

Article

Experimental and Numerical Study on Shear Performance of a Full-Scale Thin-Walled Retard-Bonded Prestressed Concrete Box Girder

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Abstract

In order to produce long-span precast bridge members and reduce the transport pressure of precast members, the lightweight design of bridge components has become a key area of research. In this study, utilizing retard-bonded prestressing technology and high-strength concrete, a full-scale thin-walled box girder measuring 30,000 mm in length and 1600 mm in height was designed and fabricated. A shear test with a shear-span ratio of 2.5 was conducted to investigate the failure mode and shear carrying capacity. Subsequently, an Abaqus finite element (FE) model was established and validated with experimental data. Based on the FE model, numerical investigations were conducted to examine the influence of bonding between prestressed steel strands and concrete, stirrup ratio, web thickness and bottom flange thickness at the end of the box girder, concrete strength and length of UHPC end zone on the shear performance of thin-walled box girder. The results indicate that the retard-bonded prestressed box girder exhibits acceptable mechanical performance. Additionally, intensifying the ends of the box girder with ultra-high-performance concrete (UHPC) can further reduce the wall (i.e., web and bottom flange) thickness of the girder, enhance its shear carrying capacity, and achieve lightweighting. This discovery provides new insights into the lightweight design of bridge components.

Keywords: box girder; retard-bonded prestressed concrete; UHPC; precast component; lightweight

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1. Introduction

1.1. Research Background

In the past two decades, to overcome various drawbacks of on-site casting construction, prefabricated assembly methods have been commonly employed. This approach effectively addresses the limitations of on-site casting, offering advantages such as high industrialization, material savings, minimal pollution, convenient construction, minimal traffic disruption, easy control of prefabricated component quality, short construction cycles, and rapid investment recovery [1–6]. However, unlike assembly buildings, bridge components are limited by transportation and lifting conditions due to their large size

and weight. For instance, the span of prefabricated small box girder bridges is typically limited to 30,000 mm, and large cantilevered cap beams also face transportation and lifting challenges. Therefore, the lightweight design of bridge components has become a current research focus.

At present, there are three ways to realize structural lightweight design: optimization of structure form, application of high-performance material and development of new construction technology. With the aim of making full use of the mechanical properties of each member and to realize the lightweight design of the structure, the structure form is very important. For example, some researchers have explored the influence of different steel bar braces and connection on the flexural performance of the composite plate [7,8]. Based on this kind of research, Deng et al. [9] put forward the precast ultra-lightweight ECC-concrete composite slab with lattice girders. In addition, the steel-UHPC lightweight composite decks proposed by Liu et al. [10] have good safety and durability, and are lighter than the traditional decks. With the rapid development of materials science in recent decades, the lightweight design of structures can also be achieved by using high-performance lightweight materials. Common high-performance engineering materials include fiber-reinforced polymer (FRP), engineered cementitious composite (ECC), high-strength concrete (HSC) and ultra-high-performance concrete (UHPC). Because of their good mechanical properties and durability, these materials are often used to strengthen flexural members and design lightweight structures. Therefore, many studies are devoted to the application of ECC and UHPC in RC structures [11–17]. For example, Lu et al. [18] investigated the mix proportion of lightweight UHPC to enhance its application in lightweight structures. Yang et al. [19] conducted bending tests on UHPC beams, and the results demonstrated a significant improvement in the crack resistance and ductility of the beams. Additionally, He et al. [20] provided a comprehensive review of the development of fully precast light steel-UHPC composite bridges and found that replacing ordinary concrete with UHPC can significantly reduce deck thickness and structural self-weight. However, the high cost of UHPC limits its practical application.

When the structure form and material are determined, the construction technology also has influence on the performance of the structure. For reinforced concrete structures, the use of prestressing techniques can effectively enhance structural load-bearing capacity and reduce concrete consumption [21,22]. Early prestressed concrete, based on the bonding method between prestressed tendons and concrete, can be classified into bonded prestressed concrete (BPC) and unbonded prestressed concrete (UPC) [23,24]. BPC structures exhibit excellent mechanical properties but require prestressing ducts. This results in the web thickness of box girders typically being no less than 250 mm in girder spans exceeding 20,000 mm, making lightweight design unachievable for large-span box girder bridges. Conversely, UPC structures are characterized by flexible arrangement and simple construction but suffer from inferior mechanical properties [25]. To address these issues, researchers have proposed the retard-bonded prestressed concrete (RBPC), which combines the construction convenience of UPC with the excellent mechanical performance of BPC. The core of this technology is the retard-bonded tendon (RBT), as illustrated in Figure 1. The structure consists of steel strands, retard-bonded material (RBM), and ribbed sheath [26]. During the early stages of construction, the RBM is in a fluid state, resulting in minimal bonding between the RBT and the concrete. Once the RBM fully expands and hardens, significant frictional forces will generate between the ribbed sheath and the concrete, ensuring a strong bonding effect between the RBT and concrete. Finally, RBPC can achieve mechanical performance similar to BPC [27–29]. However, RBM usually takes a long time to solidify, affecting the construction period. This results in the retard-bonded prestressing technology being less used in practical engineering. But prefabricated components can overcome this problem well. Since this technology eliminates the need for

corrugated ducts and grouting, it can be applied to thin-walled prestressed concrete box girder, achieving the lightweight design of bridge components. Not long ago, Xiong et al. [30,31] designed full-scale retard-bonded prestressed UHPC variable section cantilever beams for the first time, which promotes the application of retard-bonded prestressing technology in practical engineering.

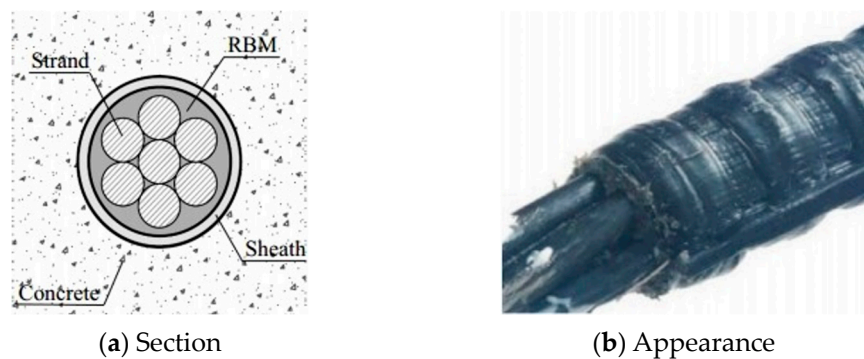


Figure 1. Details of the retard-bonded tendon (RBT) [26].

Recently, prestressed concrete small box girder bridges have gained widespread application in urban roads and highways due to their superior mechanical performance, cost-effectiveness, and simple appearance. The first highway overpass bridge made of UHPC in China consists of four prestressed box girders with a continuous span of 30,000 mm. Su et al. [32] then performed bending tests on full-scale prestressed UHPC thin-walled box beams and proposed an optimization scheme. The web thickness of the box girder is 80 mm, which reduces the weight of the whole girder, and this research content is mainly the flexural behavior of box girder. Considering the difficulty of construction and the high cost of UHPC, this may not be economical. Recent advances in three core fields provide the foundation for this study. For retard-bonded prestressing, the mechanical behavior of retard-bonded girders has been shown to depend on the retarder curing stage, lying between the unbonded and bonded limits [33], and the flexural behavior of retard-bonded prestressed UHPC beams has been experimentally established [34]. For high-performance-concrete bridge members, the application of UHPC in bridge engineering and its developments and challenges have been comprehensively reviewed [35,36], and hybrid precast girders that place UHPC in the end zones with normal concrete elsewhere have been proposed and validated [37,38]. These works, together with existing studies on prestressed thin-walled box girders, motivate the combined use of retard-bonded prestressing and high-performance concrete pursued in this study.

1.2. Research Content and Significance

Currently, existing research primarily focuses on material substitution or structural form optimization, while systematic studies on the combined application of retard-bonded prestressing technology and high-performance concrete remain insufficient. In this study, a 30,000 mm span thin-walled retard-bonded prestressed concrete box girder was designed, and a shear test with a shear-span ratio of 2.5 was conducted. Using a validated finite element model, the failure mode, shear capacity, and lightweight design schemes of the specimen were investigated. The main research significance is as follows: (1) a finite element modeling method capable of modeling the shear resistance of retard-bonded prestressed concrete box girder has been established, and its validity has been verified through full-scale experiments; (2) the influence of bonding between prestressed steel strands and concrete, stirrup ratio, web thickness and bottom flange thickness at the end of the box girder, concrete strength and length of UHPC end zone on the shear

performance of thin-walled box girder was investigated; (3) combining the finite element simulation method with an economic analysis, a concept of multi-material composite design approach involving the use of UHPC at beam ends has been proposed.

