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Flexural Behavior of Hollow Slab Beams Strengthened by a Novel Top-Anchored External Prestressing System: An Experimental Benchmark Study

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ABSTRACT: This paper introduces a novel external prestressing system specifically designed for strengthening hollow slab beams. To validate the system's feasibility and isolate flexural behavior from interface slip effects, experimental investigations were conducted on monolithically cast specimens representing an idealized, fully bonded state. By integrating theoretical analysis, experimental testing, and finite-element numerical simulation, the study examines the mechanical behavior and design principles of the proposed configuration. Results indicate that the stress development process of the externally prestressed beam (N2) exhibits strong similarity to that of a conventional internally prestressed beam (N1). Notably, the implementation of the top-anchored external prestressing tendon in specimen N2 produced a marked improvement, increasing the flexural bearing capacity by approximately 10% compared to the internally prestressed reference beam (N1). Furthermore, the monolithic anchorage zone remained intact with no observed cracks or wire rupture. These findings confirm the structural reliability of the proposed system under ideal bonding conditions, establishing a critical performance benchmark for the practical strengthening and extended service life of existing bridge infrastructure.

KEYWORDS: Bending performance; hollow slab beam; external prestressed tendon; numerical simulation; experimental research

1 Introduction

Hollow slab beams are widely employed in bridge engineering owing to their simple structure, clear force distribution, suitability for mass prefabrication, ease of installation, and cost-effectiveness. However, factors such as outdated low design load standards, insufficient bearing capacity, lack of maintenance, and disrepair have led to varying degrees of damage in many hollow slab beam bridges, gradually transforming them into potential safety hazards. Recently, various innovative strengthening techniques have been proposed to address specific structural deficiencies in existing hollow-slab systems. For instance, Du et al. [1] investigated the flexural behaviors of pre-cracked hollow core beams strengthened with core filling and unbonded prestressing steel strands, demonstrating significant improvements in load-carrying capacity. Additionally, localized grouting reinforcement methods have been recently developed to enhance the shear performance of full-scale prestressed hollow-core slabs [2]. Furthermore, to ensure reliable composite action between existing hollow-slab beams and newly added strengthening layers, recent studies have extensively evaluated the effects of various interface treatments, including surface roughening and shear dowels [3].

These recent advancements highlight the ongoing need to optimize both the strengthening configurations and the interaction mechanisms of hollow-slab bridges.

Existing strengthening techniques are broadly classified into active methods (e.g., adding prestressing tendons) and passive methods (e.g., externally bonding materials such as CFRP). Currently, passive reinforcement predominates for hollow slab bridges. For instance, Hu et al. [4] investigated CFRP reinforcement for damaged hollow slab beams. At the same time, Wang et al. [5] investigated this passive reinforcement method in testing prestressed concrete hollow-slab beams. They studied the mechanical properties of prestressed concrete hollow-core slabs strengthened with externally bonded bamboo laminates. The relationship between flexural and shear failure modes in hollow-core slabs was described. Guidance on the design of externally bonded laminated bamboo-reinforced hollow-core slabs was provided. Near-surface-mounted carbon fiber-reinforced polymers (NSM-FRP) plates have been used to reinforce slabs under repeated loading. Jasim and Daud [6] found that a hollow slab beam strengthened with the NSM-CFRP approach improved load-carrying capacity and stiffness across different failure loads. Badran and Alkloub [7] investigated how strengthening hollow slab beams with carbon fiber-reinforced polymer (CFRP) laminates and textiles, both before and after exposure to fire, affects their flexural and shear performance.

Regarding the reinforcement of hollow slabs using active methods, scholars have also conducted research. For instance, Sun et al. [8] proposed a new hinge joint reinforced with locally prestressed iron-based shape memory alloy (Fe-SMA) U-bars for a hollow-core slab bridge. The new hinge joint can effectively apply local prestressing by heating-activated Fe-SMA U-bars, thereby increasing the hinge joint's crack-penetration load in the hollow slab beam. Jiang et al. [9] proposed a hinge joint reinforcement technique by installing transverse channels at the bottom of the hollow slab beam and vertical prestressed CFRP tendons in the hinge joints. This technique was proposed to enhance the transverse collaborative working performance (TCWP) of hollow slab bridges, providing a valuable reference for reinforcing hinge joints in such bridges. In view of the problem that hinge joints are prone to cracking, Yang et al. [10] investigated the application of ultra-high-performance concrete (UHPC) on the tension side of full-scale normal concrete (NC) hollow slab beams. The findings indicate that this composite configuration effectively controls crack development and enhances overall structural performance.

However, in active reinforcement methods for hollow slabs, the low beam height and limited space between beams pose significant anchoring challenges. As illustrated in Fig. 1a, current active reinforcement techniques typically connect the anchorage zone and deviation blocks to the structure using bolts, relying on the bolts' shear strength to bear the load. This bolt connection is vulnerable to failure if inadequately secured [11]. Therefore, this paper proposes a novel external prestressing reinforcement structure. This system innovatively relocates the anchorage end to the upper part of the beam, thereby eliminating anchoring difficulties caused by the low beam height and ensuring reliable load-bearing capacity of the anchorage zone during service. In practical strengthening applications, advanced technologies such as microwave-based rebar detectors and drilling robots can be employed to create inclined holes through the existing beam without damaging internal reinforcement [12]. This facilitates the installation of external prestressed tendons and allows for rapid strengthening with minimal traffic disruption, which is of great significance for ensuring highway transport safety and promoting the sustainable use of aging highway bridges [13]. The configuration of the new external prestressing reinforcement system is illustrated in Fig. 1b.

In conventional retrofit projects, the mechanical behavior of the strengthened structure is often compromised by the bond-slip behavior at the interface between the existing concrete and the newly added anchorage block [14]. In practical retrofitting scenarios, the failure mode is often governed by the bond-slip behavior at the new-to-old concrete interface. However, prematurely focusing on interface issues may obscure the intrinsic mechanical behavior of the proposed top-anchored configuration. Therefore, this study

deliberately adopts a monolithic casting to conduct a mechanism validation test. The objective is not to simulate the actual retrofitting construction process, but to isolate and validate the load-transfer mechanism of the novel anchorage topology under idealized conditions. The results obtained represent an idealized upper-bound benchmark, and it must be emphasized that subsequent practical applications must account for the actual behavior of the post-installed anchorage interface.

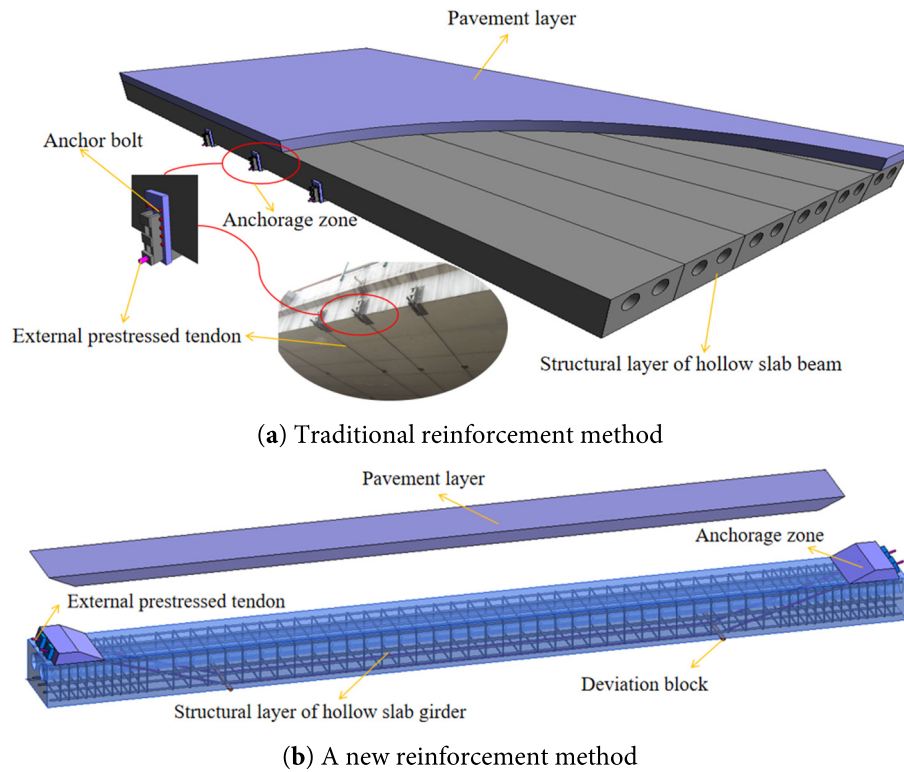


Figure 1: External prestressed reinforcement of hollow slab beam.

