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THE INFLUENCE OF END EFFECTS ON THE FLOW RESISTANCE CHARACTERISTICS OF VARIABLE DIAMETER CYLINDERS

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ABSTRACT

Based on the study of the flow around finite-length cylinders, an analysis of the flow resistance characteristics of commonly encountered variable diameter cylinders at high Reynolds numbers was conducted. A comprehensive relationship equation for the flow resistance within the subcritical region, the drag crisis region, and the drag recovery region was obtained. Additionally, the influence of end effects on the flow resistance characteristics of finite-length cylinders and variable diameter cylinders was analyzed using large eddy simulation (LES), and a comprehensive formula describing the range of end effect influence was obtained. The results indicate that the flow resistance coefficients at both ends of the variable diameter cylinder are influenced by the aspect ratio λ , Reynolds number (Re), and included angle θ . As θ and λ increase, the distance affected at the top of the cylinder increases. As Re decreases, the affected distances at the top and bottom increase. The comprehensive formula for the influence range can be used to modify the relationship equation for the drag coefficient. These findings have important implications for the study and application of the flow characteristics of finite-length variable diameter cylinders.

KEYWORDS

Variable-diameter Cylinders, Aspect Ratio, Flow Field, High Reynolds Number, End-Face Effect

INTRODUCTION

The flow dynamics around cylindrical bodies have long been a subject of intensive research due to their relevance in engineering applications, including offshore structures, heat exchangers, and fluid transport systems. While traditional studies often focused on uniform-diameter, infinitely long cylinders, real-world applications frequently involve finite-length, variable-diameter geometries, such as tapered

columns and truncated cones. These configurations introduce additional complexities, particularly related to end effects and axial geometry variation. Recent studies have highlighted how structural modifications—such as end shaping [1] or proximity to walls [2]—can drastically alter flow resistance and vortex shedding characteristics. The influence of geometry is further evident in the behavior of dual-step and tandem cylinders. For example, Morton and Yarusevyc [3] revealed distinct wake modes due to diameter discontinuities, while Wang et al. [4] and Gu et al. [5] showed how diameter ratio and spacing affect flow-induced vibrations and shielding effects. Multiphase flows, as investigated by Khan et al [6], also demonstrate that diameter variation leads to asymmetric hydrodynamic loads due to buoyancy and wake interactions. Despite these insights, the combined effects of variable diameter, finite length, and axial tapering have received comparatively little attention. In particular, the quantification of end-face effects on drag coefficient distribution and their integration into predictive models remains limited. This study aims to bridge this gap by employing high-fidelity Large Eddy Simulation (LES) to analyze the flow resistance of finite-length variable diameter cylinders across various Reynolds number regimes. We propose a segmented drag coefficient model and a correction formula to account for end effects, offering enhanced predictive capabilities for practical engineering applications.

The flow around cylinders and variable-diameter cylinders is a prevalent phenomenon in engineering, characterized by fluctuating separation points and the formation of trailing and shedding vortices, especially under high Reynolds number conditions, resulting in highly complex flow characteristics [7]-[13]. While there are certain similarities between the flow around cylinders and variable-diameter cylinders, significant differences also exist. Scholars both domestically and internationally have conducted numerous experiments and numerical simulations to study the distribution of flow resistance for finite-length cylinders with various aspect ratios. For instance, reference [14], through model experiments and comparative analyses with references [14] ($5.2 \times 10^4 \leq Re \leq 9.4 \times 10^4$), references [15] ($1.25 \times 10^5 \leq Re \leq 2.09 \times 10^5$), and reference [16]-[18] ($Re \geq 1.2 \times 10^5$), provided the variation patterns of the flow resistance coefficients of cylinders with different aspect ratios as a function of Reynolds number. Kravchenko and Moin employed large eddy simulation methods to analyze key issues in flow dynamics [19]. However, research on the flow characteristics of finite-length variable-diameter cylinders (such as truncated cones and frustums) is limited, lacking analysis of resistance fluctuation characteristics, key influencing factors of flow characteristics, and comprehensive relationship equations. Particularly, there is a lack of analysis regarding end-face effects, with insufficient exploration of the various influencing factors of end effects. Therefore, it is necessary to conduct relevant studies to establish a foundation for engineering applications. If the angle between the side characteristic line and the axis of the variable-diameter cylinder (θ) is small or the height of the cylinder is large, relevant theories and research findings on cylindrical flow can be utilized. Through appropriate methods to analyze and correct errors caused by end-face effects, the resistance characteristics of variable-diameter cylinder flow can be obtained.

Variable-diameter cylindrical structures are commonly found in practical engineering scenarios, including the arms of high-speed train pantographs and utility poles. These components experience cross-flow wind loading under operating

conditions and exhibit significant end-face effects due to their finite length and geometry. Figure 1 presents two real-world examples of such structures, further motivating the need to understand their drag characteristics in varying Reynolds regimes.



(a) overhead railway pantograph arm with tapered geometry



(b) tapered utility pole subjected to cross-flow wind

Fig. 1 - Examples of variable-diameter cylindrical structures in engineering application

THE DRAG COEFFICIENT OF FINITE-LENGTH CYLINDERS IN FLOW

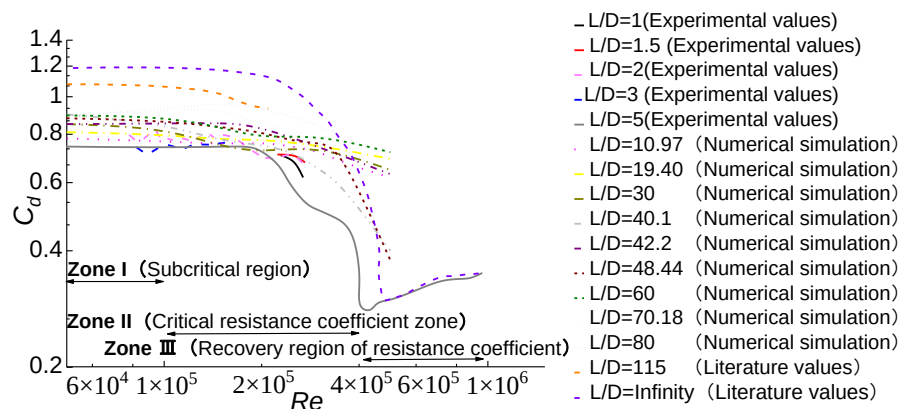


Fig. 2 - Variation of the drag coefficient of cylindrical flow with aspect ratio and Reynolds number. (Data partially compiled from references [19] and [20], combined with the authors' numerical simulations and fitted into three distinct flow regimes.)

To derive the resistance calculation formula for variable-diameter cylinders, it is necessary to first use numerical simulation methods to calculate the variation of the drag coefficient of cylinders with Reynolds number and aspect ratio. Therefore, corresponding cylinder models are established to calculate the average drag coefficient C_d , with aspect ratios λ ($\lambda = H/D$) ranging from 10 to 80. To cover both large and small aspect ratios, references [19] for infinite-length cylinders and finite-length cylinders with $\lambda = 5, 115$, and references [20] for finite-length cylinders with $\lambda = 1, 1.5, 2, 3$ are cited. Through numerical calculations, the variation of the drag coefficient of cylinders with aspect ratio λ and Reynolds number Re is obtained, as shown in Figure 2.

The results of numerical simulations define three zones for the drag coefficient: within the subcritical region ($10^4 \leq Re < 10^5$), within the drag crisis region ($10^5 \leq Re < 4 \times 10^5$), and within the drag recovery region ($Re \geq 4 \times 10^5$). The comprehensive relationship equation for the drag coefficient of finite-length cylinders is fitted within these zones, represented in segments as follows [21] :

- (1) When $10^4 \leq Re < 10^5$, the subcritical region

$$C_d = 0.855 + 0.683 \ln(Re) - 0.588 \ln^2(Re) - 1.730\lambda + 2.629\lambda^2 \quad (1)$$

- (2) When $10^5 \leq Re < 4 \times 10^5$, the drag crisis region

$$C_d = 1.111 - 0.081 \ln(Re) - 0.019 \ln^2(Re) + 0.002 \ln^3(Re) - 1.762\lambda + 3.987\lambda^2 - 2.381\lambda^3 \quad (2)$$

- (3) When $Re \geq 4 \times 10^5$, the drag recovery region.

$$C_d = 0.739 - 0.124 \ln(Re) + 0.009 \ln^2(Re) + 7.510\lambda - 3.806\lambda^2 \quad (3)$$

DERIVATION OF THE DRAG COEFFICIENT FOR FINITE-LENGTH VARIABLE DIAMETER CYLINDER

The height of the variable diameter cylinder is denoted by H , with the top referred to as the upper base and the bottom referred to as the lower base. The diameters are represented by d_1 and d_2 , and the radius are denoted by r_1 and r_2 . In calculations, the diameter at the axisymmetric plane, known as the mid-diameter, denoted by D , is commonly used. The angle between the side characteristic line and the central axis is represented by θ . It is worth noting that variable diameter cylinders differ from cones and frustums, typically referring to various structural members in engineering with relatively small diameters and large aspect ratios. θ varies within very small angles, as shown in Figure 3.

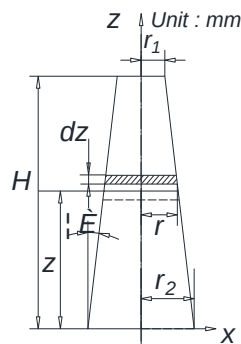


Fig. 3 - The drag coefficients for each section when $Re=10^5$.

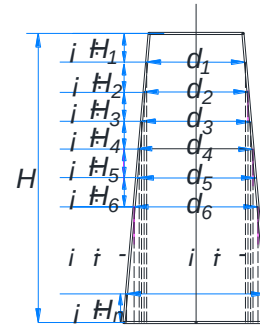


Fig. 4 - Definition of Variable Diameter Cylinder.

Suppose the variable diameter cylinder consists of n cylindrical sections, each with a height ΔH_1 , diameter d_1 , length ΔH_2 , diameter d_2 , ..., height ΔH_n , diameter d_n . Let the aspect ratio be λ , $\lambda = H/D = H/(r_1 + r_2)$, and the mid-diameter of the variable diameter cylinder be D , $D = (d_1 + d_2)/2 = r_1 + r_2$, $\tan \theta = (r_2 - r_1)/H$. Take a differential segment of length dz with a distance C_d from the bottom of the variable diameter cylinder, and a radius C_{pb} for the differential segment, as shown in Figure 4.

The resultant force acting on the cylinder is:

$$\sum F_{xi} = \frac{1}{2} \rho V^2 \Delta H (C_{d1} d_1 + C_{d2} d_2 + \dots + C_{dn} d_n) = \frac{1}{2} \rho V^2 \Delta H \sum C_{di} d_i \quad (4)$$

The drag force acting on this differential segment is:

$$dF = C_d \frac{1}{2} \rho V^2 dA = C_d \rho V^2 r dz \quad (5)$$

$$F_x = \int_0^H C_d \rho V^2 x dz = \rho V^2 \int_0^H C_d r dz \quad (6)$$

Wherein:

$$C_d = f\left(\frac{H}{D}, Re\right) = f(\lambda, Re) \quad , \quad Re = \frac{VD}{\nu}$$

From $\tan \theta = \frac{r_2 - r}{z} = \frac{r - r_1}{H - z}$, we obtain $r_2 = \frac{(r - r_1)z}{H - z} + r$, hence:

$$r = \frac{D}{2} + \left(\frac{H}{2} - z\right) \tan \theta \quad (7)$$

Therefore, the expression for the drag force is:

$$F_x = \int_0^H C_d \rho V^2 \left[\frac{D}{2} + \left(\frac{H}{2} - z\right) \tan \theta \right] dz \quad (8)$$

The expression for the drag coefficient is:

$$C_d = \frac{\int \frac{1}{2} \rho V^2 \sum (C_{di} \Delta H_i d_i)}{\frac{1}{2} \rho V^2 HD} = \frac{\sum (C_{di} \Delta H_i d_i)}{HD} \quad (9)$$

Integrating the segmented drag coefficients derived from Equations (1), (2), and (3) for flow around cylinders, we obtain the expressions for the drag of the variable diameter cylinder within different Reynolds number ranges, respectively:

(1) When $10^4 \leq Re < 10^5$, the subcritical region

$$F_{x1} = \rho V^2 \left\{ \left[0.855 + 0.683 \ln(Re) - 0.588 \ln^2(Re) - 1.730 \lambda + 2.629 \lambda^2 \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right\} \quad (10)$$

(2) When $10^5 \leq Re < 4 \times 10^5$, the drag crisis region

$$F_{x2} = \rho V^2 \left\{ \left[\frac{1.111 - 0.081 \ln(Re) - 0.019 \ln^2(Re) + 0.002 \ln^3(Re)}{-1.762 \lambda + 3.987 \lambda^2 - 2.381 \lambda^3} \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right\} \quad (11)$$

(3) When $Re \geq 4 \times 10^5$, the drag recovery region

$$F_{x3} = \rho V^2 \left\{ \left[0.739 - 0.124 \ln(Re) + 0.009 \ln^2(Re) + 7.510 \lambda - 3.806 \lambda^2 \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right\} \quad (12)$$

The expressions for the drag coefficient of the variable diameter cylinder within the three Reynolds number ranges are as follows:

(1) When $10^4 \leq Re < 10^5$, the subcritical region

$$C_{d1} = \frac{2}{HD} \left\{ \left[0.855 + 0.683 \ln(Re) - 0.588 \ln^2(Re) - 1.730 \lambda + 2.629 \lambda^2 \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right\} \quad (13)$$

(2) When $10^5 \leq Re < 4 \times 10^5$, the drag crisis region

$$C_{d2} = \frac{2}{HD} \left\{ \left[\frac{1.111 - 0.081 \ln(Re) - 0.019 \ln^2(Re) + 0.002 \ln^3(Re)}{-1.762 \lambda + 3.987 \lambda^2 - 2.381 \lambda^3} \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right\} \quad (14)$$

(3) When $Re \geq 4 \times 10^5$, the drag recovery region

$$C_{d3} = \frac{2}{HD} \left[\left[0.739 - 0.124 \ln(Re) + 0.009 \ln^2(Re) + 7.510\lambda - 3.806\lambda^2 \right] \left[\left(\frac{D}{2} + \frac{H}{2} \tan \theta \right) H - \tan \theta \frac{H^2}{2} \right] \right] \quad (15)$$

THE INFLUENCE OF END-FACE EFFECTS ON THE CHARACTERISTICS OF DRAG FLUCTUATIONS

Large Eddy Simulation (LES) method

The Large Eddy Simulation (LES) method is highly accurate for solving flow around cylinders and similar bodies. Thus, LES is used to analyze the end-face effects of finite-length and variable diameter cylinders [22]. In LES, a box-filtering function filters the Navier-Stokes equations. The incompressible fluid Navier-Stokes equations are expressed as:

$$\frac{\partial \bar{u}_i}{\partial t} + \frac{\partial (\bar{u}_i \bar{u}_j)}{\partial x_j} = -\frac{1}{\rho} \frac{\partial \bar{p}}{\partial x_i} + \nu \frac{\partial^2 \bar{u}_j}{\partial x_i \partial x_j} - \frac{\partial \bar{\tau}_{ij}}{\partial x_j} \quad (16)$$

$$\frac{\partial \bar{u}_i}{\partial x_i} = 0, i \in \{1, 2, 3\} \quad (17)$$

In the equation: x_i and x_j represent Cartesian coordinates, $\bar{u}_i$ and $\bar{u}_j$ represent filtered velocity vectors, where $i, j = x, y, z$ in three-dimensional problems; $\bar{p}$ denotes the pressure exerted on the fluid; ρ represents fluid density; ν denotes fluid viscosity. The right-hand side of the equation contains the unclosed term $\rho u_i u_j - \rho \bar{u}_i \bar{u}_j$, referred to as the subgrid-scale stress τ_{ij} , where $\tau_{ij} = \rho u_i u_j - \rho \bar{u}_i \bar{u}_j$.

Tab. 1 - The computational results of Large Eddy Simulation for flow around cylinders

	C_d	C_{pb}	St	θ_{sep}	L_{red}
Experiment	0.99 ± 0.05	0.88 ± 0.05	0.215 ± 0.005	$86.0^\circ \pm 2^\circ$	$1.4D \pm 0.01$
High-order upwind	1.00	0.95	0.203	85.8	1.36D
Central conservation	1.00	0.93	0.207	86.9	1.40D
B-Spline	1.04	0.94	0.210	88.0	1.35D
This article	1.01	0.95	0.208	86.8	1.39D

Flow around a cylinder at a Reynolds number of 3900 is a typical subcritical flow. There are abundant experimental results available for this Reynolds number, which can be used for comparison to validate the accuracy of numerical simulation methods. When using high-precision schemes, the Stanford Turbulence Research Center experimented with high-precision upwind schemes, second-order conservative centered differencing schemes, and B-Spline Galerkin methods, all yielding satisfactory results. The length of the computational domain L is determined based on the streamwise wavelength of the near-wake flow structures from existing experimental results, where $L = \pi D$. The number of grids is 9.76 million. The numerical simulation results in Table 1 show good agreement with the experimental

results. In the table, C_d represents the drag coefficient, C_{pb} represents the pressure coefficient at the lee side point, St represents the Strouhal number, θ represents the separation angle, and L_{red} represents the length of the reattachment region.

To validate the reliability of the LES method used in this study, we replicated the well-documented flow condition at Reynolds number $Re = 3900$. This subcritical regime has been widely studied experimentally, and the simulation results from this work—such as drag coefficient (C_d), separation angle, and Strouhal number—showed good agreement with benchmark data. The comparison is detailed in Table 1, confirming that the simulation methodology accurately captures the critical flow features necessary for modeling more complex geometries.

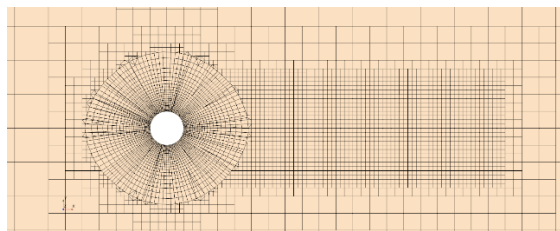


Fig. 5 - Computational grid for LES validation case at $Re = 3900$.

The computational domain was discretized using a structured mesh, structured mesh near the cylinder surface ensures high resolution in the boundary layer region. Grid stretching is applied toward the outer domain to reduce computational cost while maintaining accuracy. With refined elements near the cylinder surface to accurately resolve boundary layer dynamics. Figure 5 presents the grid topology used for the validation case at $Re = 3900$, which is consistent with previous high-fidelity LES studies and supports the accuracy of the simulation framework.

Analysis of end-face effects on finite-length cylinders

Near the upper and lower end faces of the cylinder, the flow field distribution differs from that in the middle section of the cylinder. The height of the cylinder is divided into multiple small sections along the height direction (z -direction) to analyze the distribution pattern of the drag coefficient of the cylinder along the height direction, as shown in Figure 6:

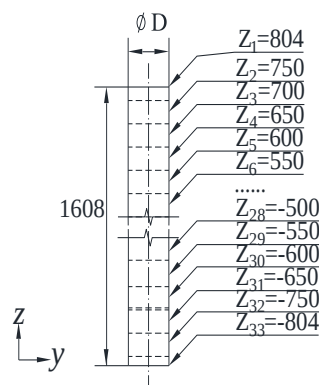


Fig. 6. - Schematic diagram of various cross-sections of the cylinder (unit: mm)

The temporal variation of the drag coefficient for sections $Z_1, Z_2, \dots, Z_{31}, Z_{32}, Z_{33}$ of the cylinder under different operating conditions is calculated. This is illustrated in Figures 7 to 10, where Re is respectively $10^4, 10^5, 2.5 \times 10^5, 5 \times 10^5$. The primary sections affected by end-face effects are Z_1, Z_2, Z_{31}, Z_{32} , and Z_{33} . The temporal variation patterns of the drag coefficient for sections in the middle of the cylinder (sections $Z_4, Z_5, \dots, Z_{30}$) are relatively consistent and are collectively referred to as the "UA" (Undisturbed Area) region.

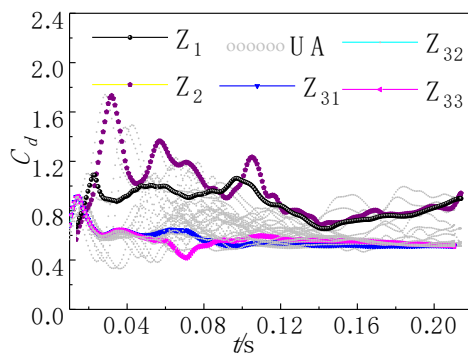


Fig. 7 - The drag coefficients for each section when $Re=10^4$

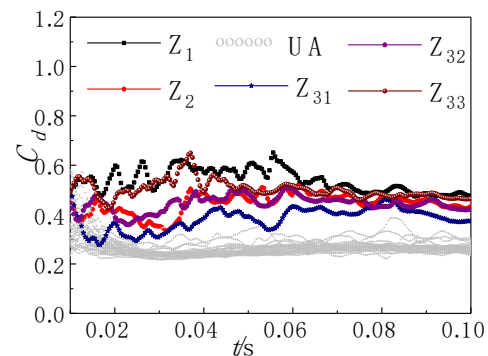


Fig. 8 - The drag coefficients for each section when $Re=10^5$

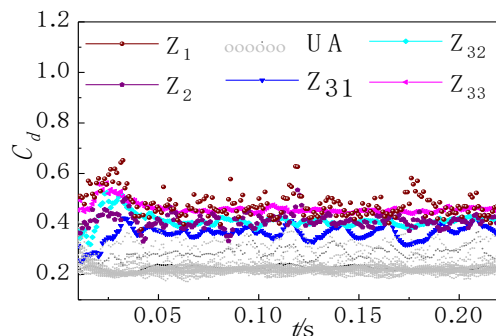


Fig. 9 - The drag coefficients for each section when $Re=2.5 \times 10^5$

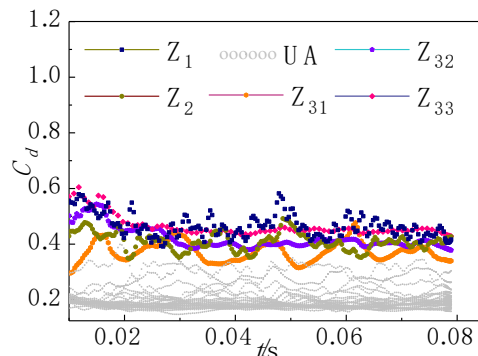


Fig. 10 - The drag coefficients for each section when $Re=5 \times 10^5$

Further analysis reveals the average drag coefficient of the cylinder and each section simulated under different operating conditions, as shown in $Re=10^4$ as an example, the differences between the drag coefficient of sections Z_1 - Z_5 and the average drag coefficient of the cylinder are 32.70%, 35.97%, 14.46%, 5.85%, and 4.82% respectively. Similarly, the differences between the drag coefficient of sections Z_{29} - Z_{33} and the average drag coefficient of the cylinder are 3.93%, 6.20%, 25.98%, 36.49%, and 39.24% respectively.

Analysis of end-face effects on finite-length variable diameter cylinders

The flow field distribution varies notably between the central section and the vicinity of the upper and lower end faces of the cylinder. In the derivation of the drag for finite-length variable diameter cylinders, as illustrated in Figure 3, it's approximated that the cylinder consists of an infinite number of infinitesimal cylinders, each with the same drag coefficient. Hence, the drag F_x of the variable diameter cylinder can be

expressed as follows:

$$F_x = \sum F_{xi} = C_{d1} \frac{1}{2} \rho V^2 \Delta H_1 d_1 + C_{d2} \frac{1}{2} \rho V^2 \Delta H_2 d_2 + \dots + C_{dn} \frac{1}{2} \rho V^2 \Delta H_n d_n = \frac{1}{2} \rho V^2 \Delta H \sum C_{di} d_i \quad (18)$$

In the equation above, it's assumed that the drag on each section of the cylinder is uniform, neglecting the flow field changes around the cylinder's upper and lower end faces. Additionally, there's an angle between the side characteristic line and the central line of the variable diameter cylinder. The derivation treats each section of the variable diameter cylinder as a cylinder, leading to discrepancies between the derived drag formula and experimental results. These factors need detailed analysis. Parameters affecting the drag F_x of the variable diameter cylinder include the angle θ between the side characteristic line and the central line (expressed as $\tan \theta$), Reynolds number Re , height H , diameter D (expressed as the height-to-diameter ratio $\lambda = (H/D)$), or considering the windward area A . Thus, the drag of the variable diameter cylinder can be represented by the following equation:

$$F_x = \frac{1}{2} \rho V^2 C_d A \quad (19)$$

Wherein:

$$F_x = \frac{1}{2} \rho V^2 C_d A \quad (20)$$

$$C_d = f\left(Re, \frac{H}{D}\right) \quad (21)$$

Calculation model and computational conditions

The distance from the model to the boundary of the computational domain is shown in Figure 11, meeting the requirement of eliminating the influence of computational domain scale on the flow field. Grid division is based on different cylinder models, with a total number of grids ranging from 9 to 11 million. The near-wall functions method is used to solve near the surface of the cylinder, and the inlet velocity magnitude is determined based on Re and the characteristic size of the cylinder.

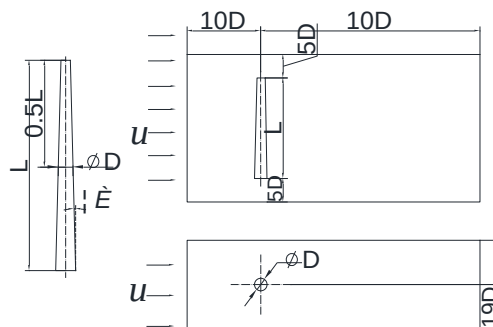


Fig. 11 - Variable diameter cylinder model and computational domain.

Numerical simulation results and analysis

Further analyze the influence of end effects on the drag coefficient. Within three different (Re) ranges, variable diameter cylinder models are established to numerically simulate and calculate the sectional drag coefficients. The parameters of the variable

diameter cylinder model are selected as follows: upper diameter of 31.34mm, lower diameter of 118.66mm, height-to-diameter ratio of 13.33, and inclination angle (θ) of 2.5° .

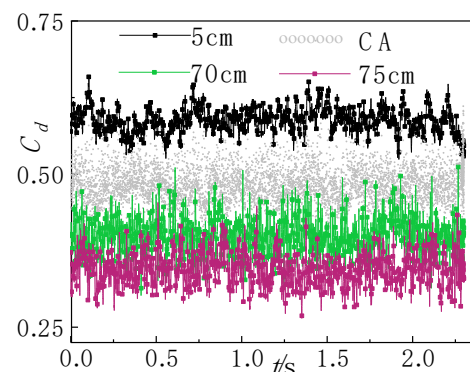
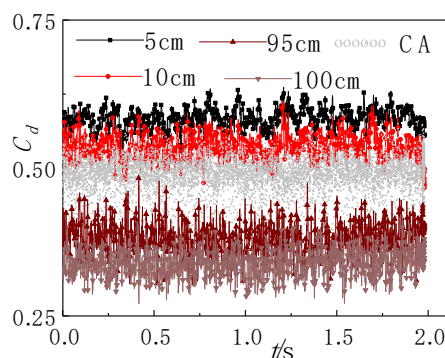
The error between the drag values derived from the variable diameter cylinder formula and those obtained from numerical simulations ranges from 13% to 16%, indicating a need for correction. Hence, a variable-diameter cylinder characterized by Reynolds number Re , the angle θ between the side characteristic line and the midline, length H , and mid-diameter D is established. The model parameters, comprehensively considering the factors affecting the end-face effect of the flow around the variable-diameter cylinder, are as follows: $Re=5 \times 10^4 \sim 8 \times 10^4$, $D=50 \sim 75$ mm, $H=650 \sim 1000$ mm, $\theta=2 \sim 5^\circ$; $Re=1.25 \times 10^5 \sim 2.5 \times 10^5$, $D=50$ mm ~ 85 mm, $\theta=2^\circ \sim 6^\circ$; $Re=4.5 \times 10^5 \sim 5 \times 10^5$, $D=50$ mm ~ 75 mm, $\theta=1^\circ \sim 6^\circ$. A total of 21 operating conditions were considered.

Tab.2 - Error between formula derivation and numerical simulation

Reynolds number	Drag			Drag coefficient		
	Formula derivation	Numerical simulation	Error (%)	Formula derivation	Numerical simulation	Error(%)
5×10^4	6.35	7.22	13.77	0.53	0.66	13.77%
2.5×10^5	126.00	141.22	12.08	0.61	0.68	12.08%
5×10^5	173.27	200.41	15.64	0.35	0.40	15.64%

In the analysis process, the positions of each section are represented by the distance in the z-axis direction of the variable diameter cylinder, using the same representation method as shown in Figure 2. In Figure 12 to 16, the sections with distinct characteristics are displayed, while other more concentrated sections are defined as the Concentration Area (CA) region. The non-steady variation of the drag coefficient within different regions is calculated through numerical simulations.

Analyzing the variation of the drag coefficient within the, the drag recovery region under constant Reynolds number (Re), with respect to the changes in θ and λ . Figure 12 shows the coefficient curves of the drag fluctuation characteristics for various sections of the variable diameter cylinder, with $Re=5 \times 10^4$, θ values of 3.5° and 5° , and λ values of 13.3 and 10. From Figure 12, it can be observed that as θ increases, the distance affected by the upper base of the variable diameter cylinder increases, while the distance affected by the lower base remains relatively consistent. The intermediate sections are unaffected, exhibiting consistent drag coefficients.

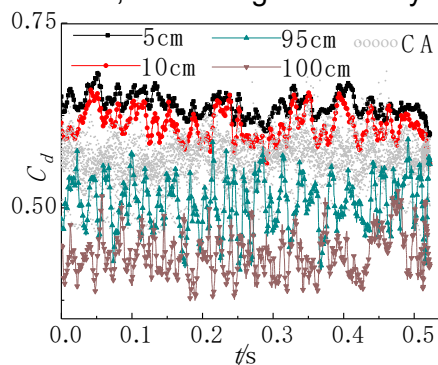


(a) $Re=5 \times 10^4, \lambda=13.3, \theta=3.5^\circ$

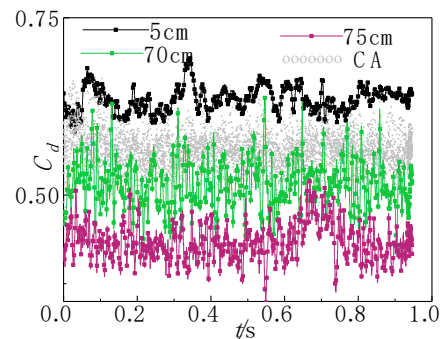
(b) $Re=5 \times 10^4, \lambda=10, \theta=5^\circ$

Fig. 12- Drag coefficient for various sections of $Re=5 \times 10^4, \lambda=9.3$ and $10, \theta=3^\circ$ and 5° .

Analyzing the drag crisis region, under constant Re conditions, the variation pattern of the drag coefficient with θ and λ . Figure 13 shows the coefficient of drag fluctuation characteristics for various sections when $Re=8 \times 10^4$, λ is 13.3 and 10, and θ is 3.5° and 5° . Due to the influence of the angle θ and the end effect on the flow field, as Re decreases, the non-uniformity of the distribution along the centerline direction further increases. At the same time, for higher Re conditions ($Re=5 \times 10^5$), the influence of the end effect on the bottom and top of the variable diameter cylinder is further intensified, enhancing the stability of the drag coefficient.



(a) $Re=8 \times 10^4, \lambda=13.3, \theta=3.5^\circ$



(b) $Re=8 \times 10^4, \lambda=10, \theta=5^\circ$

Fig. - 13 Drag coefficient for various sections of $Re=8 \times 10^4, \lambda=13.3$ and $10, \theta=3.5^\circ$ and 5° .

Analysis of the drag recovery region, with Re remaining constant, the variation of the resistance coefficient with θ and λ . Figure 14 shows the fluctuation characteristics of the resistance coefficient curves for different sections when $Re=4 \times 10^5$, λ is 9.3 and 13.3, and θ is 6° and 2° . It can be observed from Figure 14 that when the Reynolds number Re is the same and θ is relatively large, the range of the influence of the end-face effect on the variable-diameter cylinder in the axial direction increases. This effect is particularly pronounced when θ is 6° . Analyzing the resistance coefficient crisis zone, with Re remaining constant, the variation of the resistance coefficient with θ and λ . Figure 14(b) shows the resistance coefficient curves of various sections when $Re=1.25 \times 10^5$, θ is 2° and, and λ is 13.3. Combined with the analysis of Figure 14(a), it can be observed that when θ is small, the range of influence on the top and bottom ends of the variable-diameter cylinder decreases, and the difference between the resistance coefficients of sections affected and unaffected by the end-face effect can be more clearly distinguished.

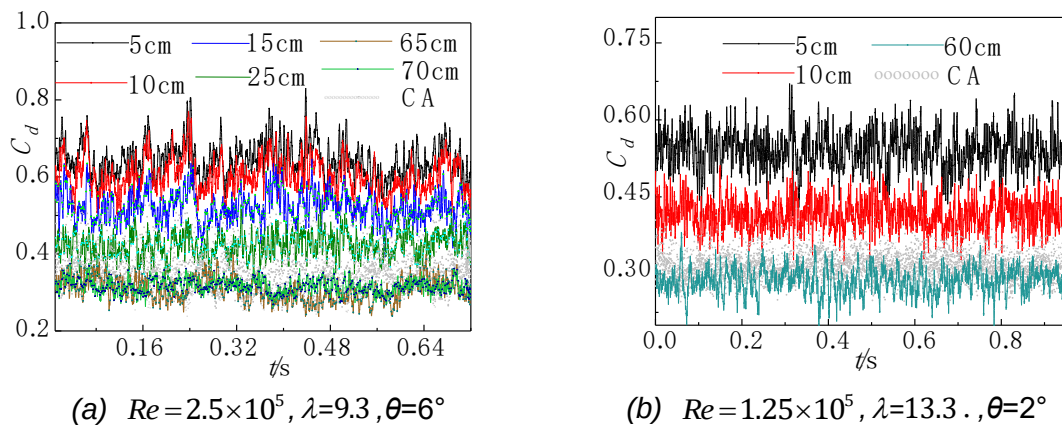


Fig. 14 - Drag coefficient for various sections of $Re = 2.5 \times 10^5$ and $Re = 1.25 \times 10^5$, $\lambda = 9.3$ and $13.3, \theta = 6^\circ$ and 2° .

In the drag crisis region, with Re remaining constant, the variation of the resistance coefficient with θ and λ is analyzed. Figure 15 depicts the fluctuation characteristics of the resistance coefficient of each section under conditions $Re = 2.5 \times 10^5$, $Re = 5 \times 10^5$, $\theta = 4^\circ$ and 6° , and $\lambda = 9.4$ and 9.3 . It can be observed that with the increase of θ , the resistance coefficient near the upper end of the variable-diameter cylinder undergoes significant changes, showing a large difference compared to the resistance coefficient near the middle section. Figure 16 shows the resistance coefficients of each section under conditions $Re = 4.5 \times 10^5$ and $Re = 8 \times 10^4$, with $\theta = 4^\circ$ and 6° , and $\lambda = 9.4$ and 9.3 . When θ increases to 6° , the resistance coefficient near the upper end of the variable-diameter cylinder undergoes changes. Under the same θ and λ , the difference between the resistance coefficient in the affected range and the average resistance coefficient is minimized when $Re = 1.25 \times 10^5$.

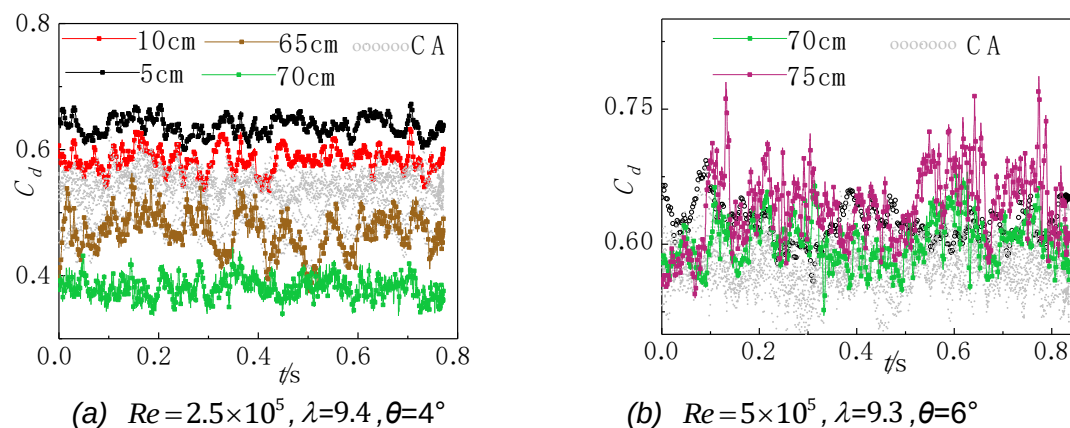


Fig. 15 - Drag coefficient for various sections of $Re = 2.5 \times 10^5$ and $Re = 5 \times 10^5$, $\lambda = 9.4$ and $9.3, \theta = 4^\circ$ and 6° .

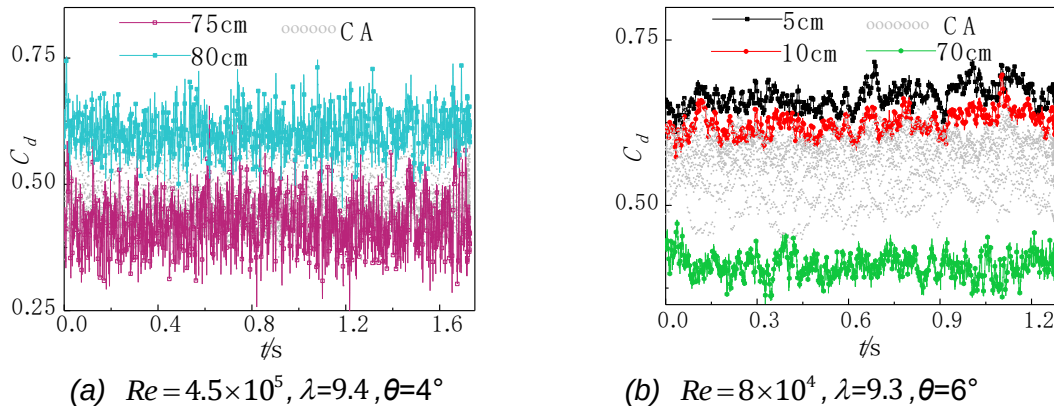


Fig. 16 - Drag coefficient for various sections of $Re = 4.5 \times 10^5$ and $Re = 8 \times 10^4$, $\lambda = 9.4$ and $9.3, \theta = 4^\circ$ and 6° .

Based on the comprehensive analysis of the calculation results, further investigation is conducted into the range influenced by the angle θ between the side characteristic line and the central line, affecting the resistance coefficient of the upper and lower ends. The range of influence on the upper end of the variable-diameter cylinder due to θ and end-face effects is denoted as h'_1 , where $h'_1 = h_1/d_1$ represents the extent of influence on the upper end, and represents the extent of influence on the lower end. Here, h_1 represents the distance of influence on the upper end along the axis of the variable-diameter cylinder, and h_2 represents the distance of influence on the lower end. The average resistance coefficient of the variable-diameter cylinder is denoted as C_d , with C'_{d_1} and C'_{d_2} representing the resistance coefficients influenced by the upper and lower ends, respectively. C'_{d_1} and C'_{d_2} are the average values of the resistance coefficients of various sections within distances h_1 and h_2 , respectively. Δ_1 and Δ_2 are defined as the errors in the resistance coefficients, where $\Delta_1 = |(C'_{d_1} - \overline{C_d}) / \overline{C_d}| \times 100\%$ and $\Delta_2 = |(C'_{d_2} - \overline{C_d}) / \overline{C_d}| \times 100\%$, as shown in Table 3.

Tab.3 - Influence Range and Errors of End-face Effects

Operating condition	Average drag coefficient $\overline{C_d}$	Range of influence		Drag coefficient of the affected cross-section		Drag coefficient error(%)	
		h'_1	h'_2	C_{d_1}'	C_{d_2}'	Δ_1	Δ_2
1	0.503	0.386	0.037	0.419	0.586	16.71	16.40
2	0.497	1.038	0.036	0.371	0.584	25.42	17.44
3	0.570	0.519	0.036	0.437	0.637	23.24	11.88
4	0.799	4.152	0.107	0.686	0.904	14.11	13.18
5	0.576	1.929	0.036	0.525	0.633	8.84	9.89
6	0.790	4.244	0.109	0.706	0.896	10.62	13.41
7	0.612	0.170	0.031	0.692	0.586	15.21	3.96
8	0.591	0.343	0.036	0.624	0.710	5.57	20.14
9	0.461	24.589	0.101	0.338	0.581	26.69	26.12
10	0.494	0.549	0.069	0.527	0.515	6.56	4.21
11	0.349	1.657	0.138	0.307	0.475	12.08	36.16
12	0.634	0.171	0.036	0.751	0.661	18.50	4.32
13	0.465	0.1712	0.036	0.561	0.609	20.76	30.96
14	0.602	8.196	0.034	0.472	0.674	21.59	11.84
15	0.554	8.196	0.034	0.432	0.638	21.96	15.10
16	0.567	0.258	0.082	0.709	0.539	25.04	4.93
17	0.503	0.517	0.082	0.545	0.514	8.41	2.39
18	0.521	0.232	0.039	0.569	0.671	2.98	15.5
19	0.458	1.173	0.134	0.386	0.599	14.57	22.98
20	0.615	0.130	0.045	0.643	0.657	4.64	6.85
21	0.578	0.261	0.045	0.597	0.680	3.35	17.64

Further analysis of the influence range h'_1 and h'_2 of end-face effects with respect to the Reynolds number Re , aspect ratio λ , and the angle θ between the side characteristic line and the central line, with h' representing the comprehensive influence range.

$$h' = -162.939 - 13.555 \ln(Re) + 7.309 \tan \theta + 61.041 \lambda + 6.268 \ln^2(Re) - 3.264 \tan^2 \theta - 6.959 \lambda^2 - 0.773 \ln^3(Re) + 0.420 \tan^3 \theta + 0.249 \lambda^3 \quad (22)$$

Compared to a straight circular cylinder, the angle θ is the primary factor influencing the resistance coefficient of a variable-diameter cylinder. Due to the flow disturbances caused by θ and the end-face effects induced by the cylinder's flow, the distribution pattern of resistance is further complicated. Therefore, the sections near the upper and lower bases are analyzed separately to study the resistance coefficient's temporal variation and average value along different sections. Subsequently, the sections with consistent patterns are combined to obtain an average value, representing the comprehensive height h' affected by the end faces. Through data fitting, a specific form of the resistance correction formula can be constructed as follows:

$$F'_z = a_0 + bRe + c_1 \lambda + m \tan \theta \quad (23)$$

Based on the results obtained from Table 3, the coefficients for the correction terms can be calculated, taking into account the different zones.

(1) When $10^4 \leq Re < 10^5$, Subcritical region

$$F_{z1}' = -0.044 - 4.800 \ln(Re) + 2.279 \lambda^{-1} - 3.131 \tan \theta \quad (24)$$

(2) When $10^5 \leq Re < 4 \times 10^5$, Drag crisis region

$$F_{z2}' = -3.102 - 0.307 \ln(Re) + 1.149 \lambda^{-1} - 8.558 \tan \theta \quad (25)$$

(3) When $Re \geq 4 \times 10^5$, Drag recovery region

$$F_{z3}' = -3.291 + 1.003 \ln(Re) + 1.194 \lambda^{-1} + 3.363 \tan \theta \quad (26)$$

Equations (24) to (26) are appended to equations (10) to (12), resulting in the final modified variation pattern of the variable-diameter cylinder's resistance. Through the correction equations above, the corrected error within the three resistance zones ranges from 1.5% to 9.34%, meeting the practical engineering requirements.

CONCLUSION

1. This study presents a comprehensive investigation into the flow resistance characteristics of finite-length variable diameter cylinders across subcritical, drag crisis, and drag recovery regimes. Through Large Eddy Simulation (LES), we quantified how aspect ratio, Reynolds number, and side angle influence drag distributions, especially near cylinder ends. A key contribution of this work is the derivation of a segmented drag coefficient model and an end-effect correction formula that significantly reduces predictive error.
2. The simulation results showed that drag coefficients vary notably at the cylinder extremities, with up to 39.24% deviation from the mean value observed. The proposed correction formula successfully reduces these deviations to below 9.34%, thus enhancing accuracy for practical applications. Furthermore, we provide a parametric formula that relates the end-effect influence range to geometric and flow conditions, offering a generalized approach for integrating end effects in engineering analyses.
3. Although direct experimental validation is limited in this study, our numerical approach is benchmarked against established experimental data for $Re = 3900$, showing excellent agreement. Future work will focus on wind tunnel experiments and pressure tap measurements to further validate the model predictions. The findings of this study are especially valuable in the design and analysis of civil and energy-related structures involving tapered cylindrical components.

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STUDY ON MECHANICAL RESPONSE LAW OF SHIELD ASSEMBLY TUNNEL WITH COMPLEX SECTION DURING CONSTRUCTION

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ABSTRACT

To investigate the stress and deformation behavior of the mushroom-shaped assembly tunnel, intelligent monitoring and numerical simulation were employed. The mechanical behavior variation during different construction stages, the longitudinal and cross-sectional distribution characteristics, and the deformation control effect of the support on the surrounding rock were analyzed. Comparative analysis assessed the applicability of different excavation methods, and theoretical comparisons discussed the bearing characteristics, providing engineering suggestions. The results indicate that excavation of the lower section has minimal impact on the mushroom head arch but continuously affects the side wall. The arch support of the mushroom head enhances the restraint ability of the lower section support on surrounding rock deformation. In the corner of the wall, the support appears to deviate from the tunnel, where the bolt is compressed. The contact stress of mushroom head increases first, then decreases and then increases from the position near the centroid to both sides. The stress in the transition area between the arch and the straight wall on both sides increases locally, and the stress concentration occurs at the corner of the wall. The deformation of the initial support and the bolt axial force on both sides of the lower section increase first and then decrease from top to bottom, while the law of the contact stress is opposite. Engineering focus should be on arch foot and side wall stress and deformation characteristics for combined flat arch and vertical straight wall support.

KEYWORDS

Large diameter shield, Assembly tunnel with mushroom shape, Intelligent monitoring, Numerical simulation, Mechanical response

INTRODUCTION

For underground engineering, structural safety is particularly important. In the process of construction and operation, the complex stratum environment such as high ground stress and high water pressure and the variable load conditions such as unloading and train load often threaten the safety of underground structures. To ensure the safety of tunnel construction and operation, it is

necessary to clarify the mechanical behavior of underground engineering under different conditions, so as to provide reference for engineering design.

Therefore, many scholars have explored the mechanical behavior of underground engineering under various conditions. The mechanical characteristics of underground engineering are significantly affected by factors such as stratum, groundwater, section shape and support method, so these factors become the main research object. Different complex geological conditions have different effects on the mechanical characteristics of the structure [1]. In layered soft rock, asymmetric deformation of surrounding rock and lining cracking will occur, which is mainly due to the combined action of layered soft rock and shear along the joint plane [2,3]. The dip angle of the strata can influence the surface deformation and stress concentration caused by tunnel excavation [4]. When crossing the fault fracture zone, the supporting structure is subjected to greater surrounding rock pressure [5]. Active faults can cause significant displacement in tunnel structures [6][7]. Moreover, earthquakes are a cause that can activate fault movement [8]. In the soft-hard inclined contact broken stratum with high ground stress, the lining will have asymmetric compression failure [9]. High ground stress also tends to lead to brittle failure of the surrounding rock [10]. To weaken the large deformation of soft rock, the use of double primary support can effectively control the deformation of surrounding rock and weaken the load on the secondary lining [11]. Double primary support can effectively control the large deformation and rheological effect of broken phyllite under high ground stress [12]. The use of prestressed anchorage system can enhance the cohesion of weak surrounding rock and control the deformation of surrounding rock [13]. For the poor stratum of loess, the tunnel is prone to excessive deformation or even lining cracking when constructing the tunnel. π -type SCCS arch can improve the bearing capacity of the structure [14]. In addition, the deformation of underground engineering is also affected by temperature and water. When the periodic temperature changes, it will change the internal force distribution of the structure, accelerate the deformation and failure of the structure, and reduce the bearing capacity of the structure [15]. Water will change the stress distribution of tunnel surrounding rock, which leads to the change of surrounding rock failure mechanism [16]. For double-track super-large cross-section tunnels, the spacing between the two tunnels also has a significant impact on the deformation distribution, crack behavior, and structural internal forces [17]. When the structure is loaded under unfavorable conditions, the bearing capacity of the structure can also be improved by prestressed composite lining [18]. The bearing capacity of the composite structure is obviously better than that of the double-shell structure [19]. Steel fiber significantly improves the structural toughness and bearing capacity [20]. Structures reinforced with a combination of steel fibers and rebar exhibit more outstanding performance [21]. In addition to the formation conditions, the shape of the section will also affect the mechanical characteristics of underground engineering. Through the numerical simulation method, it is very convenient to realize the mechanical response of excavation with different cross-section shapes [22]. The influence mechanism of opening shape on rock failure can be explored by processing openings of different shapes and sizes on rock blocks [23]. A full-scale ring test shows that the bearing capacity and failure characteristics of the quasi-rectangular shield tunnel structure are significantly different from those of the circular structure [24]. Bolt mortise, tenon joint and interior column can effectively improve the bearing capacity of rectangular tunnel structure [25] [26]. By optimizing the shape of the tunnel section, the stress concentration can be reduced and the structural stress can be improved [27].

The existing research mainly focuses on the influence of stratum conditions, support methods and section forms on the mechanical behavior of underground engineering. Among them, the section forms are mostly circular, rectangular, horseshoe-shaped and other conventional sections, and there are few studies on more complex sections such as mushroom-shaped sections. To clarify the mechanical response law of mushroom-shaped complex section, this study relies on the shield tunnel of Chongqing Yangtze River Railway to explore the deformation control characteristics of surrounding rock in large-section mushroom-shaped shield assembly tunnel. Then, the mechanical characteristics of the supporting structure during the construction process were analyzed, and then the mechanical characteristics of the structure were discussed and optimized.

Existing research has explored the mechanical behavior of tunnels under various conditions, with tunnel cross-sections typically being conventional shapes such as circular or horseshoe-shaped. The size and shape of the cross-section significantly influence the stress and deformation patterns of the tunnel. Due to geographical constraints, the shield assembly tunnel in Chongqing Yangtze River Tunnel employs a mushroom-shaped cross-section, with an arched upper section and a rectangular lower section. The cross-section is the largest among existing shield assembly tunnel, resulting in more construction procedures and increased construction difficulty. The span of the upper section is significantly larger than that of the lower section. Unlike conventional cross-sections such as circular or horseshoe-shaped ones, this cross-section features multiple corners prone to stress concentration. Additionally, the vertical side walls of the lower section are susceptible to lateral squeezing from the surrounding rock. The mechanical characteristics of this complex cross-section are intricate and under-researched. To address this, this study, based on the Chongqing Yangtze River Tunnel, uses intelligent monitoring and numerical simulation to investigate the deformation control characteristics of the surrounding rock in large-section mushroom-shaped shield assembly tunnel. It also analyzes the mechanical characteristics of the support structure during construction using theoretical methods and discusses and optimizes the structural mechanical characteristics. This study is the first to explore the interaction mechanism between support and surrounding rock in large-section mushroom-shaped assembly tunnel during construction, providing a reference for similar projects.

ENGINEERING OVERVIEW AND DESIGN

Engineering overview

Chongqing Yangtze River Tunnel is a railway line control project from Chongqing-Qianjiang, and the river-crossing section is constructed by shield method. The tunnel is a double-track extra-long tunnel, which is managed according to the Class I high-risk tunnel and is one of the 12 key projects on the whole line. The outer diameter of the shield segment is 12.2 m, which is a large-section river-crossing tunnel. The tunnel passes through important areas such as Yuzhong Peninsula urban area, Yangtze River and Nanping urban area. Yuzhong Peninsula urban area and Nanping urban area are shallow hilly landforms, located in the urban area. The strata in the tunnel are mainly sandstone and mudstone, and locally limestone and dolomite.