2. Experimental Program

2.1. Specimen Design

The specimen used in this experiment was a full-scale thin-walled box girder with a standard span of 30,000 mm, an actual length of 29,920 mm, and a height of 1600 mm. Additionally, the slope of the web side is 1:4. The thickness of the web and bottom flange of both end zones (3550 mm) is 240 mm and the wall thickness of the rest girder is 160 mm. A thickness transition zone with a length of 200 mm was built between the two parts. Due to design requirements, the top flange thickness of the box girder remains constant at 200–250 mm. The experimental girder is equipped with a total of four internal partition walls along its entire length, with three structural internal partition walls (located at the supports and mid-span) and one additional partition wall for loading. The thickness of the support partition walls and mid-span partition walls is 400 mm and 300 mm, respectively. The additional partition wall for loading is placed 4000 mm away from the centerline of the support with a thickness of 200 mm.

A total of 16 retard-bonded prestressed steel strands of 1860 MPa tensile strength were embedded in the web and bottom flange of the girder. Each strand has three steel wires with a diameter of 15.2 mm. Half of these strands were anchored at the bottom flange, while the other half were anchored in the web at the girder ends. Figure 2 provides the layout and tensioning sequence of the steel strands. The prestressed steel strands are tensioned using a 2500 kN hydraulic jack (Shanghai Road & Bridge Company, Shanghai, China). Of these, 285.6 kN tension was applied to the web prestressed steel strands, and the prestress of the steel strand was 1320 MPa. While the prestressed steel strand in the bottom flange is subjected to 554.4 kN tension and the pre-stress is 680 MPa.

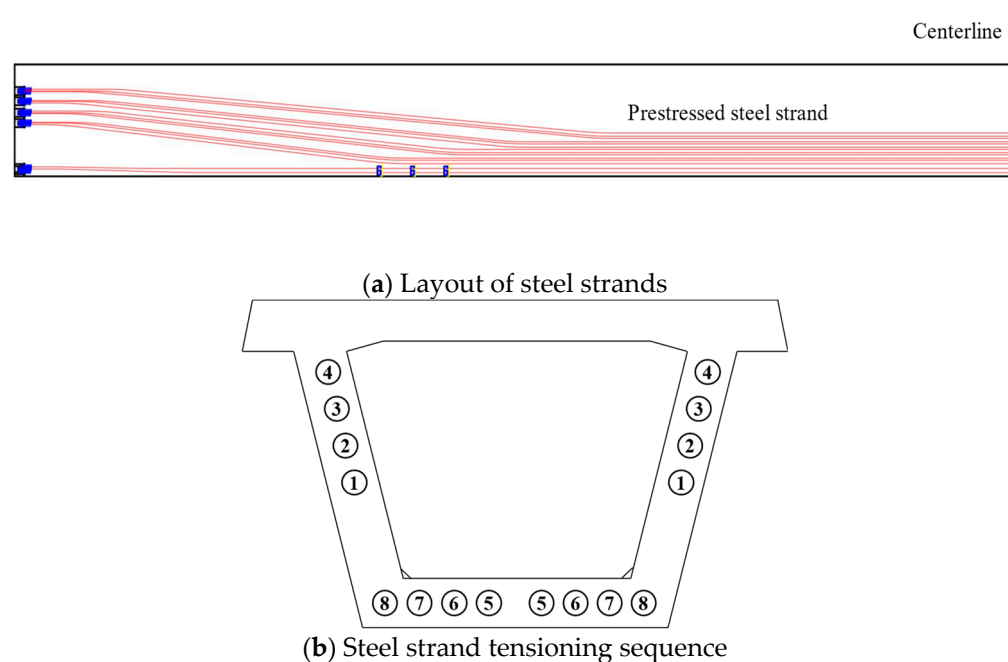


Figure 2. Details of the steel strands: the circled numbers are the tensioning order.

2.2. Material Properties

The measured compressive cylinder strength of concrete is 56.99 MPa and the elastic modulus is 37,658.9 MPa. The tensile strength of concrete adopts the suggested value of

2.85 MPa [39]. In this study, various types of steel materials were used to make the specimen, including prestressed steel strands, normal steel bars and steel plates. The material characteristics of different types of steel are presented in Table 1.

Table 1. Properties of steel materials.

Material	Yield Strength (MPa)	Elastic Modulus (GPa)	Ultimate Strength (MPa)
Steel plate	235	200	375
Prestressed steel strand	1820	195	1960
Normal steel rebars	400	201	540

2.3. Test Loading Condition

The arrangement of strain gauges and LVDTs is depicted in Figure 3. LVDTs were installed at the bottom of the loading point, supports, and a location $L/4$ from the girder end to measure the vertical displacement of the specimen. Strain gauges were placed on the upper and lower surfaces at the loading section to measure the compressive and tensile strains induced by the bending moment. Furthermore, a three-element strain rosette with angles of 0° , 45° , and 90° was affixed to the middle height of the section at support, section transition zone, and loading point. All readings from strain gauges and LVDTs were recorded and stored in a data logger for subsequent research analysis.

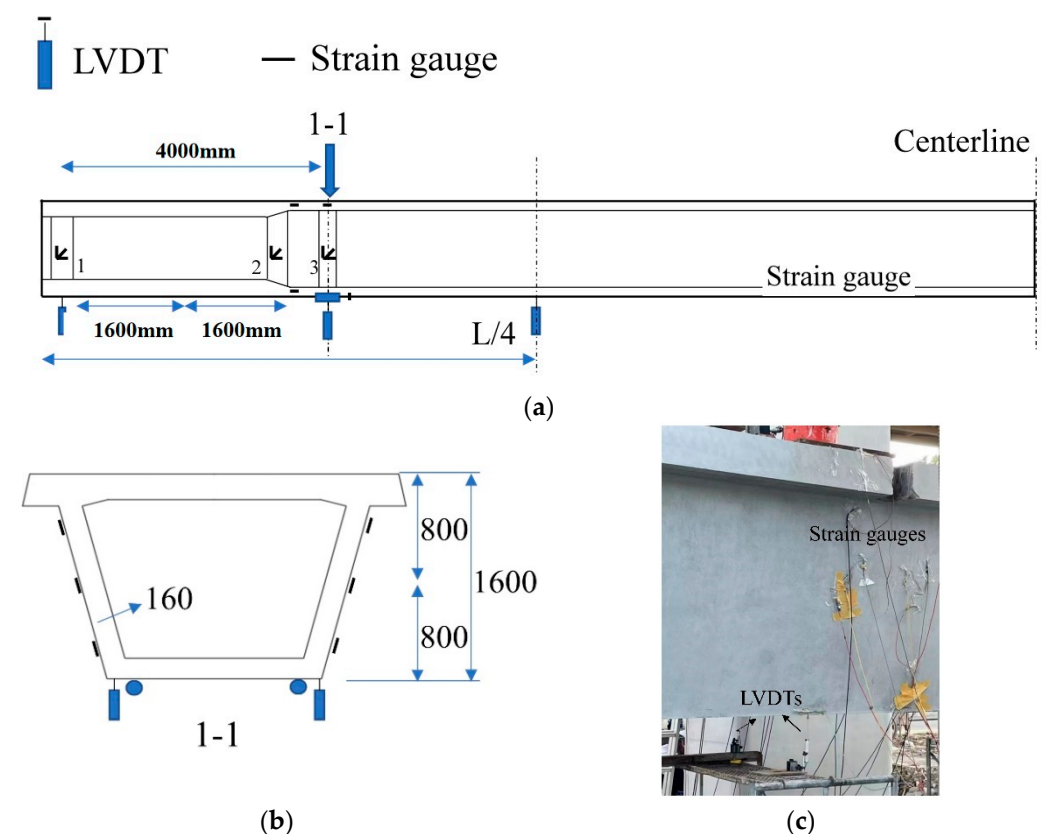


Figure 3. Instrumentation arrangement: (a) elevation showing LVDT and strain-gauge locations; (b) section 1–1; (c) photograph of the installed instrumentation.

2.4. Test Set-Up and Procedure

Figure 4 is the field photo of the full-scale thin-walled box girder specimen and the test set-up. The specimen ends are placed on two concrete supports with two rubber bearings at each side. At the upper part of the box girder, a hydraulic jack is employed to apply

vertical load. Shims are placed between the hydraulic jack and the box girder to prevent localized damage. Additionally, a steel beam supported by a concrete-filled steel tube (CFST) column is positioned above the hydraulic jack to provide reaction force and transfer the load to the foundation.



(a) Full-scale thin-walled box girder field photos



(b) Test set-up

Figure 4. Full-scale thin-walled box girder field photos and test set-up.