Through the integration of experimental testing and finite element numerical simulation, the feasibility and design principles of the system were examined. The influence of different effective prestressing stresses and tendon diameters on the cross-section stiffness of the hollow slab beam was analyzed. The results obtained from the monolithic specimens serve as a theoretical performance benchmark, providing an upper-bound reference for future practical applications involving post-installed anchorages.

2 Experiment Program

2.1 Specimen Design

The loading device and schematic diagram are shown in Fig. 2, with specific dimensions illustrated. Two prestressed hollow-slab beams, with a length of 6 m and a calculated span of 5.6 m, were constructed for this study. The specimen designated as N1 serves as the control beam with conventional internal prestressing.

Regarding the fabrication of N2, it is vital to note that this study serves strictly as a mechanism-validation test. Unlike practical retrofit applications, where anchorages are post-installed, the anchorage zones and deviation blocks of N2 were cast monolithically with the hollow slab beam in a single pour to eliminate the interference of interfacial slip. This fabrication strategy was intentionally adopted to simulate a

perfect bond condition. Consequently, the experimental behavior of N2 serves purely as an idealized upper-bound benchmark. To avoid misunderstandings regarding direct engineering applicability, it is explicitly noted that the actual behavior of the post-installed anchorage interface must be fully investigated in future practical implementations.

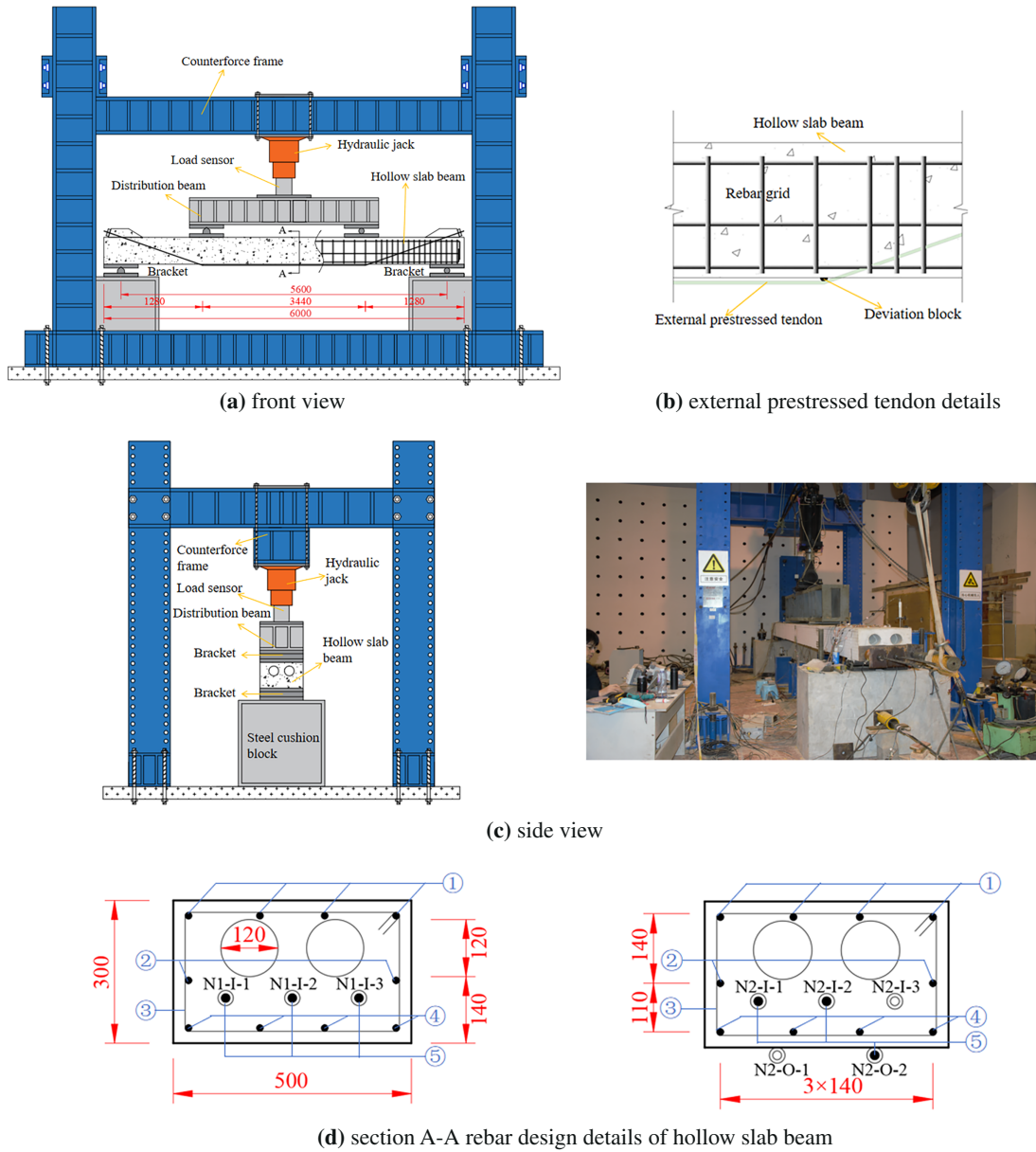


Figure 2: Loading device and schematic diagram.

N1 and N2 have identical internal reinforcement arrangements and a concrete cover thickness of 25 mm. The reinforcement design details of the hollow slab beam at section A-A are shown in Fig. 2d. In Fig. 2d, the left side corresponds to specimen N1 and the right side to specimen N2. Here, labels ① and ② both indicate HRB400 hot-rolled ribbed bars with a diameter of 10 mm, ③ denotes HRB335 bars with a diameter of 8 mm, ④ represents HRB400 bars with a diameter of 12 mm, and ⑤ refers to prestressed steel strands with a diameter of 15.2 mm. Fig. 2b shows the specific arrangement of the reinforcing bars within the beam.

Test data show that the compressive strength of concrete for specimens N1 and N2 is 26.37 MPa (3.82 ksi) and 27.14 MPa (3.94 ksi), respectively; the tensile strength is 2.78 MPa (403 psi) and 2.82 MPa (409 psi), respectively; and the elastic modulus E_c is 31.25 and 30.97 GPa, respectively.

2.2 Loading System and Device

The test loading device for the hollow slab beam is shown in Fig. 2a,c. A 100-t hydraulic jack is employed to apply a load to the hollow slab beam at two symmetrical loading points. The load is distributed to the hollow slab beam through a 2.9 m distribution beam.

Before the formal loading, the test beam was preloaded to eliminate gaps between components and ensure complete contact [15]. After verifying that there were no abnormalities in any of the instruments, the load was removed. During the formal test, each stage was loaded for 10 min, making it easier to record test observations. Before the test beam cracking, each stage was loaded in increments of 5 kN. Once the first crack in the hollow slab beam appeared, the load was increased by 10 kN per stage until the structure failed.

To measure the concrete strain distribution and deflection of the hollow slab beam [16], three concrete strain gauges (CSG) were attached to the top and two to the bottom of the hollow slab beam. Due to potential lateral loading during the test, the concrete strain gauges were symmetrically arranged on the test beam, and the average value was taken as the concrete strain value [17]. Displacement sensors (DS), concrete strain gauges (CSG), Rebar strain gauges (RSG), and hoop reinforcement strain gauges (HSG) were installed at the corresponding position on the hollow slab beam, as shown in Fig. 3.

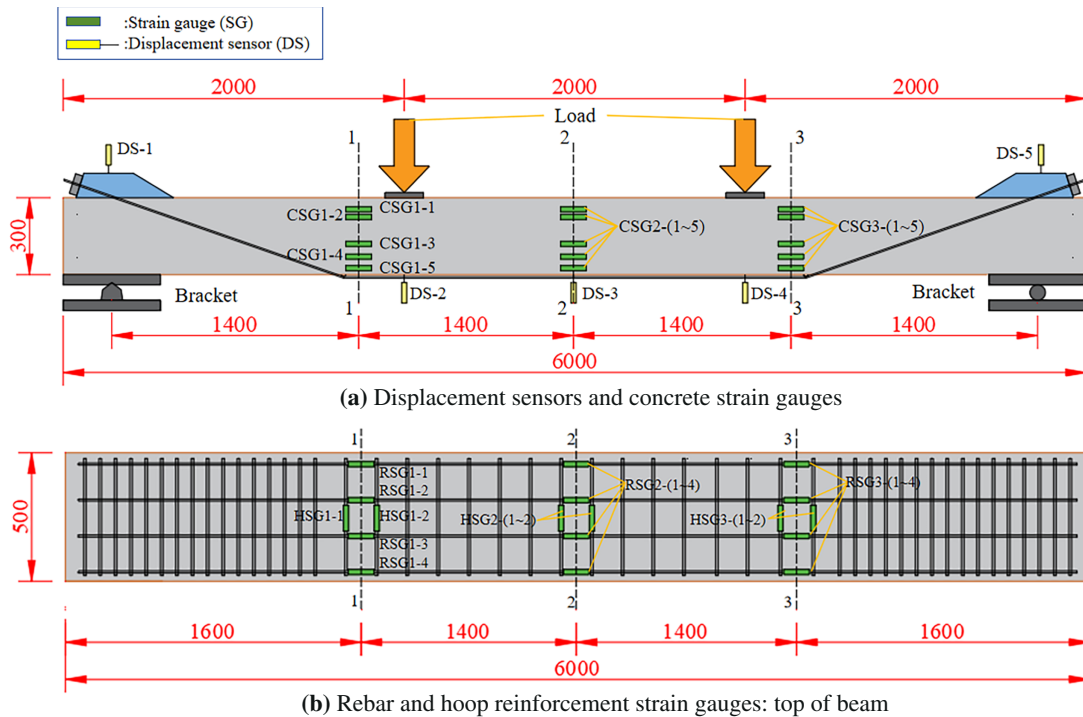


Figure 3: (Continued)

