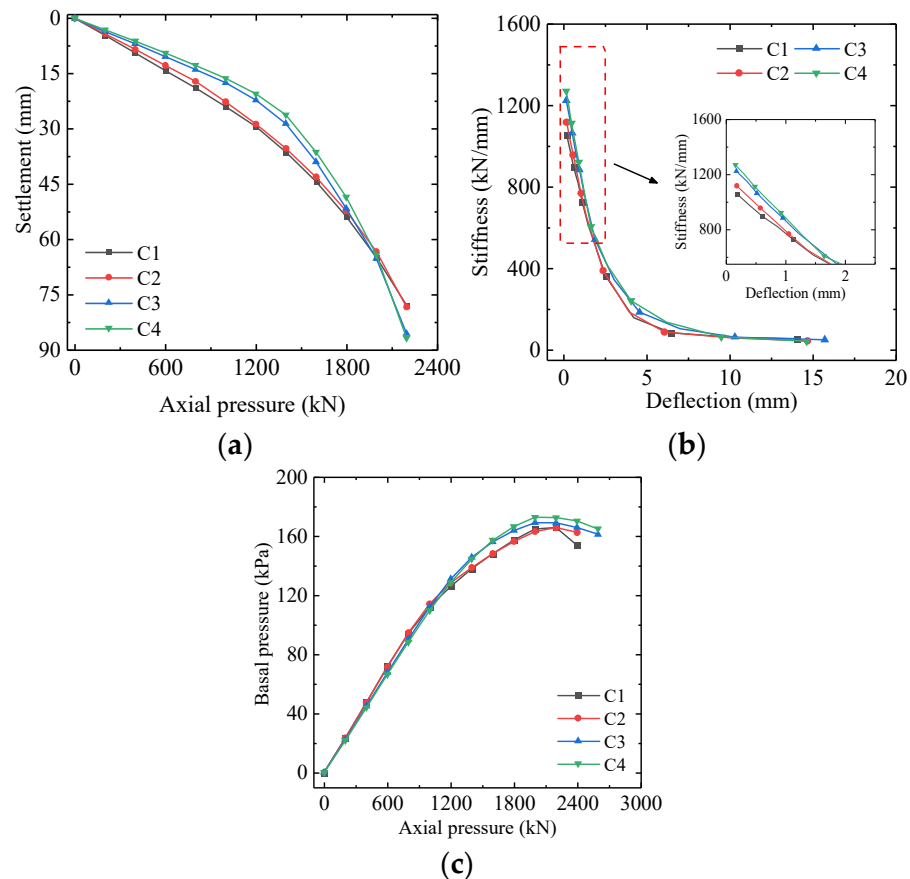


**Figure 17.** Load–displacement curves of models with different wet joint strengths: (a)  $P$ –settlement curve; (b) stiffness degradation curve; (c)  $P$ –base pressure curve.

## 6. Conclusions

The article presents a novel design scheme for prefabricated extended foundations, addressing the problems of insufficient initial stiffness and short service life in existing prefabricated foundations by incorporating concrete shear keys and wet joints between modules. The experimental results demonstrate that this design effectively enhances the initial stiffness and overall synergy of the foundation.

1. Through static load tests, this article verified the bearing capacity and settlement characteristics of the new foundation. Tests show that when the load gradually increased, the settlement of the foundation had a linear relationship with the load value. When the maximum load was reached, the foundation did not collapse, proving the feasibility of the new design.
2. During the loading process, cracks appeared in the foundation, but no through-cracks were observed, indicating that the design provides a certain level of deformation resistance and that the bending resistance of the novel foundation is relatively strong.
3. The finite element model established using ANSYS 17 is consistent with the experimental results and can effectively predict the strength and stiffness changes of the foundation, especially in the wet joint area.
4. Parametric analysis was conducted on the foundation thickness ratio, cross-sectional dimensions of shear keys, and wet joint strength. The results show that a reasonable thickness ratio (1.0 to 1.1) can improve the bearing capacity; increasing the size of the shear key can help enhance the initial stiffness of the foundation, but the size needs to be controlled to avoid causing cracks, and the use of ultra-high-performance concrete (UHPC) as a wet joint filling material can significantly improve the settlement resistance and initial stiffness of the foundation.

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