The shear-span ratio (the ratio of the distance between the load and the support to the girder height) was set to 2.5. Shear force was applied to the specimen at a distance of 4000 mm from the center of the support. Before the static load test, an initial load of 200 kN was applied twice to check the proper functioning of the instruments, and ensure tight contact between the test beam and the support/jack. Once the check was successful, the formal testing phase commenced. Based on engineering experience and preliminary finite element analysis, the initial loading phase employed force-controlled loading with a loading rate of 200 kN/min. When the load reached 3000 kN, displacement-controlled loading was initiated with a loading rate of 0.5 mm/min. Since there is a possibility of concrete spalling due to shear failure of the full-scale girder, considering the safety of the experimenters, the load process is stopped when the load–deflection curve reaches its peak and begins to drop. It should be emphasized that, because the specimen was a full-scale girder, its shear-compression failure could be accompanied by sudden and explosive spalling of concrete, for the safety of the testing personnel and equipment, loading was therefore terminated once the load–deflection curve had passed its peak and entered the descending branch. At this stage the load essentially no longer increased with deflection and the specimen had effectively attained its ultimate state, so the recorded peak load (3766 kN) can

be reasonably interpreted as the ultimate shear capacity of the specimen. The post-peak (descending) branch and the residual capacity were not fully captured in the test; this behavior was subsequently examined by means of the validated finite element model.

3. Finite Element Model of the Tested Girder

Numerical simulation by computational models has become a powerful tool in the investigation of structural mechanisms. In this study, to further investigate the shear performance of full-scale thin-walled box girders, a refined finite element model was established using the commercial software Abaqus 2023.

3.1. Constitutive Model of Materials

3.1.1. Concrete

Eight-node solid element (C3D8R) was adopted for concrete modeling and the constitutive model from the GB 50010-2010 [39] was employed. The expressions of the compressive stress–strain relationship are shown as follows:

$$\sigma = (1 - d_c) E_c \varepsilon \quad (1)$$

$$d_c = \begin{cases} 1 - \frac{\rho_c n}{n - 1 + x^n} & x \leq 1 \\ 1 - \frac{\rho_c}{\alpha_c (x - 1)^2 + x} & x > 1 \end{cases} \quad (2)$$

$$\rho_c = \frac{f_c}{E_c \varepsilon_{c,r}} \quad (3)$$

$$n = \frac{E_c \varepsilon_{c,r}}{E_c \varepsilon_{c,r} - f_c} \quad (4)$$

$$x = \frac{\varepsilon}{\varepsilon_{c,r}} \quad (5)$$

where σ represents stress; α_c is the parameter value for the descending segment of the concrete uniaxial compressive stress–strain curve, and for this study, it is taken as 2.37 according to the compressive strength of the concrete. $\varepsilon_{c,r}$ represents the peak compressive strain corresponding to f_c , and it is taken as 0.001892. E_c is the elastic modulus, which is 37,658.9 MPa, f_c is the compressive strength of concrete, taken as 56.99 MPa, and d_c represents the coefficient of compressive damage for concrete.

The equations for tensile stress–strain model are shown as follows:

$$\sigma = (1 - d_t) E_c \varepsilon \quad (6)$$

$$d_t = \begin{cases} 1 - \rho_t [1.2 - 0.2x^5] & x \leq 1 \\ 1 - \frac{\rho_t}{\alpha_t (x - 1)^{1.7} + x} & x > 1 \end{cases} \quad (7)$$

$$\rho_t = \frac{f_t}{E_c \varepsilon_{t,r}} \quad (8)$$

$$x = \frac{\varepsilon}{\varepsilon_{t,r}} \quad (9)$$

where α_t represents the parameter value for the descending segment of the concrete uniaxial tensile stress–strain curve, and for this study, it is taken as 2.55 based on the strength of concrete. $\varepsilon_{t,r}$ represents the peak tensile strain corresponding to f_t , and it is taken as 1.147×10^{-6} . f_t is the tensile strength of concrete, taken as 2.85 MPa, and d_t represents the tensile damage factor for concrete.

The Concrete Damaged Plastic (CDP) model provided by the Abaqus software was adopted in the FE model. K_c is the ratio of the second stress invariant on the tensile meridian to the second stress invariant on the compressive meridian. It can be calculated by using the empirical Equation (10) [40]. The ratio of biaxial compressive strength to uniaxial compressive strength of concrete is taken as the recommended value of 1.16 from the GB 50010-2010 [39]. Additionally, with a dilation angle of 30 degrees, a flow eccentricity of 0.1, and a viscosity coefficient of 0.005, the model satisfies both accuracy and convergence requirements. The above CDP parameters were determined by combining the recommendations of GB 50010-2010 with values reported in the literature, and were then confirmed against the test data. Specifically, the ratio of the biaxial to the uniaxial compressive strength was taken as the code-recommended value of 1.16, K_c was obtained from Equation (10), and the dilation angle (30°), flow eccentricity (0.1) and viscosity coefficient (0.005) were selected to balance predictive accuracy against numerical convergence for the normal-strength concrete. The close agreement between the simulated and measured load–deflection curves (a difference of only 4.94% at a deflection of 40 mm) confirms that the adopted parameter set reproduces the shear response with sufficient accuracy.

$$K_c = \frac{5.5}{5 + 2(f_c)^{0.0075}} \quad (10)$$

The coefficients of damage in the aforementioned calculations are distinct from the concept of the damage factor d in the CDP model. Here, d_c and d_t represent the cut-line damage along the stress–strain curve, while d denotes the unloading stiffness damage. Typically, the calculation in Equation (11) is employed for determining d [41].

$$d = 1 - \sqrt{\frac{\sigma}{E_c \varepsilon_c}} \quad (11)$$

3.1.2. Steel

In this study, various types of steel materials are involved, and their constitutive models all adopt the ideal elastic-plastic model:

$$\sigma = \begin{cases} E_s \varepsilon & \sigma < f_y \\ f_y & \sigma \geq f_y \end{cases} \quad (12)$$

The yield strength f_y and elastic modulus E_s are consistent with the experimental specimens, as detailed in Table 1. T3D2 beam elements are used for modeling the reinforcement, while S4R shell elements are employed for modeling the steel plates.

3.2. Modeling of Prestressed Steel Strand

Because the internal RBM of the specimen is not completely hardened during the experiment, the specimen is in a state of incomplete bonding. As mentioned earlier, when RBM is not fully hardened, the mechanical properties of RBPC are intermediate, between that of BPC and UPC. Therefore, the unbonded condition can be used for modeling to carry out conservative analysis. The bond of a retard-bonded tendon develops gradually as the retard-bonded material (RBM) cures, so the mechanical behavior of retard-bonded

prestressed concrete lies between the fully unbonded state (early age) and the fully bonded state (after complete polymerization) [33,34]. Since these two states constitute the lower and upper bounds of the actual partial-bond condition, the unbonded idealization yields a conservative (lower-bound) estimate of the shear capacity. As the RBM had not fully hardened at the time of testing, the specimen was close to the unbonded limit, which is further supported by the good agreement (within 4.94%) between the unbonded model and the measured load–deflection curve. First, geometric modeling is established based on the dimensions of the steel strands in the experimental specimens, using T3D2 beam elements. In simulations of unbonded prestressed steel strands using commercial finite element software, some researchers [42] have proposed two main methods during numerical simulations: the face-to-face contact method and node-to-node connect method. The former requires the establishment of a solid steel bar and the setting of contact between the steel bar and concrete, and the friction coefficient is usually very small. This method tends to be computationally inefficient. For the second method, some researchers often establish constraint relationships directly between the prestressed strands and concrete by implementing rigid springs or coupling constraints. However, selecting nodes between concrete and steel strands for constraint setting is cumbersome and challenging. To address this issue, Huang et al. [43] have proposed configuring virtual prestressed strands for spring connections. The specific process for modeling the unbonded prestressed steel strands is described below.

Firstly, two virtual prestressed steel strands were introduced as intermediate components to establish constraints between real prestressed strands elements and concrete elements. Then, the two virtual steel strands are directly embedded in concrete and arranged 5 mm away from the real steel strands. It should be noted that the virtual strand has the same mesh size as the real strand, which makes it possible to connect the corresponding nodes of the two strands with a rigid spring. To ensure that the introduction of virtual prestressed strands does not alter the overall mechanical behavior of the original structure, the cross-sectional area of virtual prestressed strands is set to 10^{-11} mm^2 , and the elastic modulus is set to 1 Pa. In this study, the simplified method was employed, assuming that the prestressed strands only experience slippage in the tangential direction and not in the normal direction. Figure 5 shows the modeling process of unbonded prestressed steel strand. After the interaction between the steel strand and concrete is established, a cooling process is applied to induce prestress. The bottom steel strand is prestressed to 680 MPa, while the web steel strands are prestressed to 1320 MPa.