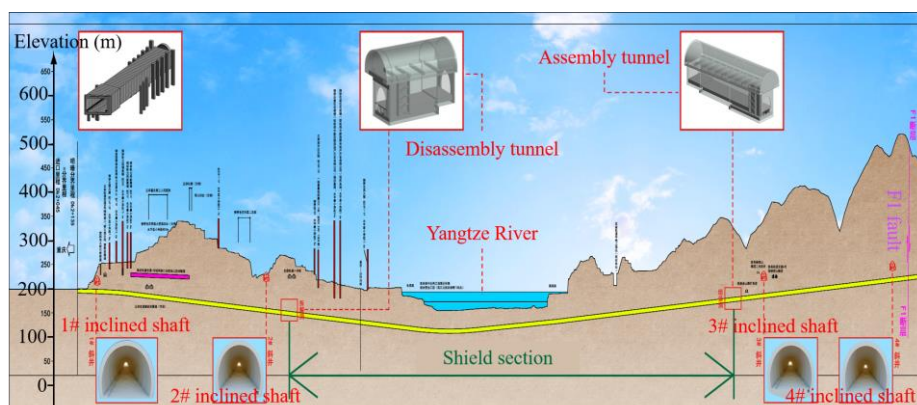


Fig. 1 – Longitudinal section line diagram of Chongqing Yangtze River Tunnel

Assembly tunnel design

The starting position of the shield is located inside the mountain, and the surface altitude is significantly higher than that of the river section. This makes the location does not have the conventional launching conditions, so the design adopts the scheme of large buried depth assembly tunnel shown in Figure 2. The starting and ending mileage of the assembly tunnel is DK7+650~DK7+700, and the buried depth reaches 149.23 m. The assembly tunnel is designed as

a two-stage structure: a transition section (15m, DK7+685~DK7+700) and a sinking section (35m, DK7+650~DK7+685). The section of the tunnel is mushroom-shaped, the upper section is mushroom head, and the lower section is rectangular. The shield components are transported to the assembly tunnel through the tunnel of the mining method section, and then the shield hoisting, trolley assembly, cutterhead installation and shield assembly are carried out. To meet the hoisting requirements, the maximum span of the tunnel is 25.54 m, and the maximum height is 29.96 m, which is the shield assembly tunnel with the largest section.

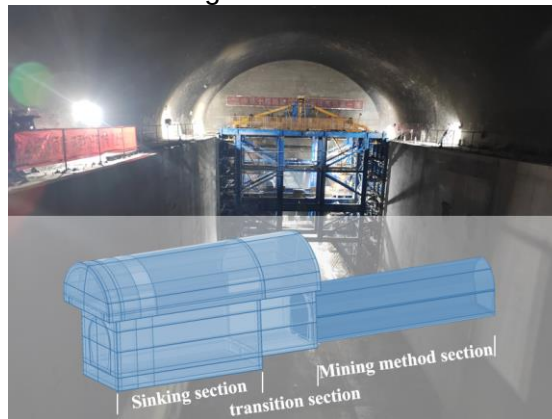


Fig. 2 – Assembly tunnel

The mushroom head of the assembly tunnel and the lower section of the sinking section are excavated in four steps, and the lower section of the transition section is excavated in three steps. The specific order is shown in Figure 3. Each excavation section is excavated to the maximum longitudinal length of the tunnel, and then the next section is constructed. After the excavation of each section is completed, spray anchor support and temporary support are applied in time. The bolt in mushroom head adopts $\phi 25$ combined hollow bolt, and the length of 4.5 m and 6.0 m is alternately arranged in plum shape. The bolt in lower section adopts $\phi 32$ HRB400 mortar bolt.

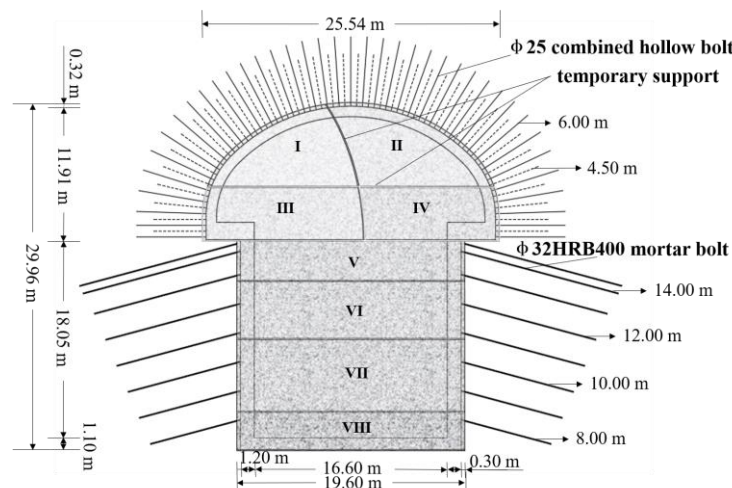


Fig. 3 – Section shape and construction design

INTELLIGENT MONITORING OF MECHANICAL BEHAVIOR IN ASSEMBLY TUNNEL

Monitoring scheme

The intelligent monitoring system is used to monitor the deformation of the initial support, contact stress and bolt axial force of the assembly tunnel, so as to clarify the variation law of the mechanical behavior of the support structure during the construction process. The mileage of the

monitoring section in this study is DK7+670, and the arrangement of monitoring elements and measuring points is shown in Figure 4. A1~A5 are deformation monitoring, of which A1 is settlement monitoring, the rest are convergence monitoring, and the monitoring instrument is laser deformation measuring instrument. B1~B5 are the contact stress monitoring of the initial support and the surrounding rock, and the monitoring instrument is the earth pressure box. C1~C5 is the monitoring of the bolt axial force, and the monitoring instrument is the bolt axial force meter. The deformation monitoring results are collected by the deformation acquisition box, and the contact stress and bolt axial force are collected by the stress acquisition box, and then uploaded to the data monitoring platform through wireless transmission to realize real-time monitoring.

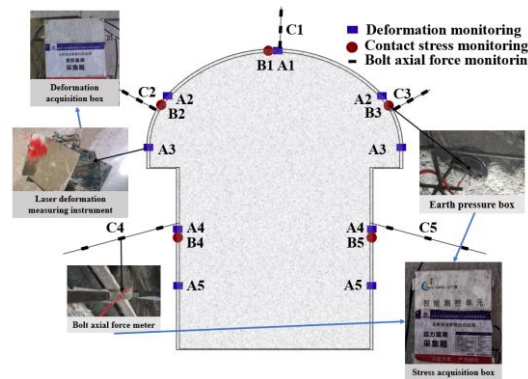


Fig. 4 – Installation of field monitoring elements and arrangement of measuring points

Diachronic characteristics of mechanical behavior in assembly tunnel

Figure 5 is the variation law of the initial support deformation during construction. The secondary lining in the mushroom head is applied before the excavation in the lower section, so the deformation in the mushroom head only analyzes the excavation process in the upper section. In addition, the deformation in the direction towards the tunnel is positive, and the deformation in the direction away from the tunnel is negative. It can be seen from the diagram that the vault (A1) settlement increases rapidly in the excavation of section II, and increases with the subsequent excavation but the increase is small. Arch foot (A2) and mushroom head side wall (A3) all produce convergence deformation deviating from the tunnel. The convergence deformation in arch foot (A2) increases during the excavation of section I and II, and decreases during the excavation of section III. This is because the section span increases after the excavation of section II, and the section height increases after the excavation of section III. The convergence deformation in the mushroom head side wall (A3) continues to increase, which is due to the increase of the IV section span. After the excavation of the section and the next section, A4 and A5 are deformed towards the tunnel, which is less affected by the subsequent excavation.

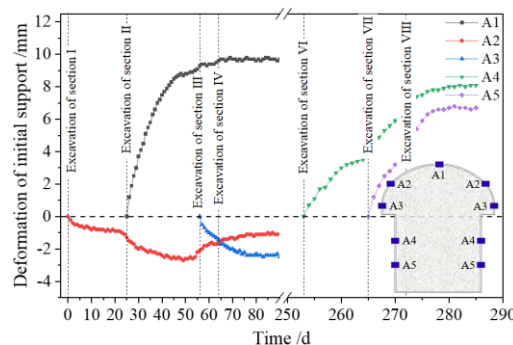


Fig. 5 – Deformation duration curve of initial support

Figure 6 is the change law of contact stress during construction. It can be seen from the diagram that the vault (B1) gradually increases after the excavation of the II section, and slowly

decreases in the subsequent section excavation. After the excavation of the section and the next section, the contact stress of the arch foot (B2, B3) increases significantly, and decreases rapidly after the excavation of the lower section. This is because the horizontal deformation of the surrounding rock squeezed by the arch foot decreases after the excavation of the lower section, and the vertical deformation towards the tunnel increases. After the excavation of the lower section, the contact stress of B2 and B3 changes slightly. The contact stress of B4 and B5 increases gradually only after the excavation of their section, and decreases rapidly after the subsequent excavation. Because the height of the section increases, the horizontal deformation of B4 and B5 increases, and the contact degree between the surrounding rock and the support decreases.

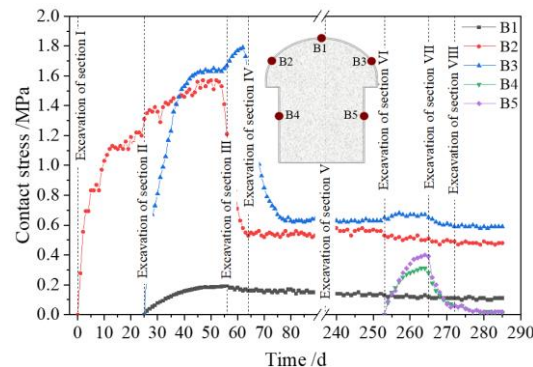


Fig. 6 – Contact stress duration curve

Figure 7 shows the variation law of the maximum bolt axial force during construction. Except for arch feet (C2, C3), the variation law of axial force at other positions is basically consistent with the deformation of initial support, and the excavation of lower section has no obvious influence on the axial bolt force in mushroom head. C2 and C3 are in a state of compression before the excavation of the lower section, and gradually become a state of tension after the excavation of the lower section. Because the horizontal deformation in the arch foot away from the surrounding rock is reduced, the vertical settlement is increased.

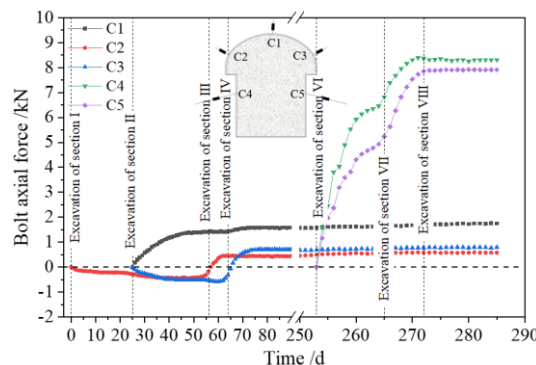


Fig. 7 – Bolt axial force duration curve

NUMERICAL SIMULATION OF MECHANICAL BEHAVIOR IN ASSEMBLY TUNNEL

Monitoring scheme

The strata where the assembly tunnel is located are mainly mudstone and sandstone. The sandstone is the main stratum, and the overlying thickness of the vault is about 74.23 m. The mudstone is the overlying stratum of sandstone, and the thickness is about 75.00 m. The Mohr-Coulomb constitutive model is one of the most widely used strength theories in geotechnical engineering. It is simple in form and effectively describes the mechanical behavior of rock and soil. It is widely used in numerical simulations of geotechnical engineering projects, including tunnels and

excavations. Geological surveys indicate that the strata in this project are relatively homogeneous. Therefore, both mudstone and sandstone adopt Mohr-Coulomb constitutive model. After the excavation of the tunnel, the support mainly includes initial support, temporary support, bolt and secondary lining. The model is shown in Figure 8. The stratum, the temporary support, the initial support and the secondary lining are solid elements, which are elastic materials. To more accurately simulate the interaction between the strata and the structure, contact elements have been employed to model these interfaces. Contact elements can effectively handle the contact, friction, and separation behaviors between the strata and support structures, thereby more realistically reflecting the mechanical behavior in actual engineering applications. The parameters of the interface mainly include the normal stiffness and shear stiffness, which can be calculated using Eq. (1). The implementation of the bolts is accomplished by utilizing the built-in cable elements. These elements, specifically designed for simulating rod-like structures, effectively model the stretching behavior of bolts under load and their interactions with the surrounding medium. Specifically, the cable elements transmit both axial and shear forces and account for the frictional and cohesive interactions between the bolt and the surrounding rock, thus faithfully reproducing the mechanical function of the bolts within the numerical model. The numerical simulation construction process is consistent with the actual situation. The strata parameters and support parameters are based on geological exploration data and engineering design, as shown in Table 1 and Table 2.

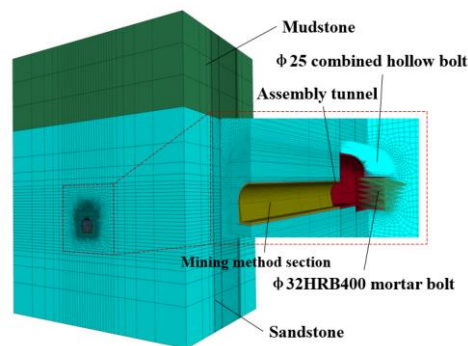


Fig. 8 – Numerical model

$$k_n = k_s = 10 \cdot \max\left[\frac{K_r + \frac{4}{3}G_r}{\Delta z_{\min}}\right] \quad (1)$$

Where k_n is the normal stiffness, k_s is the shear stiffness, $\Delta z_{\min}$ is the minimum size of the adjacent regions in the normal direction of the interface, K_r is the bulk modulus, and G_r is the shear modulus.

Tab. 1 - Strata parameters

Strata	Density / (kg/m ³)	Cohesion /MPa	Internal angle /°	friction	Elastic modulus /GPa	Poisson ratio
Mudstone	2520	2.10	38.31		2.367	0.32
Sandstone	2456	6.29	41.02		6.804	0.23

Tab. 2 – Supporting parameters

Type	Thickness /m		Elastic modulus /GPa	Poisson ratio
Initial support	Mushroom head	0.32	28.0	0.2
	Lower section	0.30		
Temporary support	0.11		28.0	0.2
Secondary lining	Mushroom head	0.9	30.0	0.2
	Lower section	1.2		

Longitudinal mechanical characteristics of assembly tunnel

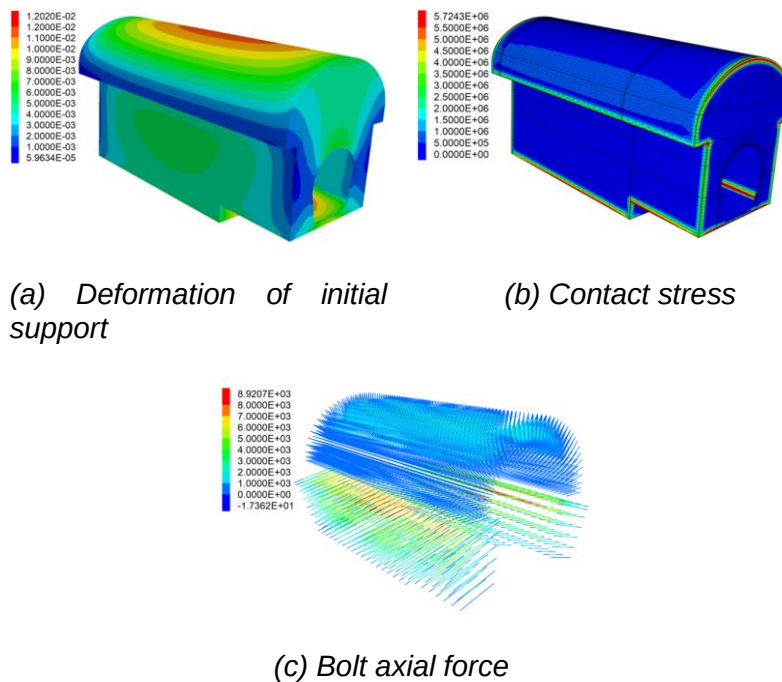


Fig. 9 – Longitudinal mechanical characteristics of assembly tunnel

Control effect of surrounding rock stability

Figure 10 is the deformation law of surrounding rock with or without support. Regardless of whether there is support or not, the deformation of vault surrounding rock is mainly vertical settlement. The deformation of the surrounding rock from the vault to the wall corner of the mushroom head gradually decreases, and the deformation direction gradually changes from vertical settlement to horizontal settlement. The bottom of the tunnel is mainly uplifted. The deformation in the upper part of the side wall in the lower section is significantly larger than that of the lower part without support, and the deformation in the middle part of the side wall is larger after support. because after the support is applied, the vertical effect of the upper load on the arch structure and the horizontal thrust of the squeezed soil are transmitted downward through the support structure, but the transmission effect is gradually weakened.

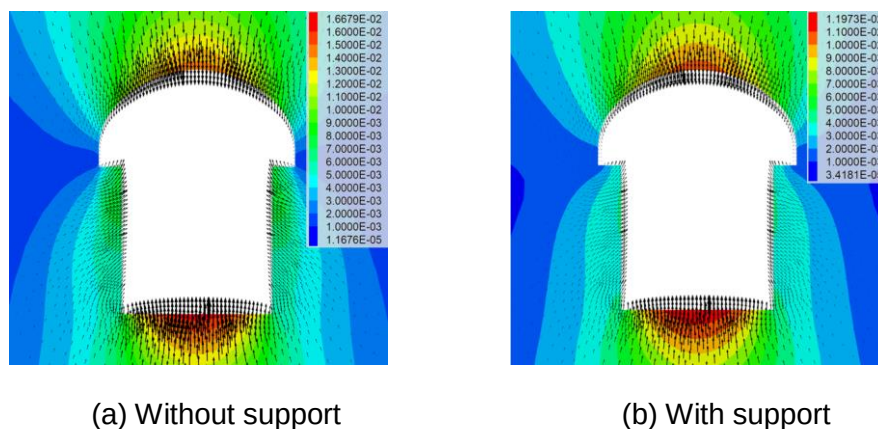


Fig. 10 – Surrounding rock deformation

To clarify the effect of support on the deformation control of surrounding rock, the deformation of typical positions was extracted, and the deformation control rate (the ratio of deformation change after support to unsupported deformation) was analyzed, as shown in Table 3. After support, the

deformation control rate of mushroom head increases first and then decreases from vault to arch foot. When there is no support, the bearing arch formed by the surrounding rock itself has a strong ability to resist deformation at the arch foot, so the deformation control rate at the arch foot after support is relatively low. The deformation control rate of the lower section gradually decreases from top to bottom, which indicates that the position of the larger deformation in the side wall moves down. On the whole, the deformation control rate of mushroom head is less than that of the lower section. On the one hand, it is due to the better self-stabilizing ability of the mushroom head arch structure when there is no support. On the other hand, the arch structure after support will enhance the constraint ability of the lower structure to the deformation of the surrounding rock.

Tab. 3 - Control effect of surrounding rock deformation

Location		Deformation value /mm		Deformation control rate /%
		Without support	With support	
Mushroom head	Vault	14.65	10.57	27.85
	Spandrel	10.17	7.08	30.38
	Arch foot	2.22	1.92	15.31
Lower section	Upper part	7.53	4.06	46.08
	Middle	6.31	3.99	36.77
	Lower part	5.30	3.75	29.25
	Bottom	16.68	11.97	24.24

DISTRIBUTION LAW OF CROSS-SECTION MECHANICAL CHARACTERISTICS IN ASSEMBLY TUNNEL

Deformation of initial support

Figure 11 shows the distribution law of initial support deformation in different excavation stages of mushroom head. After the excavation of each section, the arch is dominated by the vertical deformation towards the tunnel, and the lower wall corner is dominated by the horizontal deformation away from the tunnel. After the excavation of section II, the span of the tunnel increases by about one time, and the vertical deformation in the arch and the horizontal deformation in the corner of section I increase significantly. This shows that as the span increases, the vertical deformation in the arch increases, and the squeezing degree of the wall corner on the surrounding rock will also be aggravated. After the excavation of the section IV, compared with the excavation of the section II, the section height increases, the vertical deformation of the arch increases, and the horizontal deformation of the support on both sides decreases. The deformation at the corner of section I is 2.51 mm and 2.45 mm respectively, and the numerical difference is not significant, but the former is mainly horizontal deformation, and the latter is mainly vertical deformation. This means that the increase of the section height will weaken the extrusion degree of the structure towards the surrounding rock.

When the excavation section is an asymmetric structure, such as the excavation of sections I and III, the vertical deformation near the center of the excavated section is greater than the deformation away from the centroid, and the horizontal deformation away from the centroid will increase. After the excavation of section III, the deformation of the corner in section II, which is mainly horizontal displacement, is 3.03 mm, while the deformation of the corner of section II is 2.54 mm. When the excavation section is a symmetrical structure, the support deformation is also symmetrically distributed.

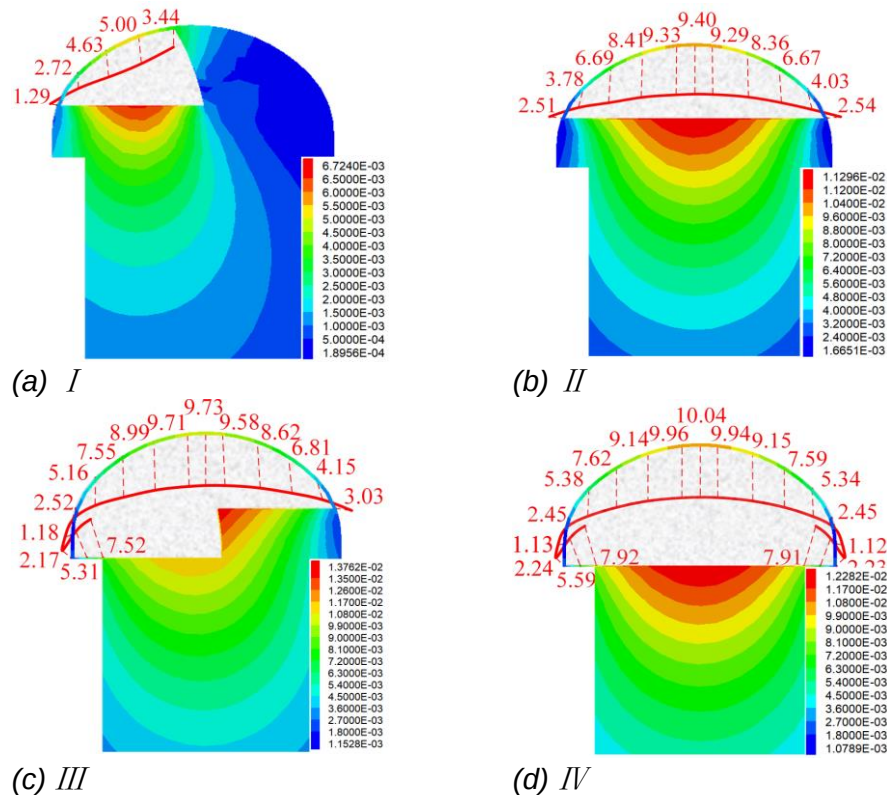
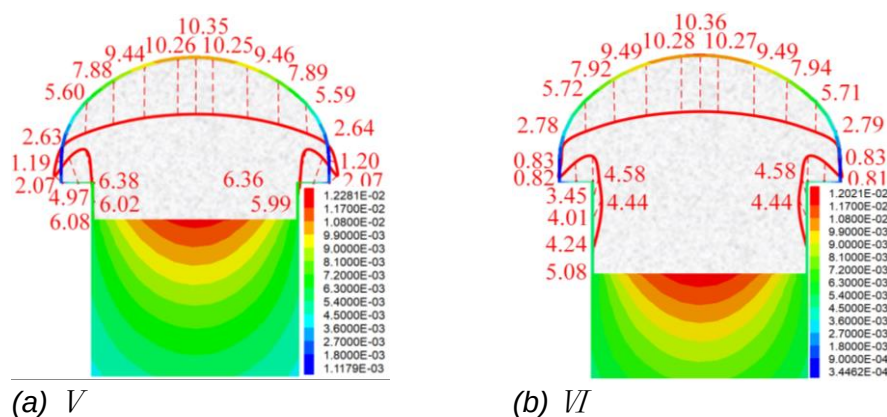


Fig. 11 – Initial support deformation during excavation of mushroom head (unit: mm)

Figure 12 is the distribution law of the deformation at different excavation stages of the lower section. The support of the lower section is a straight wall, which directly bears the horizontal extrusion of the surrounding rock on both sides. However, after the excavation of section V, the side wall does not deform horizontally towards the tunnel, but has a horizontal displacement slightly deviating from the tunnel. This shows that the thrust of the squeezed surrounding rock produced by the mushroom head support also has a significant impact on the lower section support. After the excavation of section VI, the side wall has a horizontal displacement towards the tunnel, which indicates that the horizontal thrust effect of the mushroom head support is weaker than the direct effect of the surrounding rock on the side wall. With the increase of section height, the direct effect of surrounding rock on the side wall is more significant, and the horizontal deformation of the side wall increases first and then decreases from top to bottom. In addition, the excavation of the lower section has little effect on the deformation of the mushroom head arch, and the vertical displacement increases slightly. As the section height increases, the side wall of the mushroom head is significantly affected, and the deformation from the tunnel gradually changes to the deformation towards the tunnel.



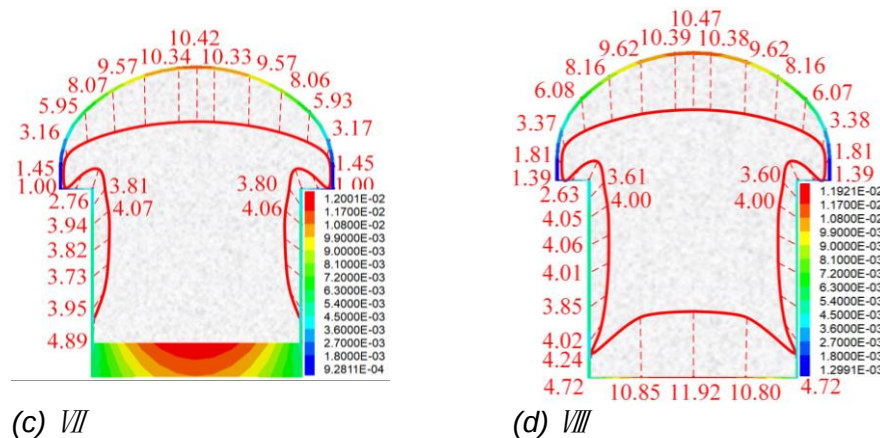
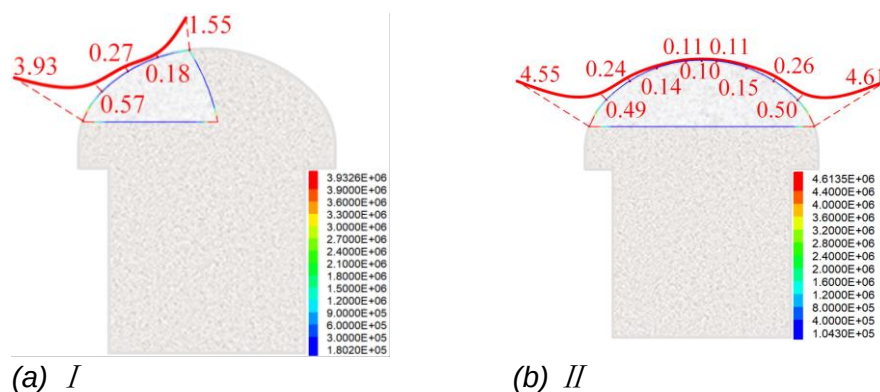


Fig. 12 – Initial support deformation during excavation of lower section (unit: mm)

Contact stress

Figure 13 shows the distribution law of contact stress between initial support and surrounding rock at different excavation stages of mushroom head. On the whole, the contact stress of the arch is small, because the arch has obvious deformation towards the tunnel, and the stress release is significant. There are sharp corners in the support of each excavation section, resulting in obvious stress concentration. On the one hand, it is because there is a sudden change in the shape of the support at these positions. On the other hand, it is because these positions produce deformation that deviates from the tunnel and squeezes the surrounding rock. The increase of the cross-section span will further release the stress of the arch and increase the stress of the corner. After the excavation of section II, the arch stress of section I decreases, while the corner stress increases from 3.93 MPa to 4.55 MPa. The height of the section increases, and then the stress of the arch and both sides decreases.

When the section is eccentrically compressed, the stress near the centroid of the excavated section is less than the stress away from the centroid. When the section is a symmetrical structure, the stress is also symmetrically distributed, and it increases first, then decreases and then increases from the vault to both sides. The stress of the arch foot increases, indicating that the horizontal thrust effect is the most significant here, and the horizontal thrust effect is weakened on the side wall.



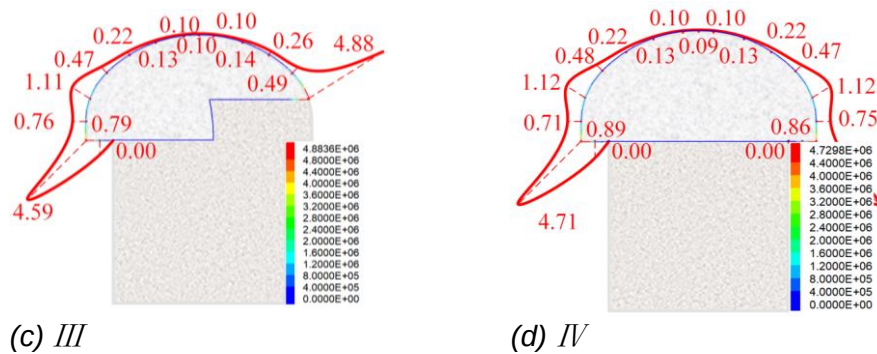


Fig. 13 – Contact stress during excavation of mushroom head (unit: MPa)

Figure 14 is the distribution law of contact stress in different excavation stages of lower section. In the excavation of the lower section, the height of the section increases, which has little effect on the stress of the arch of the mushroom head, and the influence on both sides is slightly larger. After the excavation of section V, the horizontal thrust of mushroom head is still obvious, and the stress on both sides increases. Subsequently, the height of the section continues to increase, and the stress on both sides of the mushroom head gradually decreases due to the deformation of the lower section towards the tunnel. The stress on both sides of the lower section decreases first and then increases from top to bottom, and there is a non-compression state in the middle. Because the support on both sides of the lower section is a straight wall, and the upper and lower ends are constrained by horizontal support, and the horizontal deformation is small. However, the horizontal deformation in the middle is significant, and the surrounding rock at the bottom has a squeezing effect on the straight wall, which further increases the bending degree of the middle part of the straight wall towards the tunnel, and weakens the mutual squeezing effect between the partial support and the surrounding rock.

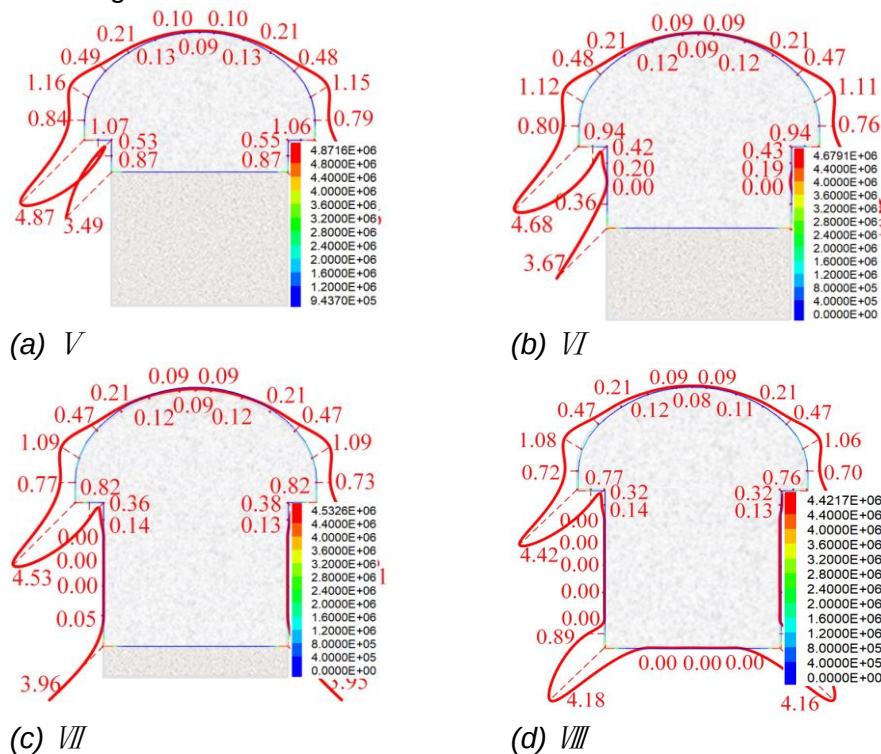


Fig. 14 – Contact stress during excavation of lower section (unit: MPa)

Bolt axial force

Figure 15 shows the distribution law of bolt axial force in different stages of mushroom head excavation. The arch bolt is in a tensile state, and the lower bolt is in a compressive state. This is because the arch produces a displacement pull bolt toward the tunnel, and the bottom produces a displacement compression bolt that squeezes the surrounding rock. Along the length direction of the bolt, the maximum axial force of the bolt appears in the middle of the bolt, which is the main action area of the bolt. Along the section direction, the maximum bolt axial force is located in the middle of the section and gradually decreases to both sides. The bolt axial force is positively correlated with the span and height of the section.

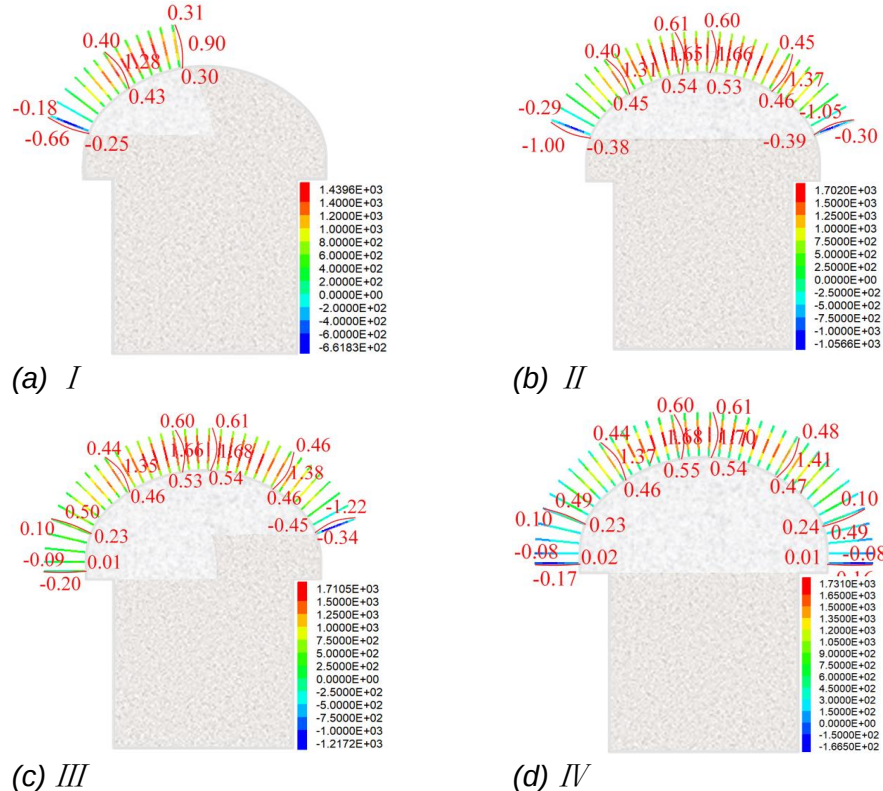
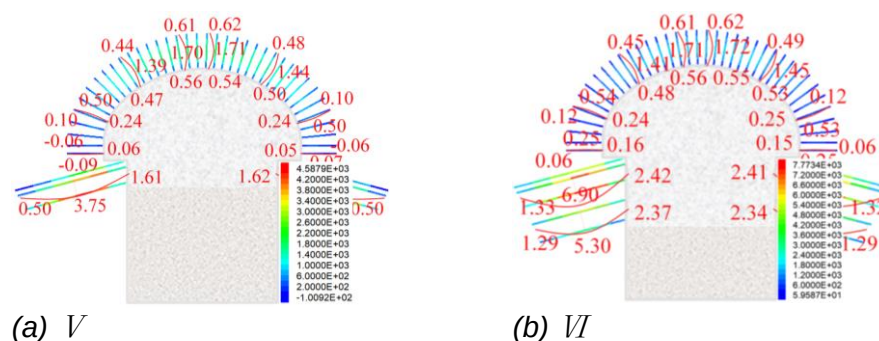


Fig. 15 – Bolt axial force during excavation of mushroom head (unit: kN)

Figure 16 is the distribution law of bolt axial force in different excavation stages of lower section. As the excavation height of the lower section increases, the bolt axial force on both sides of the mushroom head gradually increases, and the lower compression bolt gradually becomes a tension bolt. The bolt axial force in the lower section increases first and then decreases from top to bottom, and the upper part in the lower section bolt is generally greater than the lower part. This is because the horizontal deformation of the upper part towards the tunnel is significantly larger than that of the lower part, and the effect of the bolt is more significant.



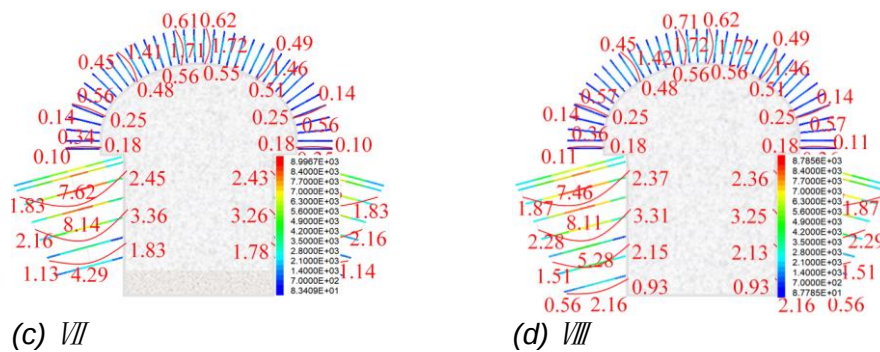


Fig. 16 – Bolt axial force during excavation of lower section (unit: kN)

DISCUSSION AND SUGGESTIONS

Excavation method

With reference to similar large-section tunnel projects under the same geological conditions, this project designed two upper section excavation methods: the center diaphragm (CD) method and the double side heading (DSH) method (Figure 17). To improve the efficiency, this project adopts the CD method with less steps. To clarify the adaptability of these two methods in this project, the mechanical response characteristics of the DSH method are further compared, as shown in Figure 18.

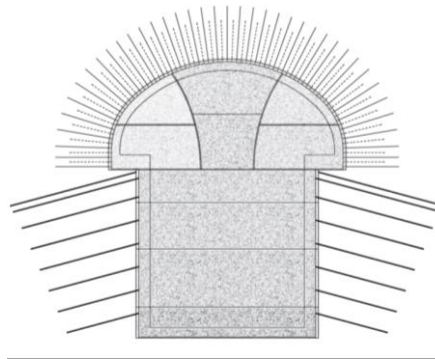
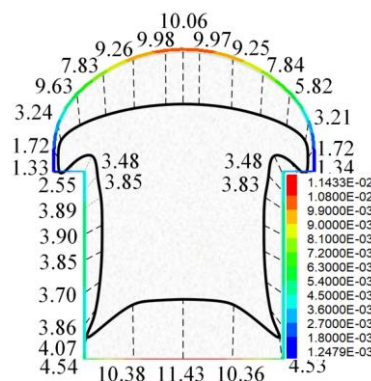


Fig. 17 – Double side heading method



(a) Deformation of initial support

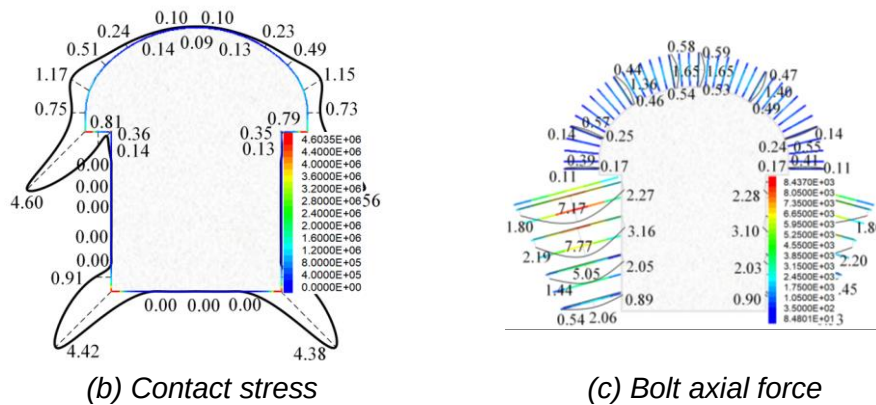


Fig. 18 – Mechanical characteristics of DSH method

Adopting the DSH method, the deformation of the initial support and the bolt axial force are relatively small, but the contact stress increases. This is because the excavation section of each process in the DSH method is small, and the initial support of each section is closed in time after excavation, so that the deformation is better controlled, and the smaller deformation also weakens the effect of the bolt. At the same time, the stress release is also weakened, so that the initial support is subjected to greater stress, and the stress concentration is more significant. As far as the deformation of the initial support is concerned, the method in this paper and the DSH method are reasonable, indicating that both methods meet the construction requirements. In similar projects, if the deformation control requirements are higher, the DSH method can be preferred. However, the DSH method has many processes, slow construction and high cost. Similar to this project, the CD method can be preferred when the deformation control is within the safe range.

Load model

Based on the above analysis, the large deformation of the assembly tunnel is located at the vault and the arch bottom, which is the main position for the stress release of the surrounding rock. The local stress at the transition position from the mushroom head arch to the straight wall on both sides is large, and significant stress concentration occurs at the wall corner. Due to the complex section form, the load model of the assembly tunnel is different from that of the conventional tunnel section. Therefore, the results of field monitoring and numerical simulation of contact stress are compared with the theoretical calculation. Due to the large buried depth of the assembly tunnel, the theoretical method adopts the 'Code for Design of Railway Tunnel' (Eqs. (2)~(5)) and the Protodyakonov's theory (Eqs. (6)~(7)). The horizontal load in the Protodyakonov's theory calculation adopts the Rankine formula (Eqs. (8)~(9)), and the calculation model is shown in Fig. 19.

$$q = \gamma h \quad (2)$$

$$h = 0.45 \times 2^{s-1} \omega \quad (3)$$

$$\omega = 1 + i(B-5) \quad (4)$$

$$e = kq \quad (5)$$

where q is the vertical load; γ is the weight of surrounding rock; B is the tunnel span; i is the coefficient related to B ; e is horizontal load; k is the horizontal load coefficient.

$$q = \gamma H \quad (6)$$

$$H = \frac{L}{2f} = \frac{B/2 + h_1 \tan \varphi}{f} \quad (7)$$

$$e = K_a(q + \gamma h_2) \quad (8)$$

$$K_a = \tan^2(45 - \varphi/2) \quad (9)$$

where H is the equilibrium arch height; L is the width of collapse arch; f is the rock firmness coefficient; h_1 is the buried depth of the tunnel; K_a is the Rankine earth pressure coefficient; h_2 is the depth of the straight wall; ϕ is the internal friction angle of surrounding rock.

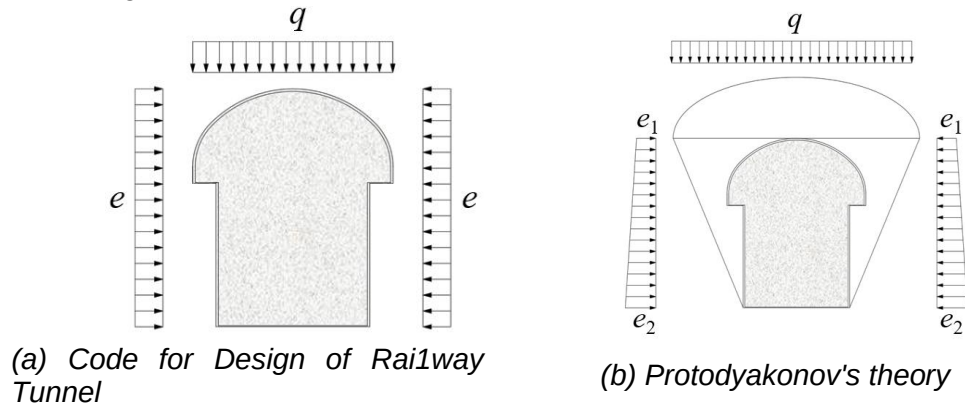


Fig. 19 – Load mode of theoretical method

Figure 20 is the surrounding rock pressure obtained by different methods, and the horizontal axis is the distance from the vault to the lower section wall corner. Based on the theoretical method, the surrounding rock pressure in the arch shows a trend of increasing first and then decreasing. However, the results of field monitoring and numerical simulation gradually increase. The numerical simulation and field monitoring results near the arch foot are larger, which are significantly higher than the theoretical method. The arch of the mushroom head is a large-span arched flat structure. Under the action of the upper load, the reverse thrust will be generated to squeeze the soil, and the effect at the arch foot is the most obvious. Compared with the circular and horseshoe-shaped sections, the vault stress of the flat elliptical section is smaller, and the stress of the upper half arch foot is larger. In addition, the smaller the flat rate is, the more significant it is, and the stress concentration occurs at the arch foot. This is the main reason why the difference between theoretical method, numerical simulation and field monitoring is more and more significant when the distance from the arch foot is closer. The surrounding rock pressure of the transition section from the arch foot to the side wall in the mushroom head decreases gradually, because the transition section with smaller curvature and vertical direction tends to produce deformation deviating from the surrounding rock, which weakens the effect of the reverse thrust of the arch structure continuing to transfer downward. In addition, the corner of the mushroom head and the lower section have significant stress concentration, and the larger value affects the comparison of the remaining positions, so it is not reflected in Figure 20. In the numerical simulation, the side wall of the lower section is not affected by the pressure of the surrounding rock except for the upper and lower ends. The field monitoring results are also significantly smaller, which is significantly different from the theoretical calculation results. This is because the upper and lower ends of the side wall are subjected to horizontal and vertical loads and are constrained. The middle part of the side wall has a large degree of deflection towards the tunnel, and the degree of squeezing the surrounding rock is weak. Based on experience and Rankine's formula, the characteristics of side wall under load and constraint cannot be considered, so there are differences with the simulation and measured results.

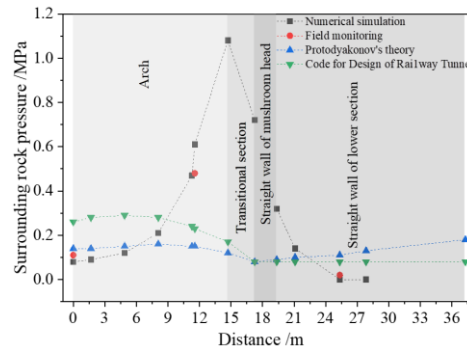


Fig. 20 – Comparison of surrounding rock pressure

Engineering proposal

Numerical simulation and field monitoring show that the stress at the arch foot and below the mushroom head is significantly greater than the theoretical results. To ensure the safety of the structure, support should be strengthened in engineering design, and monitoring should be strengthened in construction. Because the structure needs long-term bearing stress, it is appropriate to optimize the corner shape, such as optimizing the right angle into a rounded corner to weaken the stress concentration. Figure 21 is the distribution characteristics of structural mechanics after simulation optimization. The vertical deformation of the mushroom head decreases slightly, the horizontal displacement of the upper part on both sides of the lower section increases slightly, and the horizontal displacement of the lower part decreases slightly. Therefore, the contact stress of mushroom head increases slightly, and the bolt axial force decreases. The contact stress on the upper part of the lower section decreases, the bolt axial force the increases, and the axial force of the lower bolt decreases. The corner of the wall is transitioned from the corner, the displacement increases, and the contact stress decreases significantly, indicating that the corner can significantly weaken the stress concentration of the corner.

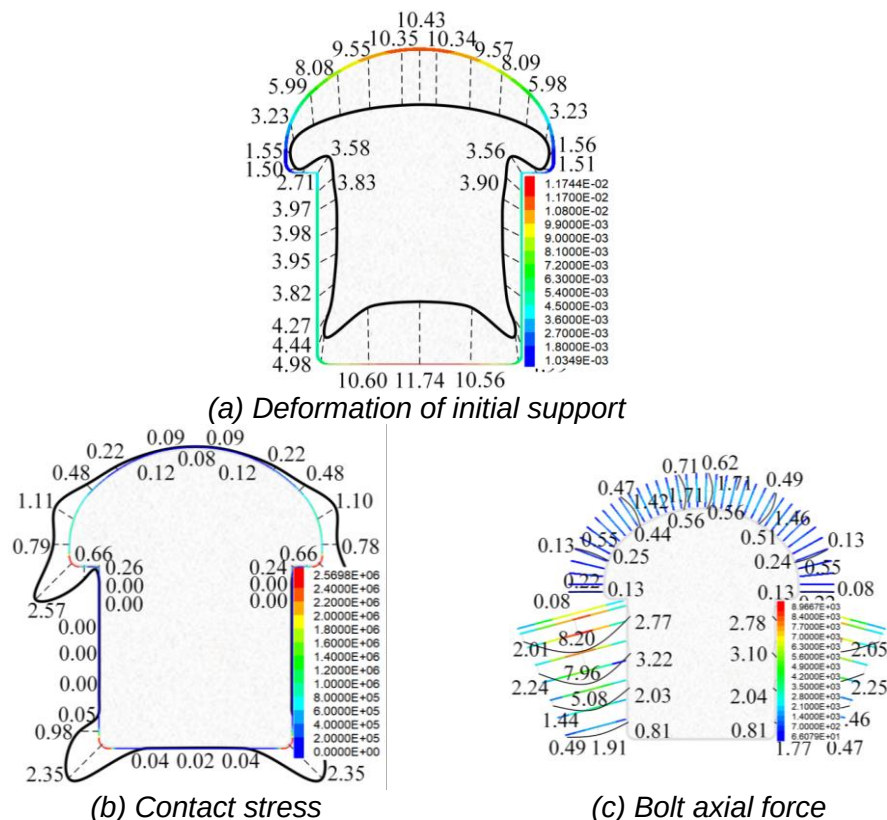


Fig. 21 – Mechanical characteristics of optimized section

In addition, the middle part of the side wall in the lower section is less loaded in the simulation and measurement. This does not mean that the structure is safe, but there is a possibility that the structure is in an unfavorable state due to excessive deflection. For the large buried deep straight wall, the structural deflection deformation should be focused on, and grouting reinforcement measures should be taken when necessary. Prestressed anchor cable, bolt encryption and other strengthening measures can also be used in the design.

CONCLUSION

Based on the Chongqing Yangtze River Tunnel, the mechanical behavior variation law and section distribution characteristics of the large-section mushroom-shaped shield assembly tunnel in different construction stages were explored. The specific conclusions are as follows:

- (1) The excavation of the lower section has little effect on the mechanical behavior of the mushroom head arch. The influence on the straight walls on both sides is slightly larger, and the influence gradually increases with the excavation depth.
- (2) The deformation control rate of the lower section is higher than that of the mushroom head, and the larger deformation position in the lower section moves down after the support, indicating that the mushroom head support can enhance the constraint ability of the lower section support to the surrounding rock deformation, and the enhancement effect gradually weakens downward.
- (3) The deformation of the initial support and the bolt axial force are positively correlated with the tunnel span. The vertical deformation of the initial support and the bolt axial force are positively correlated with the tunnel height, and the horizontal deformation is negatively correlated with the tunnel height. The contact stress of mushroom head arch is negatively correlated with the tunnel span, and the stress concentration of wall corner is positively correlated with the tunnel span. The contact stress on both sides of the mushroom head and the lower section is negatively correlated with the tunnel height.
- (4) The deformation control effect of DSH method is better, but the construction is slow and the cost is high. The method in this project with higher efficiency also meets the construction requirements. The stress of the arch foot and the corner is large, and the traditional theoretical calculation results are small. The design strength should be improved in engineering design, and the corner can be rounded to weaken the stress concentration. There is a possibility of excessive deflection towards the tunnel in the middle of the side wall of the lower section, and monitoring should be strengthened in the project.