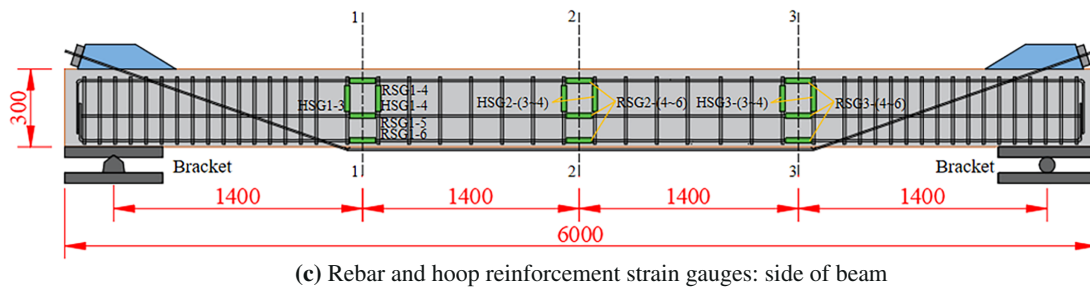


Figure 3: Layout of displacement sensors and strain gauges.

3 Experimental Results

3.1 Failure Mode and Crack Development Process

(1) Failure mode

Fig. 4 illustrates the overall failure condition of the hollow slab beams. As shown in Fig. 4a,b, both the external prestressed hollow slab beam N2 and the conventional prestressed hollow slab beam N1 display characteristic failure modes that are well-suited for reinforced beams. The beams undergo a transition from the yield of longitudinal reinforcement to eventual limit failure, demonstrating significant deformation and commendable ductility. The predominant damage characteristic observed is the compression-induced crushing of concrete in the pure bending section.

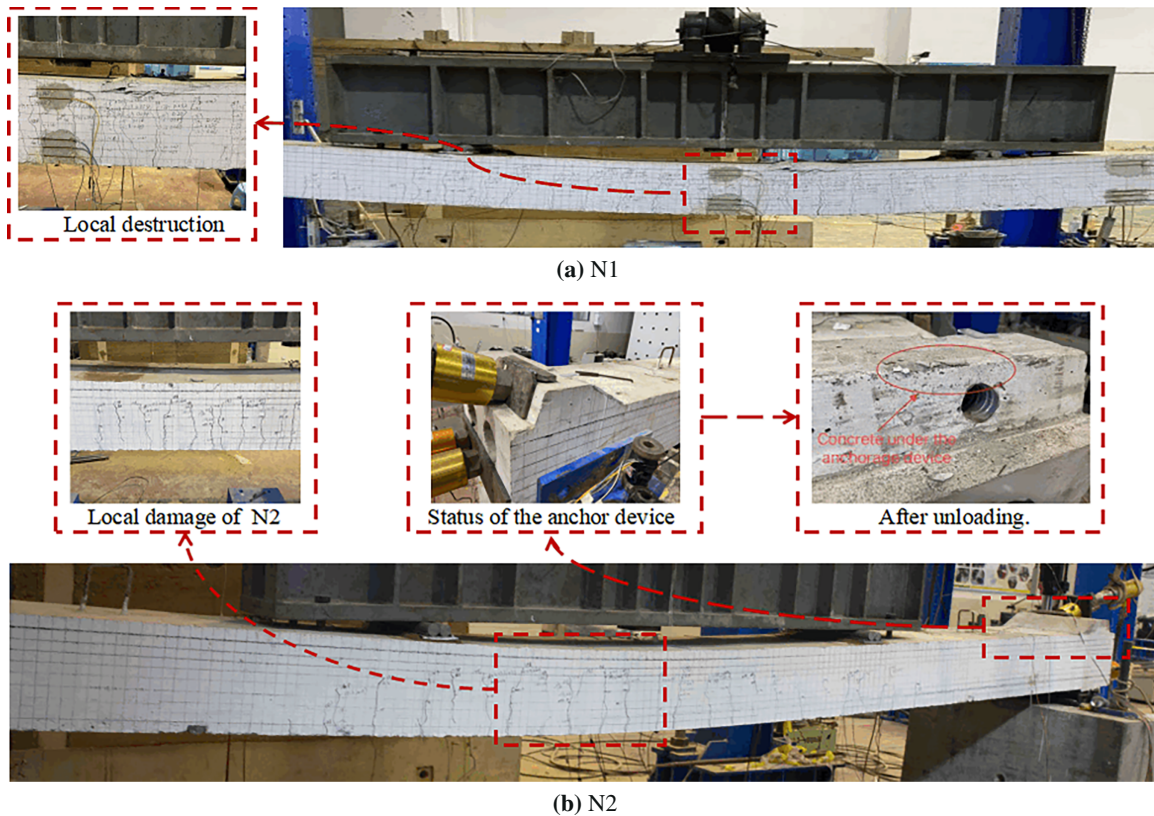


Figure 4: Overall failure condition of the hollow slab beams.

As can be observed in Fig. 4b, after the completion of the test on specimen N2, minor spalling of concrete was noted at the edge of the anchorage device upon removal of the anchor plate. This phenomenon was attributed to the uneven surface contact between the anchor plate and the concrete. Crucially, consistent with the monolithic fabrication of the specimen, no slip-induced cracking or debonding occurred at the interface between the anchorage device and the structural layer throughout the test.

While the monolithic nature of N2 precluded interface failure, the stress monitoring data provides valuable insights for practical post-installed applications. Under the action of the external prestressing force, the maximum principal stress measured in the concrete beneath the anchorage zone was only 1.61 MPa, which is lower than the characteristic tensile strength. The maximum transverse stress was merely 0.5 MPa, and the vertical and longitudinal stresses ranged from 2.62 to 11.41 MPa, all remaining well below the concrete's compressive strength. These low stress levels indicate that the geometric design of the top-anchored system effectively distributes loads. While the low stress levels observed under idealized conditions suggest that standard interface treatments (such as surface roughening and shear dowels) might be sufficient to resist these interfacial stresses [18] in practical retrofitting scenarios, this remains a theoretical projection. Because the current monolithically cast setup does not capture actual post-installed retrofit interactions, the true practical sufficiency of these interface treatments requires future experimental validation.

(2) Crack development process

As shown in Fig. 5, which contrasts the crack development in test beams N1 and N2, the crack distribution in both test beams is relatively uniform. Reinforcement beam N2, which utilizes external prestressed tendons, exhibits fewer cracks compared with beam N1. The overall crack distribution of beam N2 is more uniform, resulting in a smaller average crack width, while beam N1 has numerous fine cracks across its span. The main reason is that the external prestressed reinforcement contacts the beam body at the anchor end and the deviation block, resulting in greater constraint on the span concrete than the internal prestressed reinforcement [19]. Simultaneously, the final extension height of beam N2 is lower than that of beam N1. Therefore, the use of external prestressed tendons to replace internal prestressed tendons, along with a mixed reinforcement mode, can significantly enhance the crack resistance of hollow slab beams [20].