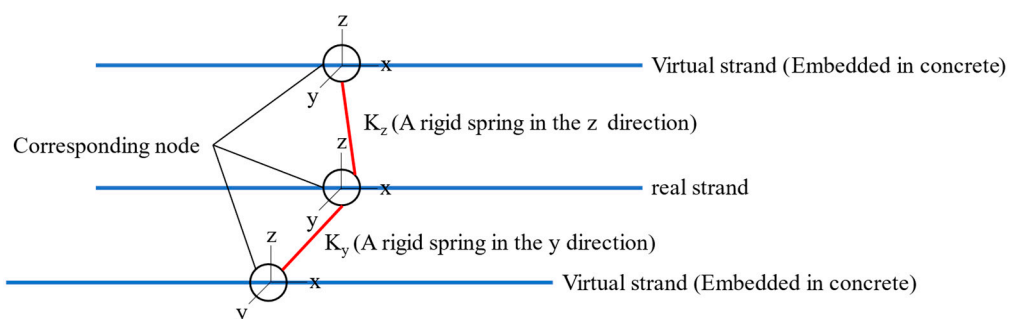


Figure 5. Modeling process of unbonded prestressed steel strand.

3.3. Boundary Condition and Load

The boundary condition of the model mimics that of a simply supported beam, mirroring the experimental specimen. Reference points are placed beneath the model at both ends, coupling the ends of the box girder to these reference points. The reference point at one end is fixed and the other end is hinged to simulate the real boundary conditions. A loading point is set at a distance of 4000 mm from one end of the girder, coupled with surface below, controlling the imposed displacement with a limit of 80 mm. Figure 6 shows the finite element model of the tested box girder. To determine an appropriate element size, a mesh-sensitivity analysis was carried out in which three concrete mesh sizes (100 mm, 150 mm and 200 mm) were compared against the experimental result, as shown in Figure 7. When the mesh is too coarse (200 mm), the predicted shear capacity is noticeably underestimated and the error is relatively large, whereas the 100 mm and 150 mm meshes yield very-close results and all three models exhibit the same failure mode. Considering both accuracy and computational cost, a concrete mesh size of 100 mm and a steel-strand mesh size of 150 mm were adopted, which provide a converged and numerically stable solution.

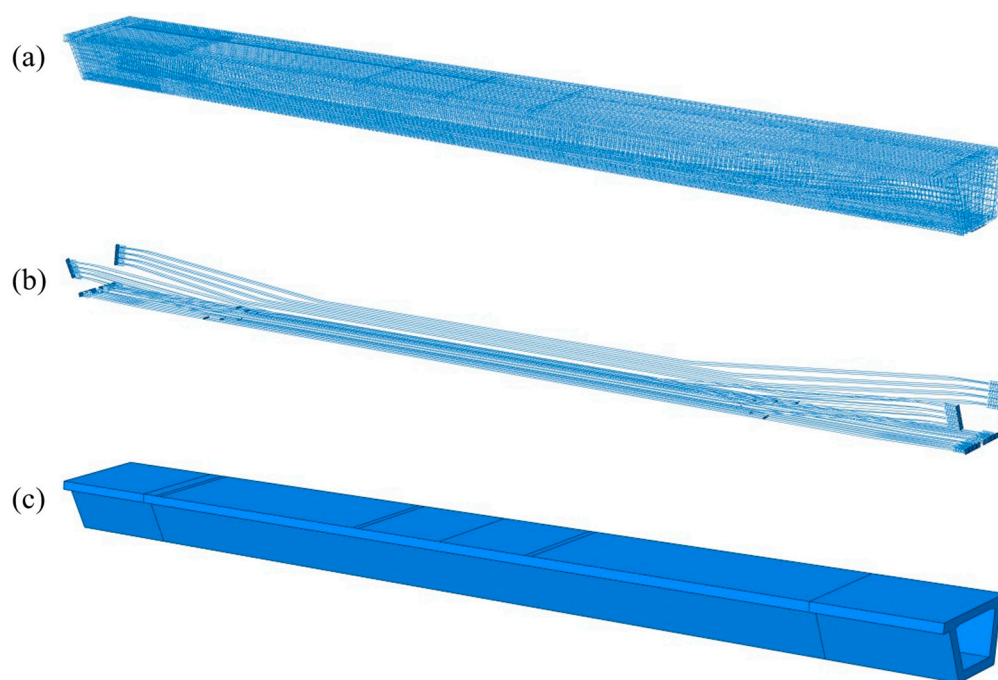


Figure 6. FE model of tested box girder: (a) normal steel bars; (b) prestressed strands; (c) concrete.

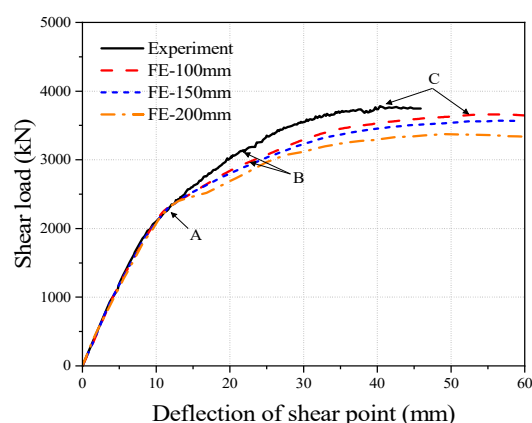


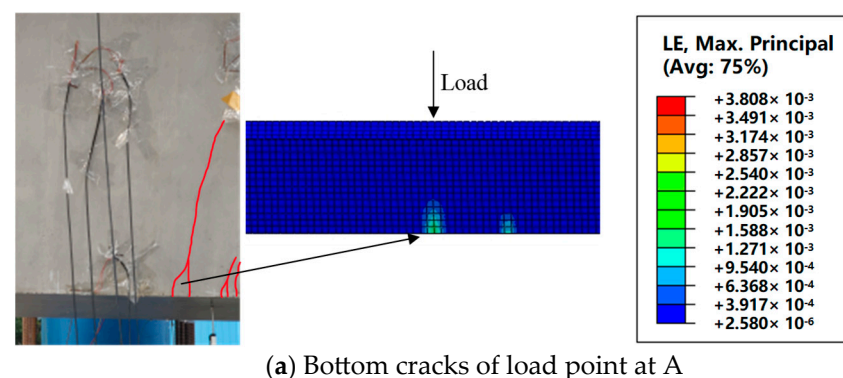
Figure 7. Shear load–deflection curves at the loading point for the experiment and the FE models with 100 mm, 150 mm and 200 mm mesh sizes: A = first cracking at the loading point; B = cracking of the transition section; C = near-ultimate (peak) state.

4. Test Results and FEM Validation

4.1. Load-Deflection Relationship and Crack Patterns

The load–deflection curves and crack patterns of the specimen are shown in Figures 7 and 8. The experimental response and the FE (100 mm) model exhibit a similar shape, with only a 4.94% difference in load at a deflection of 40 mm, which verifies the accuracy of the finite element model. In the initial loading stage, the applied load on the specimen is relatively small, the relative slip of the retard-bonded steel strands is minimal, and all parts of the structure work together, with no cracks appearing on the surface of the beam. During the crack-free stage, the stress characteristics of the girder are similar to that of an elastic body. The deflection of the specimen increased with the increasing load, and the shear load–deflection curve is roughly linear. When the load continues to increase to 2363 kN, cracking first occurred at the bottom of the loading point, with a deflection of 12.5 mm, and the specimen enters the crack-bearing phase. In this stage, some concrete gradually ceases to function, and the sectional stiffness of the component significantly decreases. The deflection growth rate accelerates, and although the slope of the curve after cracking is much smaller than before, it generally remains in a linear state. As the load further increases, the shear deflection at the loading point continues to increase. At point B, multiple cracks also appear at the bottom of the transition section, and the slope of the curve drops further. Subsequently, the load slowly increases with the increase in deflection, and part of the concrete below the loading point fails, leading to a significant overall stiffness reduction and increased overall deformation of the component. Furthermore, cracks appear subsequently above the transition section and propagate continuously towards the support. The width of the cracks also rapidly increases, forming a vicious cycle with structural deformation. Until point C, the load value basically no longer increases with the displacement, and the specimen is close to the ultimate state. The final peak load for the test is 3766 kN, with a deflection at the bottom of the loading point of 40.3 mm. Overall, the failure mode of this specimen is classified as shear-compression failure.

Figure 8 compares the crack patterns of the specimen and the FE model at three different deflection moments (A, B, C), revealing good consistency between the two. This indicates that the finite element model developed in this study can effectively predict the shear performance of the full-scale thin-walled box girder. Therefore, the proposed finite element model can be further utilized to simulate the shear behavior of thin-walled prestressed box girder with different parameters.