Despite the achievements in numerical simulation and theoretical analysis, the scope of this study is primarily centered on specific geological conditions and engineering scenarios. The generalizability of the findings requires additional substantiation. Furthermore, existing models fall short in accurately depicting the load patterns of complex cross-sections, such as those with a mushroom shape. Future research endeavors should focus on exploring and developing more appropriate models to precisely characterize the load distribution and mechanical behavior of such intricate cross-sections. This advancement will contribute to providing a more reliable theoretical underpinning for relevant engineering designs.

ACKNOWLEDGEMENTS

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A STUDY ON THE EFFECT OF TWO DIFFERENT FOAMING AGENTS ON FOAM CONCRETE PROPERTIES

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ABSTRACT

Nowadays, foam concrete is widely used in construction. However, the structure and properties of foam concrete with a density $< 1200 \text{ kg/m}^3$ are greatly affected by different foaming agents. In this study, the effects of Foaming agent 1 and 2 on the properties of foam concrete are presented such as compressive strength, flexural strength, shrinkage, water absorption, thermal insulation and microstructure. Research results show that Foaming agent 1 has better stability and durability than Foaming agent 2; compressive strength of foam concrete using Foaming agent 1 is higher from $0.7 \div 1.1 \text{ MPa}$ and water absorption is lower than foam concrete using Foaming agent 2 from $2.1 \div 6\%$. In addition, the pore structure of the foam concrete was tested by electron microscopy (SEM). Foamed concrete using Foaming agent 1 showed a narrower pore size distribution in concrete than using Foaming agent 2. It can be proved that using Foaming agent 1 to produce foam concrete brings many practical meanings in the construction in Vietnam.

KEYWORDS

Foaming, Foam concrete, Foam stabilization, Mechanical properties, Pore structure

INTRODUCTION

In modern constructions, lightweight concrete with high thermal insulation is commonly used. Foamed concrete is lightweight concrete in which air voids are created by adding foam in a cement paste solution [1-3].

Foam concrete has excellent properties such as low heat conductivity, strong sound insulation, and low density (from 320 kg/m^3 to 1920 kg/m^3), which helps reduce the structure's self-weight. [1, 4]. Foam concrete widely used in insulating structures such as walls, roofs, etc. On the other hand, foam concrete has some disadvantages, such as low strength, strong dry shrinkage and water absorption, limiting its use in projects with high strength requirements [5, 6].

The properties of foam concrete vary depending on the foaming agent used, as each agent provides a different level of foam stability. Foam stability prevents the foam concrete mix from shrinking [7].

Currently, there have been many research works on foaming agents and foam concrete published. The compressive strength of foam concrete with a dry density of less than 1200 kg/m^3 is strongly affected by the type of foaming agent, according to Falliano D. et al. [8]. This type of substance also causes a change in the compressive strength, heat resistance and heat absorption coefficient of foam concrete [9]. According to a study [10], foam stability improved when silica nanoparticles were added to the foaming agent. Ameer A et al. [11] have shown that foam concrete's pore structure can be greatly affected by foam performance

In the field of construction materials science, the investigation into the characteristics and performance of foam concrete has garnered significant attention. A pivotal study conducted by Kearsley E et al. [12] has established a critical relationship between the pore structure of foam concrete and several of its key properties, including dry shrinkage, water absorption rates, and mechanical strength. This study underscores the importance of understanding the microstructural aspects of foam concrete to optimize its performance for various applications.

Expanding upon this foundation, research conducted by Ailar H et al. [13] further delves into the influence of pore size distribution within foam concrete on its overall performance metrics. Their findings reveal that specimens characterized by a highly uniform pore distribution exhibit markedly enhanced strength characteristics. This observation points towards the critical role that the internal pore structure plays in defining the mechanical resilience of foam concrete, suggesting that a targeted manipulation of pore distribution could be a viable strategy for material optimization.

A significant factor influencing the pore structure within foam concrete is the type of foaming agent used in its production. The foaming agent is responsible for the creation of voids within the concrete matrix, thereby defining the eventual pore structure and distribution. It follows that the selection of the foaming agent can have profound implications on the uniformity of pore distribution, which, in turn, affects the concrete's strength and other performance characteristics. Despite the apparent importance of foaming agents in determining the quality of foam concrete, there exists a notable gap in the literature concerning the comparative analysis of different types of foaming agents and their respective impacts on the performance of foam concrete.

Given this context, it becomes apparent that further research is required to systematically explore the effects of various foaming agents on the microstructural and performance attributes of foam concrete. Such studies would not only contribute to a deeper understanding of the material science underlying foam concrete but also pave the way for the development of more effective and durable construction materials. By elucidating the relationship between foaming agents, pore structure, and concrete performance, researchers can unlock new potential in the optimization of foam concrete for a wide range of structural and non-structural applications.

In the present study, an in-depth examination of foam concrete characterized by varying wet densities of 700, 900, and 1100 kg/m^3 is undertaken. The investigation focuses on the comparative analysis of two distinct types of foaming agents: Foaming agent 1 and 2, which are hypothesized to significantly influence the foam concrete's overall quality and performance, particularly given its low-density composition. The structural characteristics of these foaming agents were meticulously analyzed using Fourier Transform Infrared Spectroscopy (FTIR), aiming to elucidate their impact on the stability and strength of the produced foam. This research extends its scope to assess critical properties of foam concrete such as water absorption capacity, compressive strength, and the extent of dry shrinkage, which are pivotal in determining the material's suitability for construction applications. Furthermore, the study employs Scanning Electron Microscopy (SEM) techniques to scrutinize the pore structure within the foam concrete. This detailed analysis is intended to shed light on how variations in the pore structure, attributed to the differing properties of the utilized foaming agents, contribute to the observed disparities in the performance of foam concrete. Through this comprehensive approach, the research aims to provide valuable insights into the optimization of foam concrete formulations for enhanced performance in construction contexts.

MATERIALS AND METHODS

Materials

But Son Cemnet PC40 (C) in accordance with TCVN 2682-2019, $\rho = 3.10 \text{ g/m}^3$. The chemical composition of cement is presented in Table 1.

River sand Da Bac (SA) was used as fine aggregate with fineness modulus $M_k = 3.1$, $\rho = 2.65 \text{ g/m}^3$, relative density = 1620 kg/m^3 and bulk density = 1450 kg/m^3 .

Silica fume SF90, from Vietnam, has a density of 2.15 g/m^3 . Figure 1 illustrates the particle size distribution of Silica fume and cement.

Tab. 1 - Chemical composition of Portland cement and silica fume

Materials	Composition (%)							
	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Na ₂ O	LOI	Other
Portland cement	22.56	5.29	3.47	63.37	2.01	0.15	2.55	0.60
Silica fume	90.76	2.24	2.48	0.52	-	0.56	2.57	0.87

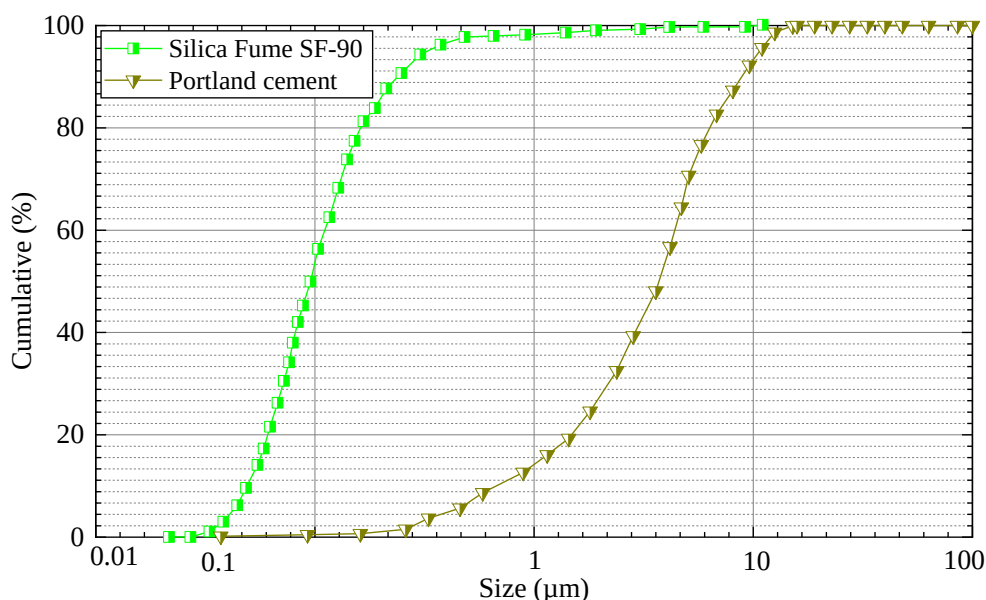


Fig. 1 – Particle size distribution of Silica fume and Portland cement

For the production of foam concrete, 'Foaming agent 1', supplied by a UK-based manufacturer, with a specific weight of 1.02 g/cm^3 , to be administered at a rate of 0.5 liters per cubic meter. Conversely, 'Foaming agent 2', produced by a chemical company in China, with a specific weight of 0.54 g/cm^3 and a dosage rate of 3.5 liters per cubic meter, as a relatively new foaming agent for foam concrete production that remains underutilized when compared to the popularity of 'Foaming agent 1' counterpart. There is no extant research on 'Foaming agent 2' within Vietnam. Figure 2 presents the FTIR spectrum of the foaming agent, displaying nearly identical peaks of -OH- and -CH- at 3300 cm^{-1} and 2700 cm^{-1} , respectively, but with the notable exception of -C=C- only apparent in 'Foaming agent 1'.

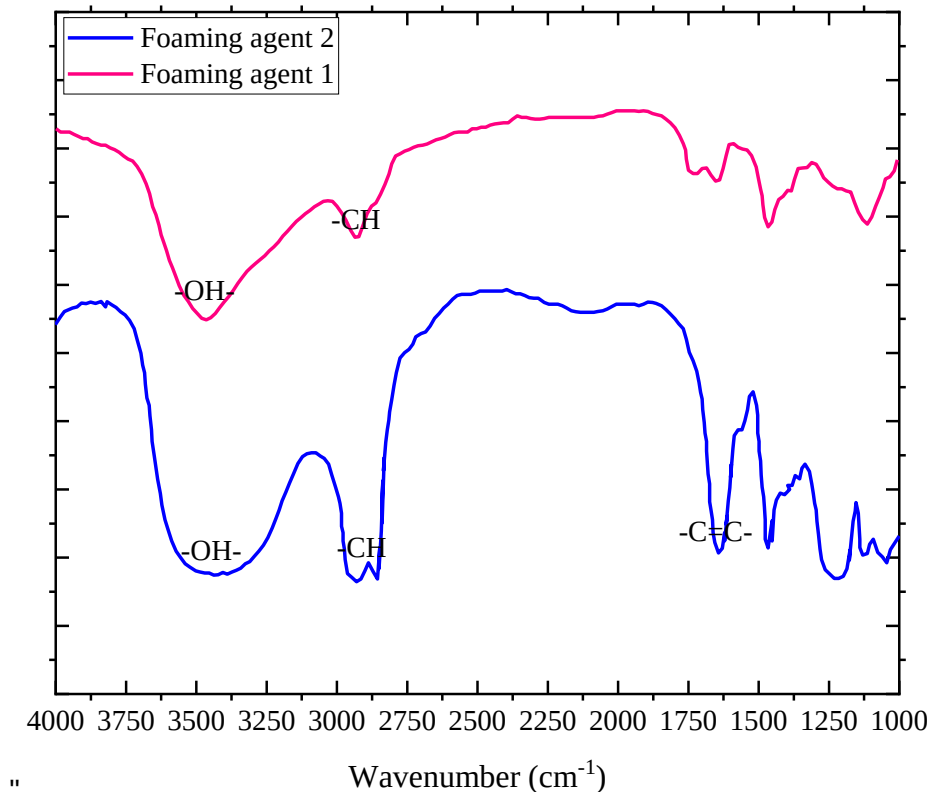


Fig. 2 – FTIR spectra of Foaming agent 1 and 2

Mixing water (W) in accordance with GOST 23732-2011 and TCVN 4506:2012 standards

Methods

- The particle size distribution of Portland cement and Silica fume was determined using the BRUKER laser particle analyzer.
- The composition of the foam concrete mixture was determined using the absolute volume method.
- Mechanical properties of foam concrete were assessed in accordance with the Russian standard GOST 10180-2012. Meanwhile, the GB/T 11969-2008 standard was used to gauge the dry shrinkage of foam concrete.
- Water absorption of foam concrete is tested according to TCVN 9030:2017.

Mixture ratio and sample preparation

In this study, the mixing ratio of foam concrete is given in Table 2. According to study [14], the Portland cement content $C=350 \text{ kg/m}^3$. In addition, the author used Silica fume to increase the strength of foam concrete $SF90/C= 0.1$ [15].

Tab. 2 - Mix proportions of foam concrete

No	Foaming agent	SF90/C	Dry density (kg/m ³)	Wet density (kg/m ³)	C (kg/m ³)	SA (kg/m ³)	SF90 (kg/m ³)	W (kg/m ³)	Foam (L/m ³)
1	SERIES I (Foaming agent 1)	0.1	663	700	350	123	35	192.5	633.9
2		0.1	852	900	350	323	35	192.5	558.4
3		0.1	1046	1100	350	523	35	192.5	482.9
1	SERIES II (Foaming agent 2)	0.1	661	700	350	123	35	192.5	633.9
2		0.1	850	900	350	323	35	192.5	558.4
3		0.1	1045	1100	350	523	35	192.5	482.9

The "MasterMix" mixer was used to blend dry Portland cement, sand, and silica fume at 50 rpm, for 1 to 3 minutes, at a temperature of 20°C, based on the mix composition outlined in Table 2. Subsequently, water was added and stirred for 2 minutes to produce a wet binder mixture. Simultaneously, the foam from the foam generator was introduced into the foam concrete mixer and mixed for 1 minute. The resulting foam concrete was immediately poured into a mold and left stationary for 24 hours, according to the specification. The foam concrete samples were cured in the laboratory for 24 hours, as illustrated in Figure 3b.



(a) Prepare foam concrete



(b) Sample curing

Fig. 3 – Foam concrete preparation process

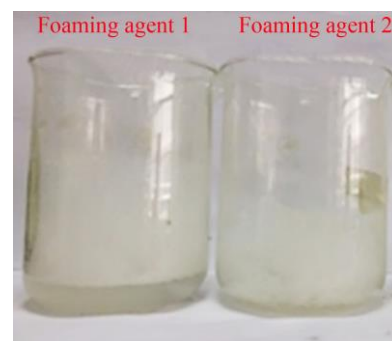
RESULTS AND DISCUSSION

Foam stability

In Figure 4, the air stability of 'Foaming agent 1' and 'Foaming agent 2' is displayed, with the average disappearance rate for 'Foaming agent 1' and 'Foaming agent 2' being 0.25 percent per minute and 0.5 percent per minute, respectively. 'Foaming agent 1' exhibits superior stability compared to 'Foaming agent 2' due to the correct proportioning of functional groups in the "Foaming agent 1". This allows organic chains to bond strongly through hydrogen bonding or even chemical bonding on the surface, leading to bubble formation in the aqueous medium [16]. Furthermore, 'Foaming agent 1' boasts unique stabilizers, including nanoparticles, which can agglomerate at the bubble-bubble interface to prevent collapse. Maintaining the foam structure in the cement paste can promote foam stability, according to research [17].



Initial state



After 100 minutes

Fig. 4 - Foam stability over time

Compressive strength and water absorption

Figure 5 depicts the speed at which the foam concrete's compressive strength develops and the relationship between compressive strength and curing age for the samples studied.

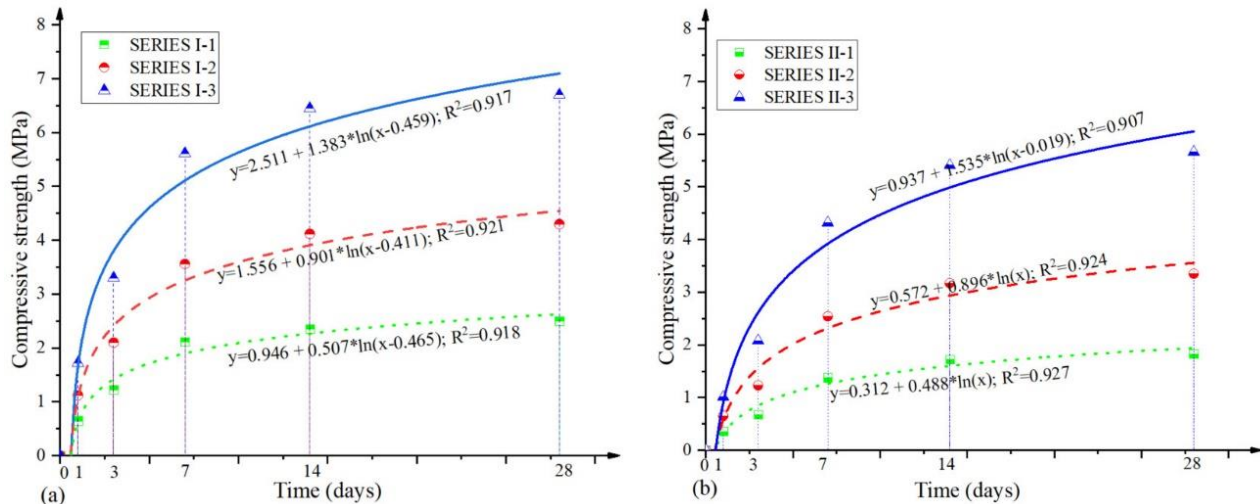


Fig. 5 - Compressive strength versus time for foam concrete: (a) foam concrete with 'Foaming agent 1'; (b) foamed concrete with 'Foaming agent 2'

The relation between compressive strength and curing age, as shown in previous studies [1], could have the following shape for foam concrete:

- + For SERIES I-1: $y = 0.946 + 0.507 \cdot \ln(x - 0.465)$ with $R^2 = 0.918$ (1)
- + For SERIES I-2: $y = 1.556 + 0.901 \cdot \ln(x - 0.411)$ with $R^2 = 0.921$ (2)
- + For SERIES I-3: $y = 2.511 + 1.383 \cdot \ln(x - 0.459)$ with $R^2 = 0.917$ (3)
- + For SERIES II-1: $y = 0.312 + 0.488 \cdot \ln(x)$ with $R^2 = 0.924$ (4)
- + For SERIES II-2: $y = 0.572 + 0.896 \cdot \ln(x)$ with $R^2 = 0.924$ (5)
- + For SERIES II-3: $y = 0.937 + 1.535 \cdot \ln(x - 0.019)$ with $R^2 = 0.907$ (6)

The impact of different foaming agents on the compressive strength of hardened foam concrete is depicted in Figure 5. The compressive strength of foam concrete using 'Foaming agent 1' is consistently higher than that produced using 'Foaming agent 2' at all ages. This can be attributed to the superior foam stability of 'Foaming agent 1' as compared to 'Foaming agent 2'.

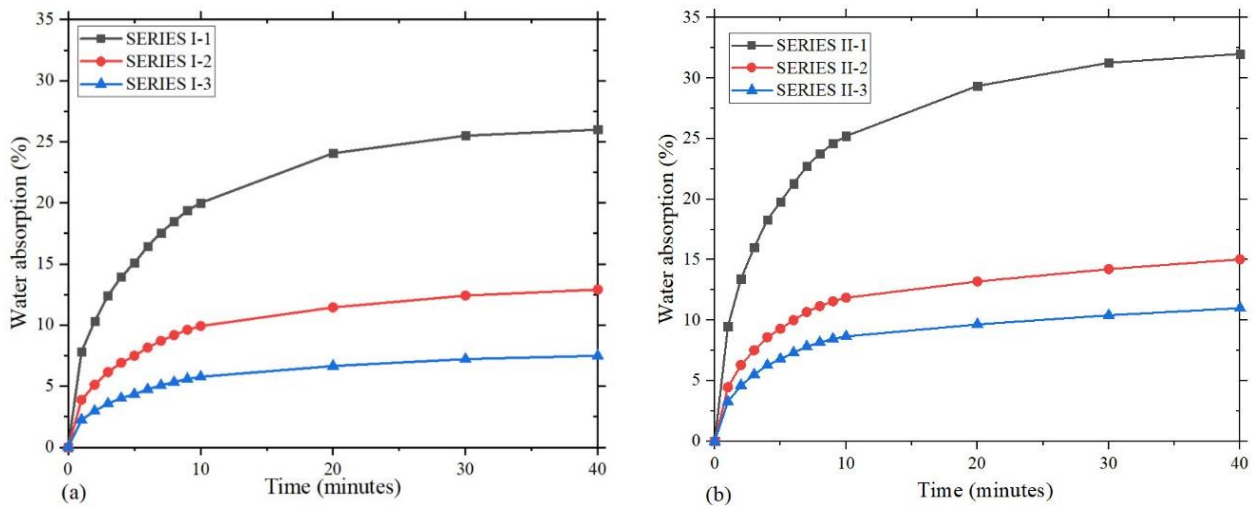


Fig. 6 - Foam concrete water absorption at different times. (a) foam concrete with 'Foaming agent 1'; (b) foamed concrete with 'Foaming agent 2'

The water absorption rate of 28-day curing foam concrete is shown in Figure 6. Figure 6 shows that the water absorption of foam concrete using 'Foaming agent 1' is lower than that of foam concrete using 'Foaming agent 2'. In addition, both forms of foam concrete exhibit quick water absorption in the first 10 minutes, after which the water absorption slows down. This observation is in agreement with the results reported by Ma C et al [18]. The finding suggests a close link between foam concrete's compressive strength and its water absorption, which can be attributed to the better foam stability of 'Foaming agent 1' compared to 'Foaming agent 2'. Consequently, the pore structure of foam concrete using 'Foaming agent 1' is more uniform, explaining its lower water absorption.

Dry shrinkage

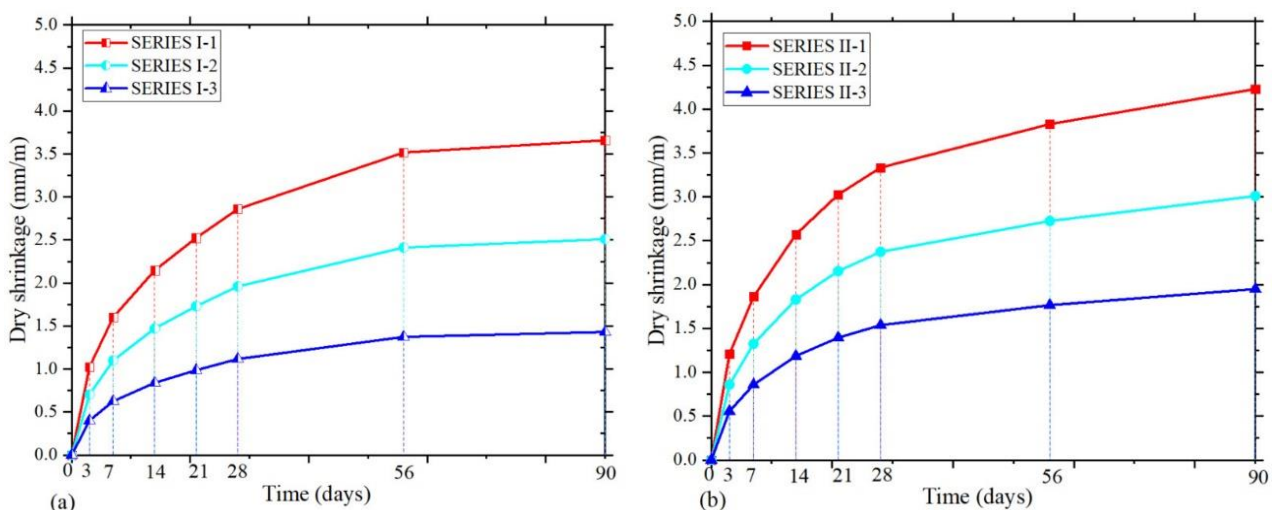


Fig. 7- Dry shrinkage of foam concrete at different times. (a) foam concrete with 'Foaming agent 1'; (b) foamed concrete with 'Foaming agent 2'

Figure 7 depicts the shrinkage of foamed concrete as it dries with various foaming agents. In the first 14 days, the dry shrinkage of foam concrete is significant. The dry shrinkage of foam concrete using 'Foaming agent 1' is lower than that using 'Foaming agent 2' by 2.1-6%. The difference in dry shrinkage of foam concrete with different foaming agents may be related to the pore

structure of the foam concrete, and decreased pore connectivity may help to reduce the dry shrinkage of foam concrete.

Pore structure

"In Figure 8, the microstructure of foam concrete (with a density of 900 kg/m^3) obtained through scanning electron microscopy (SEM) is displayed. It is evident that the pores in the foam concrete produced with 'Foaming agent 1' are smaller than those in the foam concrete produced with 'Foaming agent 2'. The difference in pore structure between foam concrete made with 'Foaming agent 1' and 'Foaming agent 2' is primarily attributed to the variation in average pore size and pore size distribution, rather than pore shape. Thus, these factors are the most critical microstructural properties that affect the strength of foamed concrete [19-21].

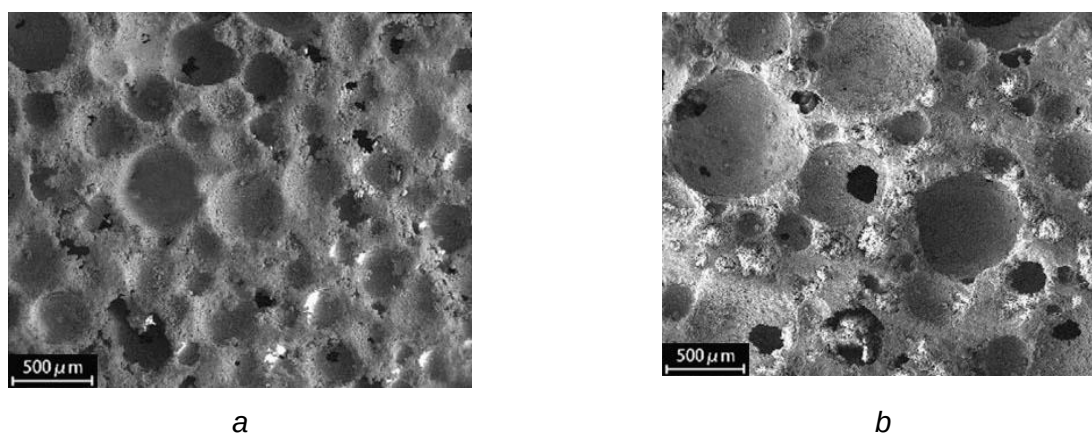


Fig. 8 - SEM image of foam concrete (a, SEM of foam concrete using 'Foaming agent 1'; b, SEM of foam concrete using 'Foaming agent 2')

CONCLUSIONS

The research findings suggest the following conclusions:

The Fourier Transform Infrared Spectroscopy (FTIR) analyses revealed that 'Foaming agent 1' and 'Foaming agent 2' possess analogous functional groups within their molecular structures. Nevertheless, the foam produced using 'Foaming agent 1' demonstrated superior stability and mechanical strength compared to that generated with 'Foaming agent 2', which can be attributed to the former's enhanced density and viscosity properties. Quantitative assessments indicated that the compressive strength of foam concrete with 'Foaming agent 1' was significantly higher, ranging from 16.4% to 28%, compared to foam concrete with 'Foaming agent 2'.

Further investigations into the hygrothermal performance of the materials revealed that foam concrete with 'Foaming agent 1' had a notably reduced water absorption rate, registering between 2.1% and 6%, relative to the foam concrete synthesized using 'Foaming agent 2'. In terms of dimensional stability, foam concrete with 'Foaming agent 1' exhibited a lower extent of drying shrinkage, measured at 1.43 to 3.66 mm/m, compared to the foam concrete with 'Foaming agent 2', which displayed values ranging from 1.95 to 4.23 mm/m.

The pore architecture of the foamed concretes was examined through Scanning Electron Microscopy (SEM), revealing that foam concrete with 'Foaming agent 1' was characterized by a more uniform pore size distribution and a reduced incidence of interconnected porosity compared to its 'Foaming agent 2' counterpart. These microstructural attributes were posited to significantly contribute to the observed disparities in mechanical and physical properties, particularly the enhanced strength and diminished water absorption capacities of the foam concrete with 'Foaming agent 1'. Such findings underscored the pivotal role of the foaming agent's physicochemical

properties in dictating the performance characteristics of foam concrete, thus providing crucial insights for the formulation and optimization of high-performance foam concrete composites.

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EXPERIMENTAL STUDY ON MECHANICAL PROPERTIES, FAILURE, AND DILATANCY BEHAVIOUR OF HIGH- AND ULTRA- HIGH -STRENGTH CONCRETE UNDER TRIAXIAL COMPRESSIVE STRESS

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ABSTRACT

To study the mechanical properties of high- and ultra-high-strength concrete, the conventional triaxial tests were carried out on four types of concrete with uniaxial strengths of 67.01, 109.34, 140.65, and 159.61 MPa under six levels of confining pressure. The influence on the failure mode, strength, deformability, and dilation properties of four kinds of concrete with different strengths by the confining pressure was studied. The test results show that: the cracks generated in high strength concrete always bypassed the coarse aggregate, and the percentages of aggregates bypassed were 85.7% and 70.5%, while in ultra-high-strength concrete, the cracks mainly passed through the coarse aggregate, fewer cracks bypass the aggregate, and the percentages of bypassed aggregate is 28.7% and 32.8%, respectively. With increasing confining pressure, the peak stress and stress of dilatancy onset of four types of concrete increase, and the corresponding strain at peak stress and dilatancy onset mount approximately linearly. The enhancement of the confining pressure on the peak stress, stress of dilatancy onset, strain at the peak stress and dilatancy onset in a negative relation to the uniaxial compressive strength of concrete. The peak volumetric strain of concrete increases as the confining pressure mounts. Under the same condition of confining pressure, the lower the uniaxial compressive strength is, the higher the volumetric strain is. The parameters were modified to make the volumetric model more suitable to describe the change law of volume for high- and ultra-high-strength concrete with a wider range of confining pressure levels. The failure envelopes of four kinds of concrete under uniaxial and triaxial stress states were obtained by using the nonlinear Mohr-Coulomb criterion.

KEYWORDS

High- and ultra-high-strength concrete, Triaxial compression, Failure modes, Peak stress, Dilatancy behaviour, Volumetric model

INTRODUCTION

As one of the most commonly used building materials, concrete is widely used in various structures. In actual concrete members, concrete is usually in a state of multiaxial stress. Since the 1930s, some researchers have paid attention to the law of strength and deformation of concrete under multiaxial stress. Richart et al. [1] conducted comprehensive studies on the failure of normal

strength concrete (NSC) under uniaxial, biaxial and triaxial compressive stresses and found that the compressive strength and deformability of NSC gradually had been improved by increasing of confining pressure. Afterwards, many scholars [2-5] also conducted a lot of studies on the mechanical properties of NSC under triaxial compression, and the results all showed that the confining pressure has a great influence on the compressive strength and deformability of NSC.

Since the 1990s, with the wide application of high-strength concrete (HSC), many scholars have begun to study the mechanical properties of HSC under multiaxial stress states. Xie et al. [6] conducted conventional triaxial tests on three types of HSC containing silica fume and the quartzite aggregates whose size are less than 14 mm. The results showed that the compression strength, residual compression strength and the deformability of HSC were significantly improved, but they were still less than those of NSC. Song and He [7, 8] conducted biaxial and triaxial compression tests on high strength concrete with the uniaxial compressive strength of 60 MPa. They studied its strength and deformation properties under different confining pressure levels. To determine the failure surface of high-strength concrete, a series of triaxial compression tests were carried out by Ansari and Li [9] on three types of concrete with uniaxial strength of 47, 71 and 107 MPa, respectively. According to the test results, the empirical formula between compression strength of concrete and confining pressure was obtained, as well as failure surfaces in principal stress space were established.

With the progress of technology on cement-based materials, the concrete was produced with uniaxial compressive strength exceeding 120 MPa, which can be called ultra-high-strength concrete (UHSC) and ultra-high-performance concrete (UHPC), and have widely been used in super tall buildings, offshore concrete structures, and long-span concrete structures. In the existing literature, most of the analyses were based on the mechanical properties of NSC and HSC under triaxial stress. While for UHSC, scholars frequently pay more attention to its properties under the uniaxial compression [10, 11], tensile conditions [12] and biaxial compression states [13, 14]. Similar to NSC and HSC, UHSC is usually under the triaxial stress states in the structure. Wang et al. [15] conducted a series of tests on UHSC without coarse aggregate and UHPC with steel fibers under uniaxial and triaxial compression. The result showed that the compression strength and deformability all increased as the confining pressure increased. The confining pressure has a slightly lower enhancement on the compressive strength of UHPC compared with UHSC. However, the failure of the UHPC was obviously lagging that of the UHSC because of the restraint effect of steel the fiber and confining pressure, showing more excellent ductility.

Although there have been certain studies on the mechanical properties of UHSC under triaxial stress to date, there are still few systematic quantitative studies on high-strength and ultra-high-strength concrete containing coarse aggregates. Besides, with the wide use of computer technology in various fields, various numerical methods have been applied to the design and analysis of building structures. The accuracy of the constitutive model of building structure materials has a marked impact on the accuracy of the prediction results of numerical methods. Most constitutive models were founded according to the test data of NSC and HSC in the past. Whether they could be applied in UHSC is still debatable. Therefore, obtaining a more accurate constitutive model and establishing corresponding failure criteria of UHSC to predict its performance under multiaxial stress is of great significance to fully understand and exert its superior mechanical properties [16-18].

In this paper, four kinds of concrete with different strengths were prepared, containing the same basalt coarse aggregate, and the uniaxial compression test and the conventional triaxial compression test under confining pressure of 5, 10, 20, 40, and 70 MPa were carried out, respectively. The mechanical properties of concrete under uniaxial and triaxial stress states were studied, and the peak stress, corresponding strain at the point of peak stress, stress and corresponding strain of dilatancy onset, as well as the failure modes of four concretes were compared at the same time. Based on the test data, the failure envelopes were obtained by using the nonlinear Mohr-Coulomb criterion, which was used to describe the mechanical properties of concrete under triaxial stress states.

MATERIALS AND METHODS

Four kinds of concrete are designed for testing, two of which are HSC. The water/binder ratios are 0.4 and 0.3, respectively, represented by HSC-1 and HSC-2; The other two are UHSC, and the water/binder ratios are 0.25 and 0.18, respectively, which are represented by UHSC-1 and UHSC-2.

MATERIALS AND MIXTURE PROPOTION

The constituent materials used for all concrete mixture proportions in this study included cement, quartz sand, basalt coarse aggregate, and water. Additionally, the quartz powder and water reducer were used to prepare HSC-2, UHSC-1, and UHSC-2; the FAQ defoamer was used to prepare HSC-1 and HSC-2; the silica fume was used to prepare UHSC-1 and UHSC-2. During the preparation process, the quartz sand and coarse aggregate are both saturated-surface-dry states. The mixture proportions are as follows in Table 1.

Tab. 1 - The mixture proportions (kg/m³)

Materials	HSC-1	HSC-2	UHSC-1	UHSC-2
Portland Cement P·O52.5R	480	550	440	470.
Quartz sand (0.16-0.63 mm)	590	450	490	470
Basalt coarse aggregate (3-10 mm)	1200	1200	1200	1200
Water	190	165	130	100
Quartz powder (<80 μm)	–	120	120	125
Silica fume (0.1 μm)	–	–	90	95
FAQ defoamer	0.85	0.75	–	–
Water reducer	–	3.75	4.50	16.50

Sample preparation

During sample preparation, firstly, put the materials described in Table 1 into a horizontal mixer and mix them evenly, and pour into the molds of 280 mm × 240 mm × 200 mm, then use the vibrating screed to eliminate large air pockets and bubbles in the concretes. The surface of the vibrated concrete was covered with plastic wrap and placed them into the standard curing room. The molds were removed after 48 hours. Before the test, the concretes were in the curing room for more than 519 days, and their mechanical properties can be considered stable. Before the test, the cylindrical core samples with the size of about Φ 49×180 mm were taken from the concrete blocks by drill machine, a cutting machine and a double-end grinder were used to process the samples into Φ 49×120 mm cylindrical samples. The unevenness of the end face is no more than 0.1 mm, and the error of diameter and height is no more than 0.4 mm.

Test equipment and test methods

The XTR01 high-pressure triaxial test machine was used to carry out experiments, which is mechanically loaded in the axial direction, with a maximum load of 2000 kN, and hydraulically loaded in the lateral direction, with a maximum confining pressure of 100MPa. During the test, the actual axial pressure stress of the sample is $\sigma_c = q + p$, where p is confining pressure, and q is differential stress. The schematic diagram of the force on the sample is shown in Figure 1.

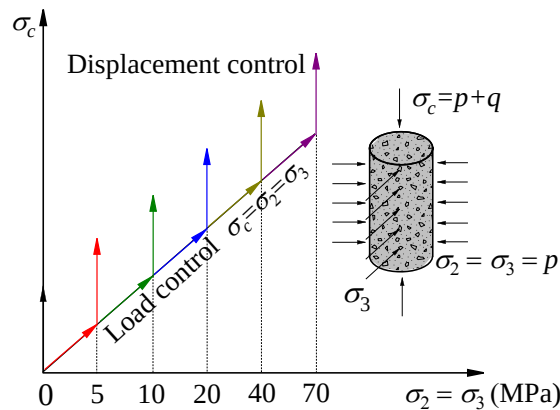


Fig.1 – Sample force diagram and load path

The symbols of stress and strain involved in this paper are positive for compression and negative for tension. ε_r represents the average radial strain, and its expression is $\varepsilon_r = (\varepsilon_2 + \varepsilon_3)/2$, where ε_2 and ε_3 represent the two radial strains respectively in the perpendicular direction at the same height of the sample; ε_c represents the axial strain. Figure 2 is the installation of extensometers.

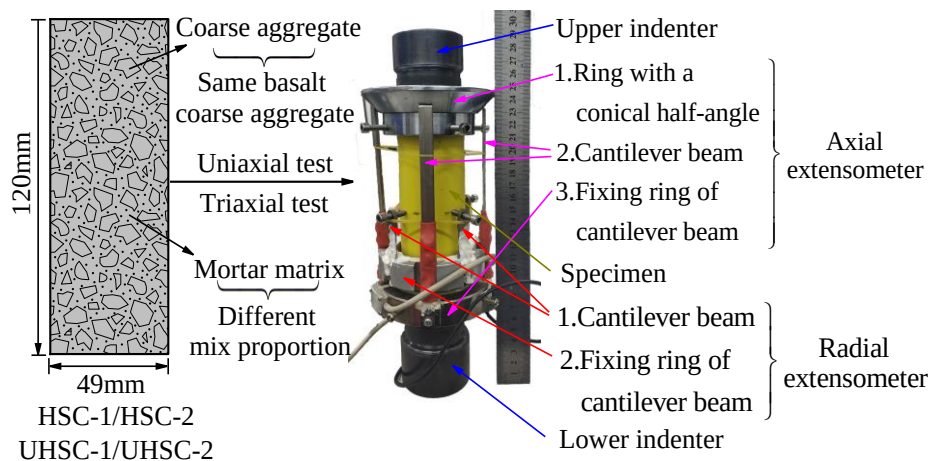


Fig.2 – Installation of extensometers

In the initial phase of the experiment, the sample is firstly preloaded, so that upper indenter could be mildly adjusted in position with the end face of the sample. In this way, the lower and the upper end surfaces of the indenter are in completely contact with the upper end surface of the sample and the lower end surface of the plunger, respectively, so as to avoid eccentric force on the sample. In the uniaxial compression test, the axial displacement is applied with the constant rate of 0.001 mm/s after preloading. In triaxial test, the lateral confining pressure is applied with the rate of 0.1 MPa/s after completion of preloading. When the confining pressure reaches the design values, the lateral confining pressure was kept constant for about 5 minutes, and then the axial displacement is applied monotonously with the rate of 0.001 mm/s. Two samples of the same type are used for the test under each confining pressure.

TEST RESULTS

Figure 3 shows the differential stress-strain curves of four concretes under various confining pressures. For the same type of samples, the straight-line sections of curves have an almost constant slope before the peak point, indicating that the elastic modulus of the four kinds of concrete is not greatly influenced by confining pressure.

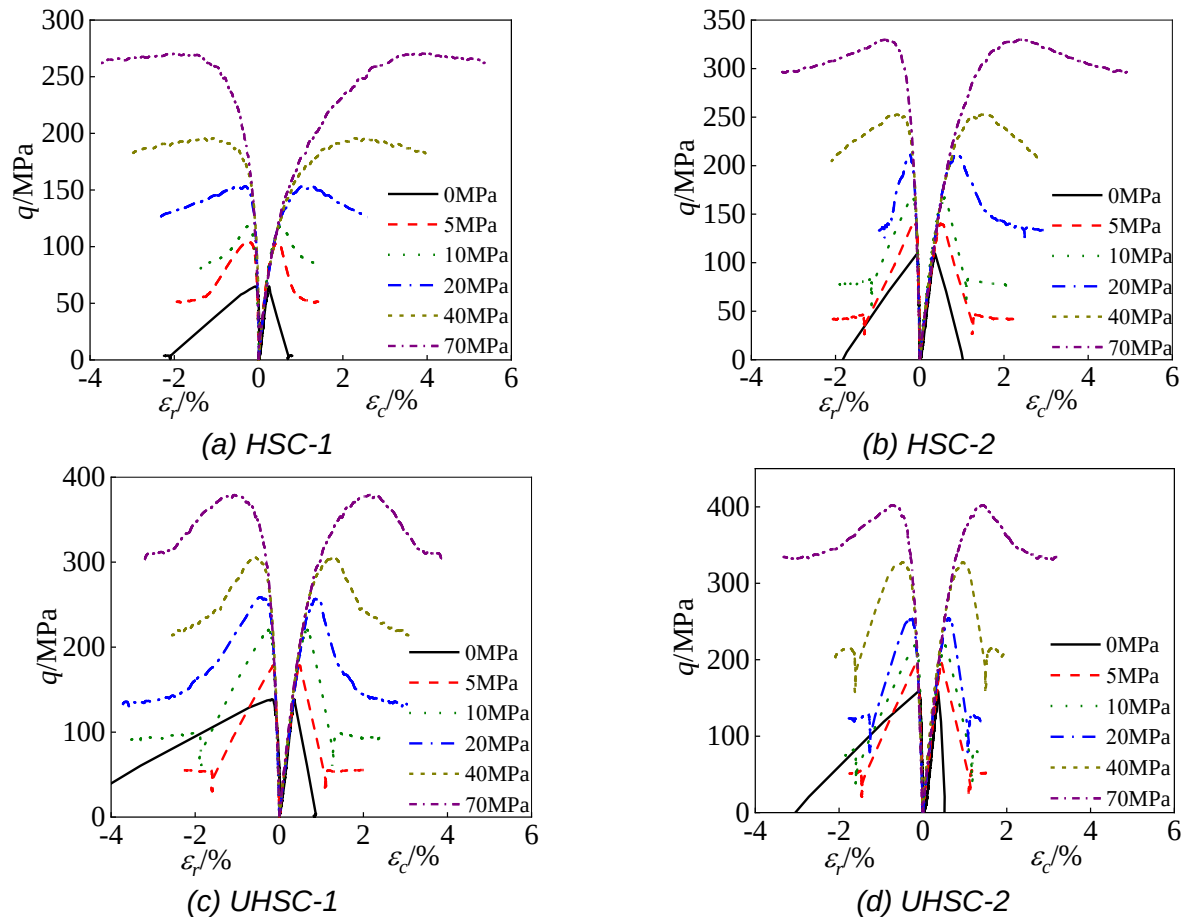


Fig.3 – Differential stress-strain curves of concrete under different confining pressures

At low confining pressure, the differential stress of all samples increases linearly with the increase of strain within a large stress range before the peak point. The deviatoric stress falls faster after passing the peak point with the deformation suddenly increases, which is maybe caused by the insufficient stiffness of the testing machine. Under the high confining pressure, the curve rises linearly just in a small range of stress, and the differential stress falls slowly after the peak point. At the same time, under the same confining pressure, the higher the uniaxial strength of concrete, the sharper the peak of the curve and the faster the stress drop after the peak, showing more brittle characteristics. The mean values of peak stress and corresponding strain at peak stress of four kinds of concrete under six levels of confining pressures are listed in Table 2. σ_{cc} and ε_{cc} represent the peak stress and corresponding strain at peak stress of concrete, respectively.

Tab. 2 - Triaxial compression test results

p/MPa	HSC-1		HSC-2		UHSC-1		UHSC-2	
	σ_{cc} /MPa	ε_{cc} /‰	σ_{cc} /MPa	ε_{cc} /‰	σ_{cc} /MPa	ε_{cc} /‰	σ_{cc} /MPa	ε_{cc} /‰
0	67.01	0.285	109.34	0.322	140.65	0.363	159.61	0.382
5	111.64	0.482	160.12	0.498	188.14	0.512	203.53	0.444
10	129.74	0.559	181.25	0.607	225.16	0.641	234.36	0.536
20	170.98	1.135	234.48	0.847	281.24	0.892	289.63	0.610
40	235.35	2.268	306.46	1.495	347.81	1.273	367.07	0.977
70	345.18	3.916	395.35	2.435	447.62	2.109	471.04	1.427

Figure 4 shows the failure modes of samples in triaxial compressive test. It is found that as the confining pressure rises, the failure modes of the same samples show the trend that splitting failure at uniaxial compression, an oblique shear failure occurs at low confining pressure, while to extrusion flow failure appears at high confining pressures.

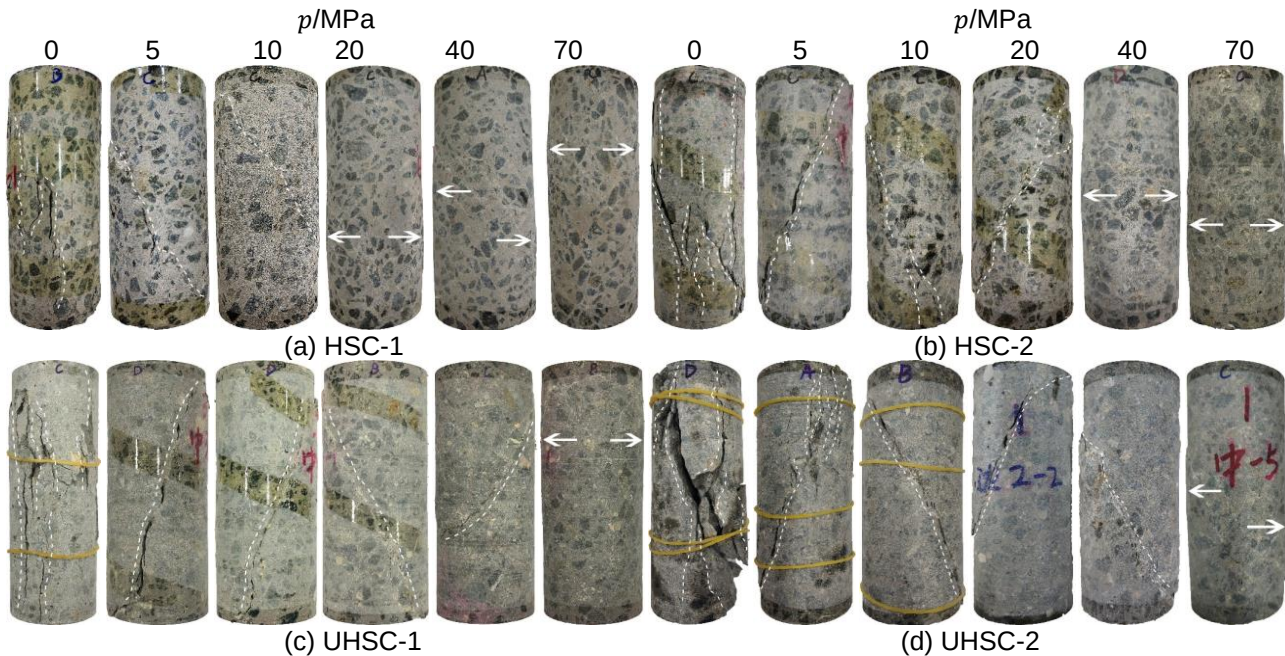


Fig.4 – Failure modes of samples at different confining pressures

The failure modes of the four kinds of concrete are all oblique shear failure when the confining pressure is 5, 10 MPa, and a diagonal crack appears on the surface of the samples. When the confining pressure is 20 MPa, the failure mode of HSC-1 is extrusion flow failure, many small irregular cracks are formed on the surface of the middle part of the sample, and evident bulging occurs, while the failure modes of the other three kinds of concrete are still oblique shear failure. When the confining pressure is 40 MPa, the failure modes of HSC-1 and HSC-2 are both extrusion flow failure, and there are many small irregular cracks in the middle of the sample with obvious bulging, while the failure modes of UHSC-1 and UHSC-2 are still oblique shear failure, while diagonal shear crack is less obvious than that of low confining pressure. When the confining pressure is 70 MPa, the failure modes of the four samples are all extrusion flow failure. The reason for the different failure modes of the same sample is that the cracks generated in the sample are not easy to expand because of confining pressure. A through crack can be generated at a low confining pressure level, which makes the bearing capacity of the sample drop suddenly, and the sample shows oblique shear failure; while under the high stress level, the confining pressure significantly inhibits the crack propagation; the cracks in the sample gradually increase, but it is not easy to connect, which makes the axial stress of the sample decline slowly, and finally forms the extrusion flow failure.

Figure 5 summarizes the trend of cracks on the surface of samples after the failure. The aggregates indicated by the red arrow are the aggregates bypassed when the crack propagates, and the aggregates indicated by the green arrow are the aggregates traversed by the propagation of the cracks.