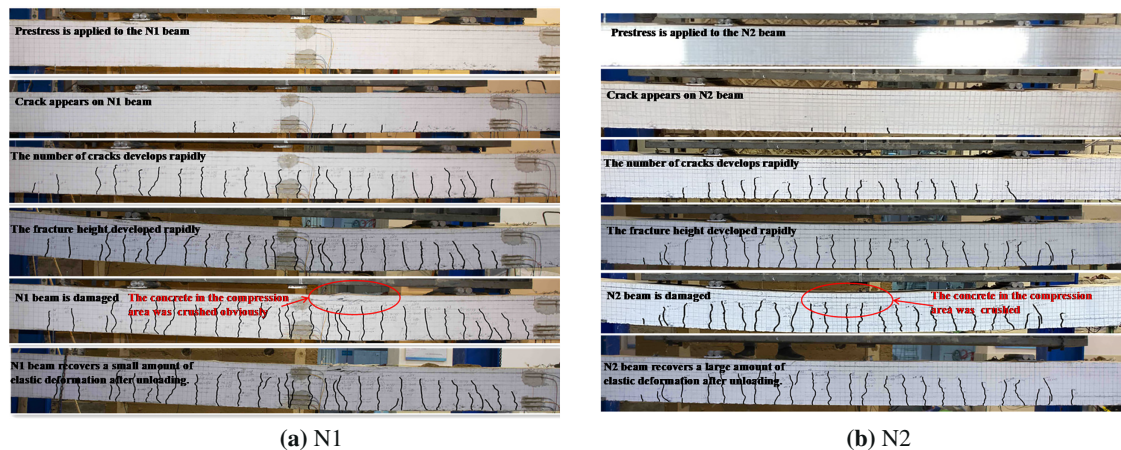


Figure 5: Crack development process.

3.2 Analysis of the Bending Bearing Capacity

Fig. 6 represents the load–deflection comparison curve of the loading process for contrast beam N1 and the externally prestressed strengthened beam N2. The curve demonstrates a three-phase linear relationship,

with two inflection points corresponding to the concrete cracking time in the tension zone at the bottom of the test beam and the yield time of the conventional reinforcement in the tension zone. Macroscopically, the load-deflection behavior of externally prestressed reinforced beams follows a similar pattern to that of ordinary internally prestressed beams [21], which can be divided into three stages as outlined. However, it is crucial to note that microscopically, the underlying strain development mechanisms diverge significantly between the two systems after cracking, primarily due to the departure of the external tendons from the classical plane-section assumption.

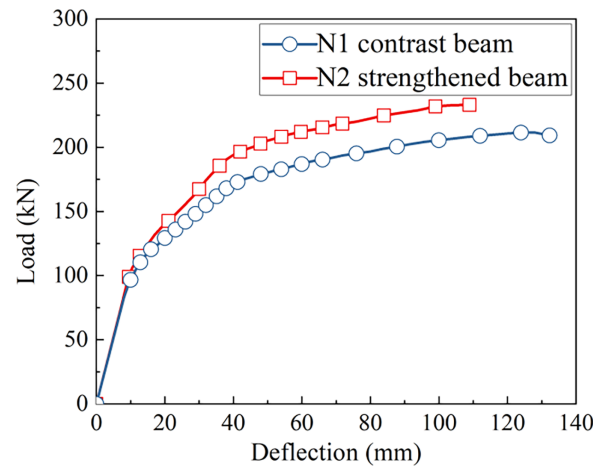


Figure 6: Comparison of the median load-deflection curve of test beams N1 and N2.

Stage I:

The elastic working phase. Due to the prestressing tendons, the test beam was in a state of tension at the top and compression at the bottom. As the load increased, the compressive stress in the lower part of the beam was progressively counteracted [22]. When the tensile stress in the concrete exceeded its ultimate tensile strength, cracks began to appear in the beam, and the corresponding external load at this point was defined as the cracking load of the beam.

Stage II:

Crack development working stage. Upon the load reaching 105 kN on beam N1, three fine cracks appeared at the bottom of the mid-span of beam N1, with a mid-span deflection of 9.8 mm. When the load increased to 171.84 kN, the tensile reinforcement in beam N1 yielded. The force process of the N2 beam was similar to that of the N1 beam. Under an external load of 114 kN, cracks appeared in the N2 beam, and the mid-span deflection was 10.6 mm. When the load reached 196 kN, the tensile reinforcement yielded, and the mid-span deflection was 42 mm. Under the same load increment, the mid-span deflection growth rate of the two hollow slab beams was significantly higher than that in the elastic stage. After cracking, the stiffness of N1 and N2 beams decreased sharply, and the crack width increased rapidly [23].

During this stage, the cracks are primarily located in the “pure bend” area and are predominantly vertical. Before the reinforcement yields, isolated diagonal cracks appear outside the “pure bend” area. The strain growth of the tensile reinforcement is notably faster than that in stage I. In this post-cracking stage, the fundamental difference between N1 and N2 becomes evident. For the internally prestressed beam N1, the tendon strain increases locally at the crack locations strictly following section compatibility. Conversely, for the externally prestressed beam N2, the external tendon strain lags behind the adjacent concrete strain because deformation compatibility is only enforced at the anchorage and deviation points. Consequently,

the strain increment of the external tendon is averaged over its unbonded length. The bending stiffness after cracking of beam N2 was greater than that of beam N1, mainly because the larger effective eccentricity of the external tendon compensates for this strain lagging effect, delaying the rapid propagation of cracks.

Stage III:

The destructive stage. When the external load on beam N1 reached 211.5 kN, the concrete in the mid-span compression zone reached its ultimate compressive strain and failed, with a maximum deflection of 102.3 mm. When the load on beam N2 reached 232.8 kN, the concrete in the compression zone was crushed, and the maximum deflection was 87.1 mm. During this stage, due to the yielding of the plain reinforcing bars in the structure, the load-deflection curve showed a second inflection point. The bending stiffness of both N1 and N2 beams further decreased. Under the same load increment, the rate of deflection increase accelerated, and loading continued until the beams failed.

During this stage, the ultimate load of the N2 beam was higher than that of the N1 beam, and the crack width and development height were lower than those of the N1 beam. This indicates the significant enhancement in the beam's load-bearing capacity by external prestressing tendons and their inhibitory effect on crack development and propagation speed [24]. The bending capacity of reinforced beam N2 increased by about 10% compared with that of beam N1. In comparison, the beam's ductility was reduced after reinforcement with external prestressing tendons. The limit deflection of N2 was reduced by 18% to N1. It should be specifically noted that the performance difference between N1 and N2 results from the coupled effects of the altered load-transfer path, the larger effective eccentricity of the external tendon, and the local stiffness variations introduced by the monolithic anchorage and deviation blocks.

3.3 Concrete Strain Analysis

Fig. 7 presents the measured strain distribution diagram for the concrete in beam N1, while Fig. 8 displays the measured strain distribution diagram for the concrete in beam N2. Specifically, the value at a height of 7.8 mm from the beam bottom represents the strain of the external prestressed tendon. While the reference hollow slab beam conforms well to the plane section assumption throughout the process from loading to structural failure, the behavior of the externally strengthened beam differs. Prior to cracking, the concrete strain varies linearly along the beam height, and the deformation of the external prestressed tendons is compatible with the beam deformation. However, after cracking, although the beam body on average still satisfies the plane section assumption, the strain of the external prestressed tendons lags behind that of the adjacent concrete. The primary reason is that deformation compatibility is only enforced at the anchorage zones and deviation points. Consequently, the strain increment of the tendons depends solely on the global relative displacement between these sections, resulting in a uniform stress distribution along the tendon length between them. Because the external tendon does not experience the localized extreme strain peaks found at the cracks of internally bonded beams, it avoids premature yielding at crack locations.

Under the limit state of normal use and bearing capacity for the external prestressed reinforced beam, the average strain change law of concrete aligns with the flat section [25]. However, the external prestressed tendon strain cannot be calculated based on the flat section assumption, mainly because the stress and deformation of the external prestressed tendons are primarily associated with the overall deformation of the beam.

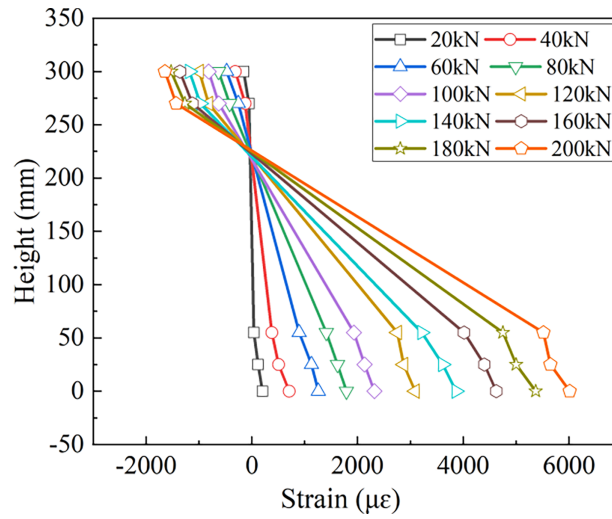


Figure 7: Concrete strain at mid-span of N1.