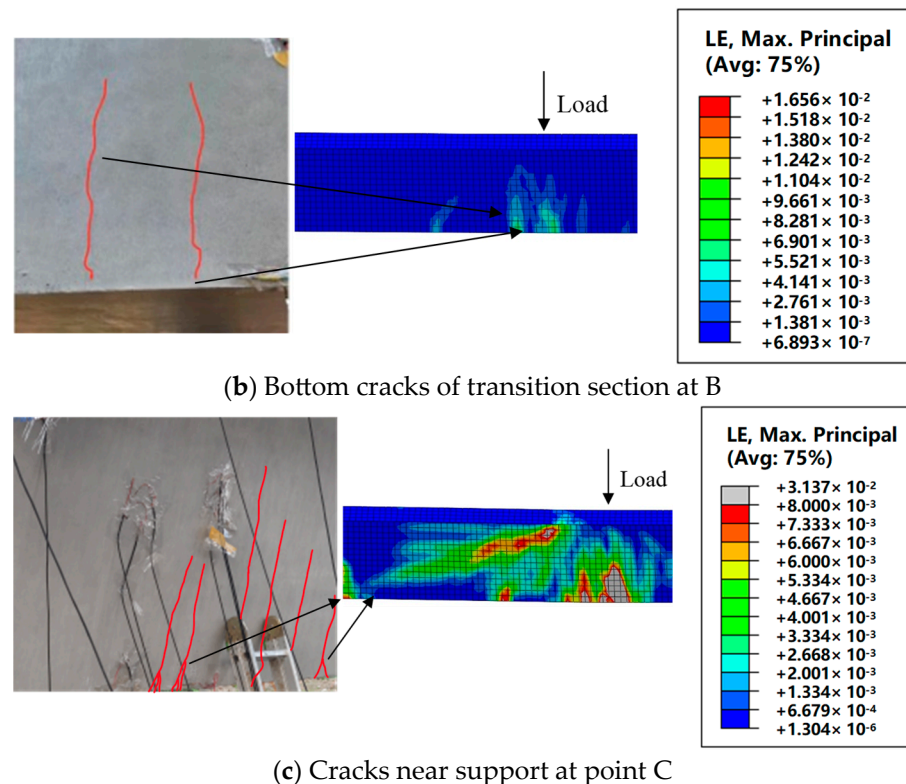


Figure 8. Crack patterns of specimen and FE model.

4.2. Load and Principal Tensile Strain

According to the measured strain from the three-element strain rosettes, the magnitude of the principal tensile strains can be calculated based on material mechanics as follows:

$$\varepsilon_{\max} = \frac{\varepsilon_x + \varepsilon_y}{2} + \sqrt{\left(\frac{\varepsilon_x - \varepsilon_y}{2}\right)^2 + \left(\frac{\varepsilon_x + \varepsilon_y - 2\varepsilon_{45^\circ}}{2}\right)^2} \quad (13)$$

where ε_x , ε_y and ε_{45° represent the readings of the horizontal strain gauge, vertical strain gauge, and the 45° inclined strain gauge, respectively. $\varepsilon_{\max}$ is the principal tensile strain.

During the loading process, the measured and simulated variations in the principal tensile strains at each measurement point are shown in Figure 9. Among these, strain gauge No.1 is above the support, strain gauge No.3 is closest to the loading point, and strain gauge No.2 is at the variable-section location. It can be found that although the values of strain at the three locations are different, the general law is basically similar. In the initial loading stage, the structure is in the elastic phase, and shear force is primarily borne by the concrete and stirrups. The relationship between the principal tensile strain and the load is approximately linear. When the load reaches 70% of the ultimate bearing capacity, strain gauge No.3, closest to the loading point, first records the cracking phenomenon. This is basically consistent with the observation in Figure 7. Subsequently, a similar situation occurs at strain gauge No.2. However, since the specimen was not loaded to complete failure, No. 1 strain gauge, at mid-height above the support, did not record obvious cracking. The finite element predictions are also plotted in Figure 9. For each measurement point, the simulated shear load–principal tensile strain curve (solid line) follows the corresponding measured curve (dashed line) closely, capturing both the initial elastic slope and the faster strain growth after cracking. The results also reproduce the observed ordering among the three locations: gauge No. 3, near the loading point, develops the largest principal tensile strains and cracks earliest; gauge No. 2, at the variable-section location, develops intermediate strains; and gauge No. 1, above the support, develops the smallest

strains and shows no obvious cracking. Apart from minor local fluctuations in the measured strain at the onset of cracking, which the smooth numerical response does not capture, this agreement in the local strain development—together with the crack-pattern comparison in Figure 8 and the load–deflection comparison in Figure 7—further confirms the reliability of the finite element model.

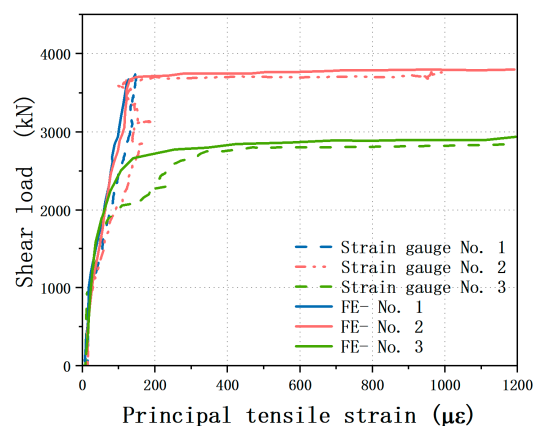


Figure 9. Shear load–principal tensile strain relationships at the three strain-rosette locations: comparison between the measured and simulated responses. (solid = FE, dashed = measured; colors = gauges No. 1/2/3).

5. Numerical Analysis

The preceding sections verified the accuracy of the model by comparing the failure modes and load–deflection curves of the FE model and experimental specimens. This section conducts relevant parameter analyses on the models to expand existing research data and obtain insights into the influence of various factors on the failure mode and shear strength of thin-walled box girders.

For RC beam components, the inherent discreteness of concrete materials makes the analysis of shear force transmission mechanisms in beams complex. Previous researchers have extensively studied factors such as reinforcement ratio, shear-span ratio, and span-to-depth ratio of beams. This study primarily investigates the impact of bonding between prestressed steel strands and concrete, stirrup ratio, wall thickness at the end zone of the box girder, concrete strength and length of UHPC end zone on the shear performance of components, supplementing existing research.

5.1. Bonding Between Prestressed Steel Strands and Concrete

In the case of retard-bonded prestressing tendons, the bonding between the strands and concrete transitions from unbonded to fully bonded over time. To compare the influence of bonding conditions on the load-carrying capacity of the box girder, steel strands were directly embedded into the entire model, resulting in a new model of bonded prestressed concrete box girder (BPCG). Figure 10 compares the load–deflection curves of the BPCG and the unbonded prestressed concrete box girder (UPCG) model established in the third section. It can be observed that, in the initial loading stage, their initial stiffness is similar, and cracking occurs at a load of around 2300 kN. The stiffness degradation of UPCG is significantly greater than that of BPCG thereafter. This is because the bonding between the concrete and the steel strands in BPCG enables effective collaboration between the materials, facilitating the redistribution of the tensile stress in concrete after cracking. This, in turn, results in a moderate development of cracks as well as a higher load-carrying capacity than that of UPCG.

Figure 11 provides stress contour plots for the steel strands in UPCG and BPCG at the peak load of the model. It is evident that the stress in the steel strands of BPCG is significantly higher than that in UPCG, and its maximum occurring at the shear location. This indicates that inclined steel strands of BPCG can work well with concrete and bear more shear load when cracks appear in the loading place. In contrast, the stress distribution in the web of UPCG is more uniform. This is because the bonded prestressed steel strands experience frictional resistance from the concrete during deformation, while the steel strands in UPCG are in an unbonded state, allowing them to slide in the tangential direction and thus distribute local stresses more uniformly across the entire structure. The friction between the steel strands and concrete in BPCG also results in larger crack zones at the ultimate state compared to UPCG. The failure modes of two models are shown in Figure 12. Overall, the bearing capacity of BPCG was 7.4% higher than that of UPCG. This 7.4% difference therefore quantifies the upper bound of the error introduced by adopting the unbonded assumption for the retard-bonded specimen; combined with the close agreement between the unbonded model and the experiment, it confirms that the simplification has only a limited influence on the predicted shear capacity.

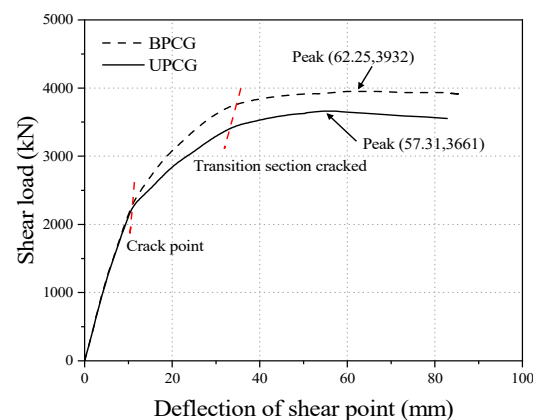


Figure 10. Shear load–deflection curve of UPCG and BPCG.