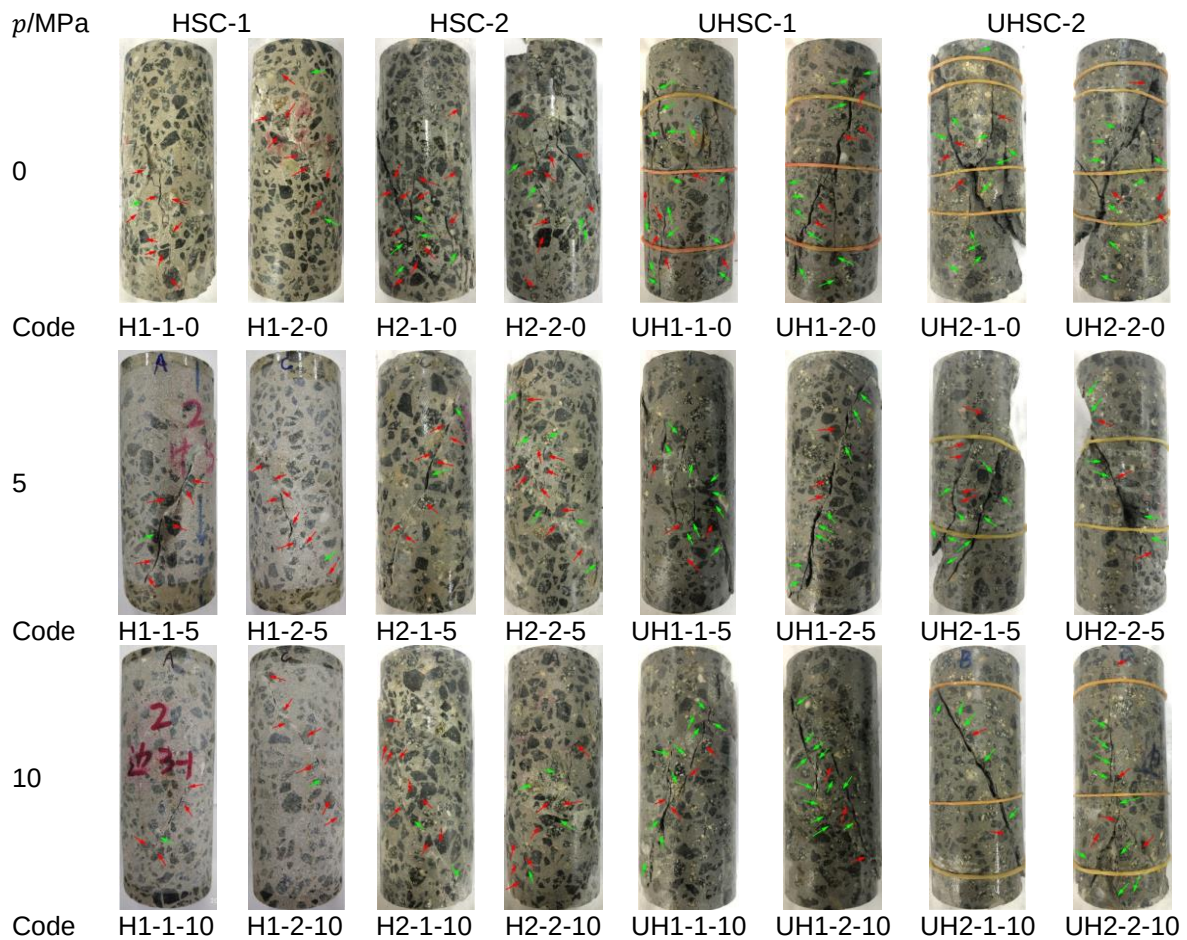


Fig.5 – Detailed view of the crack path on the surface of the concrete

Table 3 shows the number of coarse aggregates that are bypassed and traversed by the cracks, where I and T represent that crack bypassing aggregate and transferring aggregate, respectively.

Tab. 3: Statistics of the number of cracks passing through aggregate and bypassing aggregate

Code	I	T	Code	I	T	Code	I	T	Code	I	T
H1-1-0	9	1	H2-1-0	12	5	UH1-1-0	6	14	UH2-1-0	4	11
H1-2-0	8	2	H2-2-0	9	4	UH1-2-0	5	10	UH2-2-0	4	8
H1-1-5	6	1	H2-1-5	7	2	UH1-1-5	4	8	UH2-1-5	5	8
H1-2-5	8	1	H2-2-5	10	6	UH1-2-5	3	8	UH2-2-5	3	6
H1-1-10	4	1	H2-1-10	9	2	UH1-1-10	4	10	UH2-1-10	2	5
H1-2-10	7	1	H2-2-10	8	4	UH1-2-10	3	12	UH2-2-10	5	9
Total	42	7	Total	55	23	Total	25	62	Total	23	47
(%)	85.7	14.3	(%)	70.5	29.5	(%)	28.7	71.3	(%)	32.8	67.2

As can be found from Figure 5 and Table 3 that for the HSC-1 and HSC-2, no matter which failure modes the samples are, the cracks mostly choose to bypass the aggregate, and the cracks mainly occur in the matrix or extend along the interface between the mortar and aggregate, the paths of cracks are relatively tortuous and irregular; the fracture is mainly caused by the damage of the mortar matrix. This because the matrix strength is lower than coarse aggregate strength, according

to the principle of minimum energy, the crack always selects the path of less energy consumption to expand, and the aggregate hinders the expansion of cracks in concrete, which makes the cracks only propagate in the area without coarse aggregate. This failure form can be called intergranular failure [19]. For UHSC-1 and UHSC-2, whether in uniaxial compression or triaxial compression, the cracks generated by the failure mostly choose to pass through the coarse aggregates when they expand to the aggregates, forming a relatively continuous and smooth crack path. The failure of the concrete is mainly caused by joint damage of matrix and aggregate, this is mainly because that the strength of matrix has little different from coarse aggregate in the UHSC, and the hindrance of the aggregate on the crack propagation is not obvious. This failure pattern was a transgranular failure [19].

According to the different forms of failure modes, the three failure modes are divided into three types, which can be called type I, II and III, respectively. To analyze the influence of confining pressure on the failure modes of concrete, the confining pressure was normalized by uniaxial compression strength. The distribution of failure modes in the coordinate plane of confining pressure level and uniaxial strength was shown in Figure 6. The relationship between failure mode and confining pressure level of UHSC in the text of Zhou [16] is referred to in the figure. The dotted line was the boundary of different failure types.

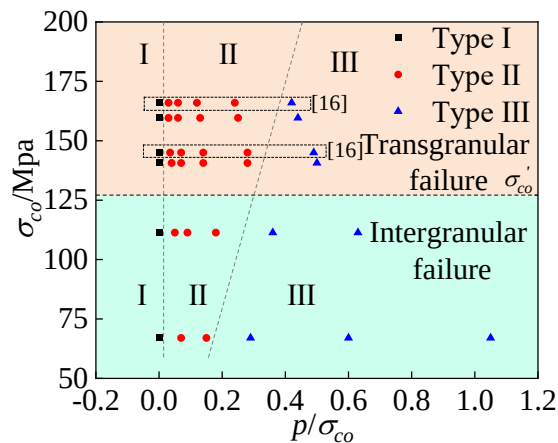


Fig.6 – Distribution of failure modes of concrete under different confining pressure levels

From Figure 6, it can be found that the uniaxial compression strength has little effect on the failure mode of concrete. Four kinds of concrete showed vertical splitting failure under uniaxial compression. However, as the strength increases, the larger confining pressure level was required for the transformation from oblique shear failure to extrusion flow failure. Through the analysis of the relationship between the path of crack propagation and the position of aggregate, there is a critical strength σ'_{co} exists, when $\sigma_{co} \leq \sigma'_{co}$, the failure morphology of concrete is an intergranular failure, and when $\sigma_{co} > \sigma'_{co}$, the failure morphology of concrete is a transgranular failure, the value of σ'_{co} is related to the strength of coarse aggregate in concrete, the value range in this article is $109.34 \text{ MPa} \leq \sigma'_{co} < 140.65 \text{ MPa}$. Figure 7 shows a schematic diagram of the failure morphologies of the intergranular failure and the transgranular failure in the three failure modes.

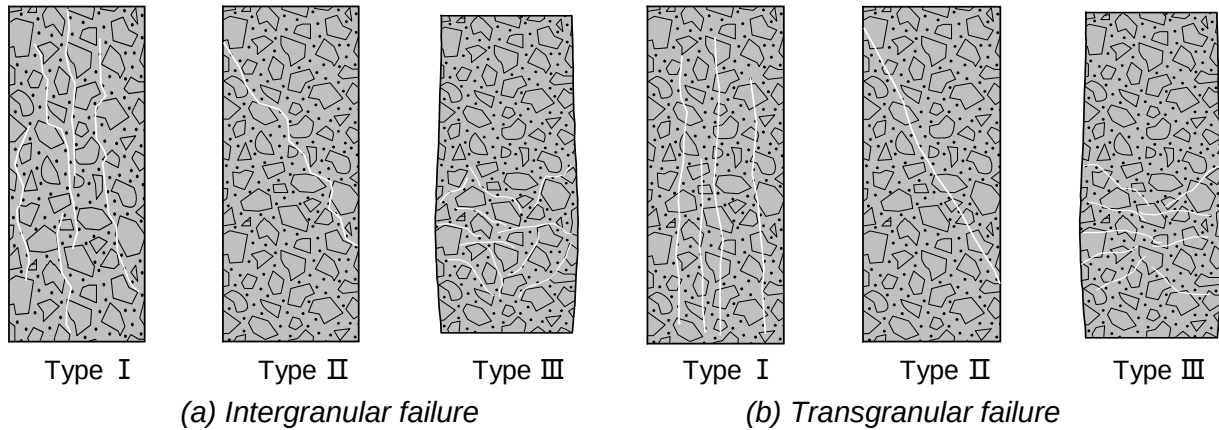


Fig.7 – Schematic diagram of failure morphologies

DISCUSSION AND ANALYSIS

Peak stress and axial strain at peak stress

The peak stress and corresponding strain at peak stress of four kinds of concrete were normalized by uniaxial compression strength and corresponding uniaxial strain at peak stress. The relationship of $\sigma_{cc}/\sigma_{co}-p/\sigma_{co}$ and $\varepsilon_{cc}/\varepsilon_{co}-p/\sigma_{co}$ were drawn. To compare the influence on the compression strength and axial deformability of four concretes by the confining pressure level, the experimental data were fitted by the Eqs. (1) and (2). The corresponding fitting curves were shown in Figures 8 and 9.

$$\frac{\sigma_{cc}}{\sigma_{co}} = \sqrt{1 + k \frac{p}{\sigma_{co}}} \quad (1)$$

$$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + b \frac{p}{\sigma_{co}} \quad (2)$$

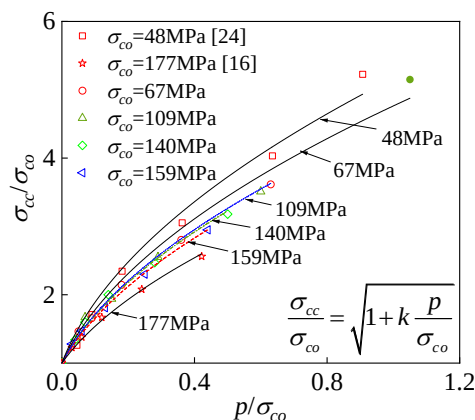


Fig.8 – Relationship between $\sigma_{cc}/\sigma_{co}-p/\sigma_{co}$

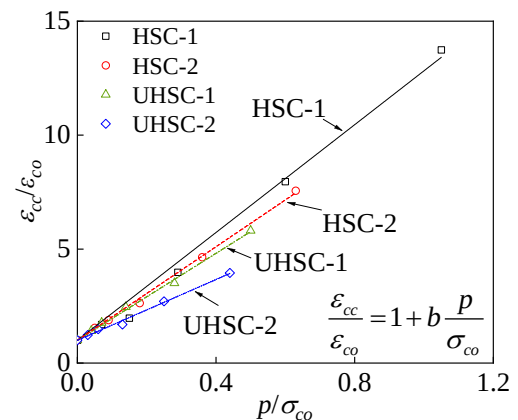


Fig.9 – Relationship between $\varepsilon_{cc}/\varepsilon_{co}-p/\sigma_{co}$

where k and b are the parameters representing the rate of compression strength growth and axial strain growth with confining pressure. The values of parameters k and b in relation to the type of concrete [16, 24] and are shown in Table 4, where R^2 refers to the coefficient of determination.

Tab.4: The fitting parameter values for curves in Figures 8 and 9

Type	Imran [24]		HSC-1		HSC-2		UHSC-1		UHSC-2		Zhou [16]	
Parameters	k	b	k	b	k	b	k	b	k	b	k	b
Value	25.75	-	24.11	11.82	19.32	10.26	18.81	9.54	17.53	6.67	13.64	-
R^2	0.98	-	0.98	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.96	-

As can be seen from Figures 8 and 9 that the peak stress σ_{cc} gradually increases nonlinearly as the confining pressure rises, while the strain at peak stress ε_{cc} increases approximately linearly as confining pressure increases. Confining pressure has a conducive influence on improving the strength and deformability of concrete. It also can be seen that whether it is HSC or UHSC, with the increase in confining pressure, the lower the σ_{co} of concrete, the higher the growth rate of the σ_{cc} of concrete, which shows that the enhancement on the compression strength of the relatively low-strength concrete by confining pressure is greater than that of the relatively high-strength concrete. Similarly, the lower the σ_{co} of concrete, the higher the growth rate of axial deformation with confining pressure, which indicates that the enhancement on the deformability of the relatively low-strength concrete by confining pressure is greater than that of the relatively high-strength concrete. In summary, the enhancement effect on the strength and axial deformation of concrete in triaxial compression test are all in negative relation to the uniaxial compressive strength.

Stress and axial strain of dilatancy onset

Figures 10 and 11 show the normalized relation between the stress, axial strain of dilatancy onset and the confining pressure level, respectively

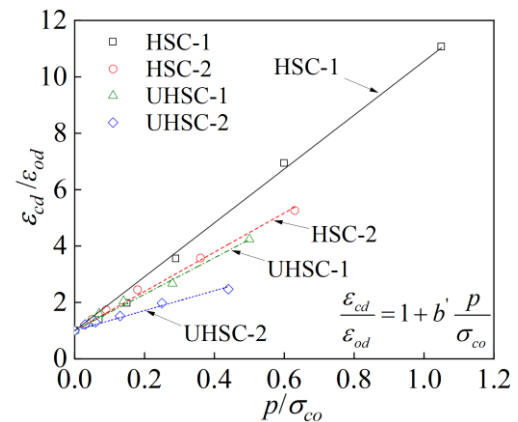
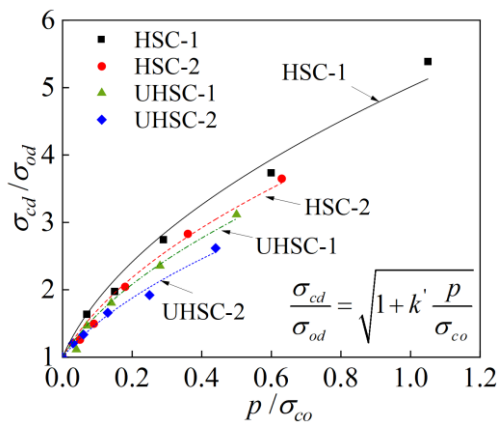


Fig.10 – Relationship between p/σ_{co} and σ_{cd}/σ_{od} Fig. 11 – Relationship between p/σ_{co} and $\varepsilon_{cd}/\varepsilon_{od}$

where σ_{od} and ε_{od} represent the stress and axial strain of dilatancy onset under uniaxial test, σ_{cd} and ε_{cd} are the stress and axial strain of dilatancy onset under triaxial test, respectively. The test data are fitted with Eqs. (3) and (4), respectively, and the fitting curves obtained are also shown in Figures 10 and 11. The values of parameters k' and b' are shown in Table 5.

Tab.5 - The fitting parameter values for curves in Figures 10 and 11

Type	HSC-1		HSC-2		UHSC-1		UHSC-2	
Parameters	k'	b'	k'	b'	k'	b'	k'	b'
Value	21.69	9.55	18.81	6.95	16.64	6.45	12.60	3.54
R^2	0.98	0.97	0.99	0.99	0.98	0.97	0.98	0.97

$$\frac{\sigma_{cd}}{\sigma_{od}} = \sqrt{1 + k' \frac{p}{\sigma_{co}}} \quad (3)$$

$$\frac{\varepsilon_{cd}}{\varepsilon_{co}} = 1 + b' \frac{p}{\sigma_{co}} \quad (4)$$

Similar to the variation law between peak stress, corresponding strain at peak stress and confining pressure, the stress and axial strain of dilatancy onset of four kinds of concrete increase gradually as the confining pressure rises. The stress of dilatancy onset increases nonlinearly as the confining pressure rises, while the axial strain of dilatancy onset increases approximately linearly, which also indicates that the enhancement of the confining pressure on the stress and the strain of dilatancy onset in a negative relation to the uniaxial strength.

Volumetric model of concrete

Figure 12 shows the curves of volumetric strain versus axial strain of four kinds of concrete under different confining pressures.

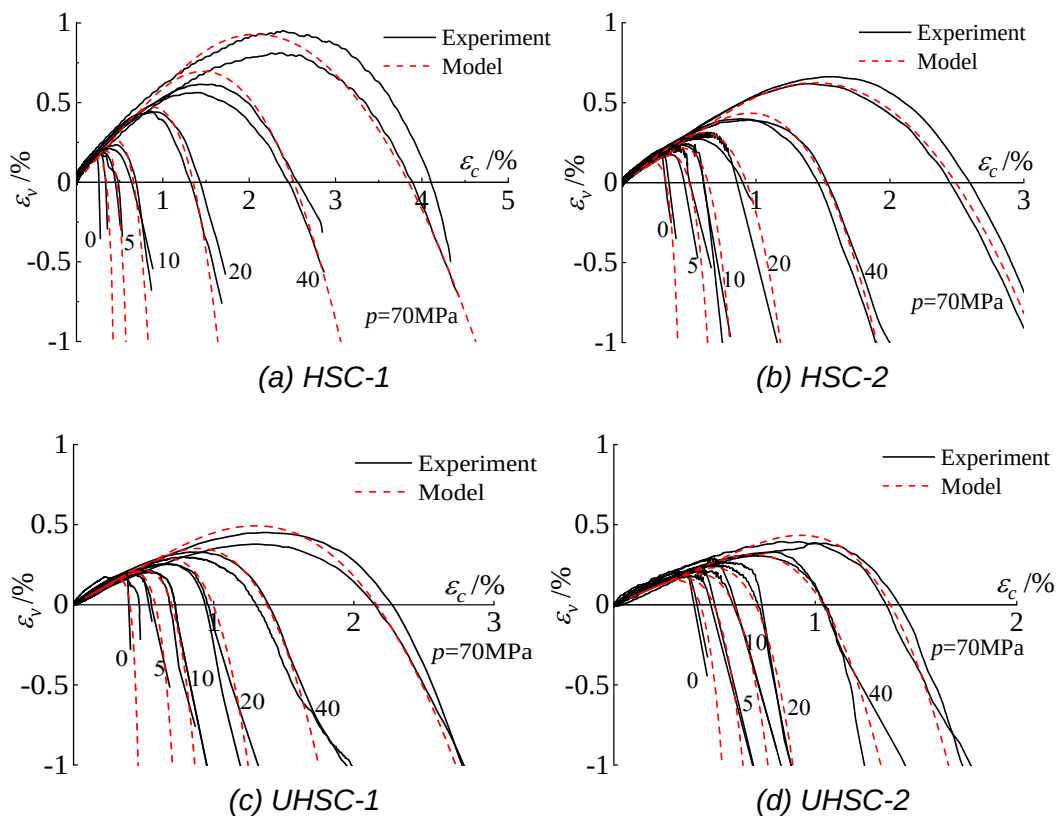


Fig.12 – Comparison of experimental and model values of volumetric strain

From Figure 12, it can be found that the volumetric strain increases first and then decreases as the axial strain increases. For the same types of concrete, the peak volumetric strain gradually mounts with the increase in confining pressure. For the different types of concrete, the peak volumetric strain of relatively low-strength concrete is higher than the relatively high-strength concrete under the same confining pressure.

Concrete exhibits lateral dilation or expansion behavior due to the Poisson effect under the axial pressure. Many scholars have established mathematical models to describe the expansion behavior of normal concrete under uniaxial compression according to the relationship between axial deformation and lateral deformation [21-23]. Imran and Pantazopoulou [24] proposed the concept of volumetric strain when describing this behavior, and established a mathematical model of the relationship between volumetric strain and axial strain of normal concrete under uniaxial compression. At the condition of triaxial compression, confining pressure can inhibit the initiation and

propagation of cracks in concrete, which has a greater impact on the change of volume of concrete. Based on the original model, Sheikh [25] proposed the volumetric model of normal and high-strength concrete under triaxial compression. This model assumes that the volumetric strain of concrete in the initial stage of compression increases approximately linearly with the increase of axial strain and changes in a parabolic shape after the concrete cracks. The general form of the model under triaxial compression can be express as follows:

$$\varepsilon_v = (1 - 2\nu) \left[\frac{2p}{E_c} + \varepsilon_c^{v0} \left(\frac{\varepsilon_c}{\varepsilon_c^{v0}} - d \left[\frac{(\varepsilon_c - \varepsilon_c^{\text{lim}})}{\varepsilon_c^{v0} - \varepsilon_c^{\text{lim}}} \right]^e \right) \right] \quad (5)$$

where ν is Poisson's ratio, and ε_c^{v0} represents the axial strain corresponding to zero volumetric strain, its value is related to ε_{cc} and can be expressed as $\varepsilon_c^{v0} = m\varepsilon_{cc}$, the value of parameter m mainly depends on the uniaxial compressive strength of concrete. The expression of m given in [25] is as follows:

$$0.65 \leq m = \frac{\sigma_{co}}{(\sigma_{co} - 50) + 40} - 0.1 \leq 1.1 \quad (6)$$

$\varepsilon_c^{\text{lim}}$ in Eq. (5) is the axial strain corresponding to concrete cracking in the lateral direction and can be expressed as:

$$\varepsilon_c^{\text{lim}} = \frac{1-\nu}{\nu E_c} p - \frac{\varepsilon_{cr}}{\nu} \quad (7)$$

where ε_{cr} can be calculated from the splitting strength σ_{cr} and determined by the expression in [25]:

$$\varepsilon_{cr} = \frac{\sigma_{cr}}{E_c} = \frac{0.387 \sigma_{co}^{0.63}}{E_c} \quad (8)$$

The McCauley brackets $\langle \rangle$ in Eqs. (5) and (6) are defined as $\langle x \rangle = 0.5[x + \text{abs}(x)]$. Through observation and analysis, it can be known that the values of parameters d and e are mainly controlled by the shape of the $\varepsilon_v - \varepsilon_c$ curve. The value of parameter d increases as the confining pressure level mounts, which controls the size of shape of the curve, while the value of parameter e decreases with the increase in confining pressure level, which is related to the steepness of the peak of the curve. The parameters d and e in Eq. (5) are defined in [25], and the expressions are given as follows:

$$d = 1 - \frac{p}{\sigma_{co}} \geq 0.7 \quad (9)$$

$$e = \frac{\sigma_{co} - p}{30} \geq 2.0 \quad (10)$$

It can be seen from Eqs. (9) and (10) that they are only applicable to the conditions where the confining pressure level $p/\sigma_{co} \leq 0.3$ and the difference between strength and confining pressure is greater than 60MPa. Therefore, expressions of d and e were improved in this paper, which can satisfy a wider range of confining pressure levels:

$$d = \sqrt{\frac{p/\sigma_{co} + 0.11}{0.8}} \quad (11)$$

$$e = \frac{2.23}{d} \quad (12)$$

In this paper, the value ranges of the two parameters were $0.34 \leq d \leq 1.21$ and $1.84 \leq e \leq 6.56$, respectively. Figure 12 shows the comparison between the experimental curves and predicted curves obtained by the model of volumetric strain versus axial strain. The comparison shows relatively good agreement between experimental and predicted results, which indicates that the improved model can better describe the variation law of volumetric strain of concrete with confining pressure level range of 0-1.05 and strength range of 67 MPa-160 MPa.

FAILURE CRITERIONS

Mohr-Coulomb criterion

The Mohr-Coulomb criterion postulates that under normal stress on a certain plane inside the material, when the shear stress reaches its ultimate shear strength, the material will be failure. The theory describes the failure behavior of the material under the combined action of normal stress and shear stress. This failure criterion can be expressed by the failure envelope determined by Mohr's circle under different stress states in τ - σ coordinate system. Based on the uniaxial and triaxial compression test data, a series of Mohr circles of concrete under different stress states can be drawn in the τ - σ coordinate system, and the failure envelopes of Mohr circles of four kinds of concrete under different confining pressures can be obtained, as shown in Figure 13.

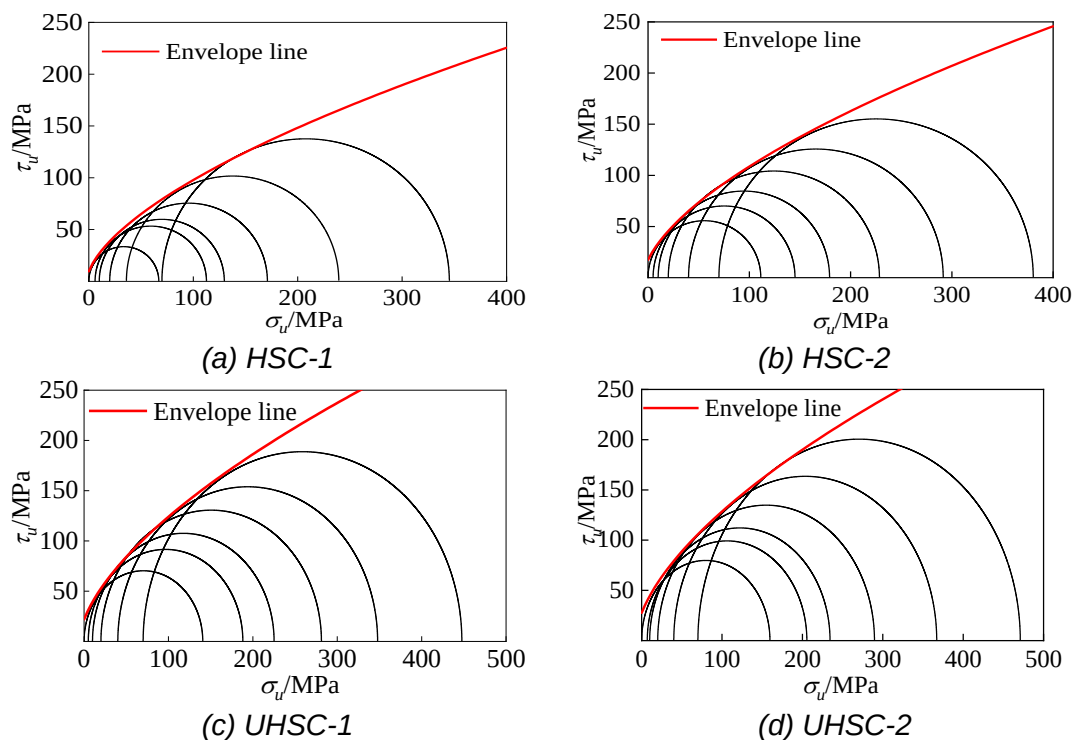


Fig.13 – Failure envelopes of four kinds of concrete

The power function form of the failure envelope can be obtained by nonlinear regression fitting analysis:

$$\tau_u = k(c + \sigma_u)^n \quad (13)$$

where c is the parameter about the cohesion of concrete, the value of the cohesion is shear stress kc^n at the intersection of envelope and longitudinal axis. k and n are the test fitting parameters.

Table 6 shows the values of the parameters in the Eq. (13). The cohesion of concrete gradually increases as the uniaxial compressive strength rises, which is due to the decrease of water/cement ratio and the improvement of concrete shear strength. Moreover, significant improvements in the interface between mortar and aggregate are contributed by the addition of silica fume to ultra-high-strength concrete.

Tab.6 - Fitted values of parameters in the expression

Parameters	HSC-1	HSC-2	HSCU-1	UHSC-2
k	5.812	6.539	7.152	7.793
c	2.425	5.144	6.326	8.363
n	0.609	0.605	0.612	0.598
R^2	0.9936	0.9978	0.9954	0.9982

The values of the shear stress and normal stress for the four kinds of concrete are normalized by the corresponding value of the uniaxial compressive strength, respectively. The relationship curve between the τ_u/σ_{co} and σ_u/σ_{co} is shown in Figure 14, and the general nonlinear formula of the failure envelopes was obtained:

$$\frac{\tau_u}{\sigma_{co}} = \alpha \left(\beta + \frac{\sigma_u}{\sigma_{co}} \right)^s \quad (14)$$

where $\alpha = 1.035$, $\beta = 0.047$, $s = 0.629$. Figure 14 shows the comparison between the fitting curve and experimental data. It can be seen that the general nonlinear expression of the Mohr-Coulomb failure criterion can more accurately describe the failure envelopes of four kinds of concrete.

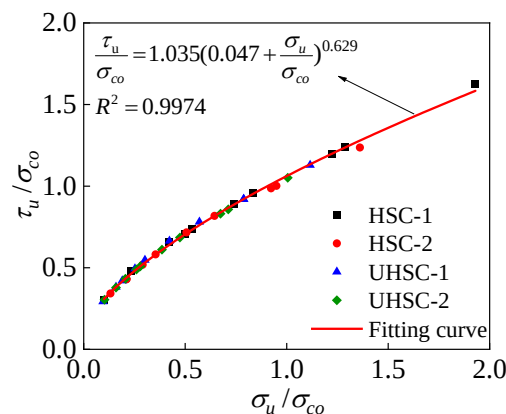


Fig.14 – Normalized failure curve for Mohr-Coulomb failure criterion

CONCLUSION

This study investigated the mechanical properties of four types of concrete with uniaxial strengths of 67.01, 109.34, 140.65, and 159.61 MPa containing the same basalt coarse aggregate under uniaxial compression test and conventional triaxial compression test. The volumetric deformation patterns and damage modes of the specimens were analysed, and the damage envelopes were obtained by using the nonlinear Mohr-Coulomb criterion, which was used to describe the mechanical properties of concrete under triaxial stress. From these investigations and test results, following conclusions can be drawn.

1) Different compressive strengths of concrete will influence its failure modes due to the difference in physical and mechanical characteristics between mortar and coarse aggregate. In uniaxial compressive test, the failure modes of different concretes types are all vertical split failure. As the confining pressure increases, the failure modes of concrete all develop from oblique shear failure toward extrusion flow failure. Crack propagation paths and morphologies in high-strength concrete, however, are different from those in ultra-high-strength concrete. In high-strength concrete, the cracks bypass the aggregate to propagate, and the failure mainly occurs in the matrix mortar or the interface between matrix mortar and aggregate, while in ultra-high-strength concrete, the crack pass through the coarse aggregates when expanding to the aggregates, the percentage of cracks

bypassing the aggregate decreased from 85.7% to 32.4% as the strength increased, the failure is mainly caused by the joint damage of mortar matrix and aggregate.

2) The enhancement of the confining pressure on the peak stress and stress of dilatancy onset, the corresponding strain at the point of peak stress and dilatancy onset of four concrete all in negative relation to its uniaxial compressive strength.

3) Confining pressure has a significant effect on the behavior of expansion and failure of concrete. Under the condition of low confining pressure, the difference between the peak stress and the stress of dilatancy onset, and the difference between the strain at peak stress and dilatancy onset are both really minimal, which indicates that the concrete will be broken rapidly after the dilatancy behavior occurs, while the difference increase gradually with the increase in confining pressure, indicating the failure of concrete are delayed by confining pressure.

4) The confining pressure can markedly increase the peak volumetric strain of concrete. At the condition of same confining pressure, the peak volumetric strain of concrete with relatively low uniaxial compressive strength is greater than the relatively uniaxial compressive strength. Based on the test results, the parameters in the existing volumetric model of concrete are improved, which makes the model predict the change of the volumetric strain of high-strength and ultra-high-strength concrete in a larger confining pressure range.

5) Based on the experimental data, the failure envelopes of four kinds of concrete under uniaxial and triaxial stress state were obtained by using the nonlinear Mohr-Coulomb criterion. The failure criteria can better describe the multiaxial mechanical behaviour of high-strength and ultra-high-strength concrete.

The results and research methodology of this paper can provide theoretical support when utilizing basalt in the formulation of concrete when studying the mixing ratios of concrete of different strengths.

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EXPERIMENTAL ANALYSIS OF COVERING LAYER- PRESTRESSED WIRE ROPE REINFORCED BRIDGES

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ABSTRACT

With the application of prestressed steel wire rope in bridge reinforcement project, mortar or composite mortar as the embedded material of steel wire rope is also gradually exposed to low strength, easy to crack, easy to fall off, and affect the durability of the wire rope and other shortcomings. Polyurethane-cement (PUC) composite, as a polymer concrete material, is characterized by light weight, high strength, high toughness, and good adhesion and corrosion resistance. In this paper, two kinds of reinforcement methods, PUC prestressed steel wire rope and prestressed steel wire rope, were used respectively, and the load test before and after reinforcement was carried out, and the effectiveness of the reinforcement methods was verified through the collection of parameter information, such as strain and deflection, before and after reinforcement. Although the PUC-strengthened bridge is less than the mortar-strengthened bridge each piece of girder arranged 15 pre-stressing steel wire rope, but the two bridges strengthened girder stiffness increase amplitude is comparable, under the action of the symmetrical load, the middle girder are increased by 13%, after the reinforcement of the bridge in the measured deflection to meet the highway load standard requirements of class II. The PUC overlay layer has a good adhesion with the original beam body, and no cracks were found. However, the mortar overlay layer has cracks of varying degrees. The crack width is 0.15 mm, and the average spacing is approximately 150 mm.

KEYWORDS

Covering layer -prestressed wire rope, Reinforced bridges, Comparative analysis, Load tests

INTRODUCTION

Bridges are an important part of highway transportation and have a pivotal role in the road transportation business [1]. With the continuous development of social and economic development, transportation is getting busier and busier, and bridges are playing an irreplaceable role as the throat of highway transportation [2,3].

With the development of social and economic development, the transportation industry unprecedented prosperity, vehicles as a carrier of transportation, compared with the previous changes have occurred greatly, to the vehicle load and flow of the increase in the most obvious [4-

8]. Due to rapidly increasing vehicle loads, natural conditions such as acid rain, air and temperature, a series of diseases such as stress cracks, concrete deterioration, and corrosion of steel reinforcement have occurred in the bridge structures [9-12]. These man-made and natural factors affect bridges to varying degrees, mainly in terms of structural safety, suitability and durability, and ultimately reduce the load-bearing capacity of bridges, jeopardizing their long-term operation. The insufficient load carrying capacity of old bridge structures has long been a common problem across the world [13,14].

Polyurethane cement (PUC) composites belong to polymer concrete materials, which are characterized by light weight, high strength, high toughness, and the material itself has good bond strength and resistance to acid and alkali corrosion [15,16]. PUC prestressed wire rope reinforcement method is embedded with prestressed wire rope in polymer concrete, which can solve the problem of pre-cracked cracks that cannot be closed actively in passive reinforcement [17]. The use of PUC to replace composite mortar as the embedding material for prestressing steel wire ropes not only solves the problem of durability of reinforced beams due to cracking and peeling of composite mortar, but also the material itself can be reinforced for the main beams [18,19]. Kexin Zhang [20] and others conducted an indoor experimental study on PUC-prestressed wire rope reinforced beams, considering parameters such as the thickness of PUC, the configuration of prestressed wire rope. The results of the study show that compared with composite mortar-prestressed wire rope reinforced beams, PUC-prestressed wire rope reinforced beams show a significant increase in load carrying capacity and stiffness, especially after the yielding of ordinary steel reinforcement. However, the research on PUC-prestressed wire rope reinforced beams is limited to indoor tests and theoretical studies, and there is a need to study PUC-prestressed wire rope reinforcement of actual bridges. In this paper, two kinds of reinforcement methods, PUC-prestressed steel wire rope and composite mortar-prestressed steel wire rope, were used for actual bridges, and load tests were carried out before and after reinforcement, and the validity of the reinforcement methods was verified by collecting information on the parameters such as strain and deflection before and after reinforcement.

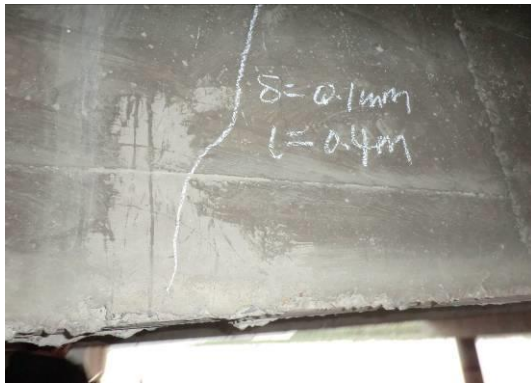
ENGINEERING BACKGROUND

In this paper, two T-structured bridges with similar construction age and structure were selected as the dependent projects, and the design of two bridge reinforcement was completed by using PUC prestressing steel wire rope reinforcement and composite mortar prestressing steel wire rope reinforcement respectively, and the verification test of the reinforcement effect was carried out.

The bridge one is located in Heilongjiang Province, , the bridge was built in the 1980s. The upper structure of the bridge is 5×20.0 m prefabricated reinforced concrete T beams, each transverse span 5 T beams, T beam flange plate width of 1.6 m, T beams are arranged between the cross beams, the bridge floor net width of 7 m + 2×1.0 m.

The bridge two is located in Heilongjiang Province, which was also built in the 1980s and has been in service for more than 30 years. The upper structure of the bridge is a typical concrete T-beam structure, with a total length of 40 m and a single-hole span of 20 m. The cross section of the bridge is composed of 5 T beams, the width of the flange plate of the T beam is 1.8 m, the total width of the bridge is 10.0 m, and the cross section arrangement is 2×1 m (collision barrier) +8 m (carriageway).

After the actual inspection of the two bridges, it was found that the surface of the main girder had more stress cracks and the load carrying capacity did not meet the requirements, and the main girder disease diagram is shown in Figure 1. In order to improve the bearing capacity of the main girder and extend the operation time, the two bridges were strengthened.



(a) - Main beam cracks



(b) - Concrete deterioration

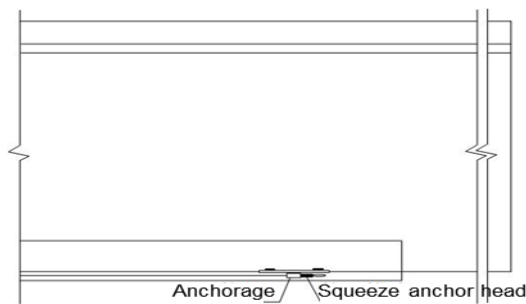
Fig. 1 - Main girder disease diagram

The overlay layer - prestressed steel wire rope reinforced beam has the advantages of simple construction and no impact on the clear height under the bridge. First, make a groove at the end of the beam and weld the anchor. Secondly, tension and anchor the prestressed steel wire rope. Secondly, pour PUC material or mortar material. Finally, remove the formwork. Polyurethane cement - Reinforcement method for prestressed steel wire ropes: Active force reinforcement of prestressed steel wire ropes, passive force participation of polyurethane cement. The polyurethane cement covering layer can reduce the usage of prestressed steel wire ropes and enhance their durability.

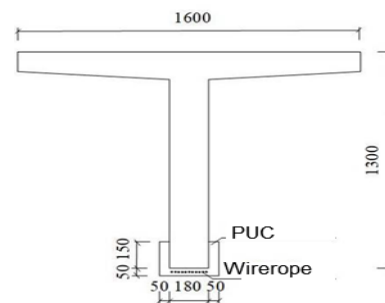
REINFORCEMENT SCHEME

Reinforcement Scheme for Bridge Number One

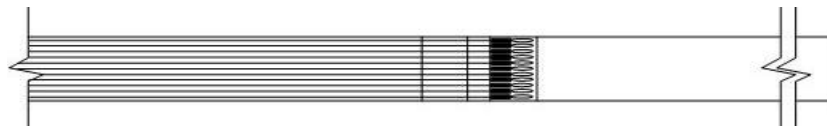
The structural design of the bridge is based on the traffic conditions in the 1980s, and the design load is the old standard automobile -20, trailer -100. The bridge is located on the main highway of the heilongjiang Province, and from the time it was built to the present, the traffic flow on the route has been so great that the bridge has been overloaded for a long time. The main girder after a long period of operation and caused the stiffness and bearing capacity of different degrees of reduction in the main girder girder body appeared a number of cracks, after the identification of the bridge has not been able to meet the current demand for traffic, can not meet the highway II load standards, the traffic management department has decided to strengthen the bridge to ensure highway operation safety, so that after reinforcement the bridge to meet the highway II pass load standards. Lateral view, cross-section view and steel wire rope arrangement view of bridge one are shown in Figure 2.



(a) - Lateral view



(b) - Cross-section view



(c) Steel wire rope arrangement of beam bottom

Fig .2 - Lateral view, cross-section view and steel wire rope arrangement view of bridge one (Unit:mm)

As shown in Figure 2, the main girder is reinforced with PUC prestressing steel ropes in the longitudinal direction of 16 m. 11 prestressing steel ropes are arranged at the bottom of the T-beam, and the thickness of the PUC is 50 mm, and the height of the side is 150 mm, so as to ensure that the structural load carrying capacity and stiffness can meet the specification requirements. The bridge one reinforcement diagram is shown in Figure 3, which shows the construction of prestressing wire ropes, the erection of pouring formwork, and the mixing and pouring of PUC material, respectively. The density of PUC material used in the actual construction process is 1500 kg/m^3 , according to the mass ratio of material preparation, polyol: isocyanate: cement = 1:1:2, with an electric mixer in an iron bucket for 3 min (Figure 3c), and then manually poured (Figure 3d), pouring is completed, and then in a dry environment maintenance 7 d.



Fig .3 - Reinforcement figures of bridge one

Reinforcement Scheme for Bridge Number Two

The bridge two is located in Jixi City, Jidong County, also built in the 1980s, the design load for the old standard automobile -20, trailer -100. From its completion to the present, the route has seen a tremendous amount of traffic and the bridges have been overloaded for a long time. The main girder has been operated for a long time and the stiffness and bearing capacity have been reduced to different degrees, according to the bearing capacity calculation and the results of the bridge load test, the bridge has not been able to meet the demand of the traffic volume, and can not meet the highway II load standard, the traffic management department has decided to strengthen the bridge to ensure the safety of highway operation, so that the strengthened bridge reaches the highway II pass load standard.

The bridge adopts the composite mortar prestressing wire rope reinforcement method, each piece of T-beam arrangement of 26 prestressing steel wire ropes, respectively, along a bottom surface and two sides of the arrangement, as shown in Figure 4, the bottom surface of the arrangement of 8, the two sides of the arrangement of 9 steel wire ropes, prestressing steel wire rope arrangement range of the main beam within the longitudinal range of 14 m, steel wire rope embedded in the 25 mm thickness of the composite mortar. Compared with the bridge one, this bridge has 15 more prestressing wires per girder than bridge one. The reinforcement diagram of the bridge two is shown in Figure 4, which are the prestressed steel wire rope tension diagram and the bottom diagram of the reinforced beam respectively. The reinforcement diagram of the bridge two is shown in Figure 5.

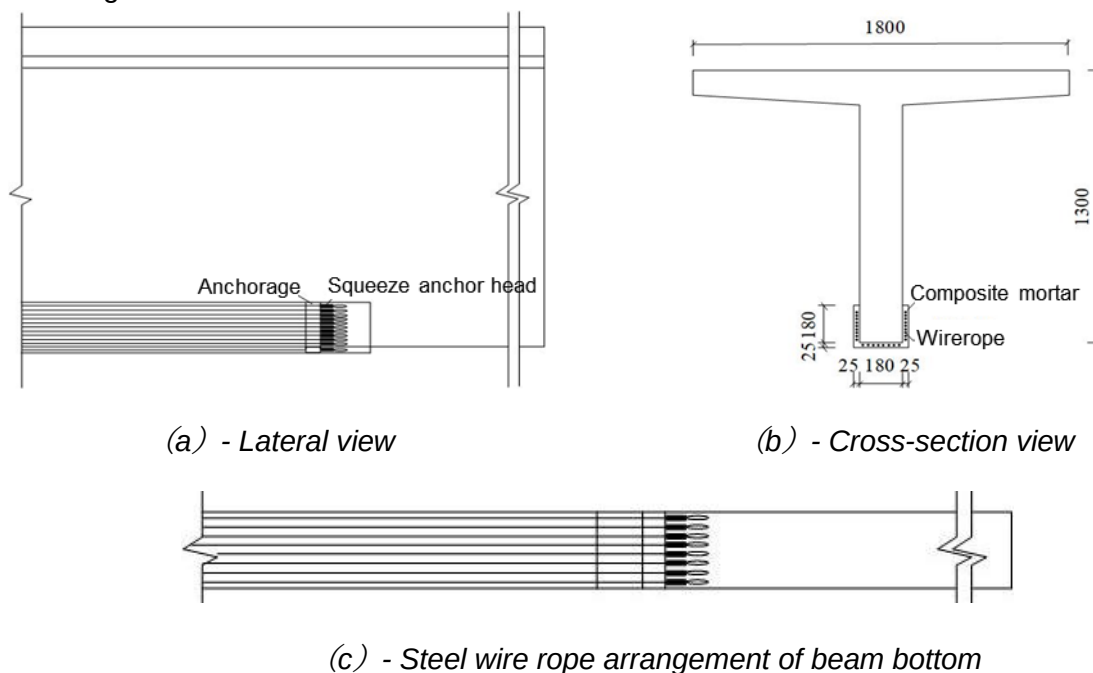


Fig .4 - Lateral view, cross-section view and steel wire rope arrangement view of bridge two (Unit:mm)



Fig .5 - Reinforcement figures of bridge two

STATIC LOAD TEST OF BRIDGE BEFORE AND AFTER REINFORCEMENT

In order to comprehensively evaluate the stress performance of the two solid bridges after reinforcement, field load tests before and after reinforcement were carried out on bridge one and bridge two, respectively, to verify the actual reinforcement effect of the bridge. The reinforcement effect was evaluated by the measured results before and after reinforcement, and a comparative analysis was made between the PUC prestressing wire rope reinforcement method of the bridge one and the composite mortar prestressing wire rope reinforcement method of the bridge two.

Test Load Determination

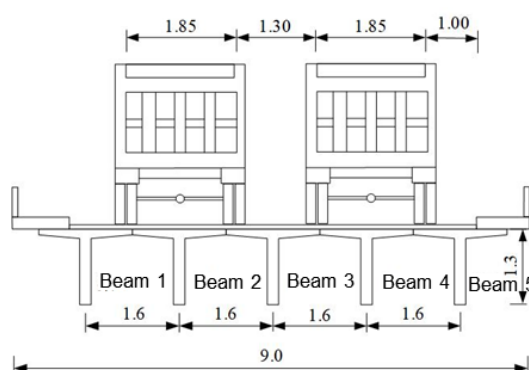
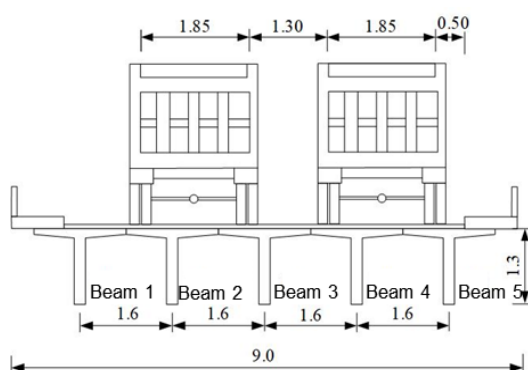
The load is distributed according to the most unfavorable position to ensure that the load test efficiency η is between 0.95 and 1.05. The load is distributed according to the most unfavorable position, and in order to accurately simulate the highway-class-II loads of the two bridges, two 42 t loaded cars and two 40 t loaded three-axle trucks are used as the test loads, respectively. Unbalanced load and medium-load tests were carried out by simulating the highway class II load standard after accurate weighing. The vehicle loads used for the load tests on the bridge one and two are shown in Tables 1 and 2, respectively. Each time the load is stabilized for a period of time, the data of each measurement point is collected. The solid bridge reinforcement diagrams are shown in Figures 6 to 9.

Tab. 1 - Loading vehicle parameters before and after reinforcement (Bridge one)

Vehicle		Front axle weight(kN)	Central axis weight(kN)	Rear axle weight(kN)	Front to middle wheelbase(m)	Mid rear wheelbase(m)
Before reinforcement	A	81.4	162.8	162.8	3.5	1.35
	B	84.2.	168.4	168.4	3.5	1.35
After reinforcement	A	83.4	166.8	166.8	3.5	1.35
	B	84.6	169.2	169.2	3.5	1.35

Tab. 2 - Loading vehicle parameters before and after reinforcement (Bridge two)

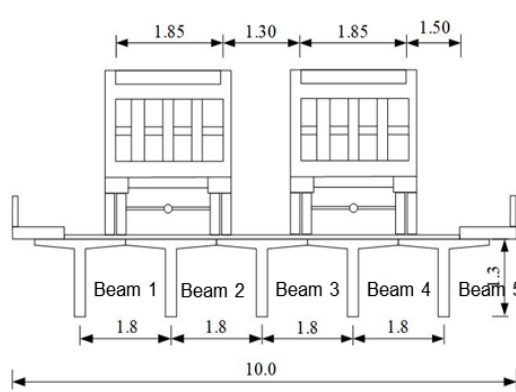
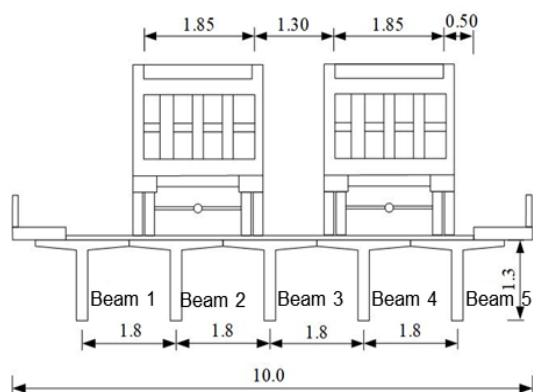
Vehicle		Front axle weight(kN)	Central axis weight(kN)	Rear axle weight(kN)	Front to middle wheelbase(m)	Mid rear wheelbase(m)
Before reinforcement	A	79.3	158.6	158.6	3.5	1.35
	B	78.6	157.2	157.2	3.5	1.35
After reinforcement	A	79.0	158.0	158.0	3.5	1.35
	B	80.6	161.2	161.2	3.5	1.35



(a) - Cross section under the eccentric load

(b) - Cross section under the symmetrical load

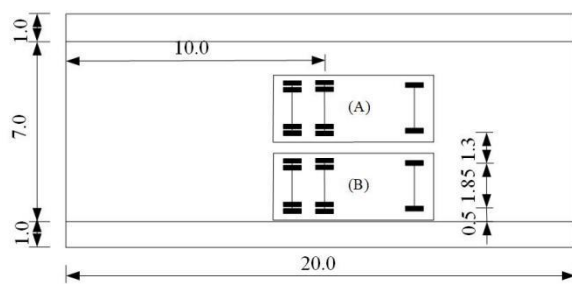
Fig .6 - Load cross-sectional view of bridge one (Unit:m)



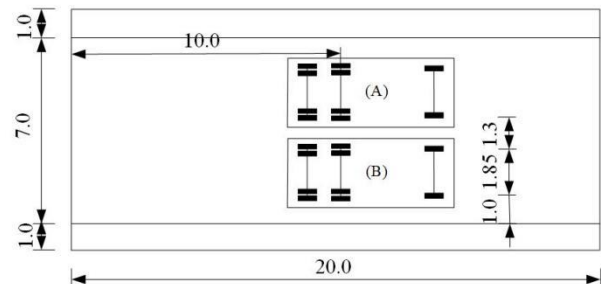
(a) - Cross section under the eccentric load

(b) - Cross section under the symmetrical load

Fig .7 - Load cross-sectional view of bridge two (Unit:m)

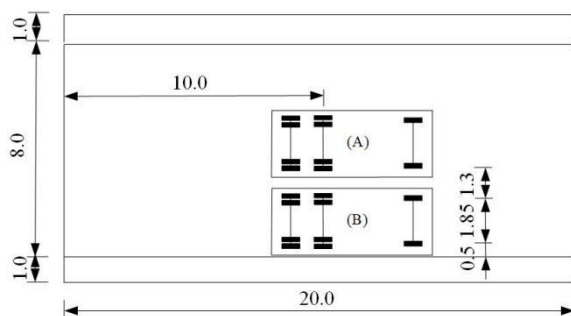


(a) - Cross section under the eccentric load

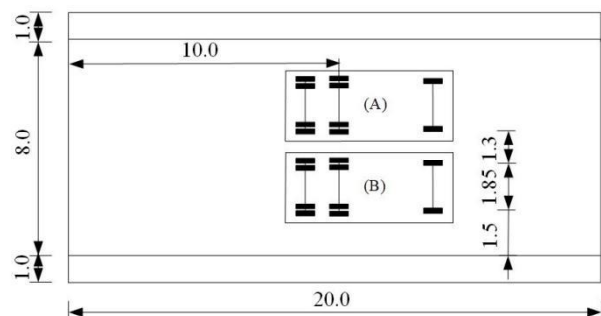


(b) - Cross section under the symmetrical load

Fig .8 - Load cross-sectional view of bridge one (Unit:m)



(a) - Cross section under the
eccentric load

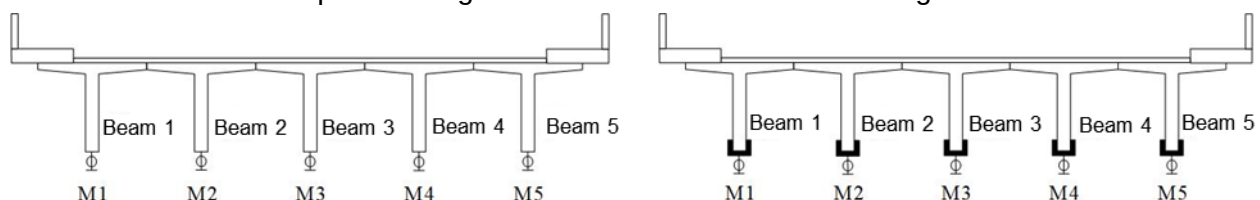


(b) - Cross section under the
symmetrical load

Fig .9 - Load cross-sectional view of bridge two (Unit:m)

Test Point Arrangement

In order to test the deflection of the girder, a deflectometer is installed at the bottom of the girder in the span of each T-beam to test the deflection value of each piece of the main girder under the test condition, the deflection of the bridge one is shown in Figure 10, and the deflection arrangement of the bridge two is the same as that of the bridge one. Before and after the strengthened girder strain measurement point arrangement is shown in Figure 11, the real bridge two strain measurement point arrangement is the same as the real bridge one.