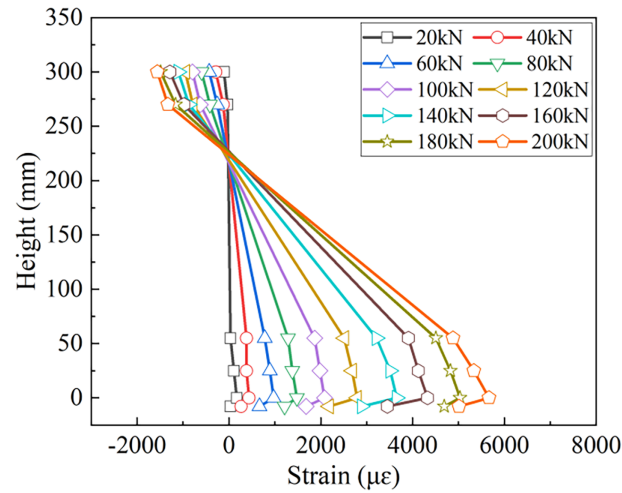


Figure 8: Concrete strain at mid-span of N2.

3.4 Strain Analysis of Ordinary Reinforcement

Fig. 9 shows the relationship curve of the reinforcement load in the tensile area of N1 and N2. Based on the strain trend, the stress performance of the reinforcement in the middle tensile area of N1 is similar to that in N2. Using the strain of the tensile bar in the beam N1 span as an example, the tensile reinforcement strain increases almost linearly during the test as the load increases [26], with the material remaining in the linear elastic stage. When the beam cracks, the strain rate of the tensile reinforcement increases. This is because the concrete in the tensile zone no longer bears tensile stress. The reinforcement entirely bore the tensile stress. As the load continues to rise, the tensile reinforcement strain increases rapidly [27]. The reinforcement strain attains a yield of 2038 microstrains, and as the load continues to increase, the concrete in the pressure area of beam N2 eventually reaches the ultimate pressure strain. At this juncture, the concrete undergoes crushing, resulting in reinforcement strain of 2986 microstrains. The strain pattern of the ordinary reinforcement closely mirrors that of the beam N1.

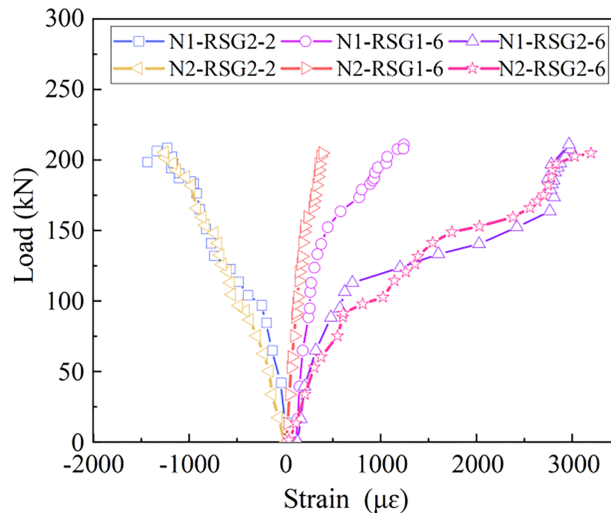


Figure 9: Load-strain relationship of the tension reinforcement.

3.5 Analysis of Stress Increment of Prestressed Steel Strand

Fig. 10 illustrates the relationship between the increase in prestressed tendon stress and the beam deflection. During loading, all prestressed tendons exhibit similar stress-increment changes. Before the test beam cracks, the change of stress increment is not obvious, as the beam remains elastic. Once cracking occurs, the rate of stress increment growth accelerates. Then, the prestressed tendons haven't yielded, stress increments and deflection are linearly related. As the load increases further and the prestressed tendons yield, the beam deflection increases faster than the stress increment, no longer conforming to a linear relationship [28].

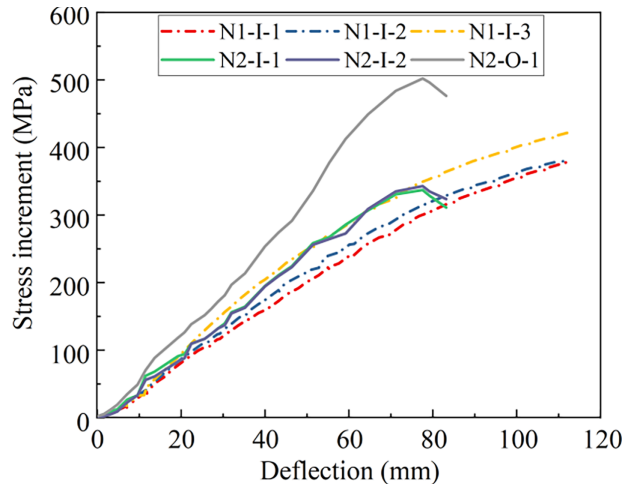


Figure 10: The relationship between prestressed tendon stress increment and beam deflection.

As the test beam approaches the limit state, all steel strands have yielded. The stress levels for the three prestressed reinforcements in beam N1 range between 378.3 and 421.6 MPa, while the stress increments of the two internal prestressed tendons in beam N2 are 311 and 323 MPa, respectively, and the stress increment of the external prestressed tendon reaches 502 MPa. Comparatively, the stress increment of the external prestressed tendon is higher than that of the internal prestressed tendon under the same displacement. This

phenomenon may be attributed to the greater eccentric distance of the external prestressed tendons, leading to a larger increase in strain at the same position in the external prestressed tendons than in the internal prestressed tendons [29]. The external prestressed tendon scheme can fully exploit material properties and offers significant advantages in improving the ultimate bearing capacity of hollow slab beams.

4 Numerical Analysis

The tests are supplemented with a non-linear finite element analysis model to elucidate the beam failure mode. Considering the material nonlinearity of external prestressed tendons and concrete, this study establishes a numerical model. Based on the numerical model, the effects of effective stress and diameter of external prestressing tendons on the flexural stiffness of hollow slab girders are analyzed.

4.1 Finite Element Model

The finite element software ABAQUS was used to develop a three-dimensional model of the external prestressed hollow slab beam for simulation, which was subsequently corrected based on test results. The finite element mesh division of the structural model used in the calculation is shown in Fig. 11. The concrete finite element mesh uses a three-dimensional eight-node hexahedron (C3D8R) element. The maximum size of the X-axis and Z-axis is 62 mm, and the maximum size of the Y-axis is 60 mm. Other elements, such as the steel anchor plate, load transfer plate, and deviation blocks, are also explicitly modeled. The space truss element (T3D2) is used to simulate both ordinary and internally prestressed steel bars.

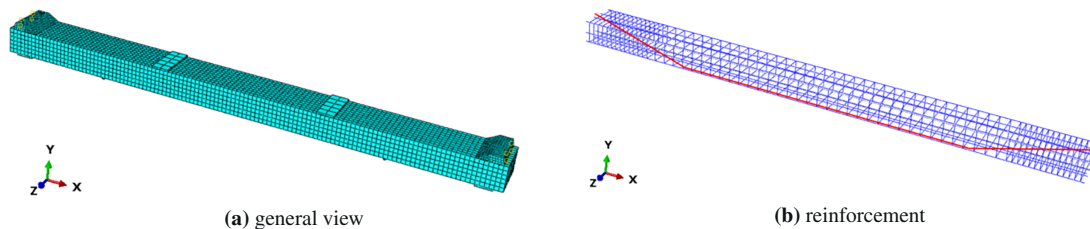


Figure 11: The finite element mesh of beam N2.

To accurately simulate the monolithic casting conditions of the experimental specimens, different interaction properties were assigned to the model interfaces. First, a Tie Constraint was applied to the interfaces between the anchorage/deviation blocks and the hollow slab beam surface. This constraint ensures that the newly added components and the beam body act as a unified whole, preventing any relative displacement at these critical load-transfer zones. This modeling strategy serves to validate the mechanical performance of the top-anchored configuration under the ideal limit state. It must be explicitly noted that this tie-constraint model is appropriate only for simulating the monolithic behavior of the idealized benchmark specimen. It does not represent actual retrofit interactions, where complex interface slip and separation behaviors might occur.

Regarding the prestressing system, the anchorage relationship between the external prestressed tendon and the anchorage area is simulated by the beam element connection (beam MPC), which effectively couples the degrees of freedom to simulate the anchorage mechanism.

4.2 Material Property Relationship

(1) Properties of concrete

The concrete damage plasticity (CDP) model uses an isotropic elastic-plastic damage model to account for tensile and compressive damage and to describe the stress behavior of concrete after entering plasticity.