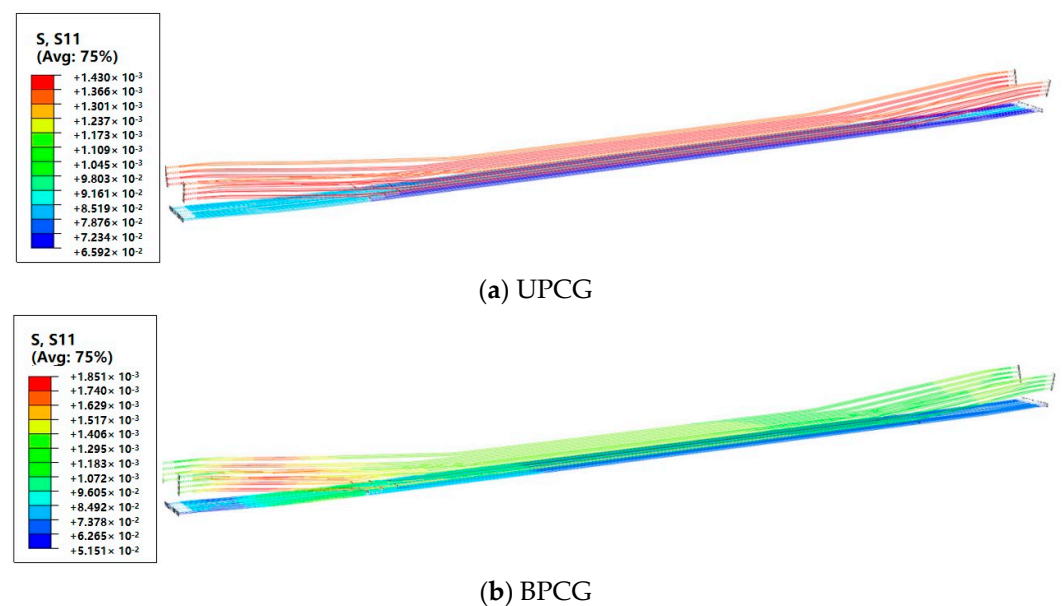


Figure 11. Stress contour plots of steel strands in UPCG and BPCG.

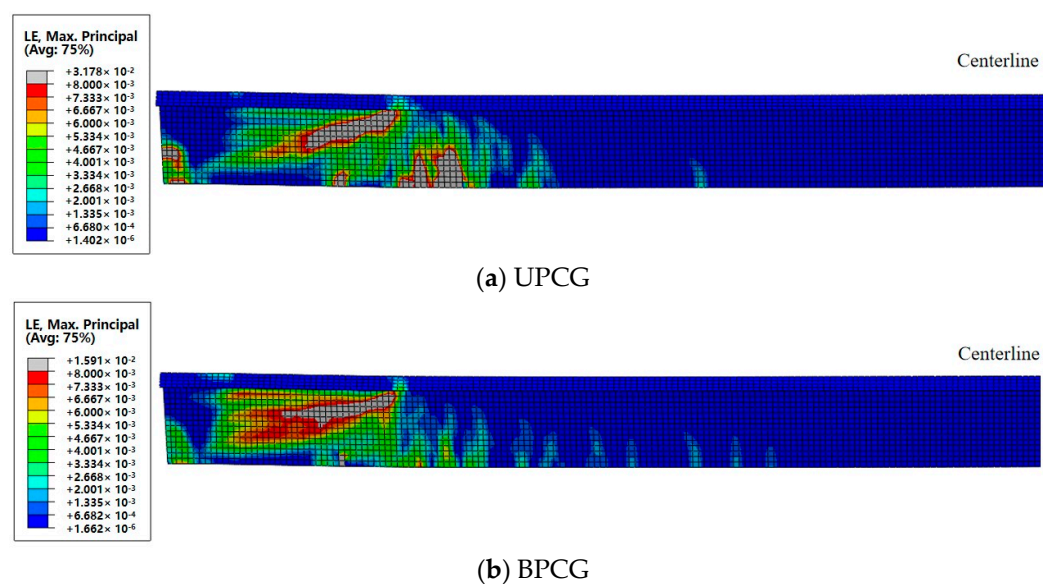


Figure 12. Failure mode of two models.

5.2. Stirrup Ratio

Due to the low stirrup ratio in many UHPC box girder bridges in Malaysia [44], the stirrup ratio in the parametric analysis is changed from 0.88% to 0.44%, 0.66% and 1.32%, in order to explore the influence of stirrup ratio on the shear performance of prestressed thin-walled box girder. Figure 13 shows the load–deflection curves of the box girder with different stirrup ratios. The data show that when the stirrup ratio decreases to 0.44%, the bearing capacity of the box girder decreases by 19.28%, and when the stirrup ratio increases to 1.32%, the shear bearing capacity increases by 7.29%, which shows that when the stirrup ratio is too high, the improvement of the bearing capacity is limited, because the failure of the specimen is mainly determined by the concrete. When the stirrup ratio is 0.44%, the ultimate load of the box girder is 1.65 times of the design live load. This indicates that a certain ratio of stirrups is needed to ensure that the ultimate bearing capacity is greater than twice the design live load.

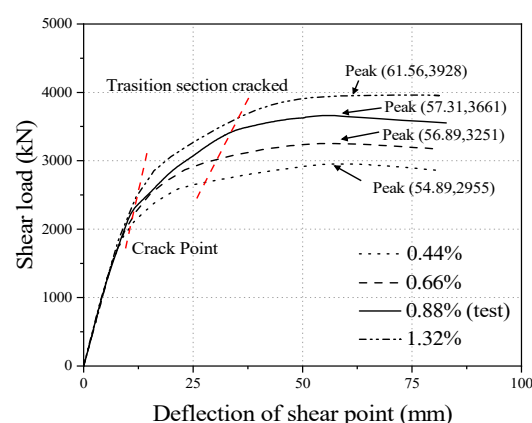


Figure 13. Shear load–deflection curve of models with different stirrup ratios.

5.3. Web Thickness and Flange Thickness at the End Zone of the Box Girder

The specimen in this experiment is variable cross-section box girder with larger wall thickness in the cross-section at the end zone. Considering that the shear load of the specimen is primarily determined by the end structures, this section explores the influence of the thickness of the web and bottom flange of the box girder ends (within a distance of

3550 mm from the supports) on shear performance of the girder. The specific division of the specimen can be seen in Figure 14.

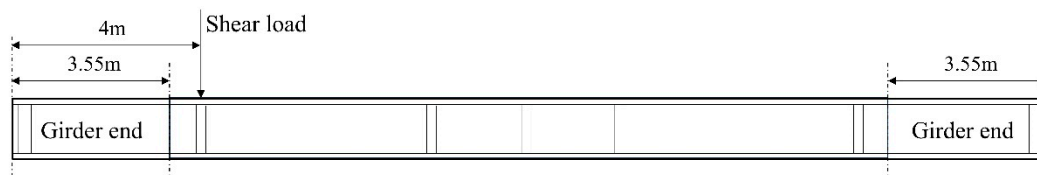


Figure 14. Schematic diagram of specimen division.

The original FE model had a wall (i.e., web and bottom flange) thickness of 240 mm at both ends. In this section, it is modified to 200 mm and 160 mm to conduct comparative analysis. In this section, different models are named based on the wall thickness at the end zone of box girder. The shear load–deflection curves for the various models can be seen in Figure 15. It can be observed that the three curves have almost identical slopes in the early stages of loading, indicating similar initial stiffness for each model. This is because the loading positions of the three models are all located in the middle zone of the span with the same wall thickness of 160 mm. As the load increases, a greater wall thickness of the box girder results in a delayed cracking of the transition zone (i.e., the variable cross-section segment of the original specimen) and a larger failure deflection. When the wall thickness at the end zone of the box girder is reduced from 240 mm to 200 mm (160 mm), the ultimate load decreases by 8.61% (19.12%), and the corresponding deflection decreases by 27.12% (41.49%). This indicates that the web thickness and bottom flange thickness at the end zone of the box girder has a significant impact on both the shear capacity and ductility of the box girder. Similarly, Su et al. [32] explored the effect of web thickness on the mechanical properties of 30,000 mm long UHPC thin-walled box girders previously, and found that when the web was reduced from 80 mm to 40 mm, the tensile stress in the web increased faster than the tensile stress in the flange, indicating that the web thickness cannot be too small. Hence, it is difficult to reduce the weight of bridge members by simply reducing the web thickness.