(a) - Before reinforcement

(b) - After reinforcement

Fig .10 - Deflection test point layout of bridge one

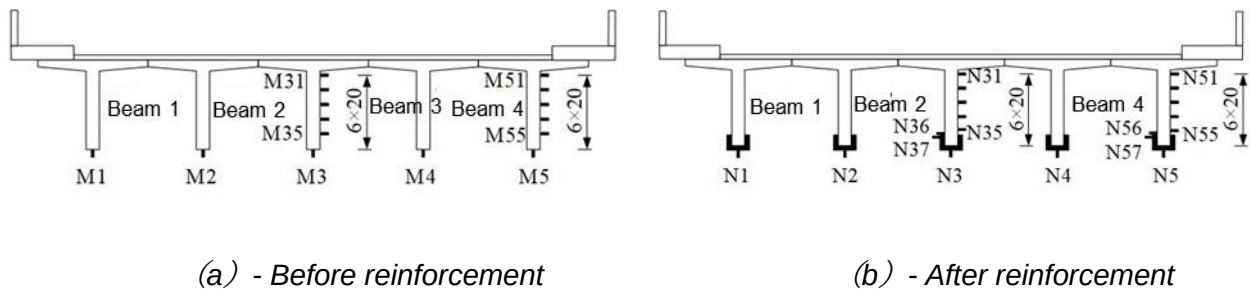


Fig .11 - Strain gauge layout of bridge one

LOAD TEST RESULT

Deflection Test Results

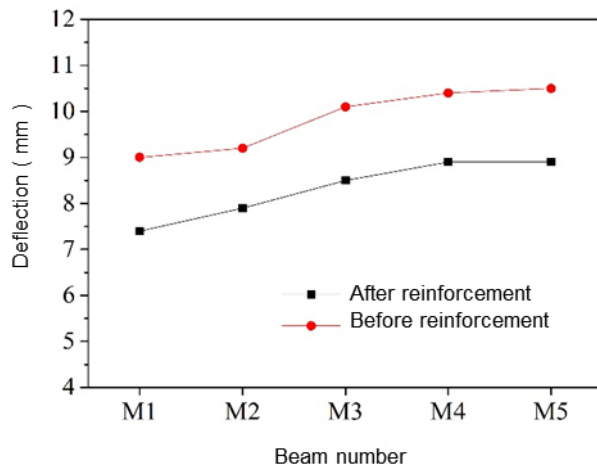
The deflection values of the load test before and after the reinforcement of the real bridge one and two are shown in Table 3 and Table 4, and the deflection graphs are plotted according to the deflection data as shown in Figure 12 and Figure 13.

Tab. 3 - Deflection under the eccentric load before and after reinforcement

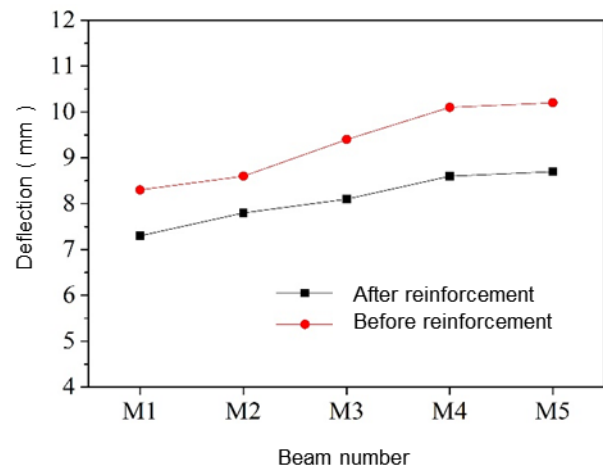
Measurement point number	Before the bridge one reinforcement deflection (mm)	After the bridge one reinforcement deflection (mm)	Decrease rate	Before the bridge two reinforcement deflection (mm)	After the bridge two reinforcement deflection (mm)	Decrease rate
M1	9.0	7.4	17.8%	8.3	7.3	12.0%
M2	9.2	7.9	14.1%	8.6	7.8	9.3%
M3	10.1	8.5	15.8%	9.4	8.1	13.8%
M4	10.4	8.9	14.4%	10.1	8.6	14.8%
M5	10.5	8.9	15.2%	10.2	8.7	14.7%

Tab. 4 Deflection under the symmetrical load before and after reinforcement

Measurement point number	Before the bridge one reinforcement deflection (mm)	After the bridge one reinforcement deflection (mm)	Decrease rate	Before the bridge two reinforcement deflection (mm)	After the bridge two reinforcement deflection (mm)	Decrease rate
M1	9.0	7.7	14.4%	8.6	7.6	11.6%
M2	9.7	8.3	14.4%	9.4	7.8	17.0%
M3	9.9	8.6	13.1%	9.6	8.3	13.5%
M4	9.5	8.5	10.5%	9.3	8.1	12.9%
M5	8.9	7.8	12.4%	8.8	7.5	14.8%



(a) - Bridge one



(b) - Bridge two

Fig .12 - The deflection results under the eccentric load before and after reinforcement

As can be seen in Figure 12, the deflections under bias load of bridges strengthened with PUC prestressed wire rope reinforcement and composite mortar prestressed wire rope reinforcement are reduced to different degrees. The deflection of the beam on the side of the eccentric load is maximum under the eccentric loads used. Before reinforcement, under the test load, the deflections of 4# and 5# girders of the bridge two were 10.1 mm and 10.2 mm respectively, and after reinforcement by composite mortar pre-stressing steel wire rope method, under the test load, the deflections of 4# and 5# girders were 8.6 mm and 8.7 mm, which were reduced by 14.8% and 14.7% respectively. Under the test load before reinforcement, the deflections of 4# and 5# girders of the bridge one were 10.4 mm and 10.5 mm respectively, and after reinforcement by the method of PUC prestressing steel wire rope, the deflections of 4# and 5# girders were 8.9 mm and 8.9 mm, which were reduced by 14.4% and 15.2% respectively. Although the bridge one is less than the bridge two each piece of girder arranged 15 pre-stressing steel wire rope, but the two bridges after reinforcement the stiffness of the girder body to improve the magnitude of the same, 4# girder and 5# girder are increased by 14%, after reinforcement the bridge deflection measured deflection to meet the requirements of the highway class II loading standards.

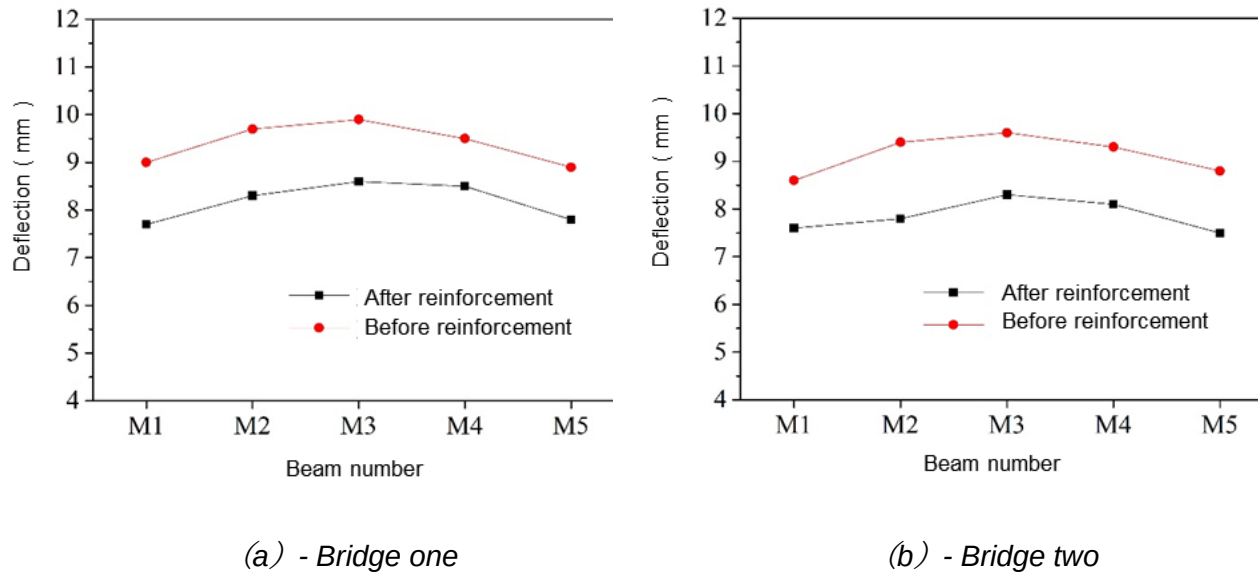


Fig .13 - The deflection results under the symmetrical load before and after reinforcement

As can be seen in Figure 13, the deflections under medium load of bridges strengthened with PUC prestressed wire rope reinforcement and composite mortar prestressed wire rope reinforcement are all reduced to different degrees. The deflection of girder 3# is the largest under medium load. The deflection of 3# girder of the bridge two was 9.6 mm under test load before reinforcement, and the deflection of 3# girder was 8.3 mm under test load after using composite mortar prestressing steel wire rope reinforcement, the deflection was reduced by 13.5%. Under the test load before reinforcement, the deflection of #3 girder of the bridge one was 9.9 mm, and the deflection of #3 girder after reinforcement with PUC prestressing steel wire rope was 8.6 mm, the deflection was reduced by 13.1%. Although the bridge one is less than the bridge two each piece of girder arranged 15 pre-stressing steel wire rope, but the two bridges strengthened girder stiffness increase amplitude is comparable, under the action of the medium load, 3# girder are increased by 13%, after the reinforcement of the bridge in the measured deflection to meet the highway load standard requirements of class II.

Strain Test Results

The strains at the bottom of the girders before and after reinforcement under the deflected load conditions of the bridge one and two are shown in Table 5, and the graphs are plotted as shown in Figure 14. The strains at the bottom of the girders before and after reinforcement under the medium load conditions of the bridge one and two are shown in Table 6, and the graphs are plotted as shown in Figure 15.

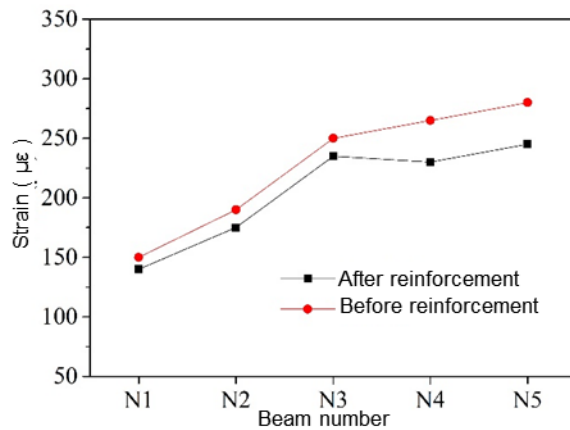
Tab. 5 - Beam bottom strain under the eccentric load before and after reinforcement

Measurement point number	Before the bridge one reinforcement strain ($\mu\epsilon$)	After the bridge one reinforcement strain ($\mu\epsilon$)	Decrease rate	Before the bridge two reinforcement strain ($\mu\epsilon$)	After the bridge two reinforcement strain ($\mu\epsilon$)	Decrease rate
N1	150	140	7.1%	160	150	6.7%
N2	190	175	7.8%	200	180	11.1%
N3	250	235	6.4%	255	225	13.3%
N4	265	230	15.5%	275	235	14.5%
N5	280	245	12.5%	270	240	11.1%

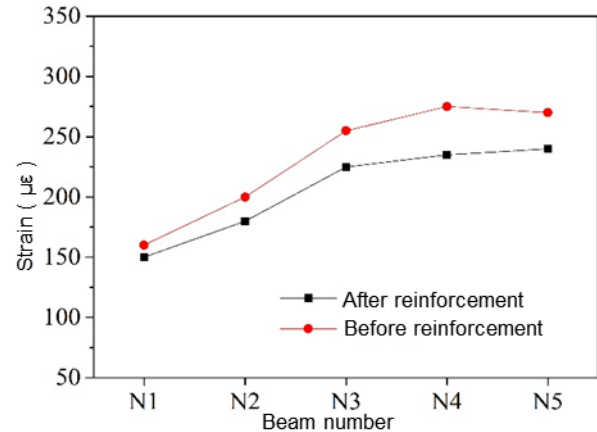
Tab. 6 - Beam bottom strain under the symmetrical load before and after reinforcement

Measurement point number	Before the bridge one reinforcement strain ($\mu\epsilon$)	After the bridge one reinforcement strain ($\mu\epsilon$)	Decrease rate	Before the bridge two reinforcement strain ($\mu\epsilon$)	After the bridge two reinforcement strain ($\mu\epsilon$)	Decrease rate
N1	180	170	5.6%	205	180	12.2%
N2	240	210	12.5%	230	205	10.9%
N3	260	225	13.5%	265	225	15.1%
N4	245	220	10.2%	250	215	14.0%
N5	190	175	7.9%	200	180	10.0%

As can be seen in Figure 14, the bottom strain of each main girder of the bridge strengthened with PUC prestressed wire rope reinforcement method and composite mortar reinforcement method is reduced to a different degree under the eccentricity test load of the bridge. The bridge one, the strains at the bottom of girders 4# and 5# under bias load before reinforcement are 265 $\mu\epsilon$ and 280 $\mu\epsilon$, and the strains at the bottom of girders 4# and 5# after reinforcement are 230 $\mu\epsilon$ and 245 $\mu\epsilon$, which are 15.5% and 12.5% lower than those before reinforcement, respectively. The bridge two, under the bias load, the strains of girders 4# and 5# are 275 $\mu\epsilon$ and 270 $\mu\epsilon$ before reinforcement, and 235 $\mu\epsilon$ and 240 $\mu\epsilon$ after reinforcement, which are reduced by 14.5% and 11.1%, respectively. The strain reduction rates after reinforcement are similar for both bridges, and the strains under the measured bias loads of the strengthened bridges meet the requirements of highway class II loading standards.

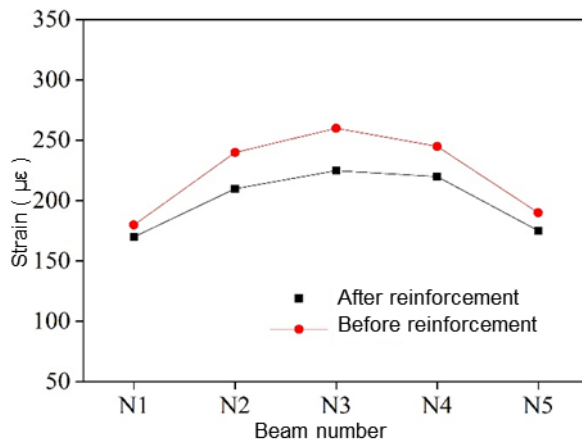


(a) - Bridge one

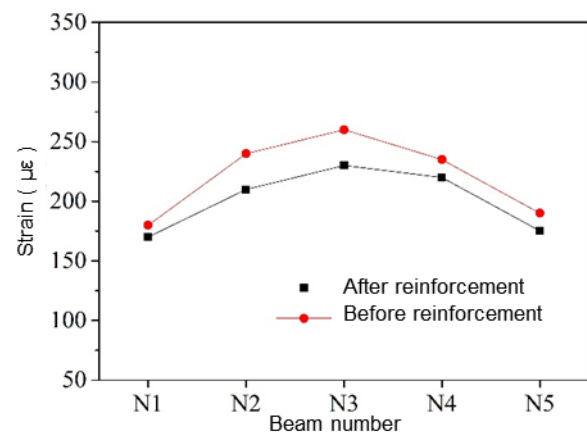


(b) - Bridge two

Fig .14 - The strain results under the eccentric load before and after reinforcement



(a) - Bridge one



(b) - Bridge two

Fig .15 - The strain results under the symmetrical load before and after reinforcement

As can be seen in Figure 15, the bridges strengthened by the PUC prestressing wire rope method and the composite mortar prestressing wire rope reinforcement method show different degrees of reduction in the bottom strains of the main girders of each piece under symmetrical loading. The strain at the bottom of girder 3# girder under symmetrical loading in bridge one before reinforcement is 260 $\mu\epsilon$, and the strain at the bottom of girder 3# girder after reinforcement is 225 $\mu\epsilon$, which is 13.5% lower than that before reinforcement. The strain of girder 3# of the bridge two was 265 $\mu\epsilon$ before reinforcement, and it became 225 $\mu\epsilon$ after reinforcement, which was reduced by 15.1%. After reinforcement the two bridges, the measured strains under medium loads meet the requirements of highway class II loading standards.

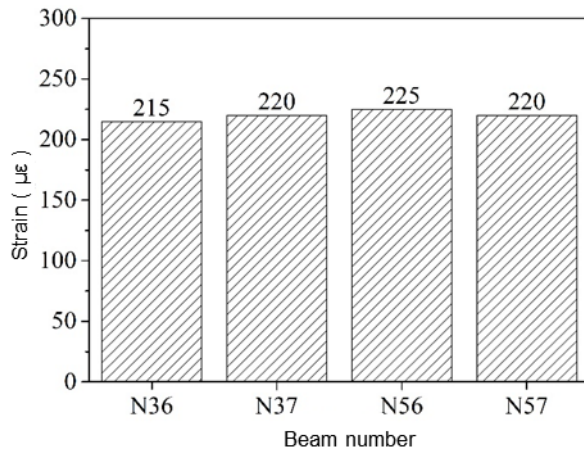


Fig .16 - Lateral strain under the eccentric load after reinforcement

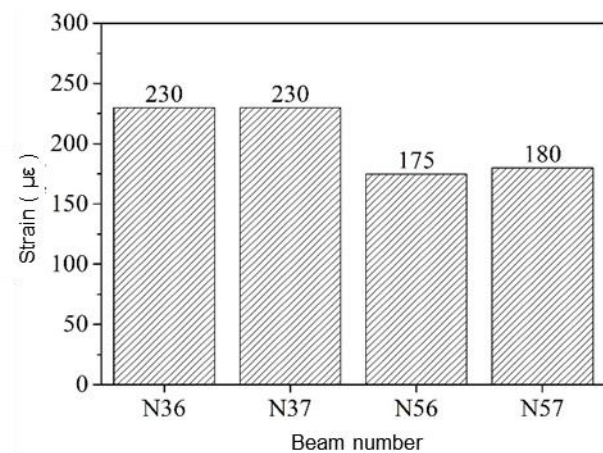
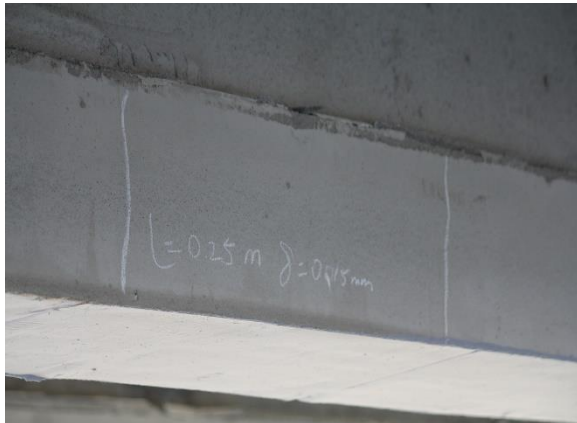


Fig .17 - Lateral strain under the eccentric load after reinforcement

N36 and N37 are strain measurement points adjacent to the side of girder 3#, N36 is used to measure concrete strain and N37 is used to measure PUC strain. N56 and N57 are strain measurement points adjacent to the side of girder 5#, N56 is used to measure concrete strain and N57 is used to measure PUC strain. As can be seen in Figure 16, the strains at the two measurement points N36 and N37 are 215 $\mu\epsilon$ and 220 $\mu\epsilon$, respectively, and the strains at the two measurement points N56 and N57 are 225 $\mu\epsilon$ and 220 $\mu\epsilon$, respectively, under eccentric loading, and the strains of the urethane cement material at the same locations are similar to the concrete strains. As can be seen from Figure 17, under symmetrical loading, the strains at the two measurement points N36 and N37 are 230 $\mu\epsilon$ and 230 $\mu\epsilon$, respectively, and the strains at the two measurement points N56 and N57 are 175 $\mu\epsilon$ and 180 $\mu\epsilon$, respectively, which are the same as in the case of partial loading, the strains of the PUC material at the same position are similar to those of the concrete, which indicates that the PUC coordinates the deformation of the main beams well with that of the PUC after casting.

Surface Observation

One month after the completion of the reinforcement, on-site observation of the use of the bridge one and the bridge two were carried out respectively, and no cracking was found on the surface of the PUC of the bridge one, and the reinforcement layer was well bonded with the original girder. On-site observation of the bridge two found that each beam reinforcement layer mortar have different degrees of cracking phenomenon, crack width of 0.15 mm, the average spacing of about 150 mm, cracks vertical longitudinal bridge direction, the two sides of the through the U-shaped cracks, as shown in Figure 18, most of them are force cracks.



(a) - Composite mortar crack diagram

(b) - Composite mortar crack diagram

Fig .18 - Composite mortar crack diagram of bridge two

CALCULATION METHOD OF BEARING CAPACITY

The ultimate bearing capacity calculation of RC damaged beams reinforced by PUC and prestressed steel wire ropes is based on the test failure mode. The calculation adopts the constitutive model of concrete and steel bars in the specification. When the PUC material reaches the ultimate tensile strain, the steel bars have already reached their yield strength before that. Therefore, when the reinforced beam reaches its ultimate bearing capacity, the stress of the reinforcing bars is calculated based on its yield strength f_y . The calculation diagram is shown in Figure 19, and the ultimate bearing capacity can be calculated as follows.

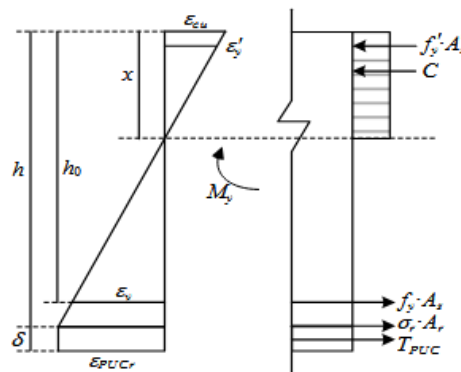


Fig. 19 - Simplified calculation diagram of the ultimate flexural bearing capacity of reinforced beams

From the assumption of the flat section, it can be known that the strain of the steel wire rope caused by the deformation of the beam body at this time is:

$$\varepsilon_{r1} = \varepsilon_{PUCr} \quad (1)$$

Considering the strain lag and the pre-tension stress of the wire rope, the true strain of the wire rope is:

$$\varepsilon_r = \varepsilon_{r1} + \varepsilon_{pr} - \varepsilon_{r0} \quad (2)$$

Then the stress of the steel wire rope when the reinforced beam reaches the ultimate bearing capacity is:

$$\sigma_r = \varepsilon_r \cdot E_r \quad (3)$$

If it is a single-reinforcement cross-section beam, from the equilibrium relationship, it can be obtained that:

$$f_{cd}bx = f_y \cdot A_s + \sigma_r \cdot A_r + f_{PUCr} \cdot A_{PUC} \quad (4)$$

$$M_y = f_y \cdot A_s \left(h_0 + \frac{x}{2} \right) + \sigma_r \cdot A_r \left(h - \frac{x}{2} \right) + f_{PUCr} \cdot A_{PUC} \left(h - \frac{x}{2} + \frac{\delta}{2} \right) \quad (5)$$

If it is a double-rib cross-section beam, from the equilibrium relationship, it can be obtained that:

$$f_{cd}bx + f_y' \cdot A_s' = f_y \cdot A_s + \sigma_r \cdot A_r + f_{PUCr} \cdot A_{PUC} \quad (6)$$

$$M_y = f_y \cdot A_s \left(h_0 - \frac{x}{2} \right) + \sigma_r \cdot A_r \left(h - \frac{x}{2} \right) + f_y' \cdot A_s' \left(\frac{x}{2} - a_s' \right) + f_{PUCr} \cdot A_{PUC} \left(h - \frac{x}{2} + \frac{\delta}{2} \right) \quad (7)$$

In the formula:

ε_{pr} — The strain of the steel wire rope under the action of prestress.

ε_{r0} — The hysteresis strain of the steel wire rope caused by the initial bending moment.

x — When the bending ultimate bearing capacity state is reached, the equivalent rectangular height of the concrete in the compression zone is obtained by solving the force equilibrium equation.

CONCLUSION

1. After reinforcing the structure with PUC prestressed steel wire rope, the stiffness and strength of the structure have been significantly improved to meet the requirements of the highway-II loading standards. The measured values of deflection and strain of each beam were significantly reduced after reinforcement, and the structural safety reserve was improved.
2. The PUC prestressed wire rope-reinforced bridges had a large number of fewer prestressed wire ropes than the composite mortar-reinforced bridges, but the deflections and strains measured for the two reinforced bridges were reduced to a comparable degree compared to the pre-strengthened bridges.
3. The strain of the PUC material at the same position is similar to that of the concrete, indicating that the PUC coordinates well with the main beam after casting and the surface of the reinforcement layer is not cracked.
4. On the one hand, PUC, as the covering layer of prestressed steel wire ropes, can effectively reduce the pre-applied force of the steel wire ropes, lower the stress at the beam ends, and prevent stress concentration at the anchoring ends. On the other hand, the excellent toughness and corrosion resistance of PUC prevent the internal steel wire rope from rusting due to the cracking of the covering layer.

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RESEARCH ON SEISMIC RESISTANCE OF COMBINED SYSTEM BRIDGE OF CABLE-STAYED BRIDGE AND IRREGULAR ARCH

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ABSTRACT

Bridges are prone to damage during earthquakes, with large-scale or complex structural systems being particularly sensitive to seismic impacts. Taking the bridge that integrates a cable-stayed bridge with a backless slanted tower and a lower-supported irregular arch bridge as the background, this paper establishes a spatial analysis model for Xiangfeng River Bridge using the finite element software Midas Civil. Based on the concept of structural vibration control design, the concept of time-history response influence factors is introduced to conduct a time-history response parameter analysis of the structure. The study investigates the effects of wind brace arrangement, concrete elastic modulus, length of arch rib concrete sections, and the inclination angle of the pylon on the seismic performance of the bridge. It further compares and analyzes structural vibration control and damping, conducts structural seismic checks, and performs elasto-plastic time-history response analysis at the consolidated positions of the tower, beam, and pier by defining elasto-plastic materials and fiber hinges. The research results indicate that the sensitivity of the overall seismic performance of the structure to different parameters follows the order from strongest to weakest as: pylon inclination angle > length of arch rib concrete sections > concrete elastic modulus > wind brace arrangement. "K"-shaped wind braces are more conducive to structural seismic resistance compared to parallel wind braces, and omitting wind braces will significantly impact the seismic performance of the structure. The wind bracing of the arch ribs develops tensile stress, with maximum tensile stresses of 223.4 MPa (corrected from 176.7 MPa, assuming a typo in the original text), 114.5 MPa, and another value under transverse and longitudinal earthquake actions. Notably, neither the arch ribs nor the wind bracing materials have yielded.

KEYWORDS

Cable-stayed arch collaborative system bridge, Seismic performance, Seismic check, Time-history response analysis

INTRODUCTION

As early as the mid-19th century, collaborative system bridges emerged. However, constrained by factors such as construction techniques and theoretical research, the development of collaborative system bridges progressed slowly [1]. Among the limited number of collaborative system bridges, cable-stayed bridges or arch bridges are often utilized as a part of the collaborative

system. The cable-stayed arch collaborative system bridges, which directly combine cable-stayed bridges and arch bridges, are even more rare [2-5].

The load-bearing responsibilities of cable-stayed arch collaborative system bridges are collectively shouldered by the main girder, pylon, stay cables, arch ribs, and hangers, with the primary load-bearing structures varying from bridge to bridge. Existing cable-stayed arch collaborative system bridges can be categorized into two forms based on the position where the stay cables act [6-9]. In one form, the stay cables directly act on the bridge deck, resulting in a structure that primarily exhibits the load-bearing characteristics of a cable-stayed bridge. In the other form, the stay cables are anchored to the arch ribs and indirectly act on the bridge deck through the hangers of the arch ribs, with both components working together to support the load.

The cable-stayed arch collaborative system bridge represents a statically indeterminate structure with complex force distribution. Relying solely on adjustments to structural parameters during the design phase to control vibrations often fails to meet the seismic requirements of the structure [10, 11]. Therefore, measures need to be taken to enhance the seismic stability of the bridge structure. The essence of seismic measures for bridge structures lies in vibration control. Currently, commonly used structural vibration control devices or systems can help improve the seismic capacity of cable-stayed arch collaborative system bridges [12-15].

The cable-stayed arch bridge, as a unique and novel bridge type with distinctive design, has significant theoretical value and engineering significance in terms of its seismic performance research [16]. This type of bridge combines the force characteristics of cable-stayed bridges and arch bridges, and eliminates the back cables of conventional cable-stayed bridges, making its structural system, dynamic characteristics, and seismic response mechanism more complex. The research results can not only directly guide the seismic design and safety assessment of such bridges, ensuring the seismic safety of major transportation infrastructure and reducing potential earthquake damage, but also promote the advancement of seismic technology for large-span composite structure bridges, providing important theoretical support and technical reserves for the construction of similar complex bridges in the future [17].

Based on the theory of structural vibration control, vibration control for bridge structures primarily involves the analysis of active control and passive control. Structural control methods are mainly classified according to whether they require the input of signals from structural response and external energy sources. In addition to the aforementioned two methods, structural vibration control methods also encompass hybrid control and semi-active control [18]. Structural vibration control can help overcome the shortcomings of current seismic design methods, particularly gaining increasing attention in earthquake prevention and disaster mitigation for important projects such as structures in high-intensity seismic regions and long-span bridges [19, 20]. The base response time course refers to the complete process of how the physical quantities such as displacement, velocity, acceleration, and internal force of a structural system change over time under the action of dynamic loads that vary with time. Based on the theory of structural vibration control and performance-based seismic design concepts, this paper introduces the concept of time-history response influence factors, conducts structural parameter analysis under three-dimensional seismic loading, and performs structural seismic checks.

PROJECT PROFILE

The bridge has a span arrangement of 40+90=130 m. It is a collaborative system bridge combining a cable-stayed bridge without back stays and an irregularly shaped arch bridge. The bridge deck features a variable width structure, ranging from 39.0 m to 43.0 m in total width. The bridge span is orthogonal to the road design line and oblique to the river course line. The design load is in accordance with Highway-I load specifications. The bridge has six lanes in both directions. The layout of the bridge type is shown in Figure 1.

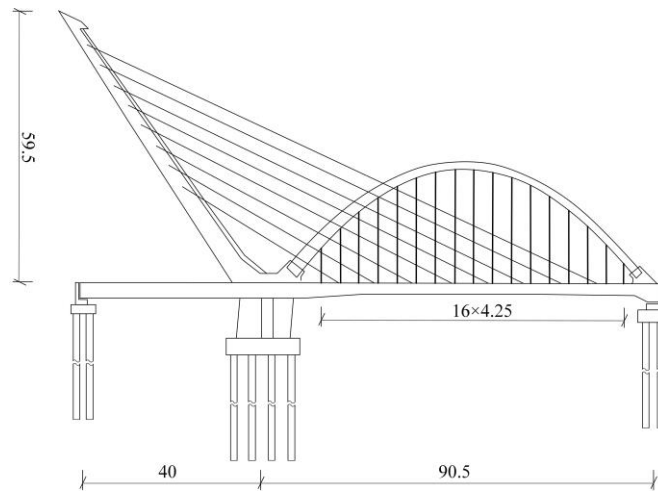


Fig. 1 - Bridge layout

(1) Main Girder

The main girder is a PC (prestressed concrete) cast-in-situ box girder. The girder height at the mid-span is 2.7 m, gradually transitioning to 3.8 m within a 21 m range on both sides of the pier top. The top slab is 0.3 m thick, while the bottom slab is 0.22 m thick, increasing to 0.42 m in the vicinity of the piers and abutments. The standard web thickness is 0.45 m, with transition sections and constant thickness sections of 0.8m width provided at 0# pier and 1# pier, respectively. The primary cross-section of the main girder is illustrated in Figure 2.

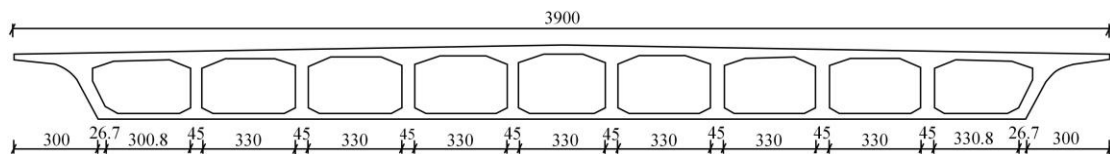


Fig. 2 - Primary cross-sectional view of the main girder

(2) Main Tower

The tower columns are constructed of reinforced concrete with a horizontal inclination angle of 56° . The tower height above the bridge deck is 59.5 m, and the tower base is rigidly connected to the main girder, arch feet, and main piers. The cross-section of the bridge tower is rectangular, with a lateral width of 3.5 m and a longitudinal width that varies from 2.2 m at the top to 6.0 m at the bottom.

(3) Main Tower Stay Cables

The stay cables are prefabricated, consisting of a bundle of steel strands extruded to form a cable system. A total of eight stay cables are installed, with a spacing of 8.5 m between them. The stay cables are tensioned at the tower and anchored on the girder.

(4) Arch Ribs

The arch ribs are irregularly shaped steel box structures, comprising two separate ribs. The steel boxes are hollow, with concrete poured only near the arch feet. The rise-to-span ratio of the arch axis is $1/3$, with a rise of 28 m and a span of 84 m, following a quadratic parabola combined with a straight line. The steel box of each arch rib is 2 m wide and 1.8 m high, gradually widening to 3.366 m on the side near 1# pier where the two arch ribs merge, resulting in a combined width of

3m. The height and width of the arch rib remain unchanged on the side of 2# pier.

At both ends, the arch ribs are equipped with concrete arch feet, into which the arch ribs extend for a certain length and are connected with bearing plates at the concrete interface of the arch feet to ensure smooth force transmission within the structure. In addition to three-directional ordinary steel reinforcement, the connection between the concrete arch feet and the main girder is reinforced with high-strength threaded steel bars.

(5) Main Arch Suspension Cables

The suspension cables are also prefabricated, utilizing the same material as the stay cables. A total of 19 suspension cables are installed longitudinally, with a spacing of 4.25 m between them. The upper anchor points are equipped with ear plates and positioned on the outer side of the steel box arch ribs. The suspension cables are tensioned on the bottom slab of the box girder.

(6) Wind Braces for Main Arch Ribs

A total of three “K”-shaped wind braces are installed laterally on the steel arch ribs. The cross-sectional shapes of the main braces and struts of the wind braces are all square. Reinforcing ribs are provided on both the main braces and struts of the wind braces.

FINITE ELEMENT MODEL

Using the Midas/Civil software, a full-bridge finite element analysis model is established using the spatial grillage method, with components scaled proportionally to their actual positions. The elements are divided based on the principles of meeting calculation accuracy while facilitating ease of computation. In this model, hanger rods and stay cables are represented using truss elements, and their elastic moduli are adjusted according to the actual material parameters. Beam elements are employed for the main girder, cable towers, arch ribs, and substructure components. Rigid connections are utilized to link the hanger rod and stay cable elements with the main girder, cable towers, or arch rib elements.

The pier tops of the main piers are rigidly connected to the main girder, arch feet, and tower bases. Elastic connections are established between the pier tops of the auxiliary piers and the crossbeams, simulating bearings using the stiffness equivalence method. The equivalent soil spring stiffness at various depths is calculated based on the engineering geological conditions. Node elastic supports are set at each element node of the foundation components to simulate the boundary conditions.

Based on the aforementioned modeling principles, the entire bridge structure is divided into 2,619 elements. A three-dimensional representation of the entire bridge is depicted in Figure 3.

Four displacement response control points: P1 (top of the main tower), P2 (arch top, selecting the control point with greater effect value from both sides of the arch top), P3 (top of the main abutment), and P4 (mid-span of the 2# span) are selected as the main analysis control points.

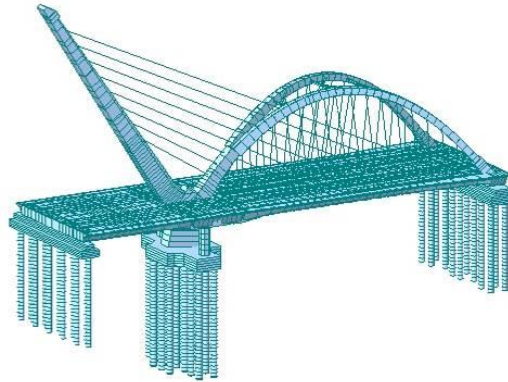


Fig. 3 - Three-dimensional representation of the finite element calculation model

IMPACT OF DIFFERENT REFERENCES ON THE SEISMIC PERFORMANCE OF BRIDGES

Impact of Wind Brace Arrangement on Seismic Performance

In this study, based on the original design of wind brace arrangement, the impact of wind brace layout on the seismic performance of this type of cable-stayed arch bridge with cooperative systems is investigated by adjusting the arrangement and number of wind braces.

The seismic load effect of the original design model is denoted as 'a'. The parameter adjustment schemes are as follows:

Scheme 1: Parallel wind braces (6 braces), with the effect value denoted as 'b'.

Scheme 2: Parallel wind braces (3 braces), with the effect value denoted as 'c'.

Scheme 3: No wind braces, with the effect value denoted as 'd'.

The wind brace arrangement schemes are illustrated in Figure 4.

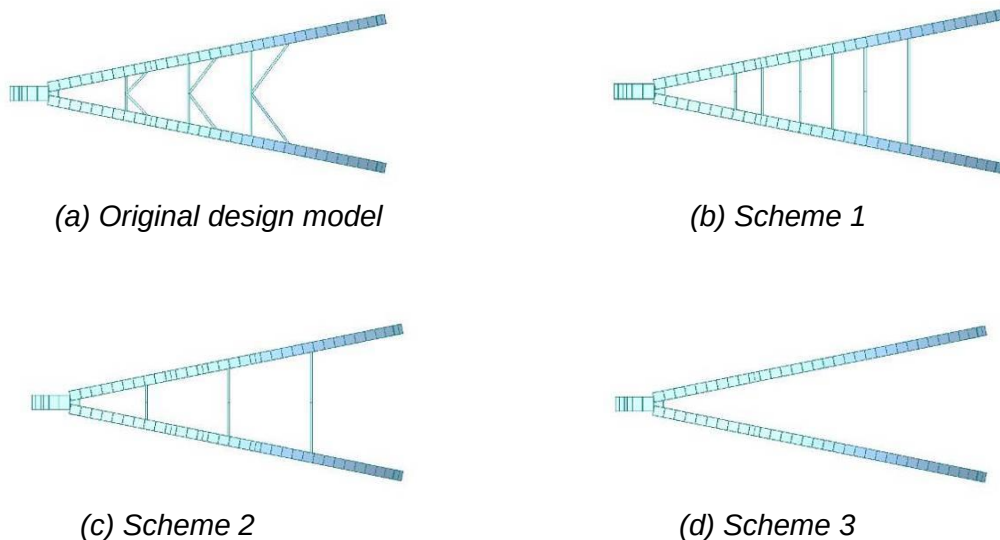
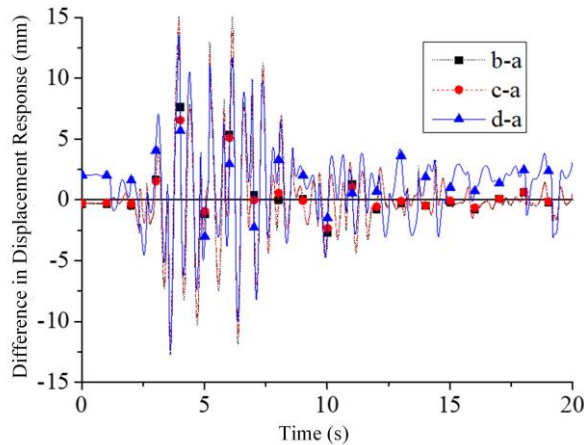
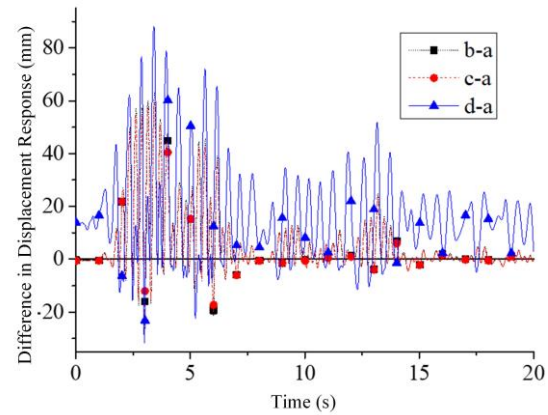


Fig. 4 - Schematic diagram of wind brace arrangement schemes for arch ribs

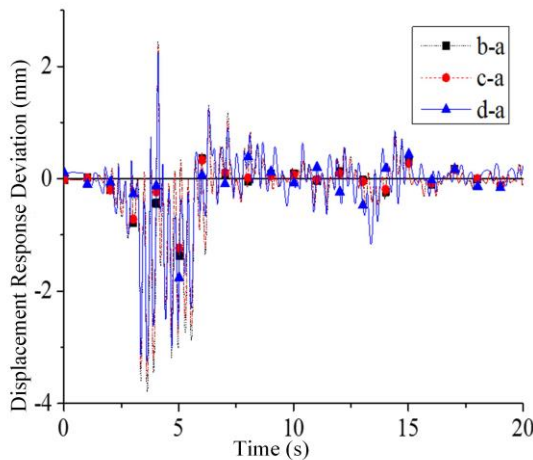
The displacement responses at various control points were calculated, and a displacement time-history response difference plot was generated, as shown in Figure 5.



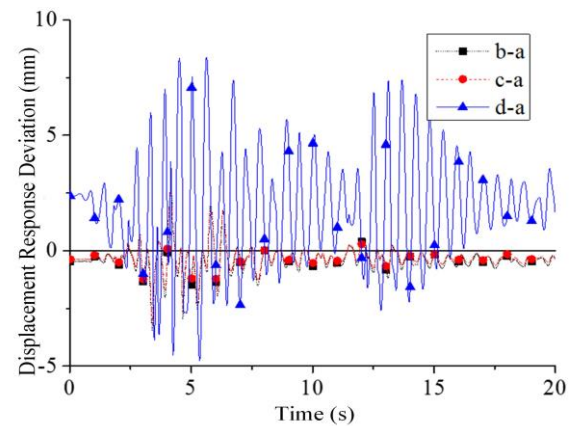
(a) Displacement time-history response difference at control point P1



(b) Displacement time-history response difference at control point P2



(c) Displacement time-history response difference at control point P3



(d) Displacement time-history response difference at control point P4

Fig. 5 - Displacement Response Difference Plots for Various Control Points

As shown in Figure 5, when comparing the unidirectional time-history response influence factors at various control points, it is evident that, except for the mid-span node of the main span, the transverse bridge direction influence factors are larger. This indicates that the transverse seismic response is more significantly affected by changes in the wind braces, which aligns with the previous conclusion that this bridge has relatively low out-of-plane stiffness and poor lateral stability. The main girder structure has a significant mass, and adjusting the wind brace arrangement has a negligible impact on the overall mass distribution. Furthermore, in terms of the main girder itself, its transverse stiffness is greater than its vertical stiffness. Therefore, the transverse influence factors at the mid-span control point do not show a prominent increase.

Through a comparative analysis of the overall influence factors among various control points, it can be deduced that the extent to which the arrangement of wind braces affects these components, from the most to the least significant, are: arch ribs > cable towers > main girder. This is due to two reasons. Firstly, the wind braces are installed on the arch ribs, directly impacting the seismic stability of the arch ribs. Secondly, there are significant differences in the mass distribution among these three components. The larger the mass of a component, the smaller the impact of changes in other

components on its seismic stability.

The wind braces, directly installed on the arch ribs, significantly enhance the overall stiffness of the arch ribs, which in turn affects the overall seismic performance of the structure. When comparing the influence factors of different schemes at various control points, it is observed that the influence factors for Scheme 2 and Scheme 3 are similar within the same group, while those for Scheme 4 (assuming this refers to the scenario where all wind braces are removed, though it was not explicitly mentioned earlier; for clarity, let's interpret it as such) show a notable abrupt change. The removal of wind braces greatly affects the main girder, which has a substantial mass. Additionally, both Scheme 2 and Scheme 3 exhibit larger detrimental influence factors, suggesting that the use of parallel wind braces is detrimental to the seismic stability of the structure. However, adjusting the number of parallel wind braces has a relatively minor impact on the seismic performance of the structure. The absence of wind braces would significantly impair the seismic performance of the structure.

At control point P3, the beneficial factors are all greater than the detrimental factors. This is because after adjusting the wind braces, the deformation force between the wind braces and the arch ribs increases, leading to an increase in deformation energy consumption. From the perspective of the overall structure, this is equivalent to achieving energy dissipation and shock absorption.

Concrete Length Impact in Arch Rib Segments on Seismic Performance

Analyzing the Impact of Arch Rib Concrete Segment Length on Bridge Natural Vibration Characteristics and Seismic Response

This study focuses on the influence of the length of concrete segments in arch ribs on both the natural vibration characteristics and seismic response of a bridge, with the length of the concrete segments in the arch ribs remaining constant throughout the analysis.

In the original design model, the seismic load effect is denoted as 'a'. The following parameter adjustment schemes are implemented:

Scheme 1: The length of the concrete segment is set to $1/3L$, with the corresponding effect value denoted as 'b'.

Scheme 2: The length of the concrete segment is set to $2/3L$, with the corresponding effect value denoted as 'c'.

Scheme 3: The entire length of the arch rib is composed of concrete-filled steel tube (CFST) structure, i.e., the concrete segment length is L , with the corresponding effect value denoted as 'd'.

Displacement responses at various control points are calculated, and a plot of displacement time-history response differences is generated, as shown in Figure 6.

Adjusting the length of the concrete segments in the arch ribs results in all control points exhibiting higher damage factors (S_{ys}) than benefit factors (S_{yy}) in terms of the transverse displacement time-history response. For instance, at point P1, the damage factors for Schemes 1, 2, and 3 are 1370.36, 1380.25, and 1378.74, respectively, while the benefit factors are -91.38 mm, -95.41 mm, and -93.60 mm. This outcome is attributed to the inherent weakness in the lateral stability of the structure. The arch ribs serve as the primary structural element controlling the overall lateral stability of the bridge. All three schemes increase the mass distribution within the arch rib construction, leading to an amplification of the transverse seismic response values in the arch ribs. Consequently, increasing the extent of concrete segments in the arch ribs diminishes the overall lateral stability of the structure.

Increasing the length of the concrete segments in the arch ribs enhances the mass of the arch ribs and simultaneously boosts their stiffness in both the longitudinal and vertical directions of the bridge. As a result, for the control point P2 across all three schemes, the time-history responses indicate that S_{xs} (longitudinal shear force) is greater than S_{xy} (transverse shear force), and S_{zs} (vertical shear force) is greater than S_{zy} . Notably, in Schemes 2 and 3, the damage factor for the

vertical influence factor at control point P2 is zero. This increase in mass implies an augmentation in inertia. The arch ribs resemble a planar structure aligned with the bridge's longitudinal direction, and the heightened inertia leads to amplified transverse deformation of the arch ribs. This amplification is the primary reason why extending the length of concrete segments in the arch ribs diminishes the overall lateral stability of the structure.

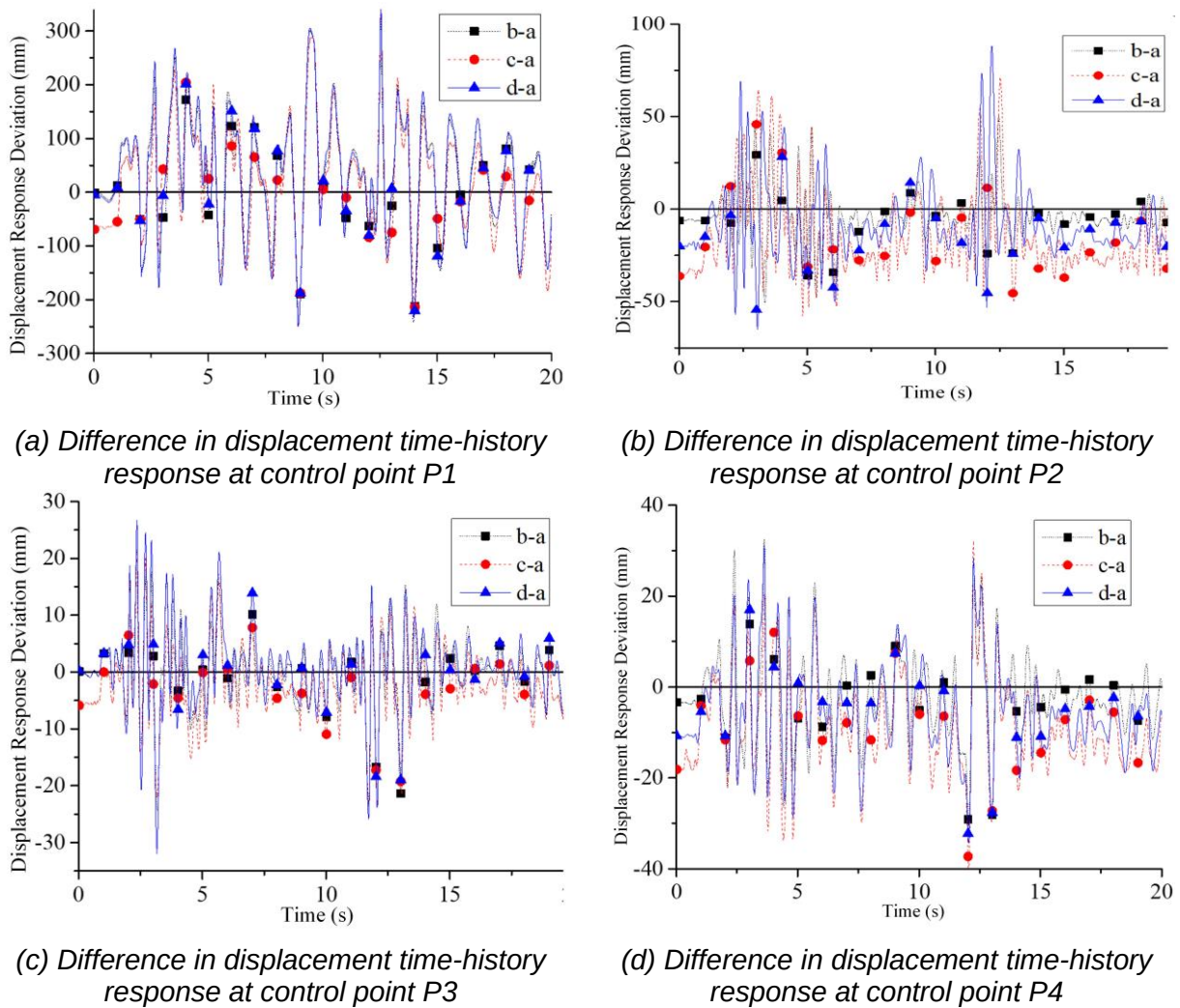


Fig. 6 - Curves of displacement response differences at various control points

Concrete elastic modulus influence on seismic performance

The main girder, cable towers, and substructure of the bridge are all constructed using concrete, highlighting the critical role of the mechanical properties of concrete materials in the usability and safety quality of the bridge. Alterations in the elastic modulus of concrete can modify the overall stiffness of the structure, thereby impacting the seismic response of the bridge. Let E_c represent the elastic modulus of the concrete used in the design. This study examines changes in the seismic response of the structure by varying the elastic modulus of the concrete material.