In the CDP model, the damage mechanisms in concrete are tensile cracking and compression failure. The evolution of the yield surface is controlled by the equivalent plastic strains in tension and compression, and the degradation of the tensile and compressive properties of concrete is analyzed using their respective damage factors.

$$\sigma = \begin{cases} f_c \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_0} \right)^2 \right] & \varepsilon < \varepsilon_0 \\ f_c \left[1 - 0.15 \left(\frac{\varepsilon - \varepsilon_0}{\varepsilon_{cu} - \varepsilon_0} \right) \right] & \varepsilon_0 \leq \varepsilon \leq \varepsilon_{cu} \end{cases} \quad (1)$$

where σ is concrete stress, ε is concrete strain, f_c is the peak stress of concrete, and ε_0 and ε_{cu} are the peak strain and ultimate strain of concrete, respectively.

Based on the material properties described in Section 2.1, the constitutive parameters of the concrete were established. Table 1 lists the plastic parameters of the concrete plastic damage model.

Table 1: Plastic parameters of the CDP model.

Dilatation Angle	Eccentricity	Strength Ratio f_{b0}/f_{c0}	K_c	Viscosity Parameter
30	0.1	1.16	0.667	0.0005

(2) Reinforcement model

The elastic-plastic constitutive model is used for ordinary reinforcements. For the prestressed tendons, linear elastic behaviour was assumed with a Young modulus of 190 GPa. The constitutive relationship of the prestressed tendon was linearly reinforced elasto-plastic.

$$\sigma = \begin{cases} E_0 \varepsilon & \varepsilon < \varepsilon_0 \\ f_y & \varepsilon > \varepsilon_0 \end{cases} \quad (2)$$

where E_0 is the elastic modulus of the steel bar, f_y is the steel yield strength, and ε_0 is the steel yield strain. Regarding the external prestressing tendons, the interaction between the tendon and the deviation block was modeled using a Surface-to-Surface Contact. In the tangential direction, a Penalty friction formulation with a coefficient of 0.2 was employed to simulate the potential forward and backward slip behavior of the external tendon during loading.

4.3 Numerical Simulation Strategy

(1) Phased analysis and prestressing application

Numerical analyses strictly follow the experimental procedure, which requires the initial prestressing force to be provided to the prestressed tendons after the component assembly is completed. Therefore, a staged analysis is required in the numerical analysis. In the experimental study, the self-weight was modelled as a uniformly distributed mass force, and the external load was applied as a concentrated force at the centre of the load transfer plate. Prestressing of the tendon is achieved using the cooling method, which exploits the tendon's negative coefficient of thermal expansion to change its temperature and thereby apply the prestressing force. The coefficient of expansion ($\alpha_c = 1.2 \times 10^{-5}/^\circ\text{C}$) is defined in the material properties, and the size of the cooling value required at the corresponding stress is calculated according to Eq. (2). It is

necessary to create a temperature field for the load module with an initial temperature of 0, and to define the value of the changed temperature in subsequent analysis steps.

$$\Delta T = \frac{\sigma_{con}}{E \cdot \alpha_c} \quad (3)$$

where ΔT is the value of cooling required to be applied, σ_{con} is the magnitude of tension control stress, E is the modulus of elasticity of prestressing tendon, and α_c is the coefficient of linear expansion of prestressing tendons.

Phase I:

Components assemble. For beam N1, the model components activated in this phase are reinforcements, the concrete beam, and boundary conditions. For the beam N2, in addition to the components of beam N1, elements such as external prestressed tendons and deviation blocks are activated.

Phase II:

Prestressing. At this stage, the prestressed tendons are subjected to the desired initial stress values using the cooling method.

Phase III:

Beam loading until failure. At the beginning of this stage, a vertical displacement (along the Z axis) constraint is activated at the concentration point. This constraint is used for kinematic (displacement-controlled) loading.

(2) Calculation program

In addition to the material nonlinearities mentioned above, the finite element computational model also considers geometric nonlinearities. The geometric nonlinearity model uses the total Lagrangian full formulation. An incremental iterative method was used to obtain a solution. The displacement control method was used in the last stage. The vertical displacements of the nodes on the top surface of the steel plate were used as displacement control points. For each load increment, the equilibrium of internal and external forces is calculated using the Newton-Raphson iterative method. Each time a displacement is generated, an iteration is performed, and the solution finally converges after successive iterations.

4.4 Results of Numerical Simulations and Validation of the Model

The experimental data and the finite element model results are compared to verify the effectiveness of the external prestressed hollow slab beam modeling. The numerical simulation results for test beams N1 and N2 are illustrated in Figs. 12 and 13. Figs. 14 and 15 show the comparison between the experimental and numerical load-deflection curves for N1 and N2 beams. Before the test beam reached the cracking load, deflection increased linearly with the load. From cracking to reinforcement yielding, the tensile reinforcements entered the plastic stage, causing mid-span deflection to rise rapidly. After yielding, the deflection growth rate continues to accelerate under a small load increment. Finally, under the ultimate bearing capacity limit state, the displacement is large, and the test beam exhibits clear signs of ductility before failure. According to the comparison between numerical calculations and experimental results in Table 2, the errors between the finite element results and the experimental results in the cracking state, yield state, and bearing capacity limit state are small, and the trends of the curves are consistent. The maximum error at the critical point is only 9.14%, which shows that the model is reasonable.

The error between the numerical simulation results and the test results is analyzed, mainly due to the following reasons, including: (1) The finite element model is relatively ideal, and the unit force is uniform, which is different from the test. (2) In finite element modeling, there are artificial assumptions and simplifications in the selection of elements and constitutive models.

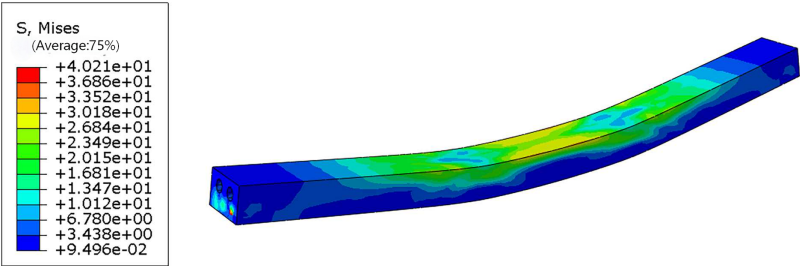


Figure 12: Concrete stress in the ultimate state of N1.

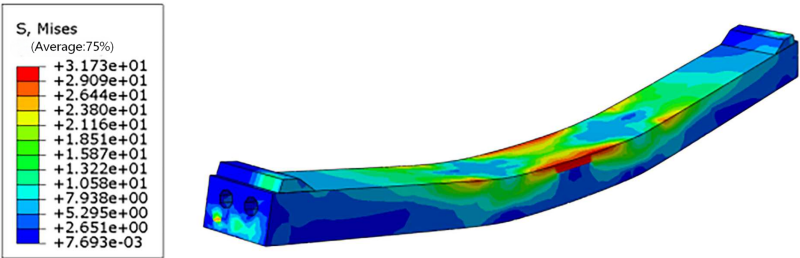


Figure 13: Concrete stress in the ultimate state of N2.

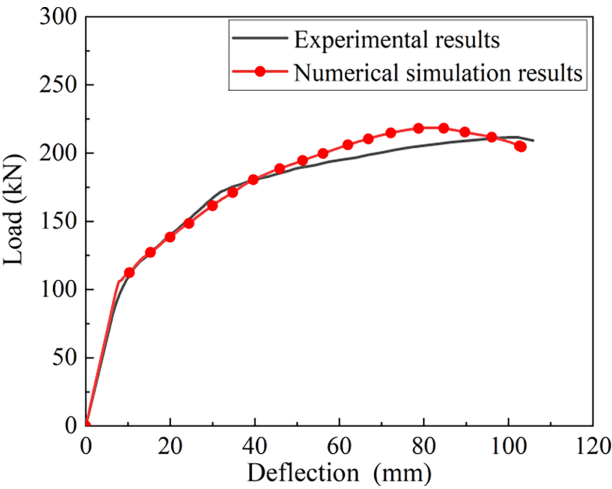


Figure 14: Load-deflection curve of N1.

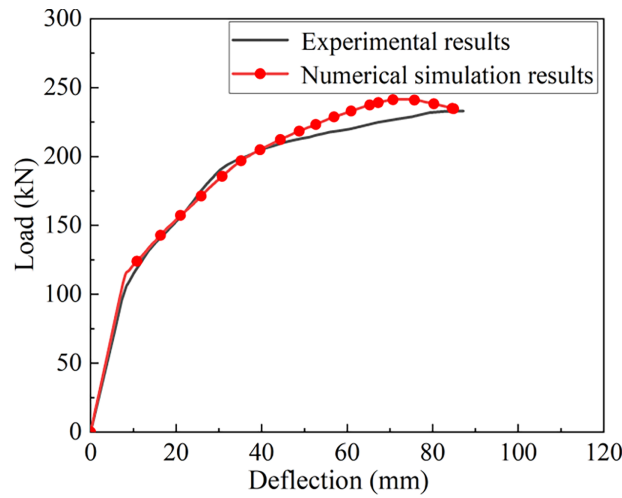


Figure 15: Load-deflection curve of N2.