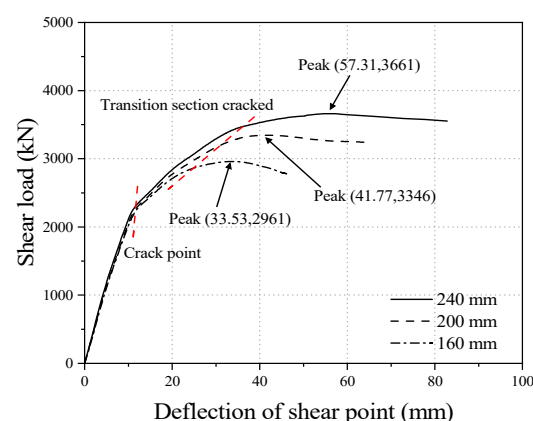


Figure 15. Shear load–deflection curves of models with different end wall thickness.

5.4. Concrete Strength

In order to ensure the bearing capacity of the box girder, reduce the thickness of the webs of the members, and realize the lightweight and uniform section design of the members, the two end zones of the 160 mm model in 5.3 are replaced by three different concrete materials with varying strength levels. Each specimen is named 160-X, where X represents the concrete grade. It is important to note that the model assumes good bonding between

different strength concrete materials. Table 2 provides the properties of different concrete materials. According to the previous study [45], the constitutive model for UHPC can be calculated using Equations (14)–(17). Additionally, CDP model is used for UHPC modeling, where $K = 0.66$ and $f_{b0} / f_{c0} = 1.1$, and the dilation angle and eccentricity are 56° and 0.1, respectively [46].

Table 2. Properties of concrete.

Number	f_{co} (MPa)	f_t (MPa)	E_c (MPa)	Poisson's Ratio
C70	70	3.5	36,000	0.2
C90	90	4.0	38,000	0.2
UHPC	155	10.43	44,400	0.18

The axial tension constitutive model of UHPC:

$$\sigma = \begin{cases} \frac{f_t}{\varepsilon_{ct}} \varepsilon & 0 < \varepsilon < \varepsilon_{ct} \\ f_t & \varepsilon_{ct} < \varepsilon < \varepsilon_{pc} \\ f_t \frac{1}{[1 + 33.3(\varepsilon - \varepsilon_{pc})]^{0.98}} & \varepsilon > \varepsilon_{pc} \end{cases} \quad (14)$$

The axial compression constitutive model of UHPC:

$$\sigma = \begin{cases} f_{co}[Ax + (5 - 4A)x^4 + (3A - 4)x^5] & 0 \leq x \leq 1 \\ \frac{x f_{co}}{8(x - 1)^2 + x} & x > 1 \end{cases} \quad (15)$$

$$A = \frac{E_c \varepsilon_{co}}{f_{co}} \quad (16)$$

$$x = \frac{\varepsilon}{\varepsilon_{co}} \quad (17)$$

where ε_{ct} = elastic tensile strain, ε_{pc} = limited tensile strain, f_t = tensile strength, ε_{co} = strain at peak compressive stress, and f_{co} = cylinder compressive strength.

The shear load–deflection curve of the models with different concrete replacement schemes are shown in Figure 16. It is worthwhile to note that after displacement control loading, the shear load–deformation curve of 160-UHPC does not show a significant decrease, but considering that the deflection of the specimen should not be exceed a certain limit and the rate of increase is minimal, it can be concluded that the box girder has reached its ultimate state. Compared to the 160 mm model, the ultimate bearing capacities of the box girders fabricated with C70, C90 and UHPC are respectively increased by 9.5%, 14.4% and 46.9%. The corresponding deflections are increased by 14.6%, 25.2% and 146.4%. It indicates that the strength of the end zone concrete has an obvious effect on the bearing capacity and deformation capacity of the box girder. For 160-C70 and 160-C90, the initial stiffness, load-bearing capacity and deformation capacity of the latter are only slightly higher than those of the former because the tensile strength of C70 and C90 is close. As the difference in concrete strength between the end zone and mid-span is small, the failure modes for 160-C70 and 160-C90 are controlled by main inclined cracks at the ends, as shown in Figure 17a,b. According to Figure 17c, the failure mode of 160-UHPC is

mainly local failure in the transition section and the loading region, this is because the transition section and the loading region are weaker than the end zone. Therefore, in the actual engineering, it needs to be strengthened in the transition area, so as to avoid local damage.

Combining the above analysis, it can be observed that, with the appropriate stirrup ratio, the shear failure of the box girder is significantly influenced by the end zone thickness and material strength. Figure 16 compares the results of previously established models with variable cross-section (i.e., 240 mm end zone wall thickness) and models with uniform cross-section with high-strength material at the end zone (i.e., 160-C90 and 160-UHPC). It is evident that by replacing the end zone with UHPC, the shear capacity and ductility are higher compared to the 240 mm model, indicating that using UHPC material at both ends has three advantages: (1) improving the shear resistance and ductility of the specimen; (2) facilitating the transition from a variable cross-section design to a uniform cross-section design, providing convenience for construction and standardization for pre-fabricated segments; and (3) reducing the end wall thickness, achieving structural lightweighting.

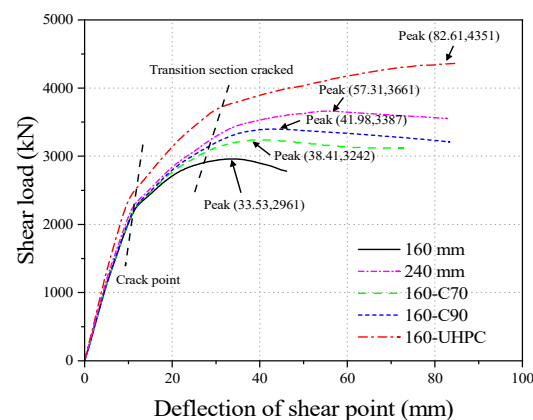
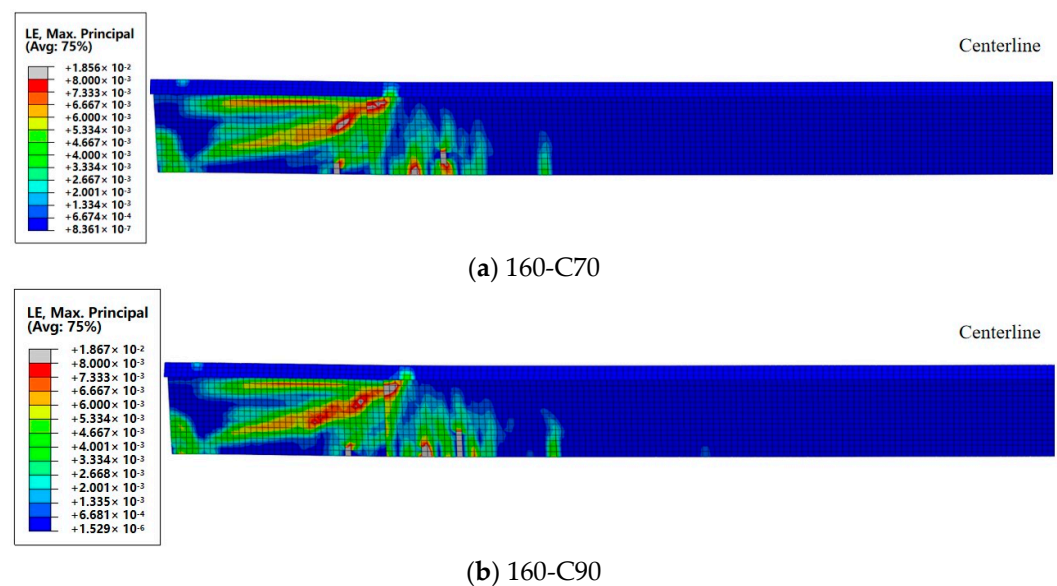


Figure 16. Shear load–deflection curve of the models with different concrete replacement schemes.



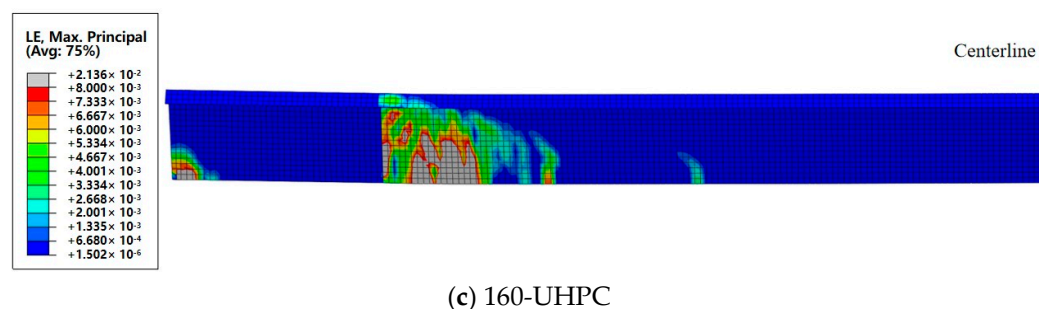


Figure 17. Failure mode of the models with different concrete replacement schemes.