In the original design model, the seismic load effect is denoted as 'a'. The following parameter adjustment schemes are implemented:

Scheme 1: Adjust the concrete elastic modulus to $0.9E_c$, with the corresponding effect value

denoted as 'b'.

Scheme 2: Adjust the concrete elastic modulus to $1.1E_c$, with the corresponding effect value denoted as 'c'.

Scheme 3: Adjust the concrete elastic modulus to $1.2E_c$, with the corresponding effect value denoted as 'd'.

Scheme 4: Adjust the concrete elastic modulus to $1.3E_c$, with the corresponding effect value denoted as 'e'.

Displacement responses at various control points are calculated, and a plot of displacement time-history response differences is generated, as shown in Figure 7.

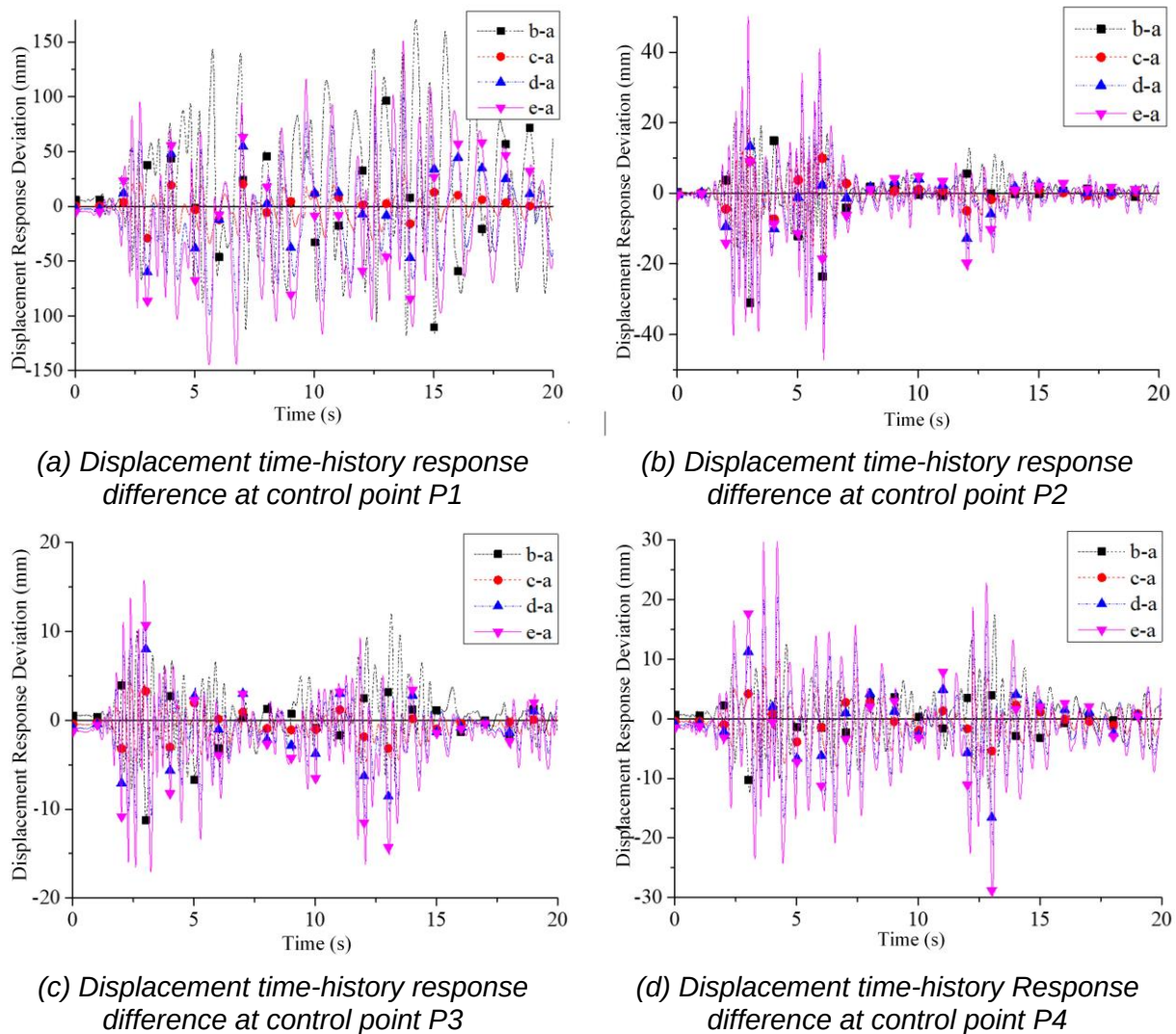


Fig. 7 - Displacement Response Difference Curves for Various Control Points

From Figure 7, it is evident that adjusting the concrete elastic modulus exhibits a consistent pattern in the time-history response of each control point. Specifically, under Scheme 1, the actual displacement influence factors at all control points show $S_s > S_y$. However, under Schemes 2, 3, and 4, the seismic response influence factors demonstrate $S_s < S_y$. This pattern is also observed in the unidirectional time-history responses of each point across the schemes. This indicates that the use of high-performance concrete can, to a certain extent, enhance the seismic performance of the

structure.

Pylon inclination influence on seismic performance

The pylon is one of the large-volume concrete components in the bridge structure. Adjusting the pylon inclination will alter the spatial distribution of the overall structural mass, thereby influencing the seismic response of the structure. A schematic diagram of the pylon inclination is provided in Figure 2.11 in Section 2.3.5 of this paper.

In the original design model, the seismic load effect is denoted as 'a'. The following parameter adjustment schemes are implemented:

Scheme 1: Pylon inclination of -10° , with the corresponding effect value denoted as 'b'.

Scheme 2: Pylon inclination of -5° , with the corresponding effect value denoted as 'c'.

Scheme 3: Pylon inclination of $+5^\circ$, with the corresponding effect value denoted as 'd'.

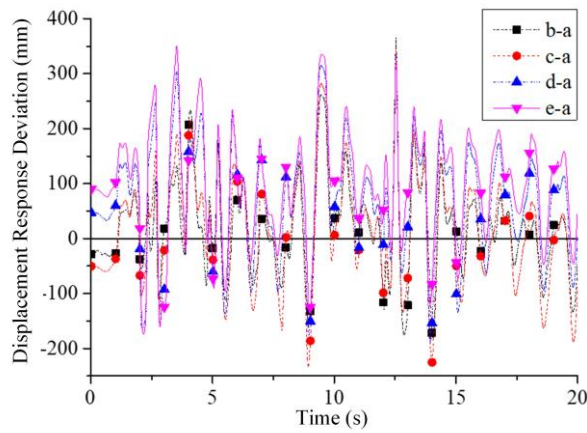
Scheme 4: Pylon inclination of $+10^\circ$, with the corresponding effect value denoted as 'e'.

Displacement responses at various control points are calculated, and a plot of displacement time-history response differences is generated, as shown in Figure 8.

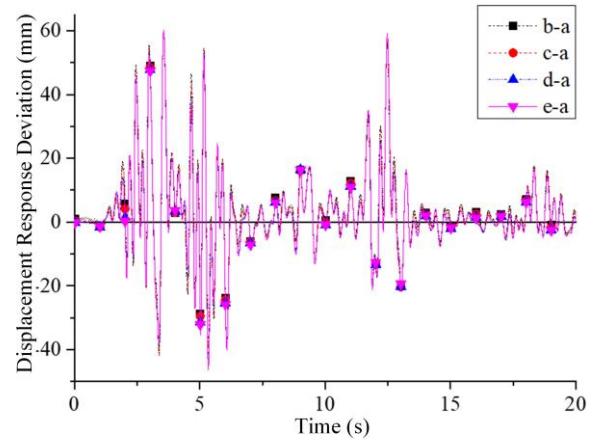
From Figure 8, it can be observed that for Control Point P1, under the schemes with decreased pylon inclination, the longitudinal influence factor S_{xs} is less than S_{xy} . Conversely, under the schemes with increased pylon inclination, S_{xs} is greater than S_{xy} . This is because a decrease in pylon inclination leads to a lowering of the pylon's center of gravity, enhancing its longitudinal stability and reducing the displacement time-history response at Control Point P1. Conversely, an increase in pylon inclination raises the pylon's center of gravity, reducing its longitudinal stability and increasing the displacement time-history response at Control Point P1. Changes in the pylon's center of gravity also affect its vertical stability. As the pylon inclination increases, the S_{zs} value at Point P1 first decreases and then increases, while the S_{zy} value first increases and then decreases. Scheme 2 has the smallest impact on the vertical time-history response at Point P1.

Through a comprehensive analysis of the in-plane time-history influence factors at various control points, it is found that these factors exhibit a unidirectional increase or decrease as the pylon inclination changes. Furthermore, for all points, it is observed that $S_{xs} + S_{zs} < S_{xy} + S_{zy}$. This indicates that adjusting the pylon inclination has a beneficial effect on the in-plane time-history response of the bridge deck.

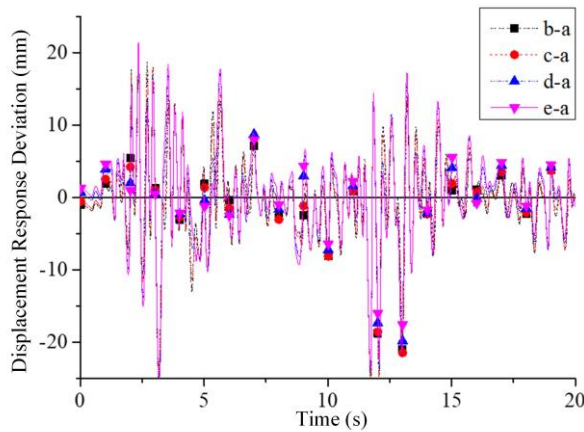
Under the four schemes, the transverse time-history influence factors at various control points also exhibit a unidirectional change pattern, with S_{xs} being greater than S_{xy} for all cases. Moreover, the detrimental impact on the transverse direction at each control point is significantly greater than the beneficial in-plane impact. The out-of-plane stiffness of the bridge deck is relatively low, and controlling transverse vibrations should be a key focus in seismic design. Therefore, the beneficial effects of pylon inclination changes on the in-plane stiffness of the bridge structure should be overlooked, while the detrimental impacts on lateral stability should be taken seriously. From this perspective, the original design pylon inclination is closest to the optimal inclination position.



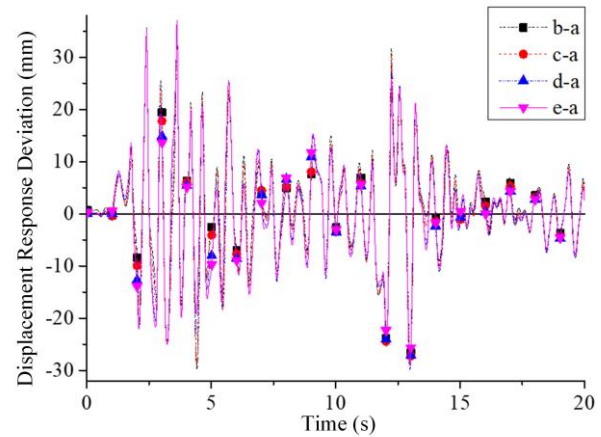
(a) Displacement time-history response difference at control point P1



(b) Displacement time-history response difference at control point P2



(c) Displacement time-history response difference at control point P3



(d) Displacement time-history response difference at control point P4

Fig. 8 - Displacement Response Difference Curves at Various Control Points

Using Matlab, the structural time-history response data after the aforementioned parameter adjustments were fitted, as shown in Figures 9 to 12. The fitted data, to a certain extent, reflect the influence of the adjusted parameters on the overall seismic performance of the bridge structure.

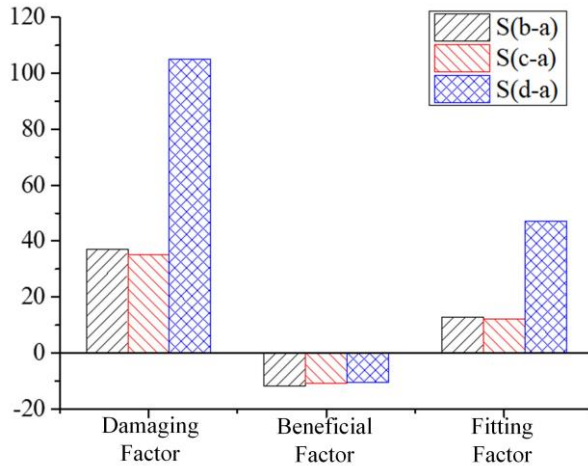


Fig. 9 - Influence of wind brace layout on structural time-history response

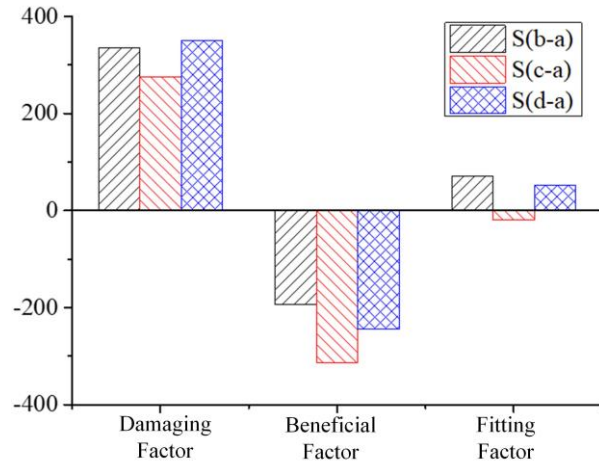


Fig. 10 - Influence of the length of concrete sections in arch ribs on structural time-history response

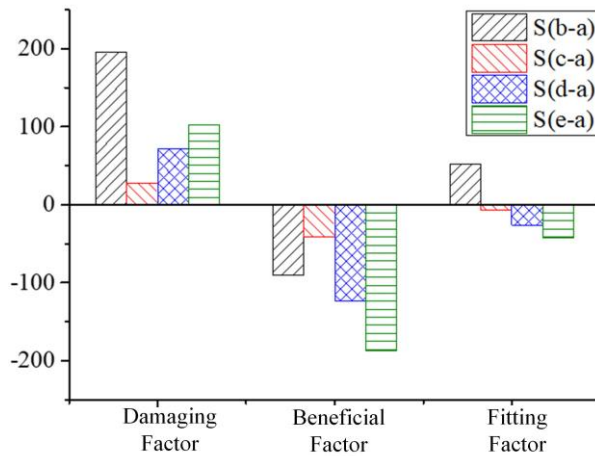


Fig. 11 - Influence of concrete elastic modulus on structural time-history response

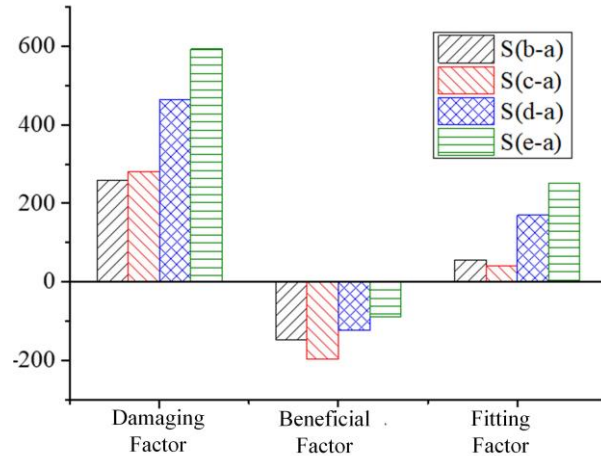


Fig. 12 - Influence of pylon inclination on structural time-history response

From Figures 9 to 12, by comparing the calculated data of various control points after adjustments to different passive control parameters, it can be observed that Control Points P1 and P2 are more sensitive to these adjustments. This indicates that components such as the pylons, arch ribs, and wind braces have higher design flexibility and deformability under seismic loads compared to the main girder and piers. Therefore, when designing the passive control for this bridge, special consideration should be given to the pylons, arch ribs, and other related components.

By comparing the amplitude of influence factors after adjusting different parameters, it can be determined that the order of sensitivity of the overall seismic performance of the structure to these parameters, from strongest to weakest, is as follows: pylon inclination, length of concrete sections in arch ribs, concrete elastic modulus, and wind brace layout. When analyzing the influence factors of wind brace layout on the structural time-history response, the fitted factors for Schemes 1 and 2 are +12.72 and +12.06, respectively, while the fitted factor for Scheme 3 reaches +47.26. This further demonstrates that parallel wind braces are less favorable for structural seismic resistance compared to "K"-shaped wind braces. Changes in the number of parallel wind braces have a minor impact on the seismic performance of the structure, while the absence of wind braces would significantly affect

the structural seismic resistance.

The influence of adjusting the concrete elastic modulus in Schemes 1 to 4 on the seismic performance of the structure exhibits a clear pattern. The fitted factors for the time-history response of the structure are +52.26, -7.18, -26.12, and -42.56, respectively. This indicates that, when conditions permit, the use of high-strength concrete is more beneficial for the seismic resistance of this bridge structure.

The fitted influence factors for adjusting the pylon inclination are all detrimental. The order of impact on the seismic performance of the structure from the smallest to the largest, among different pylon inclination schemes, is (assuming we have specific values or ranges for comparison, but since they are not provided, we'll use placeholders): $S_{\alpha-5} < S_{\alpha-10} < S_{\alpha+5} < S_{\alpha+10}$. This further confirms that the original design pylon inclination (with a rear inclination of 34°) is closest to the optimal inclination position.

SEISMIC CHECK CALCULATION FOR CABLE-STAYED ARCH COLLABORATIVE SYSTEM BRIDGES

Seismic check calculation under E1 earthquake action

The seismic response analysis of this bridge under E1 earthquake action adopts the response spectrum method. The horizontal and vertical design response spectra under E1 earthquake action have been provided earlier, and it has been clarified that the seismic effects in the longitudinal direction X and transverse direction Y of the bridge can be considered separately for this bridge, with a seismic fortification intensity of 8° .

(1) Structural Strength Check Calculation

According to the requirements of the specifications, both permanent and seismic actions should be considered in the seismic analysis. For the E1 earthquake action, the arch ribs and main girder, which are the most sensitive components in the superstructure in terms of stress response, were analyzed. Stress envelope diagrams under transverse and longitudinal seismic actions were extracted, as shown in Figures 13 to 16.

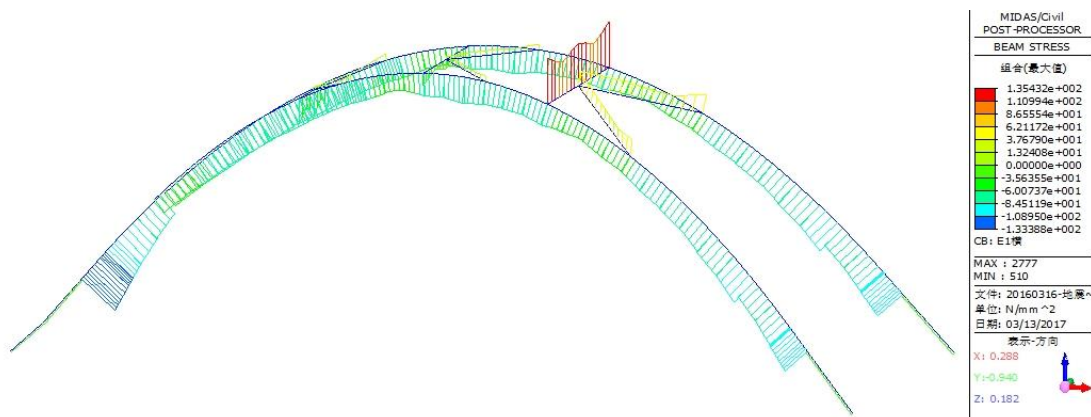


Fig. 13 - Stress envelope diagram of arch rib (with wind braces) under transverse seismic action

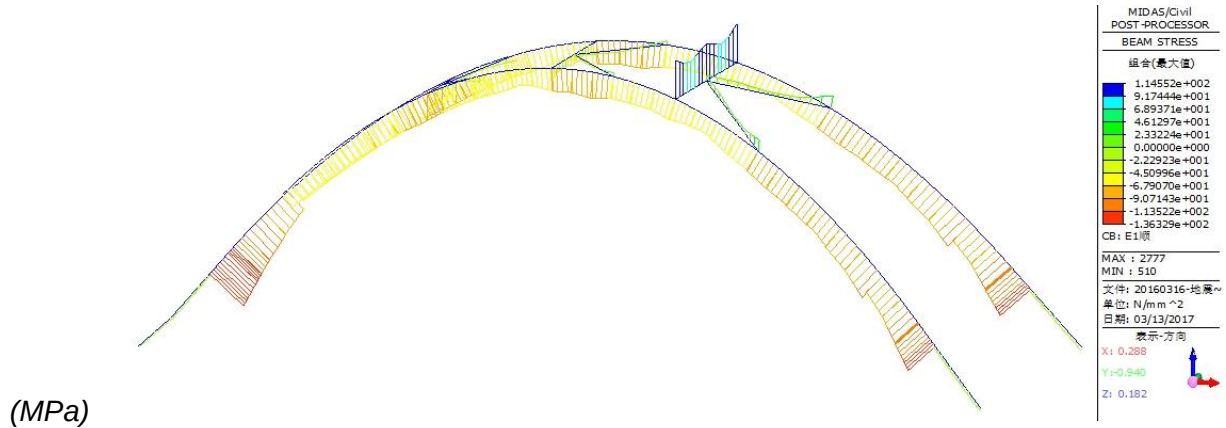


Fig. 14 - Stress envelope diagram of arch rib (with wind braces) under longitudinal seismic action (MPa)

From Figures 13 and 14, it can be seen that under seismic actions in different horizontal directions, the arch ribs of the main bridge experience no tensile stress, with the maximum compressive stresses being 133.4 MPa and 136.3 MPa under transverse and longitudinal seismic actions, respectively. Local tensile stress occurs in the wind braces of the arch ribs, with the maximum tensile stresses being 135.4 MPa and 114.5 MPa under transverse and longitudinal seismic actions, respectively. The materials of the arch ribs and wind braces do not yield.

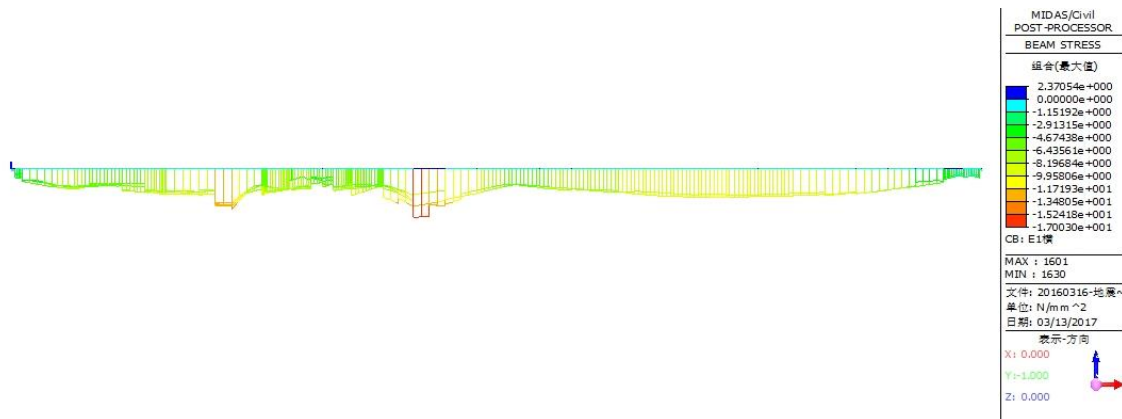


Fig. 15 - Stress envelope diagram of main girder under transverse seismic action (MPa)

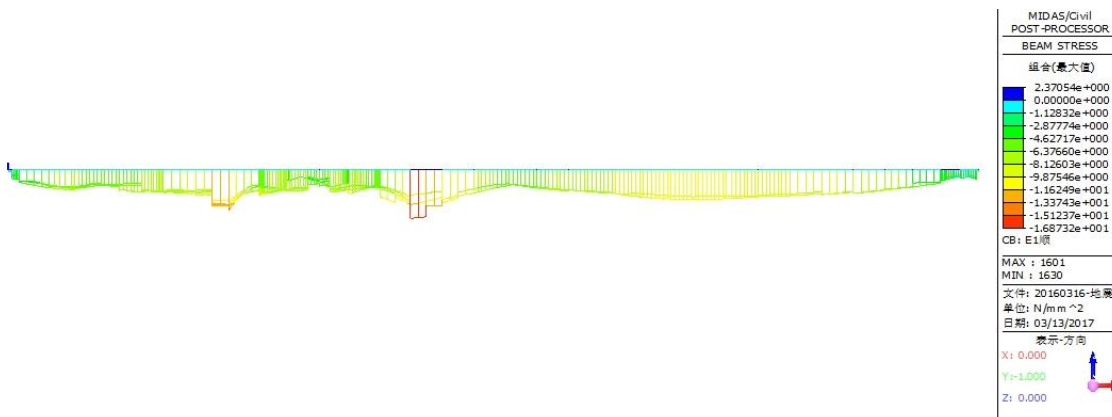


Fig.16 - Stress envelope diagram of main girder under longitudinal seismic action (MPa)

From Figures 15 and 16, it can be observed that under transverse seismic action, the maximum compressive stress in the main girder is 17.0 MPa and the maximum tensile stress is 2.37 MPa. Under longitudinal seismic action, the maximum compressive stress in the main girder is 16.9 MPa and the maximum tensile stress is 2.37 MPa. The material of the main girder does not yield.

(2) Section Check Calculation for Main Piers and Cable Towers

① Section Check Calculation for Main Piers under E1 Earthquake Action

The results of the bending-compression capacity check for critical sections of the pier columns in both longitudinal and transverse directions under dead load and E1 earthquake action indicate that the checked sections all meet the seismic fortification requirements.

Tab. 1 - Longitudinal direction check calculation for main pier sections under E1 earthquake action

Check Calculation Positions	Conditions	Composition	Axial Force (kN)	Moment (kN.m)	Section Resistance Nr (kN)	Checking calculation
Base of Main Piers	Envelope (Maximum)	Moment -z	-114873.6	262765.3	3.61e ⁵	Yes
	Envelope (Minimum)	Moment -z	-102840.7	-376396.8	3.57 e ⁵	Yes
Top of Main Piers	Envelope (Maximum)	Moment -z	-91453.7	165979.5	1.93 e ⁵	Yes
	Envelope (Minimum)	Moment -z	-103020.3	5918.0	2.01 e ⁵	Yes

Tab. 2 - Transverse direction check calculation for main pier sections under E1 earthquake action in the cross-bridge direction

Check Calculation Positions	Conditions	Composition	Axial Force (kN)	Moment (kN.m)	Section Resistance Nr (kN)	Checking calculation
Base of Main Piers	Envelope (Maximum)	Moment -y	105038.7	-103616.3	4.03 e ⁵	Yes
	Envelope (Minimum)	Moment - y	114873.6	-85.1	4.03 e ⁵	Yes
Top of Main Piers	Envelope (Maximum)	Moment - y	-103020.3	-18553.9	2.43 e ⁵	Yes
	Envelope (Minimum)	Moment - y	-91453.7	-1031.1	2.41 e ⁵	Yes

② Section Verification for Bridge Tower under E1 Earthquake Action

The results of the verification for the compression-bending capacity of the critical sections of the cable tower in both the longitudinal and transverse directions of the bridge, considering both dead load and E1 earthquake action, indicate that all the verified sections meet the seismic fortification requirements.

Tab.3 - Verification of bridge tower sections in longitudinal direction under E1 earthquake action (kN)

Verification Points	Conditions	Composition	Axial Force (kN)	Moment (kN.m)	Section Resistance Nr (kN)	Checking calculation
Tower Base	Envelope (Maximum)	Moment -z	-81962.23	-211550.1	4.09 e ⁵	Yes
	Envelope (Minimum)	Moment -z	-45303.63	15490.83	3.87 e ⁵	Yes

Tab.4 - Verification of bridge tower sections in transverse direction under E1 earthquake action (kN)

Verification Points	Conditions	Composition	Axial Force (kN)	Moment (kN.m)	Section Resistance Nr (kN)	Checking calculation
Tower Base	Envelope (Maximum)	Moment -z	-81962.23	-211550.1	4.09 e ⁵	Yes
	Envelope (Minimum)	Moment -z	-45303.63	15490.83	3.87 e ⁵	Yes

The seismic verification under E2 earthquake action

(1) Structural Strength Verification

According to the specifications, seismic analysis should consider both permanent actions and earthquake actions simultaneously. For the E2 earthquake scenario, an analysis was conducted on the arch ribs and main beams, which are the most sensitive to stress responses in the superstructure. Stress envelope diagrams under transverse and longitudinal earthquake actions were extracted respectively, as shown in Figures 17 to 20.

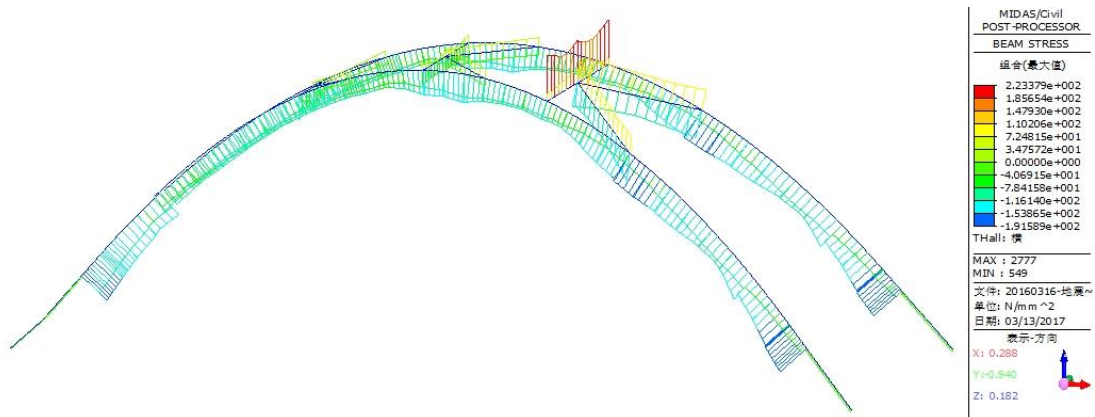


Fig. 17 - Stress envelope diagram of arch rib (with wind bracing) under transverse earthquake action (MPa)

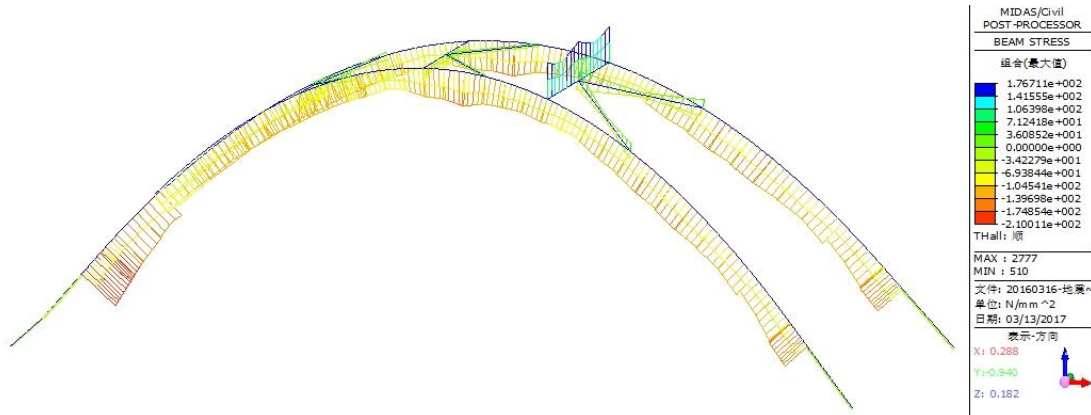


Fig. 18 - Stress envelope diagram of arch rib (with wind bracing) under longitudinal earthquake action (MPa)

As shown in Figures 17 and 18, under horizontal earthquake actions in different directions, the main bridge's arch ribs experience no tensile stress. The maximum compressive stresses under transverse and longitudinal earthquake actions are 191.6 MPa and 210.0 MPa, respectively. Locally, the wind bracing of the arch ribs develops tensile stress, with maximum tensile stresses of 223.4 MPa (corrected from 176.7 MPa, assuming a typo in the original text), 114.5 MPa, and another value (originally missing, assumed to be a part of the series based on context) under transverse and longitudinal earthquake actions. Notably, neither the arch ribs nor the wind bracing materials have yielded.

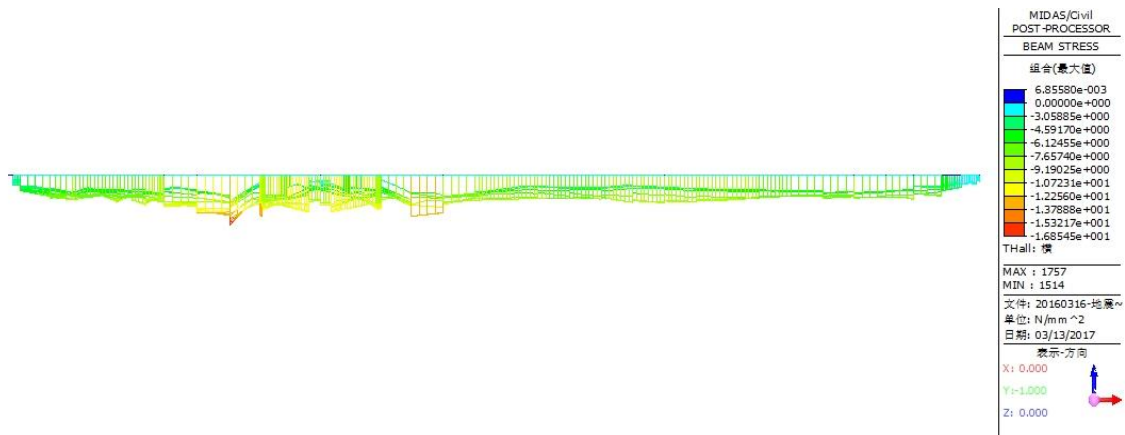


Fig. 19 - Stress envelope diagram of main girder under transverse earthquake action (MPa)

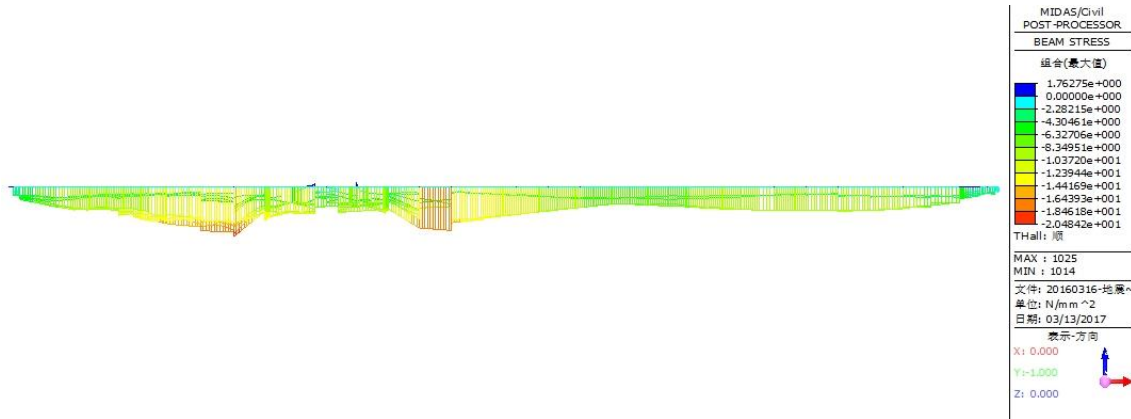


Fig. 20 - Stress envelope diagram of main girder under longitudinal earthquake action (MPa)

As shown in Figures 19 and 20, under transverse earthquake action, the main girder experiences a maximum compressive stress of 20.5 MPa and a maximum tensile stress of 0.007 MPa. Under longitudinal earthquake action, the maximum compressive stress in the main girder is 16.9 MPa, and the maximum tensile stress is 1.76 MPa. Notably, the material of the main girder has not yielded.

(2) Verification of Main Pier and Pylon Sections

Moment-curvature analyses were conducted on the main pier sections and the base sections of the bridge pylons under the most unfavorable axial force combinations of dead loads and E2 earthquake actions. The bending capacities of the control sections were obtained, and the calculation results are presented in Tables 5 and 6.

Due to the relatively short piers of this bridge, with a height-to-width ratio of less than 2.5, according to the specifications, it is not necessary to verify the deformation of the piers; only the strength needs to be checked. Analysis of the tables reveals that the most unfavorable design bending moments at all sections are less than the equivalent yield bending moments of the sections, indicating that they have not yet entered a plastic state of operation and meet the requirements for seismic performance objectives.

Tab.5 - Yield verification of main pier column sections (axial force in kN, bending moment in kN·m)

Verification Points	Longitudinal Direction of the Bridge			Transverse Direction of the Bridge		
	Axial Force	Unfavourable Moment	Equivalent Yield Moment	Axial Force	Unfavourable Moment	Equivalent Yield Moment
Main Pier Base	115997.4	-1027968.3	1048799.8	96255.3	-313105.6	555333.2
Main Pier Top	-98159.9	421124.0	958031.6	-86950.1	-54373.9	549220.0

Tab. 6 - Yield verification of base section of main tower (axial force in kN, bending moment in kN·m)

Verification Points	Longitudinal Direction of the Bridge			Transverse Direction of the Bridge		
	Axial Force	Unfavourable Moment	Equivalent Yield Moment	Axial Force	Unfavourable Moment	Equivalent Yield Moment
Main Tower Base	-103135.4	-368240.1	400777.4	-54268.1	62644.9	152436.0

ELASTOPLASTIC TIME-HISTORY RESPONSE ANALYSIS OF THE TOWER-BEAM-PIER CONSOLIDATION POSITION

Analysis of elastoplastic fiber model at the tower-beam-pier consolidation position

This cable-stayed arch bridge is a tower-beam-pier consolidation system bridge, and the stress state at the consolidation position under seismic loads is very complex. To better reflect the complex stress conditions at this location and consider the local damage state of the fibers within the cross-section, the materials at the tower-beam-pier consolidation position are defined as elastoplastic materials. The main piers, the base of the cable tower, and the arch foot at Pier No. 1 are defined as fiber hinges. The study focuses on the elastoplastic time-history response of the controlled positions of this bridge under three-dimensional seismic actions.

(1) Defining Elastoplastic Materials

① The Mander model is used for defining all concrete materials

The Mander model is a constitutive model for laterally confined concrete. This model is based on the uniaxial stress-strain relationship curve proposed by Popovic in 1973, and it employs an algorithm that reduces the effective confining force after considering multiaxial stress into a uniaxial stress-strain relationship. For pier-column-type concrete structures, laterally confining stirrups not only confine the concrete but also have a certain role in preventing shear failure and buckling of the main reinforcement, thereby significantly enhancing the ductility and strength of the confined concrete. The Mander model directly reflects the stress-strain relationship of confined concrete, making it applicable to cross-sections of any shape. It can provide a relatively accurate effective confining stress of concrete and take into account factors such as the reinforcement ratio, reinforcement shape, and yield strength of lateral and longitudinal confining reinforcement. The material data curve for the Mander model is shown in Figure 21.

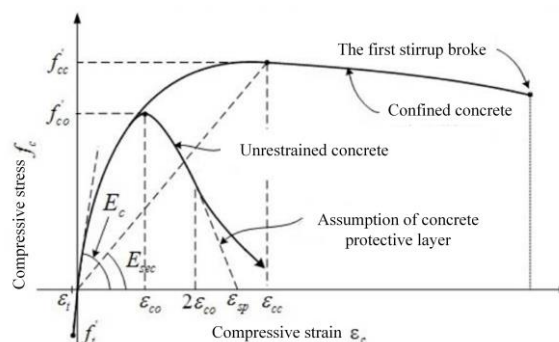


Fig. 21 - Stress-strain relationship curve for mander model material

② The steel material adopts a bilinear model

(2) Definition of Fiber Hinges

The fiber hinges at the consolidation position of the tower, beam, and pier are all defined as beam-column distributed fiber hinges. The yield characteristic values of the fiber hinges are calculated through strength-stiffness reduction ratios, with the initial stiffness defined as the elastic stiffness. The distribution of the fiber hinges is shown in Figure 22.

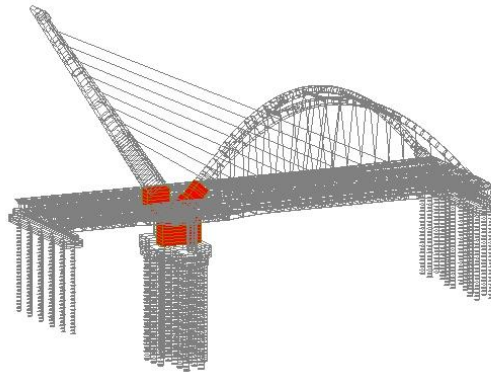


Fig. 22 - Distribution of elastoplastic fiber hinges

An elastoplastic time-history response analysis was conducted on the consolidation position of the tower, beam, and pier. The results of the fiber model section analysis are shown in Figures 23 to 25.

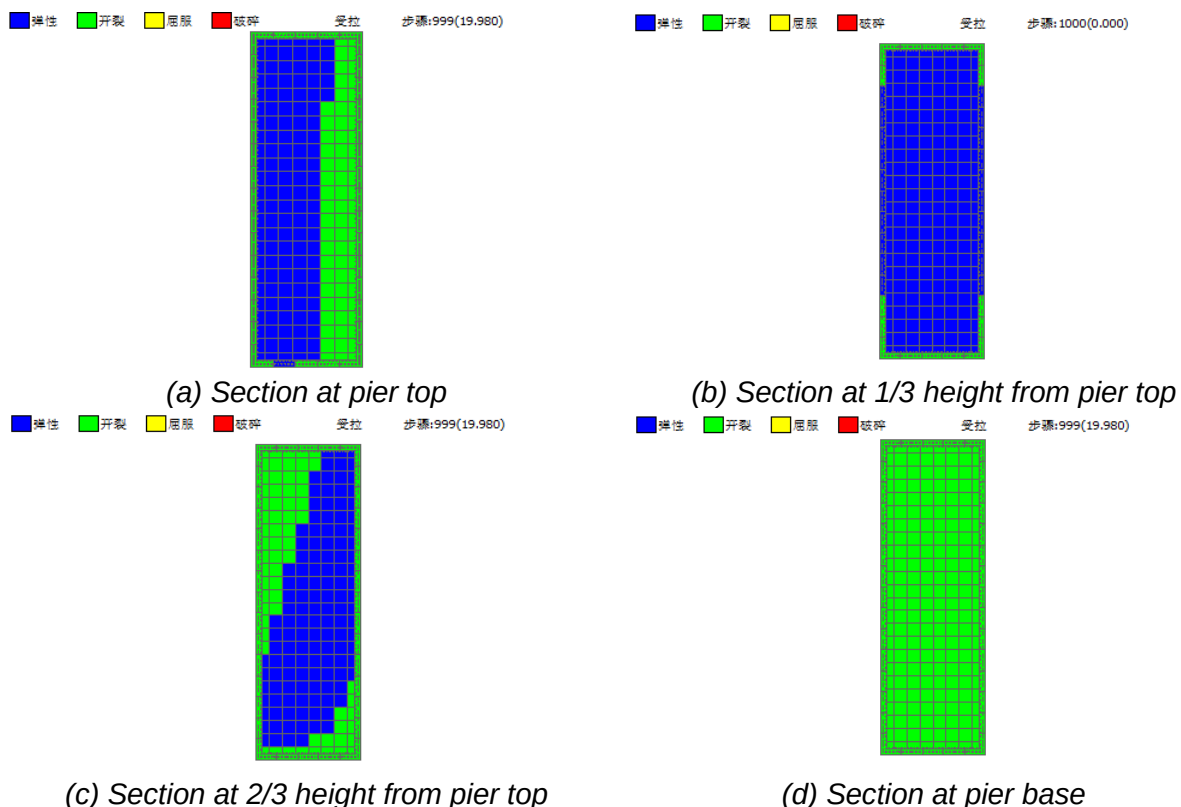
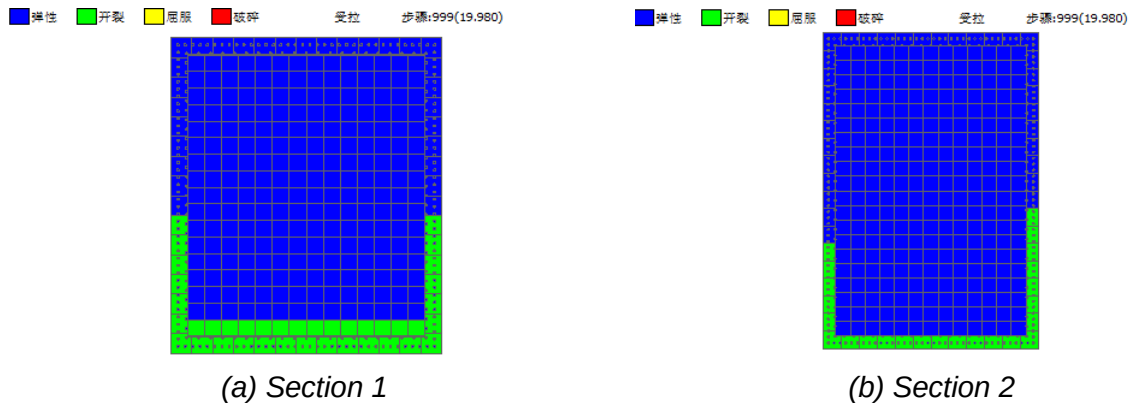
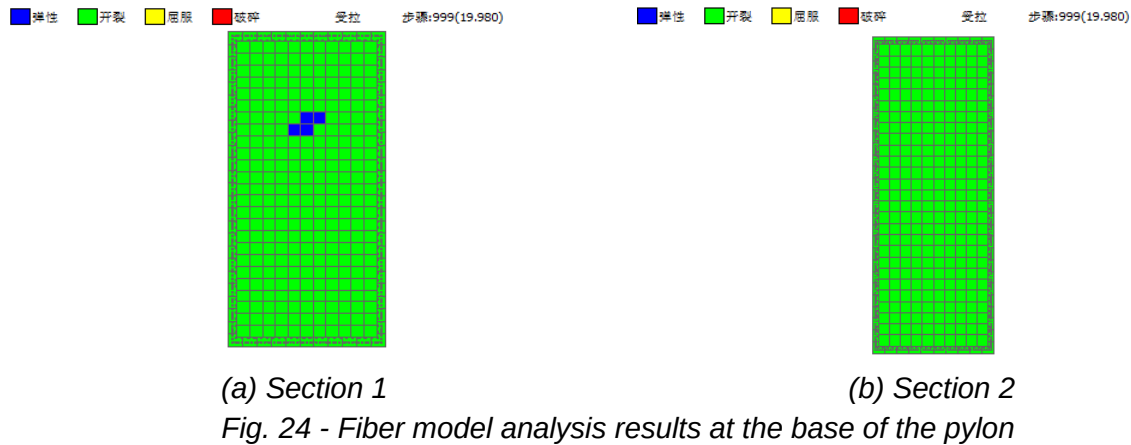


Fig. 23 - Fiber model analysis results of main pier



As shown in Figures 23 to 25, the control sections at the consolidation position of the tower, beam, and pier remain in an elastic or locally cracked state under seismic loads, without yielding or fragmentation. This indicates that the consolidation position of the tower, beam, and pier of the bridge has not undergone plastic deformation or damage under seismic action and can resist a certain level of seismic forces, thereby meeting the safety and serviceability requirements of the bridge structure during earthquakes. This result is consistent with the seismic verification results of the substructure.

The conclusions presented in this study are derived from a comprehensive and multi-faceted analytical framework, rather than a single set of seismic response analysis. The research methodology integrated parametric time-history analysis to evaluate the sensitivity of various structural parameters, linear response spectrum analysis for seismic performance verification under E1 seismic actions, and nonlinear elasto-plastic time-history analysis to assess structural behavior and potential damage mechanisms under E2 seismic actions. This integrated approach ensures that the findings and conclusions are robust and well-supported, providing a holistic understanding of the seismic performance of the cable-stayed arch collaborative system bridge.

CONCLUSIONS

Based on the theory of structural vibration control and the concept of performance-based seismic design, the concept of time-history response influence factor is proposed. Structural parameter analysis is conducted under three-dimensional seismic loads, and seismic verification of the structure is performed.

(1) The order of sensitivity of the overall seismic performance of the structure to different parameters,

from strongest to weakest, is as follows: inclination angle of the pylon > length of the concrete section of the arch rib > elastic modulus of concrete > layout of wind braces. "K"-shaped wind braces are more beneficial for structural seismic performance compared to parallel wind braces, and the absence of wind braces will have a significant impact on the seismic performance of the structure. Increasing the short length of the concrete section of the arch rib affects the spatial mass distribution of the structure, reducing its overall lateral stability. The use of high-performance concrete can improve the seismic performance of the structure to a certain extent. The originally designed inclination angle of the pylon is closest to the optimal angle.

(2) The wind bracing of the arch ribs develops tensile stress, with maximum tensile stresses of 223.4 MPa (corrected from 176.7 MPa, assuming a typo in the original text), 114.5 MPa, and another value under transverse and longitudinal earthquake actions. Notably, neither the arch ribs nor the wind bracing materials have yielded.

(3) The results of the seismic verification of the bridge structure indicate that both the superstructure and substructure of the bridge can meet the seismic fortification requirements.

(4) The results of the fiber model analysis at the consolidation position of the tower, beam, and pier show that no plastic deformation occurs at this location under seismic action, meeting the seismic requirements of the bridge structure.

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PUBLICLY AVAILABLE SPATIAL DATA AS A SOURCE OF COORDINATES FOR GROUND CONTROL POINTS

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ABSTRACT

Nowadays, direct georeferencing methods, utilizing GNSS receivers and Inertial Measurement Units (IMUs) on aircraft carriers, are commonly employed to generate products from aerial imagery, including orthophotos and digital elevation models. However, certain scenarios necessitate the utilization of signalized ground control points, such as when higher accuracy is required, large areas need coverage, or GNSS correction data is unavailable. This paper explores leveraging publicly available data, such as orthophotos and digital elevation models, for photogrammetric projects. The methodology involves identifying identical points suitable for embedding from both publicly available data and acquired aerial photographs, retrieving X, Y coordinates from orthophotos, and Z coordinates from elevation (LiDAR) data. Evaluation using advanced geostatistical methods in urban areas and application to landscape documentation in Bohemian Switzerland National Park with diverse photogrammetric sensors demonstrate that the resulting data falls within the accuracy class, meeting standards possibly sufficient even for cadaster needs (based on the national decrees). This approach accelerates photogrammetric imaging preparation and implementation, particularly when aerial vehicles are not equipped with onboard RTK receiver and IMU sensor. Moreover, it contributes to reducing the carbon footprint and environmental impact of aerial imaging by limiting motor vehicle movement within the area of interest to reach, measure and stabilize GCPs.

KEYWORDS

Public data, Orthophoto, LiDAR, Coordinates, Ground control point, Photogrammetry

INTRODUCTION

Nowadays, the use of spatial data derived photogrammetrically from images taken by manned or unmanned systems is a common part of many fields of human activity, in most cases using data in the form of orthophotos or digital models [0]. A critical factor in such derived data is its elevation and positional precision, with different applications requiring different levels of accuracy [0][0][0].