Table 2: Comparison of experimental results and numerical simulation loads.

Model	Cracking Load (kN)		Error	Ultimate Load (kN)		Error
	Numerical Simulation	Experimental Result		Numerical Simulation	Experimental Result	
N1	105	96.2	9.14%	218.7	211.5	3.35%
N2	114	106	7.55%	241.6	232.8	3.52%

5 Parameter Analysis

5.1 Effect of Effective Stress on Section Stiffness

Based on the established finite element model of external prestressed hollow slab beam N2, the prestress is applied to the finite element model by the cooling method. Different initial effective stresses in external prestressed tendons are achieved by varying the reduced temperature. Initial effective stress of external prestressed tendons is set at 0, 0.15, 0.3, 0.5, and $0.65 f_{ptk}$, respectively. The mid-span load-deflection curve of external prestressed hollow slab beams under different effective initial stresses is shown in Fig. 16.

Before cracking, the slope of the load-deflection curve under the initial effective stress of different external prestressed tendons hardly changes, and there is basically no difference in the stiffness of the beam. This shows that the change of the effective stress of the external prestressed tendon has no significant effect on the stiffness before the cracking stage, mainly due to the large bending stiffness ($E_c I_0$) of the hollow slab beam before cracking. In addition, to reduce discrepancies with the test beam and ensure the credibility of the finite element model, only one external prestressed tendon is included, consistent with the N2 beam.

With the increase in the effective stress of the external prestressed tendon, the section stiffness increases under the same load, and the section stiffness does not change before cracking. When the external load exceeds the cracking load, the same load increases the initial effective stress of the external prestressed tendon, resulting in greater structural stiffness. This trend becomes more pronounced as the external load increases. Therefore, the initial effective stress of the external prestressed tendons of the hollow slab beam can be appropriately increased to improve the stiffness of the hollow slab beam after cracking. The load-stiffness

relationship curves of the beam under different initial effective stress are shown in Fig. 17. The stiffness of the beam with different initial effective stresses is shown in Table 3.

Fig. 18 shows the load-stiffness reduction factor curves of the beam under different initial prestress values. According to the stiffness degradation curve, the stiffness of the hollow slab beam decreases sharply after cracking, then gradually decreases as the load continues to increase, and finally slows. When the load is constant, as the effective stress of the external prestressed tendon increases, the stiffness attenuation coefficient increases and the stiffness damage decreases. Therefore, increasing the initial effective stress of the external prestressed tendon can better address stiffness degradation after cracking.

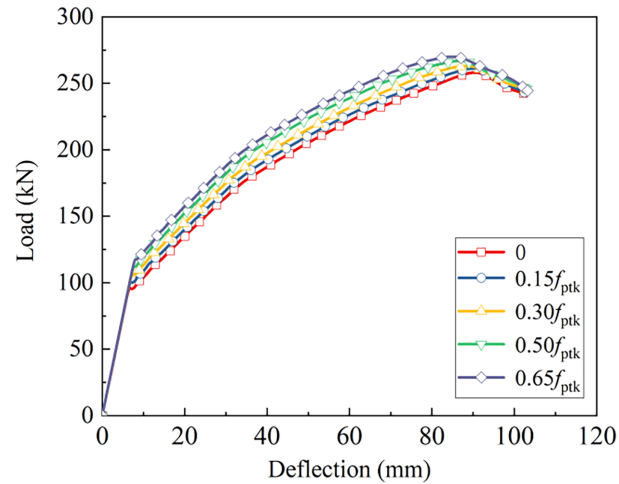


Figure 16: Load-deflection curves with different initial effective stresses of N2.

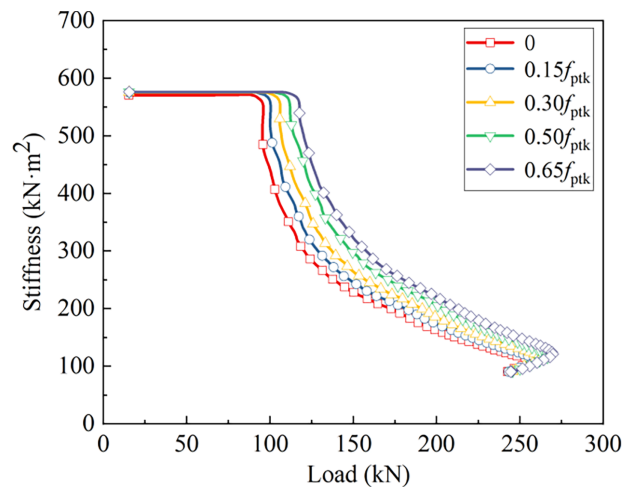
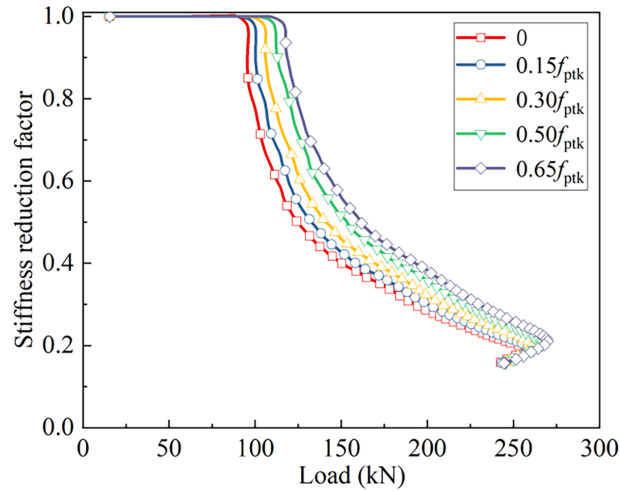


Figure 17: Load-stiffness relationship curves.

Table 3: Stiffness of beam with different initial effective stresses.

Load (100 kN)	Stiffness of Beam (kN·m ²)				
	0	0.15 f_{ptk}	0.3 f_{ptk}	0.5 f_{ptk}	0.65 f_{ptk}
0.15	570.18	575.25	575.52	575.71	575.91
1.0	442.48	556.70	572.08	575.74	575.93
1.1	357.96	407.33	463.95	567.48	575.35
1.2	301.67	339.75	390.01	456.54	502.67
1.3	269.98	295.36	328.78	383.04	411.74
1.4	246.18	266.66	290.36	331.04	362.90
1.5	228.58	245.31	263.80	296.38	323.19
1.6	215.24	228.12	269.47	269.47	293.37
1.7	202.68	214.83	229.21	251.37	268.27
1.8	189.16	201.90	216.11	233.97	250.55
1.9	174.29	187.06	201.37	220.90	235.37
2.0	162.30	173.15	184.47	205.89	220.20
2.1	151.75	161.29	171.22	188.96	203.35
2.2	142.28	150.60	159.58	175.16	186.76
2.3	133.21	140.63	149.10	162.60	172.71
2.4	124.54	131.47	139.17	150.66	160.66
2.5	116.48	122.68	129.80	140.71	149.09
2.6		113.28	119.81	129.71	138.27
2.7					122.27

**Figure 18:** Load-stiffness reduction factor curves.

5.2 The Influence of the Area of External Prestressed Tendon on the Section Stiffness

The influence of the area of the external prestressed tendon on the section stiffness was analyzed by changing the diameter of the external prestressed tendon. The different diameters of the external prestressed

tendon are $d_1 = 9.5$ mm, $d_2 = 12.7$ mm, $d_3 = 15.2$ mm, $d_4 = 17.8$ mm, $d_5 = 21.6$ mm. The mid-span load-displacement curves of hollow slab beams with different diameters of external prestressed tendons are shown in Fig. 19.

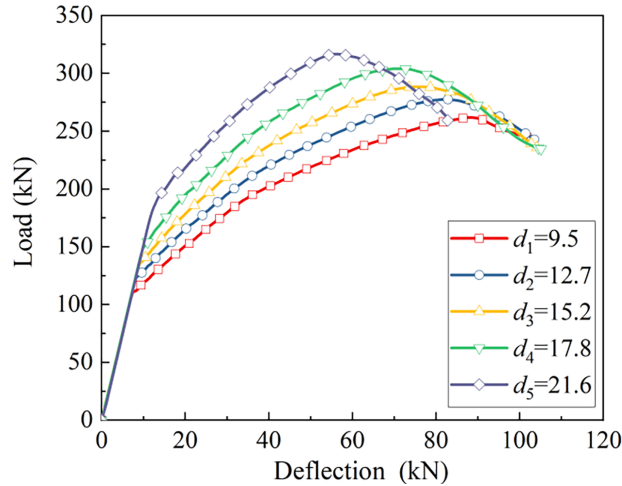


Figure 19: Load-deflection curves under different diameters of the external prestressed tendon of N2.