5.5. Length of UHPC End Zone

Based on the above studies, to explore the influence of the UHPC end zone length on the shear capacity of the box girder, eight different FE models are established in this section. The geometry of these models is the same as that of the 160 mm model in Section 5.3, while UHPC of different lengths are applied at both ends. The shear load–deflection curves for each model are shown in Figure 18a. It can be found that the initial shear stiffness of the box girder will be improved when UHPC is used to strengthen the two ends of the box girder. However, with the increase in the length of the UHPC end zone, the initial shear stiffness of the box girder is almost unchanged, which indicates that the former has little correlation with the latter. In order to comprehensively consider economic efficiency and mechanical properties, this paper introduces a cost–performance ratio I :

$$I = \frac{F}{P} \quad (18)$$

where F and P present peak shear load and the total cost of the box girder material, respectively. Prices for various materials can be seen in Table 3.

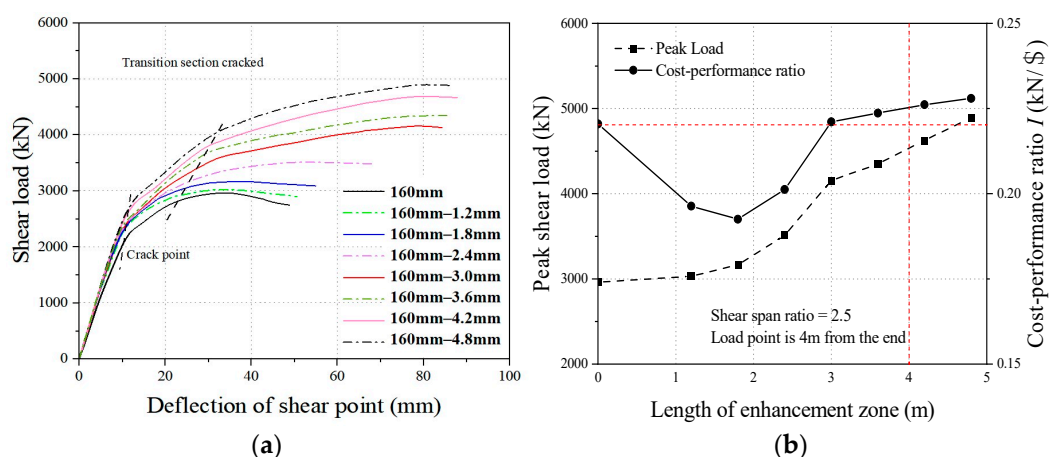


Figure 18. Effect of length of reinforcement zone on the shear capacity of box girder. (a) Shear load–deflection curve; (b) economic efficiency and mechanical properties.

Table 3. Prices for various materials.

Material	Unit Price (US\$)
Steel plates	550
Normal steel rebar	580
Retard-bonded prestressed steel strand	1600
NC ($f_{co} = 56.99$ MPa)	120
UHPC ($f_{co} = 155$ MPa)	780

According to Figure 18b, the longer the length of the UHPC end zone is, the higher the shear bearing capacity of the box girder is. While the length of the enhanced region is less than 2000 mm, the effect of UHPC end zone on the shear capacity of the box girder is not remarkable, and the cost–performance ratio decreases with the increase in the length of UHPC end zone. This is because the box girder is mainly damaged at the loading area. When the length of the UHPC end zone is between 2000 and 3000 mm, the shear bearing capacity of the box girder and cost–performance ratio increases rapidly with the increase in the UHPC length, which is due to the development of diagonal cracks at both end zones being inhibited and changes in the failure mode of the box girder. When the UHPC length exceeds 3000 mm, the improvement of the shear capacity of the box girder becomes weak with the increase in the length of the UHPC end zone. It should be noted that the economic analysis only considers the material cost, excluding the constructability challenges and associated costs of UHPC. In practice, the cost–performance ratio I defined here reflects only the material cost and should therefore be regarded as a first-order, relative comparison; the true economy of a UHPC end zone is additionally governed by construction complexity, curing requirements, casting sequence, quality control, preparation of the UHPC–NC interface, and labor cost, as well as the potential long-term durability and maintenance benefits of UHPC. Commercial UHPC is generally about 10–20 times the unit cost of standard high-performance concrete, and a reliable UHPC–NC interface requires dedicated surface roughening and a controlled casting sequence [37]; the higher unit cost and fabrication demand of UHPC are the primary reasons for confining it to the shear-critical end zones [35,36]. A comprehensive life-cycle cost assessment incorporating these factors is beyond the material-level scope of the present study and is recommended as future work. Taking into account the technical complexities of large-volume UHPC construction and the results derived from the cost–performance ratio’s marginal benefits, it is recommended to use UHPC for 3000–4000 mm at both ends of the box girder, while employing ordinary concrete for the remaining sections. From a design and construction perspective, adopting a 3000–4000 mm UHPC end zone enables the original variable cross-section (with 240 mm end walls) to be replaced by a uniform 160 mm section, which simplifies formwork, standardizes the precast segments and reduces the self-weight of the girder, while confining the higher UHPC cost to the shear-critical end regions; a comparable end zone UHPC strategy has been reported for hybrid precast girders [37,38]. It should nevertheless be emphasized that this recommendation is derived mainly from the finite element analysis and is supported by limited experimental evidence; further experimental verification, including the influence of the UHPC–NC interface, is therefore required before design application, as noted in the Conclusion section.

6. Conclusion

This study focuses on the lightweight design of superstructures in bridges. A 30,000 mm long full-scale thin-walled retard-bonded prestressed concrete box girder was fabricated, and experimental and numerical investigations were conducted on its shear performance. The specific conclusions are as follows:

- (1) The failure mode of the specimen with a shear-span ratio of 2.5 was classic shear-compression failure. The measured first-cracking load and peak load of the specimen were 2363 kN and 3766 kN, respectively. This conclusion is drawn directly from the full-scale experiment, whereas conclusions (2)–(5) below are obtained from the validated finite element model and therefore constitute numerical predictions that still await further experimental verification.
- (2) A finite element model was established and validated by the experimental data. Through the validated numerical modeling method, the shear performance

differences between BPC box girder and UPC box girder were analyzed. It was found that the BPCG and UPCG have similar initial stiffness. With the increase in load, due to the fact that the steel strands of BPCG can effectively collaborate with the concrete, the shear capacity of BPC is 7.4% higher than that of UPC.

- (3) The FE results show that in order to ensure the shear bearing capacity of the box girder, a certain stirrup ratio is needed. However, the high stirrup ratio has little contribution to the improvement of the bearing capacity of the box girder. In addition, the wall thickness has a significant impact on the bearing capacity of the box girder. When the end zone wall thickness of the box girder is reduced from 240 mm to 200 mm (160 mm), the ultimate load decreases by 8.61% (19.12%), and the corresponding deflection decreases by 27.12% (41.49%).
- (4) Numerical investigations were conducted on effect of concrete strength and concrete layout schemes on the shear performance of the box girder. When the wall thickness is 160 mm, the ultimate bearing capacity of the box girders fabricated with C70, C90 and UHPC are 9.5%, 14.4% and 46.9% higher than that of original model. The corresponding deflection was increased by 14.6%, 25.2% and 146.4%. It indicates that the strength of the end zone concrete has an obvious effect on the bearing capacity and deformation capacity of the box girder.
- (5) Comparing the 160-UHPC model (i.e., the end zone of the box girder is made of UHPC with a wall thickness of 160 mm,) with the 240 mm model (i.e., 240 mm end zone wall thickness), it is found that the shear capacity and corresponding deflection of the former are 18.8% and 44.15% higher than that of the latter. This result revealed that using UHPC at both end zones of the box girder can reduce the end zone web thickness, and realize equal-section design and lightweighting. At the same time, the influence of the length of the UHPC end zone on the shear capacity of the box girder is explored. Considering the economic efficiency and mechanical properties, it is suggested that the length of the UHPC end zone should be 3000–4000 mm.

It should be noted that the conclusions of this paper mainly come from the numerical simulation of the verified finite element model, and there is a lack of more experimental data. In the future, a part of the reduced-scale test can be considered to explore whether the bond between UHPC and NC has an impact on the bearing capacity of the component. Moreover, when using UHPC to replace the original variable-section box girder side concrete, the influence of the length of UHPC installation on the shear performance of the specimen and the optimal retrofitting scheme need further investigation.

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