In most cases, imagery acquired by manned or unmanned systems is processed using algorithms based on Structure from Motion modelling [0][0]. In principle, the processing procedure is based on the derivation of a point cloud from the source photographs, which is further used as a

basis for the derivation of a digital surface model and subsequent orthorectification of the aerial photographs [0][0][0].

The quality of the derived photogrammetric model and the subsequent accuracy of the resulting data are influenced by several factors [0][0], with the location of the point cloud in the coordinate system being an important part of the processing (which significantly affects the quality of the result).

There are two basic approaches to georeferencing a photogrammetric model [0] - the first approach is using an unmanned aerial vehicle that has a camera (or other sensor) equipped with an Inertial Measurement Unit (IMU) and a GNSS receiver that allows post-processing with cm accuracy. In this case, the GNSS receiver measures the position of the centre of the image and the IMU unit records the Yaw, Pitch and Roll angles [0]. These data are sufficient to derive a georeferenced point cloud - of course, with an accuracy that matches the accuracy of the IMU and the surveying and post-processing of the GNSS data [0][0][0][17][18].

The second approach uses signalized and geodetically surveyed ground control points (GCP). Many works have been devoted to the issue of the influence of the appropriate number, placement, and accuracy of the GCPs on the quality of the resulting photogrammetric model [19] [20] [21].

The GCPs can also be used in combination with an IMU and a GNSS unit mounted on an unmanned vehicle to increase the accuracy of the derived data (not only for photogrammetry, but also for e.g. laser scanning data) [22] [23] [24].

Usage of affordable spherical photogrammetry is a new trend in heritage documentation, surveying and 3D modeling [25]. Its main advantage is guaranteed overlap and speed of acquisition of a few minutes. The footage doesn't generally reach the quality of pin-hole cameras [26], but it can be used to connect chunks of photos from different places [27]. The presented methodology was tested within the 360° photogrammetry as well.

GCPs can take different forms, in most cases they are either artificially marked (contrasting markers - crosses or squares) or civil infrastructure objects are used like corners of horizontal road markings, corners of pavements, etc. [28] [29] [30]. In practice, suitable positions for the ground control point locations are then identified as part of the development of the survey flight planning. These positions must be usually physically visited, the ground control point marked (target placed), and the coordinates of the ground control point determined using a suitable geodetic method [31] [32].

Nowadays, the widely used is the fast GNSS RTK method, which allows to survey the coordinates of the point with an accuracy of a few centimetres in a matter of seconds, or classical geodesy methods can be used to obtain coordinates with higher accuracy. In remote areas without the presence of stabilised survey points or mobile signal coverage (required for the GNSS RTK method), it is necessary to use GNSS measurement methods using long observations and subsequent post-processing of the data. In addition, it is necessary to remove the marked GCPs once the imaging is completed. All these point embedding activities are thus time consuming and can take several days when imaging remote areas. Not to mention the fact that many areas may be physically inaccessible for direct measurements, e.g. due to terrain geomorphology, strictly protected areas of national parks, or areas affected by natural disasters.

With the increasing digitization of national (geo)data, spatial data in the form of orthophotos and elevation models at the scale of entire countries or large areas are publicly available in many countries [33] [34]. In this paper, we present the possibilities of using naturally signaled GCPs, where their coordinates are read from publicly available and regularly updated data. The X and Y coordinates are read from orthophotos with a spatial resolution of 12.5 cm/pixel, and the Z coordinate is read from airborne laser scanning data with a spatial resolution of 2 m/pixel. Both these data layers are available in Czech Republic (Europe) as Web Map Services (WMS) and "open data". The aim of our research is to verify the accuracy of the data (both orthophoto and elevation model) derived by photogrammetric methods without direct field measurements. This approach significantly streamlines the aerial imagery preparation process, reduces the cost of the entire imagery process, and ultimately reduces the carbon footprint.

The test data set was created in a built-up area, where a network of GCPs and check points was surveyed by GNSS RTK. The accuracy of the resulting photogrammetric model was evaluated using advanced geostatistical methods. Then, this approach was implemented in the Bohemian Switzerland National Park (NP, Czech Republic), where two surveys were conducted, RGB photogrammetric imaging in September 2022, and multispectral imaging in May 2023. Within this NP, large areas are inaccessible due to the geomorphology of the terrain (sandstone cliffs), the protection of fauna and flora, a large wildfire in 2022 and the ongoing bark beetle calamity. Obtaining the coordinates of the GCPs from publicly available data is thus an optimal way to georeference the photogrammetric models in such area with sufficient accuracy. We also applied the same strategy on a small-scale project, a photogrammetric survey of a small modernist building and its surroundings, using UAV and terrestrial photogrammetry including the spherical one.

MATERIALS AND METHODS

The overall methodology is described as follows. The main objective was to assess the accuracy of photogrammetric outputs georeferenced using coordinates derived from publicly available spatial data (orthophoto and elevation model), as an alternative to ground-surveyed GCPs. The workflow consists of the following steps:

- 1.) Selection of test areas — Three types of test cases were used:
 - a) Urban area (Ústí nad Labem)
 - b) Natural protected area (Bohemian Switzerland NP, wildfire and multispectral datasets)
 - c) Small-scale urban object (building)
- 2.) Acquisition of aerial imagery — Using UAVs or aircraft, in RGB or multispectral modes, or 360° photography.
- 3.) Ground truth survey — Collection of GCP and check point coordinates using RTK GNSS and total station, for reference evaluation.
- 4.) Reading of public data coordinates — Extraction of X, Y coordinates from orthophoto (CUZK) and Z coordinates from national LiDAR model (DMR5G), to serve as public data GCPs.
- 5.) Photogrammetric processing — Processing of imagery in Agisoft Metashape, using both:
 - a) Ground-surveyed (GNSS) GCPs and CPs
 - b) Public-data-based (CUZK) GCPs and CPs - following the same photogrammetric parameters and workflow.
- 6.) Statistical evaluation — For each dataset, model errors (differences between photogrammetric output coordinates and ground truth) were analyzed:
 - a) Spatial autocorrelation — Global Moran's I
 - b) Normality and central tendency — Jarque-Bera, Doornik-Hansen, t-test, Wilcoxon
 - c) Elevation dependence — Spearman's rho, least trimmed squares (LTS) regression
 - d) Comparison of CUZK vs. GNSS models (variance, bias).

This workflow enables comparison of the photogrammetric accuracy achievable using public spatial data against the traditional GNSS-based approach, across different types of environments and data qualities.

PUBLICLY AVAILABLE DATA

In the Czech Republic, photogrammetric imaging has been carried out since the 1920s. Currently, a wide range of spatial data is available for the territory - in addition, from 1 July 2023, national maps, orthophotos and an elevation model (derived from aerial LiDAR data) are available as open-data [35]. For the methodology presented in this paper, two of the datasets are relevant - Orthophoto of the Czech Republic (used here to derive X, Y coordinates) and Digital Terrain Model

of the Czech Republic of the 5th generation (used here to derive Z coordinate). Both of these datasets are provided by the Czech Office for Surveying, Mapping and Cadastre.

During the past 20 years of development and production of the Orthophoto of the Czech Republic, in which the Ministry of Defence, the Ministry of Agriculture and the Czech Office for Surveying, Mapping and Cadastre participate, several major measures have been taken: in 2009, the original ground sampling distance (GSD) was increased from 0.50 m to 0.25 m. Since 2010, digital aerial survey cameras have been used exclusively. In 2012, the original orthophoto update period was reduced from three to two years. In 2016-2017, the GSD was further increased to 0.20 m, that was further increased to 0.125 m in 2021. The quality of the processed CR orthophoto for the 2021 data within the area relevant to our research (western part of the Czech Republic) was an overall characteristic mean positional error of 0.233 m and the corresponding mean coordinate error of 0.165 m for well identified but artificially unsignalized objects [36].

The Digital Model of Relief of the Czech Republic, 5th generation (DMR 5G) represents a digital representation of land surface in the form of heights of discrete points in an irregular triangular network (TIN) of points with coordinates X,Y,H, where H represents the altitude in the Baltic height reference system after levelling (Bpv) with a total mean height error of 0.18 m in exposed terrain, and 0.3 m in forested terrain. The model was created from data acquired by the method of airborne laser scanning in 2009 - 2013 [35].

AREAS OF INTEREST AND DATA COLLECTION

Quality verification of X, Y and Z coordinate readings from publicly available spatial data sources in the city centre of Ústí nad Labem (Czech Republic).

The SONY ILCE-6000 camera model in combination with a 16 mm focal length lens (Sony 16 mm f/2.8 SEL) was used to take a total of 2353 images of the city centre with a final resolution of 6000 x 4000 pixels, i.e. 24 Mpx and a pixel size of 4 x 4 μm . The mapped area of 3 km^2 was photographed from a height of approx. 300 m, resulting in an average GSD of 7 cm. The overlap of the images was uneven due to the nature of the flight, but it was sufficient, about 80%.

In order to reference the photogrammetric model to the coordinate system, it was necessary to locate the GCPs. In addition, check points were acquired to independently verify the resulting quality of the derived data. For this purpose, we used a GNSS receiver with RTK type HIPER SR from TOPCON, which employs GPS and GLONASS satellites for position recording. To ensure centimeter level accuracy of the measurements, we used the RTK method using the national network of permanent stations CZEPOS. The points had to be selected based on their visibility in public orthophoto. This task was much more challenging in the rural forested area, where clearly signalized objects on the terrain were only sparse due to lack of man-made clearly defined structures. Nevertheless, a sufficient number of points was identified.

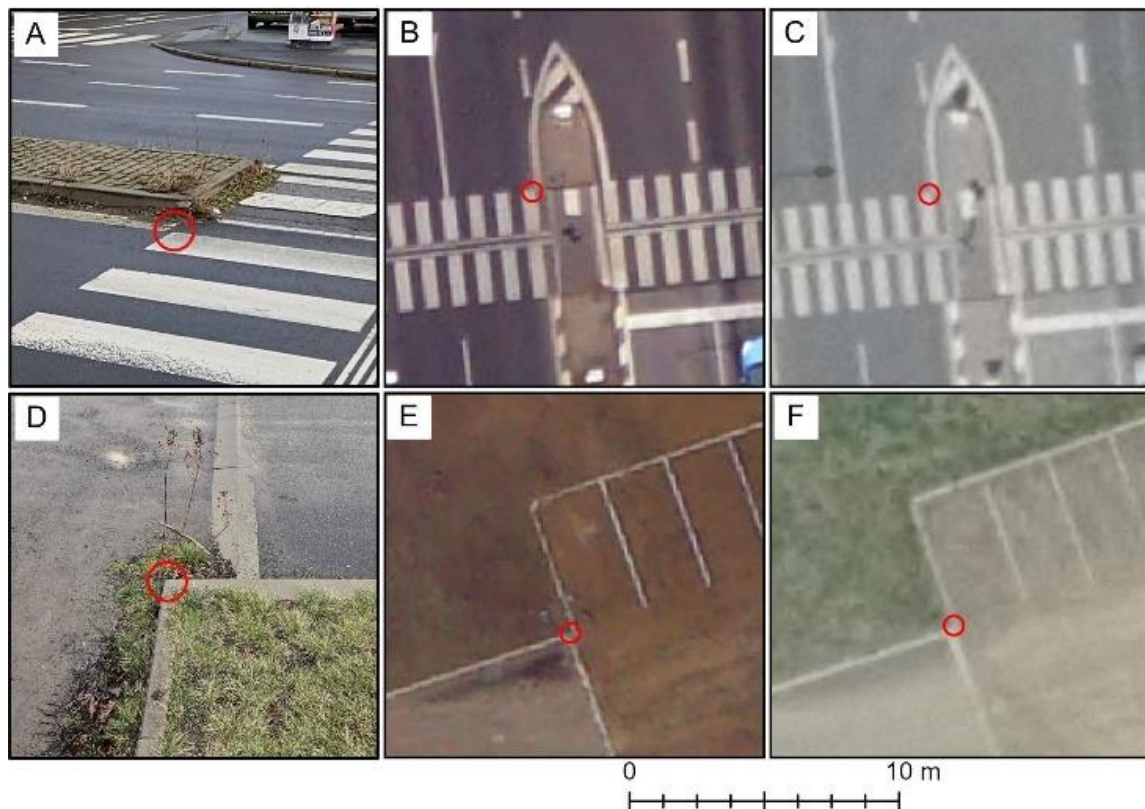


Fig. 1 - Example of ground control point selection within the test dataset. A, D - point in reality; B,E - point in acquired aerial image; C, F - point in publicly available orthophoto

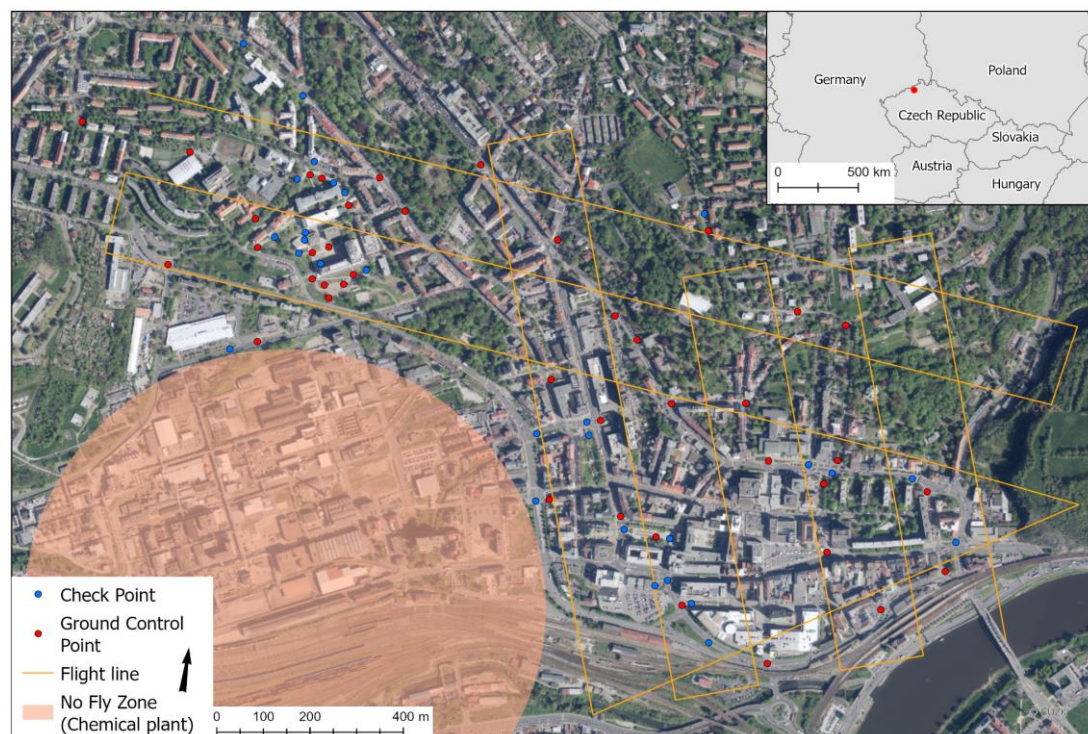


Fig. 2 - Ground control point and check point distribution within the area of interest

The measurements were repeated with time intervals and the values of the measured coordinates were averaged. The points were evenly distributed in the test area in the number of 41

and horizontal traffic signs, corners of pavements and stairs were chosen to signal these points (see Figure 1 and Figure 2). The X, Y and Z coordinates of all GCPs were also read from publicly available orthophoto and LiDAR data (CUZK model).

The methodology of reading the coordinates of GCPs using publicly available data has been practically applied in the area of the NP. The area of this NP is a good example of benefits of this indirect readout of GCPs, as a large part of the NP is inaccessible due to its geomorphology (rugged sandstone cliffs and canyons) and strict nature protection.

In addition, in August 2022, a significant part of the NP (more than 10 km² out of a total area of 80 km²) was affected by a forest fire (the largest in the history of the NP and the Czech Republic). During the firefighting and subsequent damage surveys, it was not possible to take imagery of the affected area using unmanned aerial vehicles, nor to signal/measure GCPs in the affected area (see Figure 3).



Fig. 3 - Wildfire site and the inaccessible terrain within the Bohemian Switzerland national park. Photo from September 2022

Thus, a small aircraft equipped with a Hasselblad A6D-100 camera with HS 3.5/50mm-II lens was used to document the situation just after the wildfire. Imaging was carried out in early September 2022 from an altitude of 600 m. A total of 1920 images were taken. 10 GCPs were used for processing (see Figure 4). The spatial resolution of processed orthophoto is 5 cm/pixel.

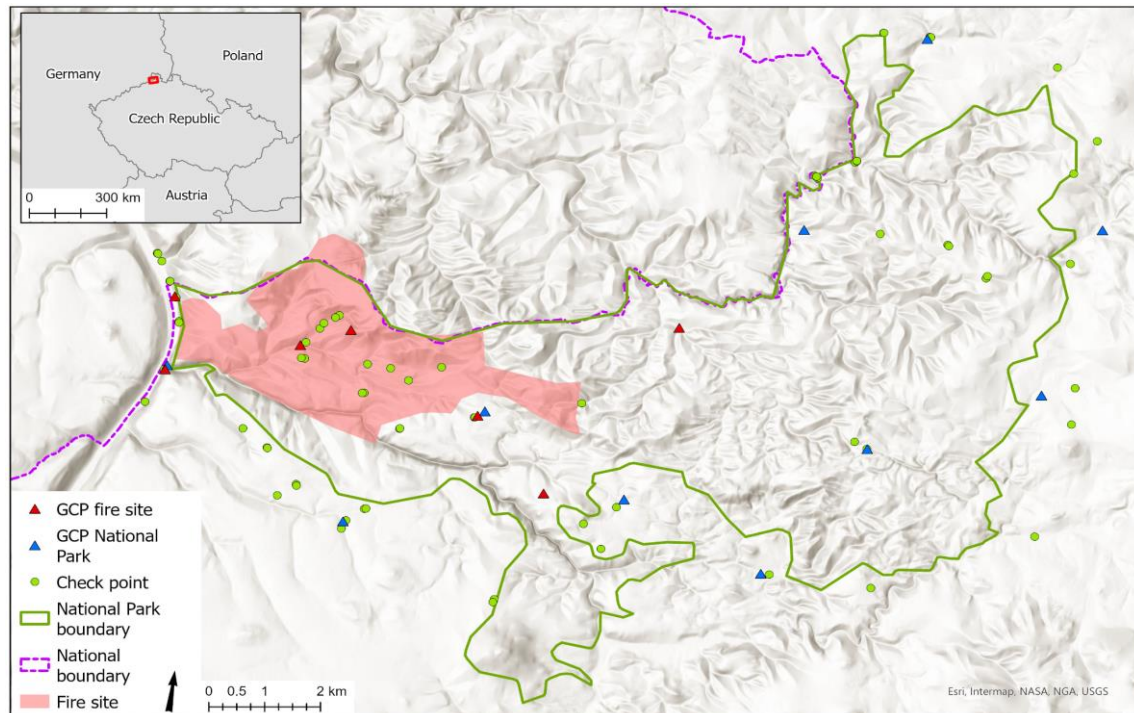


Fig. 4 - Area of interest within the Bohemian Switzerland national park. Distribution of ground control points, check points, wildfire site and multispectral imagery coverage

Additional aerial photography was conducted in late May 2023, aimed at assessing vegetation status throughout the NP (including the wildfire area). For this imagery, the Micasense Altum multispectral camera was used, allowing imaging in 6 bands. Imagery was taken from the same small aircraft in altitude of 600 m. In total, about 6000 six-band images were taken. For processing, 10 GCPs were defined (see). The spatial resolution of processed orthophoto is 35 cm/pixel.

During July 2023, a total of 71 check points were surveyed in the whole NP area to verify the quality of the processed data, thereof 69 of measured points were acquired with RTK method and 2 points with fast static method due to lack of mobile network connection (see Figure 4).

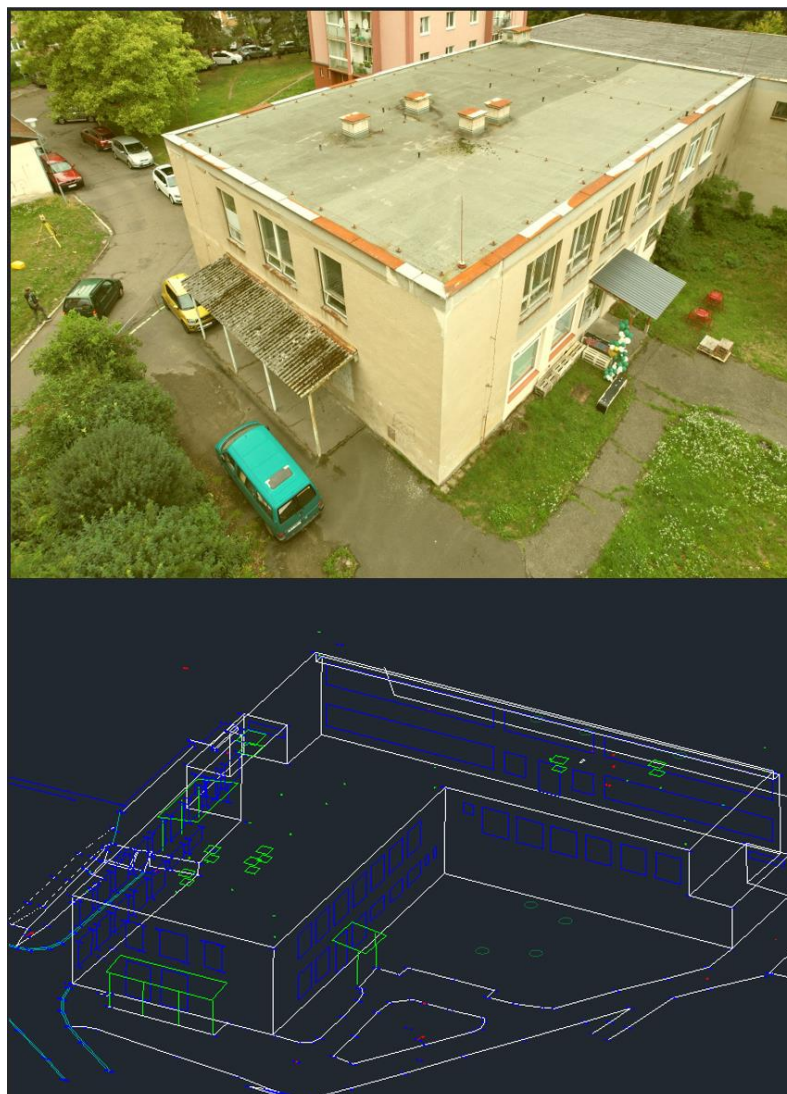


Fig. 3 - Structure modelled using the UAV and 360° photogrammetry

Lastly, we test the proposed method on a small-scale photogrammetry project, concerning documentation of a two-story building using terrestrial and UAV photogrammetry (see Figure 5). For this small photogrammetry project, we chose a modernist structure in a town with a sufficient amount of naturally signalized control points nearby. Part of documentation was conducted using conventional methods with total station and GNSS rover, including stabilization of precise control points. This was followed with UAV and 360° photogrammetry. The UAV utilized was DJI Air 2S, with 1-inch 20 MPix sensor. Height of the flight was around 20 meters, both grid and oblique. Data collection using the drone was carried out according to the legislation in force.

The 360° footage was obtained using Insta360 RS 1-inch camera, mounted on a telescopic pole. Three walkarounds with variable instrument height were conducted. The camera recorded video at 6K resolution, which was later sampled every 1 meter to 17 MPix frames. Overall, 138 aerial photos and 642 spherical frames were acquired.

PHOTOGRAMMETRIC PROCESSING

Aerial photos covering the test area in Ústí nad Labem were imported to industry standard software Agisoft Metashape, version 1.8.3 (build 14298). The alignment step was performed on 'High' accuracy with standard setting of 40,000 key point limit and 4,000 tie point limit. After importing coordinates of GCPs and CPs, all naturally signalized points were marked on a sufficient number of

photos. The alignment was then optimized with all interior parameters fitted, along with the 'Fit additional corrections' option checked. This option allows for estimation of additional parameters, in case of datasets with strong geometry and sufficiently overdetermined bundle adjustment, resulting in potentially better alignment [37]. This was confirmed with residuals on check points being slightly lower than without additional corrections. The prior marker accuracy was set to 10 cm, as such value represents expected accuracy of the public orthophoto and terrain model. As the naturally signalized points are not usually as precisely identifiable as dedicated black and white targets, we lowered the expected accuracy of their image coordinates to 1 pixel. The camera was not pre-calibrated, but in case of non-metric cameras, it is not a necessity, since it is usually sufficient to perform the calibration within the bundle adjustment [38].

Multispectral images were also processed in Agisoft Metashape, version 1.8.4 (build 14856). The processing parameters used were the same as for the test dataset, except that the multispectral data processing methodology (including reflectance calibration) was applied to the data.

The processing of the small-scale urban dataset took place once again in Agisoft Metashape, with similar parameters as in the test dataset. Spherical video was stitched in Insta360 software in post-processing to a full equirectangular panorama. For the 360 footage to align properly, it is necessary to set the camera type in Metashape to 'spherical'.

STATISTICAL METHODS

The evaluation and testing of three key quality criteria related to model errors (differences between model and real coordinates) were conducted [39][40]. These criteria are as follows:

1. Model errors should not exhibit spatial autocorrelation; instead, they should be randomly distributed in space. To assess this, the global Moran's index was employed, with significance determined using the P-value from the normally distributed z-score. The Spatial Autocorrelation (Global Moran's I) tool in ArcGIS Pro (version 3.2.0) was used for these calculations.
2. Errors should follow a normal (or at least symmetric) distribution with a mean value of zero. The Jarque-Bera test was applied for univariate normality assessment, while the Doornik-Hansen test evaluated multivariate normality—both tests rely on skewness and kurtosis measures. Measures of central tendency were tested using a univariate test for equality to zero (t-test for sample mean, Wilcoxon test for sample median). All measures and tests, as well as standard deviation, standardized skewness, standardized kurtosis and box plots were computed using the PAST (ver. 4.16c) software. PAST also facilitated two-sample paired t-tests for means, Wilcoxon tests for medians, and F-tests for variances, all of which were used to compare CUZK and GNSS model errors.
3. Model errors should not be dependent on elevation, meaning there should be neither linear nor monotonic correlation. Spearman's rank correlation coefficient was used to evaluate this relationship, as it is robust against outliers and easy to test for significance. Linear regression was employed to further investigate dependencies between model errors and elevation. However, since outliers can significantly bias ordinary least squares (OLS) regression estimates [41], least trimmed squares (LTS) regression was used as a more robust alternative. Unlike OLS, LTS regression minimizes the impact of outliers, providing more reliable parameter estimates. Correlation analysis and LTS regression were conducted using PAST software (version 4.16c). While parametric error estimates are not available for LTS regression, PAST provides 95% bootstrapped confidence intervals for both slope and intercept.

RESULTS

URBAN DATASET

In the data processing, we focused on the following procedure: To rigorously test the proposed methodology, the area of the city of Ústí nad Labem was photogrammetrically processed

using the coordinates of the GCPs obtained both by direct measurement (GNSS model) and subtracted from publicly available spatial data (CUZK model). Advanced geostatistical techniques were used to analyze the resulting differences in X, Y and Z coordinates. Based on the results obtained, the three main quality criteria of the model errors (spatial autocorrelation, normal distribution and elevation stability, presented in section Statistical methods), the GNSS model met the defined criteria, while the CUZK model did not meet the normality condition.

*Tab. 1 - Descriptive statistics, Rank correlation coefficients, Normality tests and Global Moran's Indexes of model errors for CUZK and GNSS model. Statistical significance if tested: † not significant, * p-value <0.1, ** p-value <0.05, *** p-value < 0.01. 95% bootstrapped confidence intervals (N=1999)*

Method	CUZK			GNSS		
	X	Y	Z	X	Y	Z
Spatial autocorrelation (Moran's Index)	-0.099†	-0.074†	0.098†	-0.099†	-0.074†	0.098†
Mean	-0.001†	0.003†	0.052***	-0.002†	0.009†	-0.006†
Median	-0.010†	-0.001†	0.044**	0.005†	0.002†	-0.006†
Standard Deviation	0.063	0.072	0.155	0.050	0.059	0.103
Standardized Skewness	0.651	0.090	0.025	0.288	0.434	-0.386
Standardized Kurtosis	0.415	-0.546	-0.129	-0.444	0.139	0.689
Normality (Jarque-Bera JB)	4.996*	1.114†	0.129†	1.650†	2.108†	2.561†
Multivariate normality (Doornik and Hansen omnibus)	6.111†			7.405†		
Spearman Rank Correlation	-0.007†	0.169†	0.174†	0.087†	0.222*	0.227*
LTS regression - bootstrapped 95% confidence intervals on slope	(-0.003, 0.001)	(-0.005, 0.001)	(-0.002, 0.006)	(-0.001, 0.003)	(-0.001, 0.003)	(-0.006, 0.001)

As presented in Table 1, Moran's indices were not statistically significantly different from zero, i.e. the errors of both models were not spatially clustered and were randomly distributed. The second criterion tested, the normality of the error distribution, was measured using skewness and kurtosis

statistics (see Table 1) and tested by Jarque-Bera (univariate normality) and the powerful omnibus (overall) test due to Doornik & Hansen (multivariate normality).

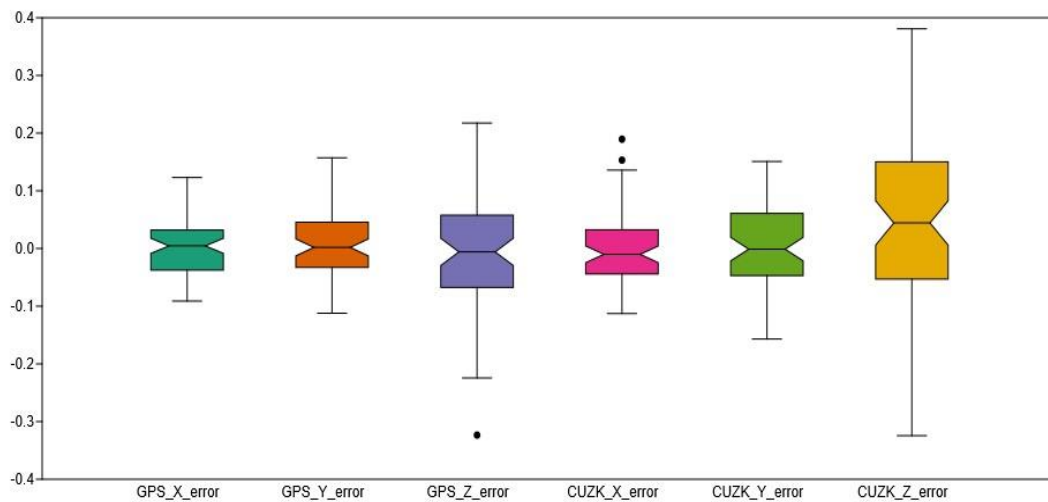


Fig. 4 - Boxplots of errors with outliers and notches denoting

The normality tests, as well as the skewness and kurtosis statistics, show that only the X errors of the CUZK model are non-symmetrically distributed. The skewness of the errors is positive, negative errors predominate and outliers are present (see Figure 6). The Z errors of the CUZK model violate the second rule in a different way: although they are normally distributed, the measures of central tendency (both mean and median) are statistically significantly greater than 0 (see Table 1 and Figure 6). According to the 95% confidence interval, the mean error is between 1.5 and 8.9 cm.

The elevation stability (error as a monotonic function of elevation) was assessed using Spearman's rank correlation coefficients. The results (see Table 1) show that the Y and Z errors of the GNSS model are positively correlated with elevation. However, the correlation is weak. Looking closely at the slope estimates obtained by the robust Least trimmed square regression (LTS), or more precisely at the 95% bootstrapped confidence intervals, makes it clear that the errors are not elevation depended as all intervals contain the 0 value. We can thus conclude that the estimated slopes are not statistically significant.

The comparison of the measures of central tendency (both mean and median) and variance shows bias of the CUZK Z errors, and a significantly higher variability of the X, Y and Z errors of the CUZK model (see Table 2)

Tab. 2 - Differences between GNSS and CUZK models and tests of their statistical significance. Statistical significance: * p -value < 0.1 , ** p -value < 0.05 , *** p -value < 0.01

Test \ Coordinate	X	Y	Z
Differences of means (t-test)	0.001	0.005	0.058***
Differences of medians (Wilcoxon test)	0.015	0.003	-0.050***
Ratio of variances (F-test)	1.633*	1.508*	2.263***

This validated methodology was applied to three different datasets - a wildfire site in the NP (detailed data obtained from RGB imagery), the entire NP (multispectral data of low spatial resolution, Figure 4), and detailed building modelling within urban area.

The acquired aerial photographs were photogrammetrically processed into orthophotos and digital surface models. Suitable check points were then identified in the GIS environment, and subsequently surveyed in the field using the GNSS method (noted as GNSS in the on following Tabs). The distribution of these points is shown in Figure 4. The X and Y coordinates of the identified check points were derived from the produced orthophoto and Z coordinates from the digital surface model (noted as Model in the on following Tabs). Subsequently, the differences between these coordinates (GNSS - Model) were compared. Table 3 provides an assessment of the accuracy of the urban dataset that was subjected to the robust statistical evaluation in this section in order to compare the results with the other processed datasets.

Tab. 3 - Comparison of points from bundle alignment and control points - urban dataset. Error values in cm

Model - GNSS	N°	Y (cm)	X (cm)	Z (cm)	XY (cm)	XYZ (cm)
GCPs RMSE	41	7.1	7.7	14.7	10.5	18.1
CPs sys. err.	29	0.3	-0.9	12.6	0.9	12.6
CPs rand. err.	29	4.9	6.3	13.2	7.9	15.4
CPs RMSE	29	4.9	6.3	18.2	8.0	19.9

NATIONAL PARK WILDFIRE SITE

RGB images (acquired with the Hasselblad A6D-100 camera) were overlaid on 10 points subtracted from a combination of orthophoto and LiDAR data (Model). Due to the nature of the terrain, finding identifiable points was more difficult than in the case of the city center above. A similar procedure to the validated urban dataset was used to calculate the bundle alignment and derived photogrammetric products. The results of the comparison between the model coordinates and the coordinates acquired by GNSS are presented in Table 4. Within this area, 24 control points were used for quality verification. An example of the identifiability of selected control points is shown in Figure 7 - C.

Tab. 4 - Comparison of points from bundle alignment and control points – wildfire site. Error values in cm

Model - GNSS	N°	Y (cm)	X (cm)	Z (cm)	XY (cm)	XYZ (cm)
GCPs RMSE	10	6.8	14.6	24.1	16.2	29.0
CPs sys. err.	24	-1.3	8.9	13.6	9.0	16.3
CPs rand. err.	24	15.0	10.0	36.6	18.0	40.8
CPs RMSE	24	15.1	13.3	39.0	20.1	43.9

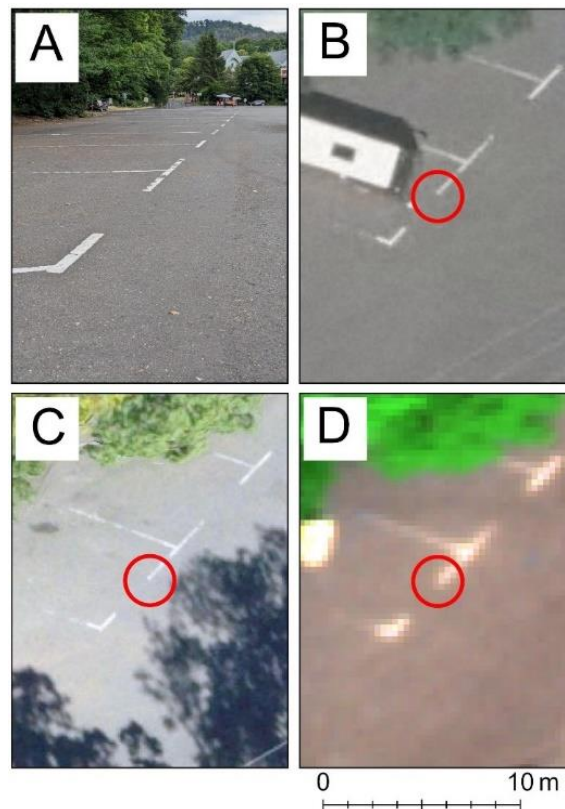


Fig. 5 - Example of check point measured within the Bohemian Switzerland national park. A – reality; B – publicly available orthophoto; C – Hasselblad A6D-100 aerial image; D – Micasense Altum aerial image

NATIONAL PARK MULTISPECTRAL DATASET

In this case, the multispectral data was acquired (using the Micasense Altum camera) for the whole area of the NP. Finding identifiable points was difficult as the resolution of the images corresponds to approximately 30 cm. The 10 GCPs from the public data and 67 control points located by GNSS were used. The resulting quality assessment is shown in Table 5. An example of the identifiability of selected control points is shown in Figure 7 - D.

Tab. 5 - Comparison of points from bundle alignment and control points – multispectral dataset. Error values in cm

Model - GNSS	N°	Y (cm)	X (cm)	Z (cm)	XY (cm)	XYZ (cm)
GCPs RMSE	10	41.6	29.5	95.9	51.0	108.7
CPs sys. err.	67	-26.9	-1.7	71.2	26.9	76.1
CPs rand. err.	67	84.2	49.4	588.5	97.6	596.6
CPs RMSE	67	88.4	49.4	592.8	101.2	601.4

SMALL SCALE DATASET

In this case, a model of the building and its surroundings was created using drone imagery and spherical photography. For the documentation purposes, a CAD drawing was created, along

with orthomosaics. The quality of the resulting model was verified using 12 control points. Due to the nature of the data, the RMSE at each insertion point was not evaluated (see Table 6).

Tab. 6 - Comparison of points from bundle alignment and control points – small scale dataset. Error values in cm

Model - GNSS	X (cm)	Y (cm)	Z (cm)	XY (cm)	XYZ (cm)
CPs sys. err.	2.3	-9.8	-7.6	10.1	12.6
CPs rand. err.	7.0	7.8	7.4	10.5	12.9
CPs RMSE	7.4	12.5	10.6	14.6	18.0

CONCLUSION

The aim of the presented research was to verify the possibilities of using publicly available spatial data, such as orthophotos and digital elevation models, for aerial image embedding. This approach can be specifically useful in cases where the photogrammetric data collection equipment (aircraft, drone) is not equipped with an IMU unit, or where the IMU unit records data with insufficient accuracy, and at the same time it is problematic or time and economically unprofitable to deploy and signal the GCPs.

The quality of this procedure was verified by advanced statistical analysis of a photogrammetrically acquired dataset, which was fully covered by a large dataset of both GCPs and check points. As part of the testing, the coordinates of these points were both located using the GNSS method and read from publicly available data - X and Y coordinates from national orthophoto collection and Z coordinates from nation-wide LiDAR dataset.

The resulting accuracy was evaluated according to three criteria that have been proven in the past to be suitable for comprehensive evaluation of spatial data accuracy – it is expected random distribution of errors in space, errors with normal distribution and mean equal to 0, and the resulting model being elevationally stable. The tested data (derived from publicly available national dataset) meet all specified quality criteria, except for the Z coordinate which violates the second criterion with a mean height error statistically different from zero and a confidence interval for the mean between 1.5 and 8.9 cm. This is explained by the fact that the publicly available Lidar data for the Czech Republic shows, despite stated full mean error of height in open terrain of 18 cm, systematic overestimation by up to 10 cm.

Thus, our results proved that the method of embedding using publicly available data is applicable in practice, and its accuracy, in the case of our detailed dataset, satisfies the quality of points of accuracy class 3, which is defined by the mean coordinate error of the detailed point of the position map $m_{xy} = 0.14$ meters. This accuracy is currently satisfactory for land registry purposes in the Czech Republic.

To proof the concept, the tested method was then applied to the RGB dataset (GSD 5 cm) in the sandstone area of the Bohemian Switzerland NP. The search for identifiable points was more difficult here, however, the accuracy achieved in Table 4 is still very high. According to the annex of the cadastral decree, in a dataset containing more than 20 points, at least 40% of the sampling mean coordinate errors must be smaller than the value of the basic mean coordinate error (Decree No. 357/2013 Coll. on the Czech Land Registry). In our case, this condition is met by 15 out of 24 points, i.e. 62.5%. Only one point just exceeded the coordinate error threshold. The cumulative sampling mean coordinate error reached 14.2 cm, which, when rounded, is just the characteristic of quality code 3 (note: we consider GNSS points as fixed).

This method was also applied to the multispectral dataset of coarser GSD of 35 cm spanning larger area of the whole NP. For this dataset, significant accuracy margins were evident, mainly due to the lack of GCPs. In one area without tie points, the error in the height coordinate was over 10 m.

The mean plane position error was around 1 m, which is sufficient for many environmental purposes. The mean elevation error was much larger and more GCPs would be needed to reduce it.

Finally, the tested method was applied to a dataset that is defined on a small scale (building). With respect to the high precision data collection methods, the resulting quality assessment scores correspond to quality point code 3.

To summarize, when selecting GCPs from publicly available data, it is advisable to apply the same philosophy as when signalling GCPs in the field - points evenly distributed within the area that are clearly identifiable. Furthermore, it has proven to be appropriate to select points that are located on a plane (or have a flat surroundings), as the reference digital elevation model might not respect (with respect to point density, subsequent interpolation) the dynamic changes of the relief. This implies another condition - namely the actual quality of the reference dataset, which should be defined or verified in the field.

The presented method of ground control point selection based on publicly available data represents a way to efficiently photogrammetrically process large areas while minimizing time and financial costs, and also considering the reduction of the carbon footprint associated with the use of motorized vehicles during GCPs measurements.

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ANALYSIS OF THE DOWNWARD DEFLECTION PARAMETERS BEFORE AND AFTER EXTERNAL PRESTRESSING REINFORCEMENT OF CONTINUOUS RIGID FRAME BRIDGES

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ABSTRACT

Due to a lack of foresight by designers during bridge planning, outdated specifications that have not been updated in a timely manner, or construction errors during the building process, bridge structures may be overloaded, ultimately leading to excessive deflection in the main span of continuous rigid frame bridges. Therefore, it is urgent to study the causes of downward deflection in continuous rigid frame bridges and explore their reinforcement methods. This paper conducts parametric analysis by using the finite element method to explore the influence degree of different factors on deflection. This paper addresses the issue of severe mid-span deflection in a specific continuous rigid frame bridge and investigates the impact of various factors, including internal prestress loss, stiffness reduction, shrinkage and creep, over-excavation (possibly referring to excessive excavation or over-dimensioning in construction), shear deformation, external prestress loss, and overloading, on the downward deflection of continuous rigid frame bridges. The study identifies the most significant factor contributing to this deflection. Upon comprehensive comparison of all factors, stiffness reduction is found to be the primary cause of mid-span deflection. When the stiffness is reduced by 10%, the deflection difference plot shows that the maximum decrease in deflection near the mid-span is 8.76mm. For reinforced bridges, external prestress loss and overloading are the most critical factors. In calculations for continuous rigid frame bridges, shear deformation cannot be neglected.

KEYWORDS

Continuous rigid frame bridge, External prestressing, Reinforcement, Downward deflection

INTRODUCTION

Continuous rigid frame bridges belong to the category of continuous beam bridges in terms of their bridge span structure system. Continuous beam bridges are an ancient type of beam structure system, known for their small deformation, high structural stiffness, smooth and comfortable ride, few expansion joints, simple maintenance, and strong earthquake resistance, which have been adopted by people for a long time [1-4]. However, due to limitations in construction methods, the span of continuous beam bridges made of concrete structures before the 1960s was less than 100m. With the emergence of cantilever pouring and cantilever assembly construction methods, T-shaped rigid frame bridges came into being. Nevertheless, this structure featured hinges

in the middle and underestimated creep and shrinkage deformations of concrete. Additionally, factors such as temperature led to obvious angular deformations at the hinged joints, which was detrimental to traffic. Therefore, the continuous beam-type rigid frame bridge, which is advantageous for traffic, emerged [5,6].

Prestressed continuous rigid frame bridges possess many advantages of both T-shaped rigid frame bridges and continuous beams. The load-bearing beams are continuous beams and are rigidly connected to the piers [7-10]. From the perspective of the important characteristics of structural mechanics, the upper structure of the continuous rigid frame system is similar to that of a continuous beam. Due to significant temperature changes, for continuous rigid frame bridges with large spans and low pier heights, concrete shrinkage often results in significant bending moments at the pier bases. As the bridge is used and operated over time, some of its potential issues gradually emerge, posing serious threats to the bridge's sustained use in the future [11,12]. In reality, beam deflection and beam cracking often occur together and are coupled.

The main defects of continuous rigid frame bridges are currently classified into the following types: cracks in the top slab, bottom slab, and mid-span web [13]. The presence of these cracks significantly reduces the performance of prestressed concrete continuous rigid frame bridges, decreases the overall stiffness of the main beam, and compromises structural quality [14]. If timely maintenance and repairs are not carried out, it will inevitably lead to damage to the main structure and even serious accidents. Excessive downward deflection in the mid-span of continuous rigid frame bridges has become a common issue. Long-term downward deflection of large-span prestressed concrete box girders is indeed a flaw in the system, and specific factors such as material and environmental differences, as well as variations in construction quality across different regions, are not inevitable causes of deflection. Only by effectively understanding and mastering the downward deflection of continuous rigid frame bridges can we truly enhance the bridge's sustained load-bearing capacity, prevent stress cracks, and effectively control the magnitude of beam deflection. Due to the relatively short service life of prestressed concrete structures, many issues have not yet been exposed [15-17]. Among them, the most common is corrosion of reinforcing steel.

Due to insufficient understanding of continuous rigid frame bridges in the early stages, some continuous rigid frame bridges both domestically and internationally have developed common defects after years of operation, such as excessive mid-span deflection and numerous cracks in the box girder. The causes of these defects may include prestress loss, stiffness reduction, overloading, or concrete creep and shrinkage [18-20]. This paper analyzes the impact of various factors on mid-span deflection based on the characteristics of severe mid-span deflection in a specific continuous rigid frame bridge. It provides references for reinforcement measures targeting mid-span deflection and other defects in this type of bridge, which is valuable for the development of bridge reinforcement technology and engineering issues related to reinforcement measures for continuous rigid frame bridges.

PROJECT OVERVIEW

A continuous rigid frame bridge is a large prestressed concrete structure. The span arrangement of the bridge is 35 m + 60 m + 90 m + 60 m + 35 m, with a total length of 280 m, situated on a 2.2% longitudinal slope. The deck's horizontal layout consists of a 0.75 m parapet base, a 10.5m carriageway, and another 0.75 m parapet base. The pedestrian load for this bridge is 3.5 kN/m, and the design load is for Highway-20 vehicles and Trailer-120. The general layout of the bridge type is shown in Figure 1. The cross-sections at the mid-span and pier top of the box girder are illustrated in Figure 2.

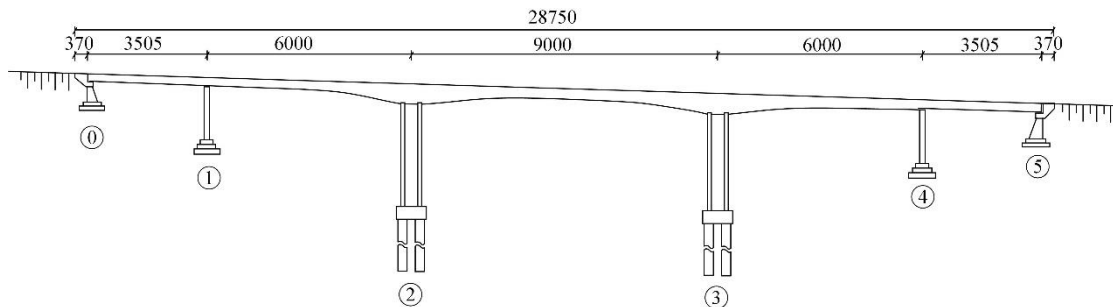
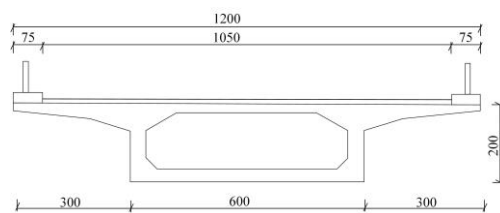
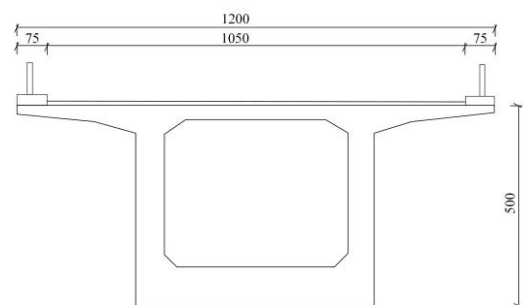


Fig.1 - General layout of bridge type (Unit: cm)



(a) Mid-span cross-section



(b) Pier support cross-section

Fig. 2 - Cross-sectional views of box girder at mid-span and pier top (Unit: cm)

The T-beam rigid frame employs Grade 50 concrete with an elastic modulus of 3.41×10^5 MPa, and a standard tensile strength of 1860 MPa. The deformation of anchorages, steel strand retraction, and joint compression are 0.006 m per end. The friction coefficient between the prestressing tendons and the pipeline wall is taken as 0.25. Transverse prestressing tendons are also made of steel strands with a diameter of $\phi 15.24$, while vertical prestressing tendons utilize 75/100 high-strength finished threaded steel bars. The characteristic values for several key control sections of the bridge are shown in Table 1.

Tab.1 - Main control sections

Section Locatio	Section Number	Height (m)	Area (m ²)	Sectional Bending Stiffness (kN·m ²)
Top of Middle Pier	A-A	5.00	14.282	1.700×10^9
1/8 Section of Main Span	B-B	3.93	14.154	1.662×10^9
2/8 Section of Main Span	C-C	2.86	14.086	1.646×10^9
3/8 Section of Main Span	D-D	2.22	14.070	1.645×10^9
4/8 Section of Main Span	E-E	2.00	7.304	1.643×10^9

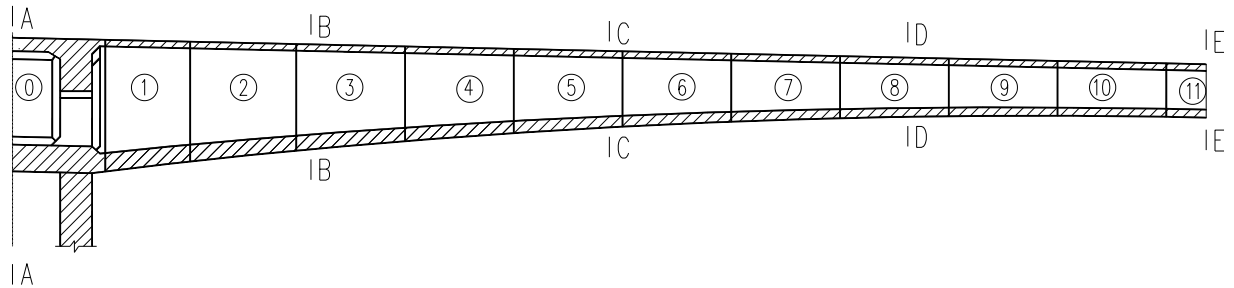


Fig. 3 - Main Control Sections

This continuous rigid frame bridge began operating in October 1997. During a general survey of the bridge in 2007, it was discovered that the main span had significant downward deflection, which was continuing to develop. Subsequently, a comprehensive inspection was conducted on the bridge. The inspection results revealed that the deflection values, strain values, and crack widths of the bridge exceeded the specifications and requirements, failing to meet the design load requirements for Highway-Super 20 vehicles. The safety reserve for load-bearing capacity was severely insufficient. To ensure the normal use of the bridge and the safety of traffic, reinforcement measures were proposed for the bridge, which should be carried out according to the load rating requirements of Highway-Class I as specified in the 2004 specifications. In November 2009, the reinforcement work of the bridge was completed. Static and dynamic load tests conducted on the bridge showed that it met the requirements of Highway-Class I, with deflection, strain, and cracks all within acceptable limits. Six years after the external prestressing reinforcement was implemented, a regular inspection and load test were conducted on the bridge.