Before cracking, the slope of the curve under different diameters of external prestressing tendons is almost unchanged, and there is basically no difference in the stiffness of the beam. It shows that changes in the diameter of the external prestressed tendons have little effect on stiffness before cracking. In calculating the transformed section moment of inertia I_0 , the net section moment of inertia of the hollow slab beam is much larger than the moment of the external prestressed tendon to the centroid of the beam section. The numerical simulation and theoretical analysis results for stiffness are consistent. Thus, changes in the cross-sectional area of external prestressed tendons have little effect on the beam's stiffness before cracking.

With the increase in the diameter of the external prestressed tendon, the section stiffness increases under the same load. Before cracking, the section stiffness increases linearly with the diameter of the external prestressed tendon. When the external load is greater than the cracking load, under the same load, the structural stiffness increases with the increase of the diameter of the external prestressed tendon, and this trend is more obvious with the increase of the external load. Therefore, the external prestressed tendon diameter of the hollow slab beam can be appropriately increased to improve the stiffness of the hollow slab beam after cracking. The load-stiffness relationship curves of the beam with different diameters of external prestressed tendons are shown in Fig. 20. The stiffness of the beam with different diameters of external prestressed tendons is shown in Table 4.

The stiffness degradation process of hollow slab beams with different external prestressed tendon diameters is analyzed. Fig. 21 shows load-stiffness reduction factor curves of the hollow slab beam with different diameters of external prestressed tendons. According to the stiffness degradation curve, the stiffness of the hollow slab beam decreases sharply after cracking, then gradually slows as the load continues to increase, and finally approaches a slow rate. When the load is constant, as the external prestressed tendon diameter increases, the slope of the stiffness-damage and stiffness-reduction curves gradually decreases. Therefore, increasing the area of external prestressed tendons can better address the stiffness attenuation after cracking in hollow slab beams.

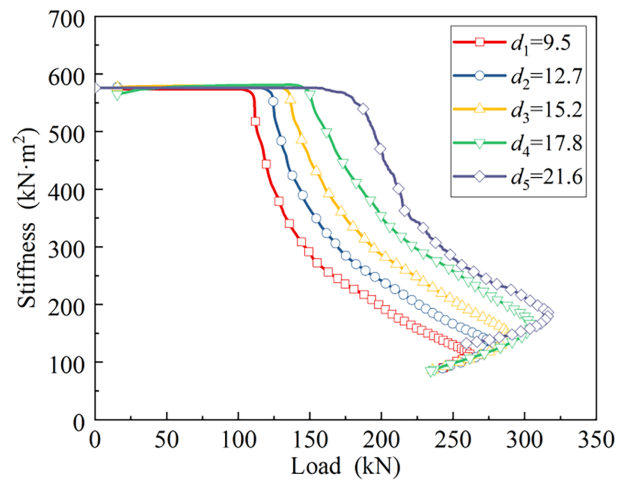


Figure 20: Load-stiffness relationship curve.

Table 4: Stiffness of beam with different diameters of external prestressed tendon.

Load (100 kN)	Stiffness of Beam (kN·m ²)				
	$d_1 = 9.5 \text{ mm}$	$d_2 = 12.7 \text{ mm}$	$d_3 = 15.2 \text{ mm}$	$d_4 = 17.8 \text{ mm}$	$d_5 = 21.6 \text{ mm}$
0.15	574.08	577.12	577.05	564.69	575.93
1.0	574.09	577.14	579.61	580.55	575.93
1.1	564.40	577.11	579.63	580.55	575.93
1.2	440.44	572.69	579.68	581.03	575.90
1.3	368.73	483.87	576.63	581.16	575.90
1.4	322.73	413.59	513.69	580.63	575.90
1.5	289.83	364.34	448.40	564.84	575.90
1.6	261.62	328.01	404.91	507.05	573.44
1.7	243.87	298.25	365.75	456.66	568.23
1.8	227.60	274.22	334.70	419.15	557.51
1.9	214.31	256.51	308.57	385.40	524.98
2.0	196.54	241.34	286.24	353.24	464.67
2.1	179.53	225.76	270.70	324.90	410.32
2.2	165.41	209.56	255.02	304.83	352.18
2.3	152.82	194.05	237.95	287.71	330.59
2.4	141.08	179.27	221.75	273.64	304.48
2.5	129.99	166.46	206.98	258.13	280.16
2.6	116.97	155.18	191.54	242.16	263.39
2.7		143.58	178.51	224.94	248.37
2.8			165.75	210.00	235.78
2.9			146.06	195.33	225.52
3.0				179.65	212.14

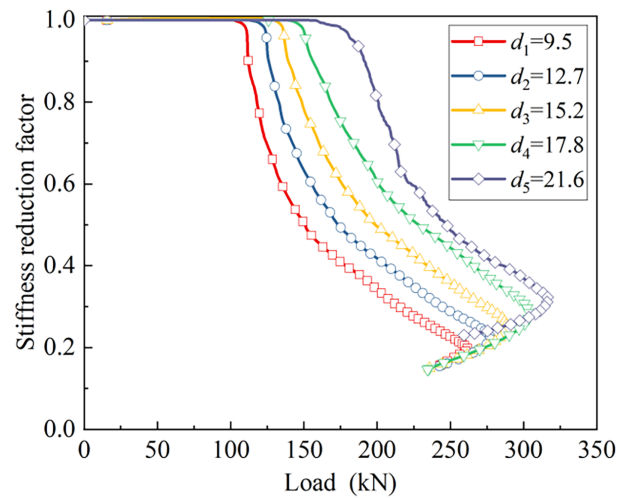


Figure 21: Load-stiffness reduction factor curve.

6 Conclusion

This study presents an experimental and numerical investigation into a novel top-anchored external prestressing configuration for hollow slab beams. To evaluate the fundamental mechanical feasibility of this system without the influence of construction-induced interface defects, monolithically cast prototype specimens (N2) were tested and compared with conventional internally prestressed beams (N1). Based on the results, the following conclusions and design recommendations are drawn:

1. The stress evolution of the externally prestressed beam N2 is consistent with that of the internal prestressed beam N1. The ultimate flexural capacity of N2 increased by approximately 10% compared to N1. The flexural ductility (ultimate displacement) of N2 decreased by 14.82%, while the system maintained typical reinforced concrete failure characteristics.
2. The monolithically cast anchorage zone remained intact at ultimate failure. No cracking or crushing was observed in the anchorage region, and the concrete stress levels remained within design limits. However, it must be noted that this result merely confirms that the geometric design of the top-anchored system effectively transmits prestressing forces strictly under idealized, perfect-bonding conditions as a benchmark demonstration.
3. An increase in the initial effective stress of the external tendons effectively improved the post-cracking stiffness of the hollow slab beam. Although stiffness before cracking was unaffected, the section stiffness after cracking increased proportionally with tendon stress under identical loads, demonstrating the effectiveness of higher initial prestress levels.
4. Increasing the tendon diameter also enhanced beam stiffness both before and after cracking. Before cracking, cross-sectional stiffness increased almost linearly with tendon diameter. After breaking, greater tendon diameters led to more pronounced improvements in stiffness under equivalent loading, particularly at higher external load levels.

In summary, this study integrates theoretical analysis, benchmark testing, and numerical simulation to demonstrate the fundamental mechanical feasibility of the proposed top-anchored external prestressing configuration. The findings establish a theoretical upper-bound performance benchmark for the system under idealized bonding conditions. It must be explicitly noted that this study intentionally eliminated the critical complexities associated with post-installed anchorage components, particularly the interaction mechanism at the new-to-old concrete interface. Therefore, to fully validate the potential of this configuration

as a viable retrofitting solution for aging bridges, in-depth investigations into these practical interface dynamics are still required. Future research will focus on the long-term behavior of the new-to-old concrete interface and the fatigue resistance of the post-installed system under repeated loading, thereby laying a solid foundation for its widespread application in practical bridge strengthening projects.

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Availability of Data and Materials: All data, models, and code that support the findings of this study are available from the corresponding author upon reasonable request.

Ethics Approval: This study does not involve human participants, animals, or identifiable personal data. Therefore, ethical approval was not required.

Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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