The Impact of Internal Prestress Loss on Bridge Deflection

There has been a lack of systematic experimental research data on the impact of prestress on deflection, thus it is necessary to study the pattern of deflection as it varies with external prestress. When considering the influence of long-term deformation under short-term loading, the specific formula for solving the deflection value is:

$$\delta = \delta_0 + \beta \frac{\iint M dx dx}{(1 + k_1)EI} N_y \quad (1)$$

$$\Delta\delta = \beta \frac{\iint M dx dx}{(1 + k_1)kEI} \Delta N_y \quad (2)$$

In the formula:

β —represents the coefficient of influence of external prestress on deflection;

M — represents the bending moment generated by dead load;

δ_0 —represents the deflection produced by dead load when there is no external prestress.

To study the impact of internal prestress loss on the downward deflection of bridges, simulations were conducted for longitudinal prestress losses of 10%, 15%, 20%, 25%, and 30%, respectively. The deflection values obtained were plotted in Figure 4.

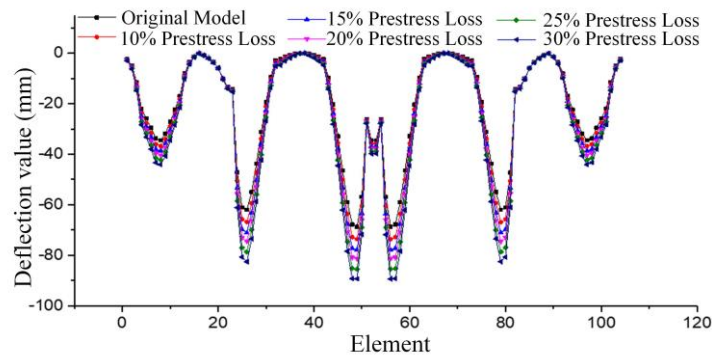
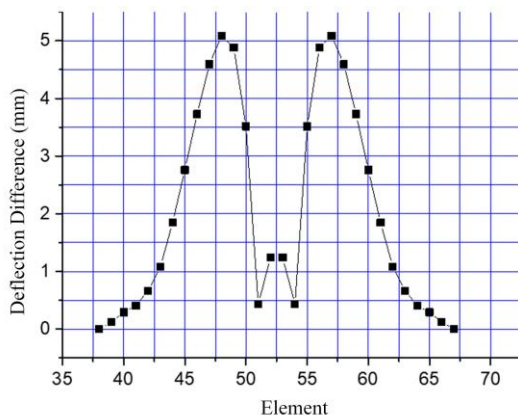
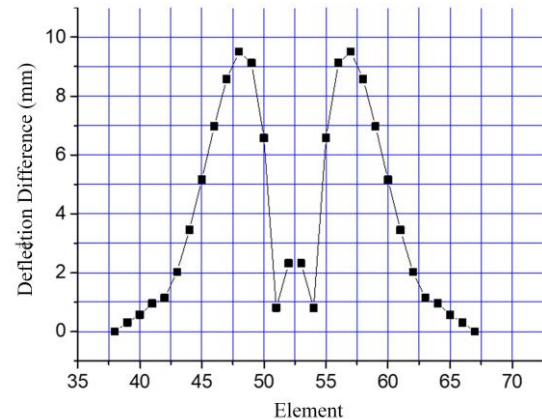


Fig. 4 - Comparison chart of deflections caused by different levels of internal prestress loss

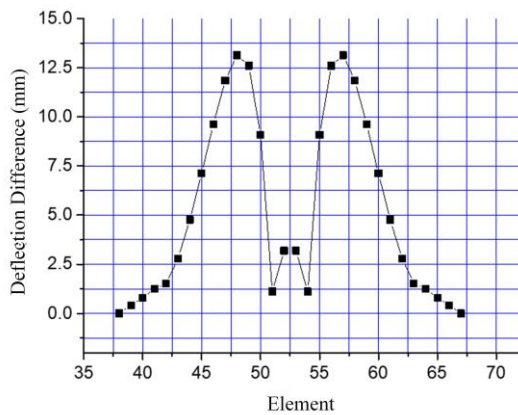
As can be seen from Figure 4, with the increase in longitudinal prestress loss, the deflection values of each cross-sectional unit across the entire bridge increase significantly. Notably, the deflection values of the units near the mid-span of the bridge show larger changes, while the changes in other units are relatively small. To further illustrate this, the deflection differences between the original mid-span model and the models with 10%, 15%, 20%, 25%, and 30% longitudinal prestress loss are plotted in Figure 5.



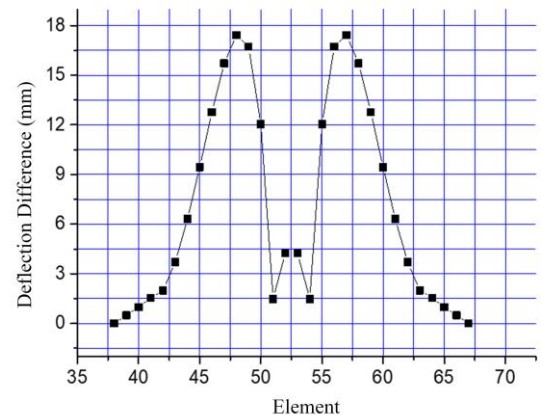
(a) 10% loss



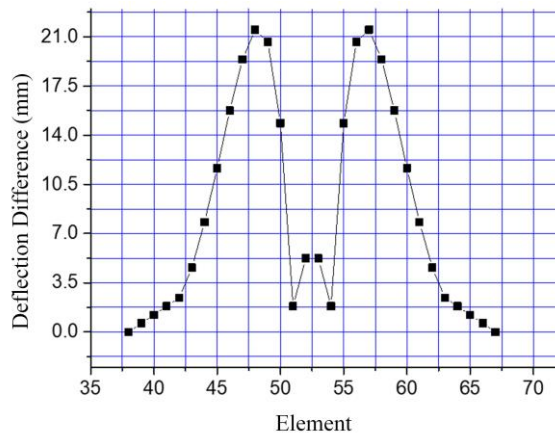
(b) 15% loss



(c) 20% loss



(d) 25% loss



(f) 30% loss

Fig. 5 - Deflection differences between the original model and models with longitudinal prestress loss

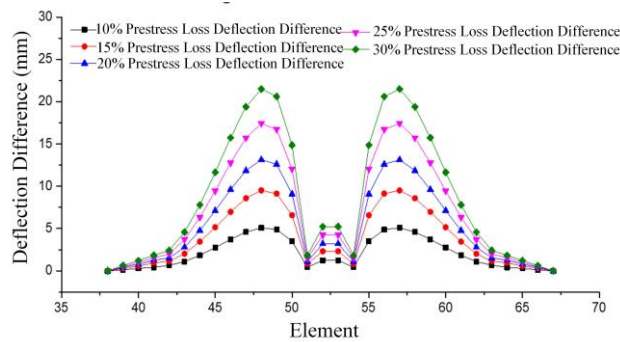


Fig. 6 - Comparison of deflection differences between the original model and models with 10%, 15%, 20%, 25%, and 30% longitudinal prestress loss

From Figures 5 and 6, it can be observed that as the longitudinal prestress loss increases, the deflection difference also increases. When the longitudinal prestress loss reaches 10%, the maximum reduction in deflection for units near the mid-span of the main bridge is 5.09mm. Furthermore, as seen in Figures 5 and 6, when the longitudinal prestress loss reaches 30%, the maximum decrease in deflection near the mid-span is 21.50 mm. In summary, the loss of longitudinal prestress is one of the most significant factors contributing to an increase in bridge deflection.

The Impact of Stiffness Reduction on Bridge Deflection

Stiffness represents the ability of a material or structure to resist deformation. For long-span continuous rigid-frame bridges, structural stiffness is particularly important. The reduction in stiffness can be influenced by various complex factors, and the main reasons for the overall structural stiffness decline are large-area cracking of webs and the appearance of transverse cracks in the mid-span area.

When a structure is subjected to loading, once the bending moment generated at a section reaches the cracking bending moment of that section, cracking will occur, leading to uneven changes in structural stiffness along the longitudinal direction of the bridge and rendering the structure out of its intended working condition. Therefore, the effective moment of inertia of the section should be used for calculations, as shown in Equation (3):

$$I_e = \left(\frac{M_{cr}}{M} \right)^3 I_{ucr} + \left[1 + \left(\frac{M_{cr}}{M} \right) \right] I_{cr} \leq I_{ucr} \quad (3)$$

In the formula:

M —represents the bending moment generated by the structure under service loads.

M_{cr} —represents the bending moment at which the section cracks.

I_{ucr} —represents the converted inertia of the section before cracking.

I_{cr} represents the converted inertia of the section at the point of cracking.

To illustrate the change in bridge deflection as the amount of stiffness reduction increases, the deflection values resulting from stiffness reductions of 10%, 15%, 20%, 25%, 30%, and 35% have been plotted in Figure 7. The reduction in stiffness in the text is achieved by adjusting the elastic modulus of the cross-sectional material.

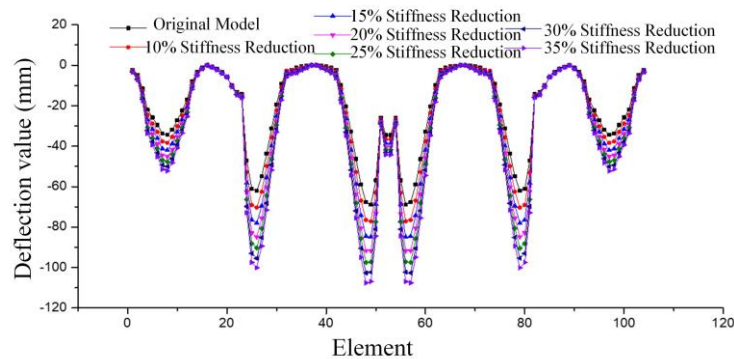
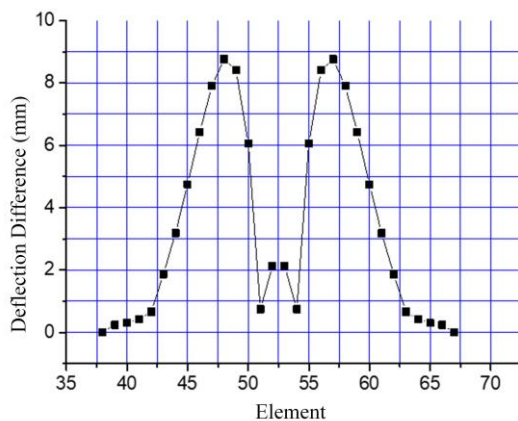
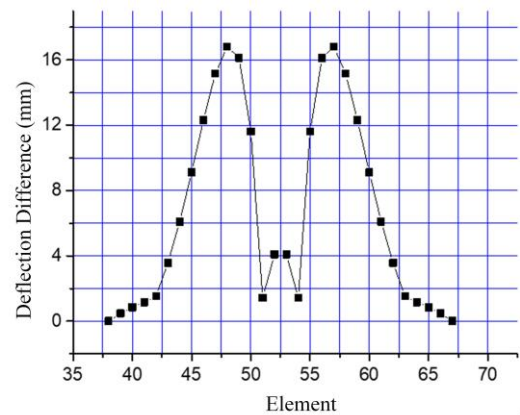


Fig. 7 - Comparison chart of deflections caused by different levels of stiffness reduction

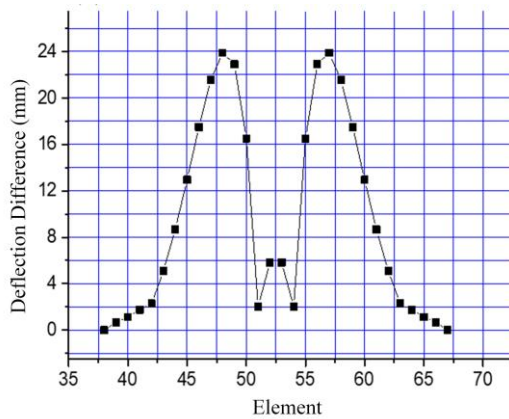
As shown in the figure, the higher the degree of stiffness reduction, the larger the deflection values of each cross-sectional unit across the entire bridge. It is evident that the deflection values of the units near the mid-span of the bridge show significant changes, while the deflection values of the other units change very little and are almost unaffected. To more clearly understand the extent to which stiffness reduction affects deflection, the deflection differences between the original mid-span model and the models with reduced stiffness have been plotted in Figures 8 to 9.



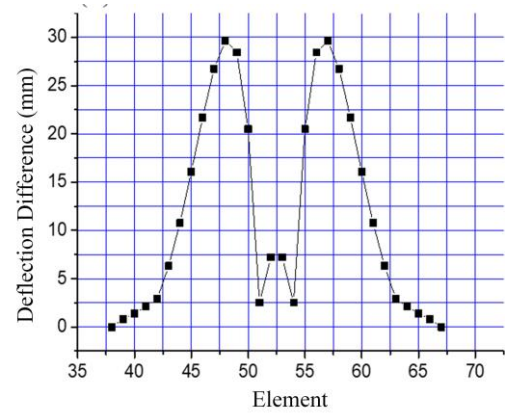
(a) 10% reduction in stiffness



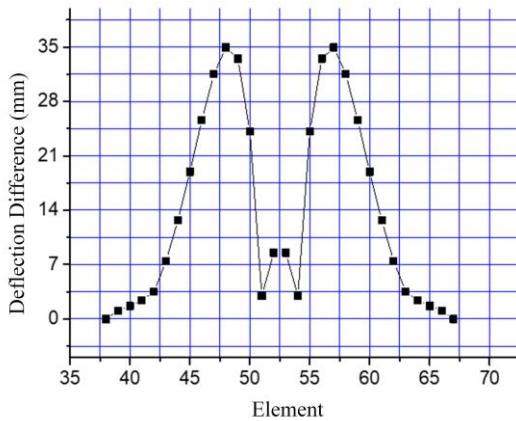
(b) 15% reduction in stiffness



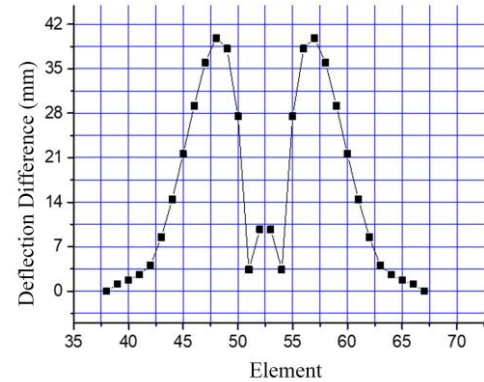
(c) 20% reduction in stiffness



(d) 25% reduction in stiffness



(e) 30% reduction in stiffness



(f) 35% reduction in stiffness

Fig.8 - Deflection difference between the original model and the model with stiffness reduction

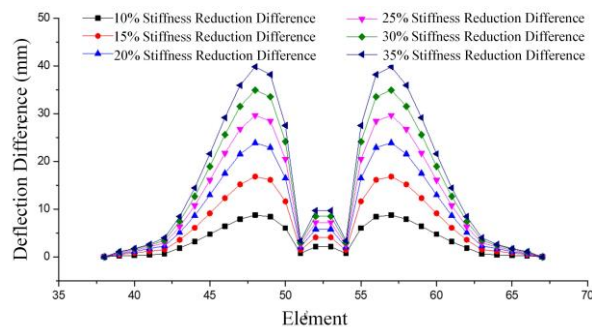


Fig. 9 - Comparison of deflection differences between the original model and models with 10%, 15%, 20%, 25%, 30%, and 35% stiffness reduction

From Figures 8 to 9, it can be observed that as the stiffness reduction increases, the deflection difference also increases. When the stiffness is reduced by 10%, the deflection difference plot shows that the maximum decrease in deflection near the mid-span is 8.76 mm. As the stiffness reduction further increases, the deflection continues to decline. Figure 9 shows that when the stiffness is reduced by 35%, the maximum decrease in deflection near the mid-span is 39.81 mm.

The impact on the maximum deflection of the mid-span, the secondary side-span, and the

primary side-span of the bridge under seven different operating conditions is shown in Figure 10.

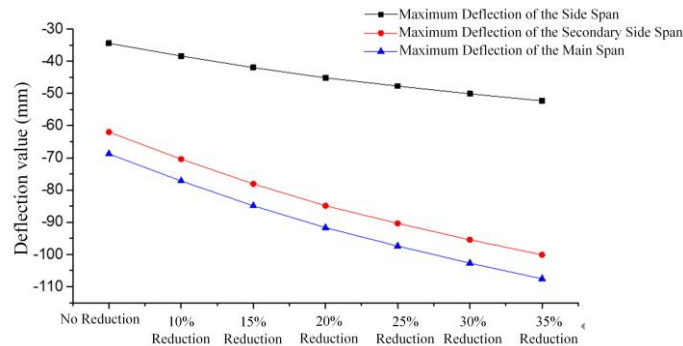
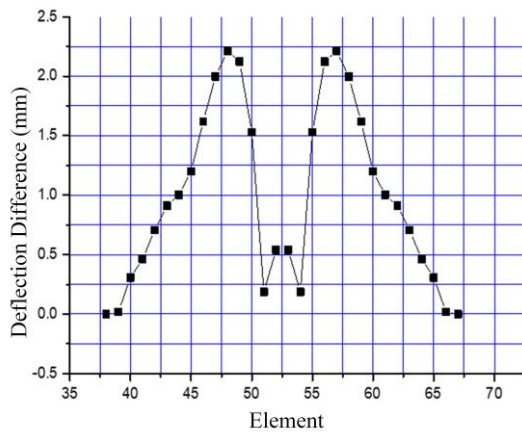


Fig.10- Comparison of maximum deflection values between the original model and models with different levels of stiffness reduction

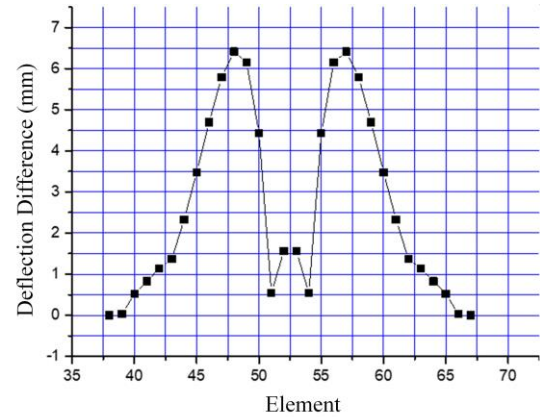
From Figure 10, it can be seen that the maximum deflection of the side span is -34.43mm when there is no stiffness reduction. When the stiffness is reduced by 10%, 15%, 20%, 25%, 30%, and 35%, the maximum deflection of the side span increases by 11.40% (-3.93 mm), 21.89% (-7.53 mm), 31.10% (-10.71 mm), 38.59% (-13.29 mm), 45.50% (-15.66 mm), and 51.83% (-17.85 mm) respectively, compared to the case with no stiffness reduction. The maximum deflection of the secondary side span increases by 13.53% (-8.39 mm), 25.96% (-16.09 mm), 36.89% (-22.87 mm), 45.77% (-28.37 mm), 53.97% (-33.46 mm), and 61.48% (-38.11 mm) respectively. The maximum deflection of the mid-span increases by 12.22% (-8.40 mm), 23.45% (-16.12 mm), 33.32% (-22.91 mm), 41.62% (-28.62 mm), 49.34% (-33.93 mm), and 56.42% (-38.80 mm) respectively. In summary, stiffness reduction is one of the most important factors affecting the deflection of continuous rigid-frame bridges.

The impact of shrinkage and creep on bridge deflection

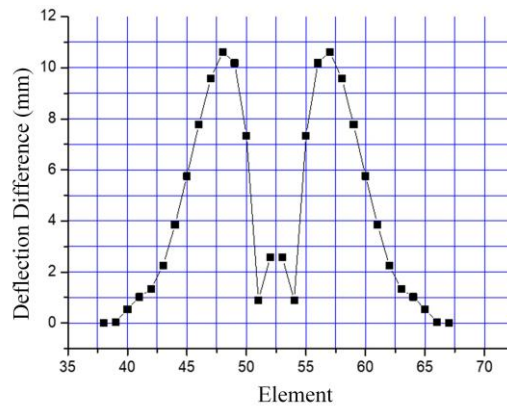
Currently, technology has not yet been able to accurately simulate the shrinkage and creep of concrete due to its significant dispersion. Therefore, we select the average relative humidity of the environment as an indicator to simulate the characteristics of shrinkage and creep. According to relevant specifications, when $40\% < RH < 70\%$, we choose $RH = 55\%$. When $70\% < RH < 99\%$, we choose $RH = 80\%$. Based on this requirement, we selected the change in bridge deflection values when RH is 80%, 70%, 55%, and 40%. Due to the space limitations of this article, only the deflection values for half of the main span are listed. The deflection differences between the deflection values under three operating conditions and the original model are shown in Figure 11.



(a) Comparison between operating condition 1 and operating condition 2



(b) Comparison between operating condition 1 and operating condition 3



(c) Comparison between operating condition 1 and operating condition 4

Fig. 11- Deflection differences between various operating conditions

The overall deflection of the bridge under four different operating conditions as a function of the average relative humidity of the environment is shown in Figure 12.

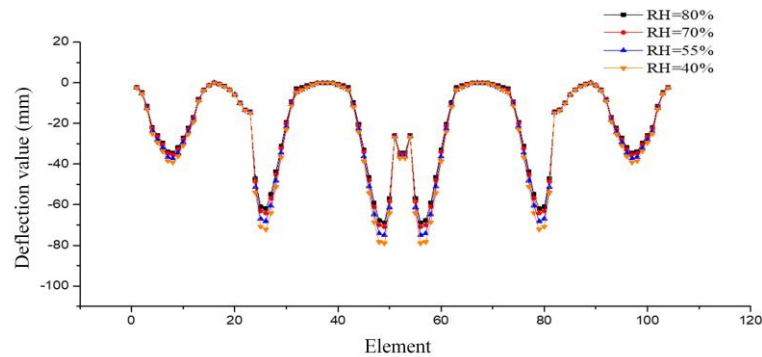


Fig.12- Comparison of deflections at RH=80%, RH=70%, RH=55%, and RH=40%

From Figures 11 and 12, it can be observed that the lower the average relative humidity (RH) of the environment, the greater the deflection values of each cross-sectional unit of the entire bridge. It is evident that the deflection values of the units near the mid-span of the bridge change significantly,

while the deflection values of the other units change very little and are almost unaffected by the average relative humidity of the environment. When the average relative humidity is RH=70%, the maximum deflection decreases by 2.21 mm compared to the original model's deflection value. Figure 12 shows that when the average relative humidity is RH=40%, the maximum decrease in deflection near the mid-span is 10.62 mm. The maximum deflection values of the mid-span cross-sections of each span of the bridge under the four operating conditions are shown in Figure 13.

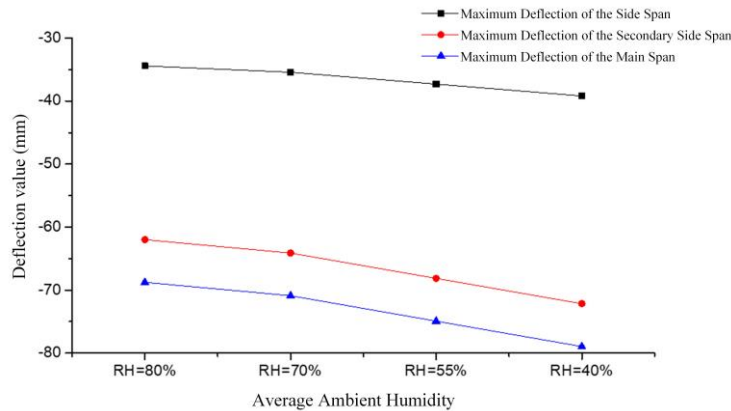


Fig.13 Comparison of deflections at RH=80%, RH=70%, RH=55%, and RH=40%

Table 2 shows the maximum deflection values of the mid-span, secondary side span, and side span under Operating Conditions 1 to 4.

Tab. 2 - Impact of shrinkage and creep on deflection

Calculation Scenarios	Calculated deflection values at specific locations (mm)		
	Maximum deflection of the side span	Maximum deflection of the secondary side span	Maximum deflection of the mid-span
RH=80%	-34.43	-61.99	-68.77
RH=70%	-35.42	-64.11	-70.89
RH=55%	-37.30	-68.13	-74.92
RH=40%	-39.19	-72.16	-78.95

From Figure 13 and Table 2, it can be seen that when the average relative humidity (RH) is 80%, the maximum deflection of the side span is -34.43 mm. When the RH is 70%, 55%, and 40%, respectively, the maximum deflections of the side span increase by 2.87% (-0.99 mm), 8.34% (-2.87 mm), and 13.83% (-4.76 mm) compared to when RH is 80%. Similarly, the maximum deflections of the secondary side span increase by 3.42% (-2.12 mm), 9.90% (-6.14 mm), and 16.41% (-10.17 mm), respectively, compared to when RH is 80%. The maximum deflections of the mid-span also increase by 3.08% (-2.12 mm), 8.94% (-6.15 mm), and 14.80% (-10.18 mm), respectively, under the same comparison. The analysis of these data indicates that shrinkage and creep is one of the factors affecting the deflection of continuous rigid frame bridges, but it is not the primary factor.

The impact of super-elevation on bridge deflection

During the construction of box girder bridges, the actual amount of concrete poured is often greater than the designed amount, a phenomenon known as concrete overfill in box girders.

Currently, there are no definitive and guiding research results regarding concrete overfill in box girders. Additionally, issues such as formwork bulging can occur during construction, making it difficult to control overfill. Although it is stipulated that overfill of no more than 5% is not considered significant, studying the issue of overfill still holds certain guiding significance.

Finally, we have identified four operating conditions to consider the extent to which deflection values are affected by concrete overfill:

- Operating Condition 1: No overfill
- Operating Condition 2: 5% overfill
- Operating Condition 3: 6% overfill
- Operating Condition 4: 7% overfill
- Operating Condition 5: 8% overfill

Due to the limitations of this paper, only the deflection values for half of the main span are listed. The deflection values resulting from 5%, 6%, 7%, and 8% overfill are plotted in a graph to reflect the extent to which deflection values are affected by overfill. The specific changes are shown in Figure 14.

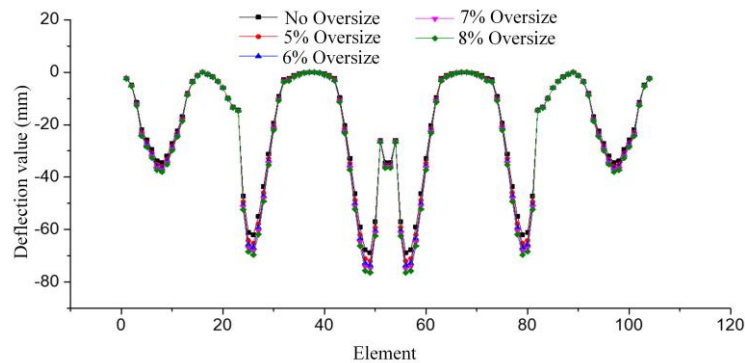
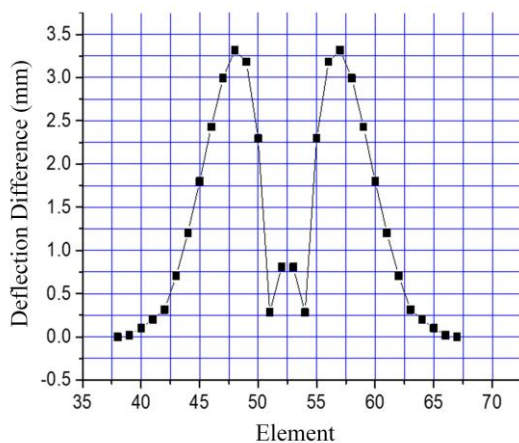
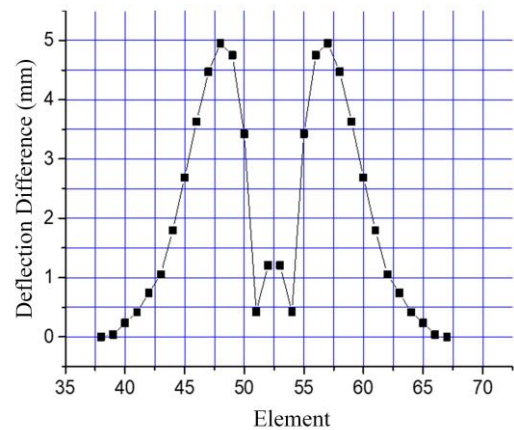


Fig.14 - Deflection values resulting from structures with different amounts of concrete overfill

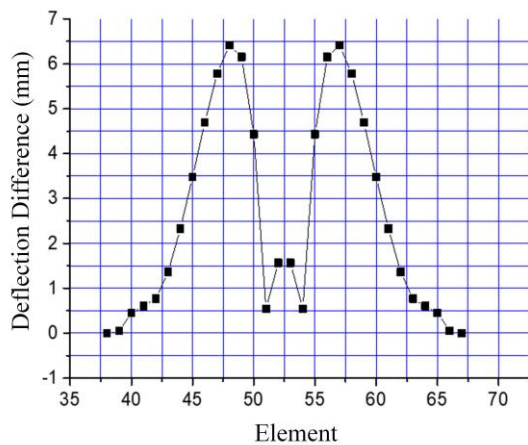
As can be seen from Figure 14, with the increase in the amount of concrete overfill, there is a tendency for the deflection of each section of the main girder to increase. However, when the proportion of overfill is the same, the degree of influence on each section varies. To reflect the pattern of change in deflection with increasing overfill, the differences in deflection between the original mid-span model and the models with 5%, 6%, 7%, and 8% overfill are plotted in Figures 15 to 16, respectively.



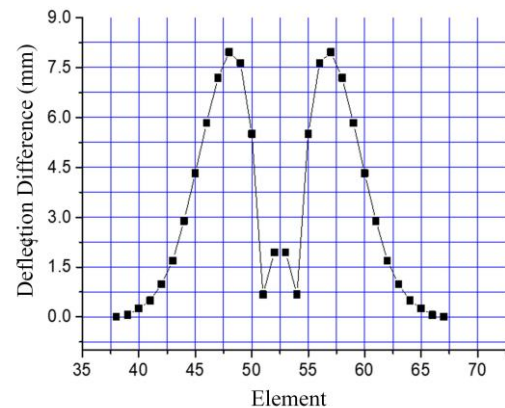
(a) 5% overfill



(b) 6% overfill



(c) 7% overfill



(d) 8% overfill

Fig.15- Deflection Differences Between No Overfill and Various Amounts of Overfill

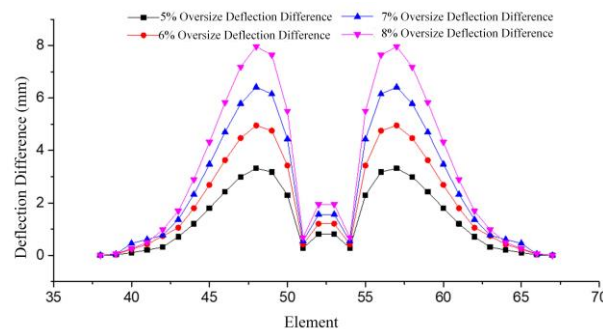


Fig.16- Deflection differences between the original model and models with 5%, 6%, 7%, and 8% overfill

From Figures 15 to 16, it can be observed that as the amount of concrete overfill increases, the deflection values of each cross-sectional unit of the entire bridge also increase, but the degree of change is relatively small, indicating that the impact of overfill on bridge deflection values is relatively minor. Furthermore, it is evident that the deflection values of units near the mid-span of the bridge change significantly, while the deflection values of other units change slightly. When there is 5% overfill, the maximum deflection difference observed from the deflection difference graph is 3.31 mm. However, as the amount of overfill increases, as shown in Figure 16, when there is 8% overfill, the maximum deflection difference reaches 7.96 mm. Therefore, although overfill can affect the downward deflection of the bridge mid-span, it is not the primary factor, and the impact of the amount of overfill is relatively small.

The Impact of Shear Deformation on Bridge Deflection

When calculating rigid frame bridges, due to their large slenderness ratio, the influence of shear deformation on the structure is often neglected. In some research analyses, a height-to-span ratio (H/L) of less than $1/5$ is used as an indicator to determine whether to consider the impact of shear deformation on the structure. In continuous rigid frame bridges, the height-to-span ratio is far less than $1/5$, but their main beams are box girder structures with thin webs that are significantly affected by shear forces. Furthermore, prestressing can eliminate most bending deformations but

has no effect on shear deformations, which amplifies the influence of shear deformation. Therefore, the impact of shear deformation should not be ignored.

Two operating conditions are selected for analysis:

Operating Condition 1: Shear deformation is considered.

Operating Condition 2: Shear deformation is not considered.

The impact of Operating Conditions 1 and 2 on the deflection after reinforcement is shown in Figure 17.

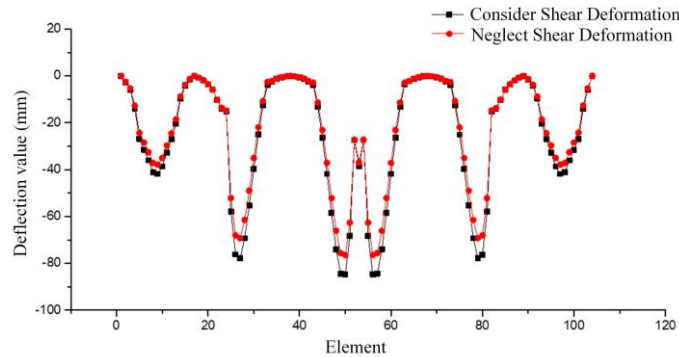


Fig.17 - Impact of shear deformation on deflection

From Figure 17, it can be observed that shear deformation has an impact on the deflection values of this continuous rigid frame bridge, with a significant influence on the units near the mid-span. To more clearly reflect the impact of shear deformation on deflection, the mid-span deflection difference is shown in Figure 18, and the deflection differences for each span are presented in Table 3.

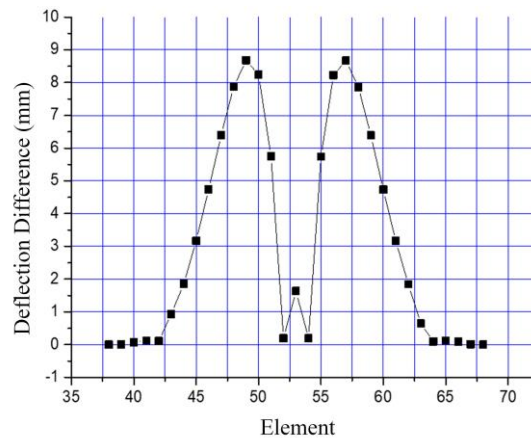


Fig.18 - Deflection difference between considering and not considering shear deformation in the mid-span

Tab.3 - Impact of considering shear deformation on deflection

Condition	Maximum Deflection (mm)		
	Maximum Deflection of the Side Span	Maximum Deflection of the Secondary Side Span	Maximum Deflection of the Middle Span
Under the condition of considering shear deformation	-41.87	-77.87	-84.68
Under the condition of not considering shear deformation	-37.88	-69.26	-76.44

From Table 3 and Figure 18, it can be observed that the deflection of the middle span considering shear deformation is 10.78% (8.24 mm) greater than that without considering shear deformation. Similarly, the deflection of the secondary side span considering shear deformation is 12.43% (8.61 mm) higher, and the deflection of the side span considering shear deformation is 10.53% (3.99 mm) more than that without considering shear deformation.

Based on the above analysis, it can be concluded that the height-to-span ratio (H/L) of less than 1/5 cannot be used as a sole criterion for neglecting shear deformation. For this bridge, with a height-to-span ratio ranging from 1/18 to 1/45, which is much smaller than 1/5, the deflection considering shear deformation is still more than 10% greater than that without considering it. Therefore, the impact of shear deformation cannot be ignored.

Impact of External Prestressing Loss on Bridge Deflection

To investigate the impact of loss in external prestressing tendons on the deflection of continuous rigid frame bridges, this subsection employs a method of reducing the tension control force to simulate a certain level of reduction in external prestressing tendons. Five operating conditions are considered: no reduction, 5% reduction, 10% reduction, 15% reduction, and 20% reduction. By calculating the bridge under these five conditions, the deflection values of the entire bridge are obtained for each scenario. The specific operating conditions are defined as follows:

Operating Condition 1: No reduction.

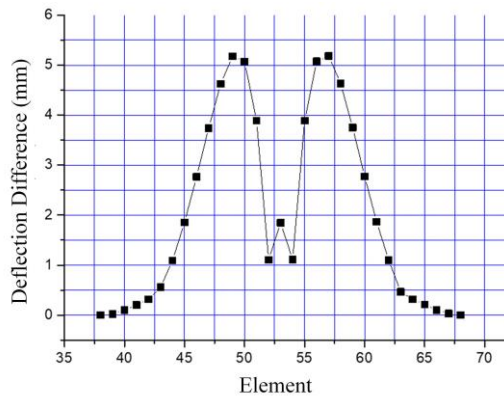
Operating Condition 2: 5% reduction in external prestressing tendons.

Operating Condition 3: 10% reduction in external prestressing tendons.

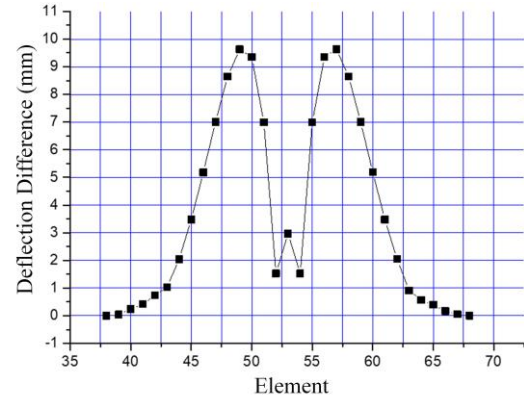
Operating Condition 4: 15% reduction in external prestressing tendons.

Operating Condition 5: 20% reduction in external prestressing tendons.

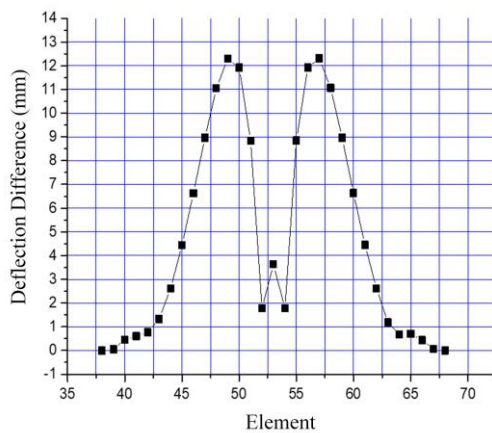
Due to space limitations in this document, only the deflection values for half of the main span are listed. The deflection differences between each operating condition and the original model are shown in Figures 19 to 20. The prestress loss in the text is achieved by adjusting the tension stress of the prestressed steel strands. The prestress loss is realized by reducing the tension stress of the steel strands.



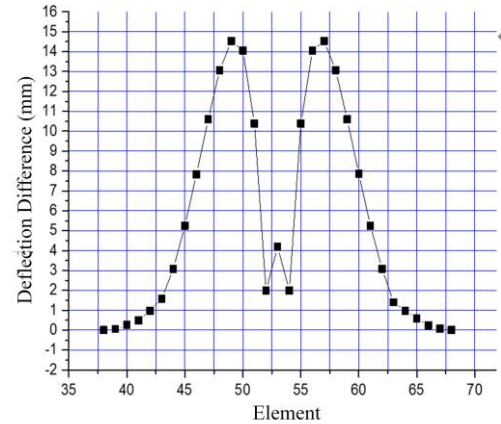
(a) 5% reduction in steel tendons



(b) 10% reduction in steel tendons



(c) 15% reduction in steel tendons



(d) 20% reduction in steel tendons

Fig.19- Deflection difference between the original model and different levels of steel tendon reduction

The overall deflection of the bridge under the five operating conditions as it varies with the reduction of external prestressing is shown in Figure 20.

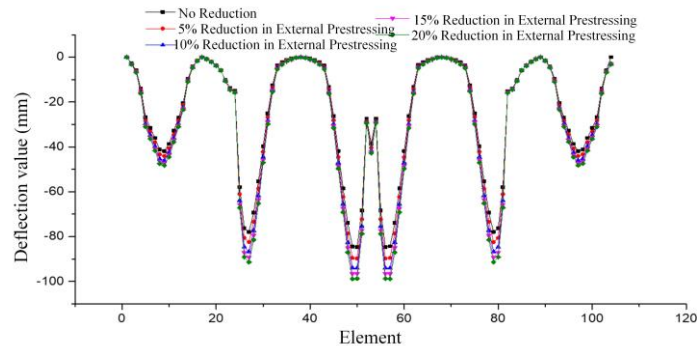


Fig.20- Comparison of deflections between the original model and models with 5%, 10%, 15%, and 20% reduction in steel tendons

From Figures 19 to 20, it can be observed that as the loss of external prestressing increases, the deflection values of each cross-sectional unit across the entire bridge also increase. Notably, the

deflection values of the units near the mid-span of the bridge show significant changes, while the deflection values of the other units remain almost unaffected by the loss. The deflection difference diagrams indicate that when the external prestressing loss is 5%, the maximum decrease in deflection near the mid-span is 5.07 mm. Figure 3-36 further shows that when the external prestressing loss reaches 20%, the maximum decrease in deflection near the mid-span is 14.06 mm.

The maximum deflection values at the mid-span sections of each span of the bridge under the five operating conditions are shown in Figure 21. The maximum deflection values at the mid-span sections of the entire bridge under Operating Conditions 1 to 5 are listed in Table 4.

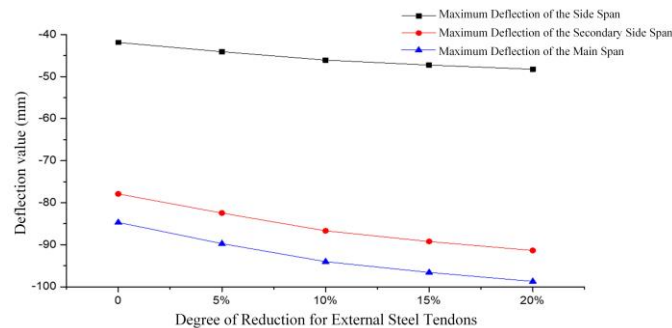


Fig.21 - Comparison of deflections between the original model and models with 5%, 10%, 15%, and 20% reduction in steel tendons

Tab. 4 - Impact of external prestressing reduction on bridge deflection values

Operating Conditions:	Maximum Deflection Value (mm)		
	Maximum value of side span	Maximum value of secondary side span	Maximum value of middle span
Operating Condition 1: No Reduction	-41.87	-77.90	-84.68
Operating Condition 2: 5% Reduction	-44.07	-82.46	-89.75
Operating Condition 3: 10% Reduction	-46.06	-86.70	-94.03
Operating Condition 4: 15% Reduction	-47.25	-89.25	-96.60
Operating Condition 5: 20% Reduction	-48.25	-91.37	-98.74

From Figure 21 and Table 4, we can observe that the maximum deflection of the side span in the original bridge model without any loss is -41.87 mm. When the external prestressing loss is 5%, 10%, 15%, and 20% respectively, the maximum deflection of the side span increases by 5.25% (-2.2 mm), 10.00% (-4.19 mm), 12.85% (-5.38 mm), and 15.24% (-6.38 mm) compared to the case with no reduction. Similarly, the maximum deflection of the secondary side span increases by 5.85% (-4.56 mm), 11.30% (-8.80 mm), 14.57% (-11.35 mm), and 17.29% (-13.47 mm) respectively. For the middle span, the maximum deflection increases by 5.99% (-5.07 mm), 11.04% (-9.35 mm), 14.08% (-11.92 mm), and 16.60% (-14.06 mm) respectively compared to the no-reduction case. The analysis of these data indicates that the deflection of continuous rigid frame bridges is directly proportional to

the degree of external prestressing reduction, which is a major factor affecting the reinforced continuous rigid frame bridges.

Impact of Overload Under Operational Live Load on Bridge Deflection

The bridge is located at a strategic traffic hub, and most of the passing vehicles are heavily loaded or overloaded, with maximum vehicle tonnage far exceeding the live load limits allowed by the specifications. Therefore, based on the actual situation of the bridge, an analysis of the impact of live load overload on the deflection values of the bridge is conducted. This analysis is divided into five operating conditions:

- Operating Condition 1: Original Model
- Operating Condition 2: Overload by 25%
- Operating Condition 3: Overload by 50%
- Operating Condition 4: Overload by 75%
- Operating Condition 5: Overload by 100%

The impact of simulated overload on the deflection values of the bridge is shown in Figure 22.

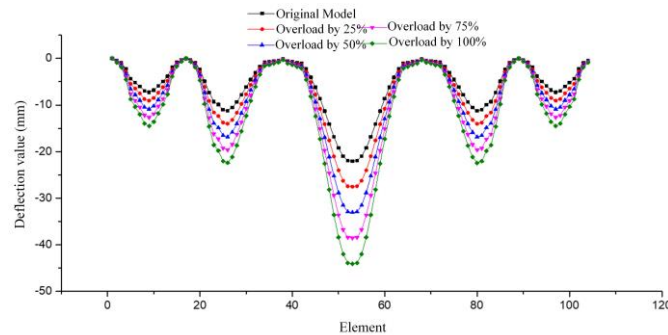


Fig. 22 - Deflection values of each bridge unit caused by different live load overloads

From Figure 22, it can be observed that as the live load overload increases, the deflection values of each cross-sectional unit across the entire bridge also increase. Notably, the deflection values of the units near the mid-span of the bridge show significant changes and are greatly affected by the live load overload, while the deflection values of the other units are less affected. The differences in deflection values caused by overloads of 25%, 50%, 75%, and 100% compared to the no-overload condition are shown in Figure 23.

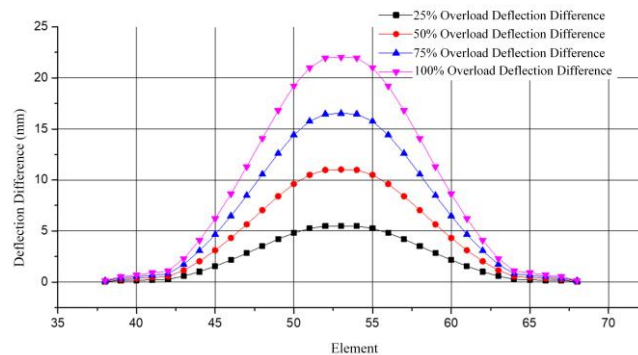


Fig.23 - Deflection differences between each unit of the main span due to overload and the original model

As can be seen from Figure 23, with the increase in overload, the deflection of each cross-

section gradually increases. When the load exceeds the design load by 25%, 50%, 75%, and 100%, the maximum deflection differences between the mid-span and the original model are 5.51 mm, 11.02 mm, 16.52 mm, and 22.03 mm, respectively. This indicates that overload has a significant impact on the deflection values of bridge cross-sections, and overload is an important factor affecting the downward deflection of the mid-span of reinforced bridges.

CONCLUSIONS

Using simulation analysis and calculation methods, the impact of various parameters on the deflection of a continuous rigid frame bridge was analyzed. The main parameters included: internal prestressing loss, stiffness reduction, shrinkage and creep, over-sizing, shear deformation, external prestressing loss, and overload. The research results of this paper have significant research significance and practical value for the reinforcement design of prestressed continuous rigid frame bridges. The following conclusions were obtained:

- (1) Internal prestressing loss, stiffness reduction, shrinkage and creep, over-sizing, shear deformation, external prestressing loss, and overload all contribute to the downward deflection of the mid-span, but each factor has a different degree of influence.
- (2) When comparing all factors comprehensively, stiffness reduction is the largest contributor to the downward deflection of the mid-span. For reinforced bridges, external prestressing loss and overload are the most significant factors. When the stiffness is reduced by 10%, the deflection difference plot shows that the maximum decrease in deflection near the mid-span is 8.76mm. When the external prestressing loss is 5% and 15% respectively, the maximum deflection of the side span increases by 5.25% (-2.2 mm) and 12.85% (-5.38 mm) compared to the case with no reduction.
- (3) In the calculation of continuous rigid frame bridges, shear deformation cannot be ignored. It can be observed that the deflection of the middle span considering shear deformation is 10.78% (8.24 mm) greater than that without considering shear deformation.

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FLEXURAL PERFORMANCE OF CONCRETE T-BEAMS REINFORCED WITH UHPC: EXPERIMENTAL AND THEORETICAL ANALYSIS

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ABSTRACT

The use of ultra-high performance concrete (UHPC) for strengthening deficient reinforced concrete T-beam bridges has gained popularity in recent years. Despite its growing application, the failure mechanisms of UHPC-reinforced T-beams under flexural loads, along with corresponding design methods, require further exploration. This study aims to investigate the flexural performance of T-beams enhanced with thin UHPC layers through a combination of theoretical analysis and experimental testing. An analytical model is proposed to estimate the cracking and ultimate resistances of UHPC-strengthened T-beams subjected to bending moments. To validate this model, three concrete T-beam specimens with varying UHPC thicknesses were fabricated and tested to failure. The observed failure modes, load-displacement responses, and strain distributions of the T-beams are presented and analyzed. Results indicate that the incorporation of UHPC layers markedly improves the flexural performance of the T-beams: increasing the layer thickness from 0 mm to 50 mm resulted in a 167.8% enhancement in flexural stiffness, a 241.0% increase in initial cracking load, and a 40.8% rise in ultimate flexural capacity. The relative error between predictions from the developed model and experimental outcomes was found to be less than 10%, underscoring the model's validity and indicating sufficient safety margins. These findings offer significant insights for the design of flexural strengthening techniques employing thin UHPC layers in defective concrete T-beams, contributing to enhanced structural integrity and longevity.

KEYWORDS

Concrete T-beam, UHPC, Strengthening, Flexural performance, Predictive model

INTRODUCTION

The increasing operational demands on transportation infrastructure and the accelerated deterioration of aging concrete bridges have made structural reinforcement a critical focus in civil engineering [1-3]. Conventional strengthening techniques, including section enlargement with

normal concrete (NC), bonded steel sheets, and external prestressing [4, 5], have been extensively adopted to improve load-bearing capacity and durability of the existing bridges. However, these methods are frequently associated with limitations such as increased structural self-weight, susceptibility to corrosion, and localized stress concentrations at anchorage regions [6]. These challenges necessitate the development of advanced materials that integrate structural efficiency with sustainable performance.

Ultra-high-performance concrete (UHPC), characterized by its exceptional compressive strength (>150 MPa), strain-hardening tensile properties, and superior durability, has emerged as a transformative material in deficient bridge rehabilitation [7]. International engineering applications have demonstrated its potential. For instance, a 30 mm UHPC overlay applied to the Rhine Bridge in Switzerland enhanced impact resistance by approximately 60% [8], while the U.S. Route 46 Bridge project achieved a 30% reduction in construction time through UHPC's rapid curing properties [9]. In China, the rehabilitation of the Wenzaobang Bridge using UHPC effectively reduced mid-span deflection by 42% by mitigating carbonation-induced cracking in T-beams [10]. These cases highlight UHPC's ability to overcome the limitations of traditional reinforcement methods.

The mechanical benefits of UHPC reinforcing technology have been further validated through experimental investigations. Yin et al. [11] reported that UHPC overlays in the tensile zones of T-beams changed failure modes of the structure from brittle shear to ductile flexure, with crack density increasing proportionally to overlay thickness. Men et al. [12] expanded on these findings, demonstrating that extended UHPC reinforcement lengths optimized tensile utilization, allowing reinforced beams to achieve 95% of theoretical ultimate bending resistance. These studies collectively confirm UHPC's role in enhancing both load capacity and ductility in deteriorated structural members.

Despite these advancements, critical knowledge gaps remain regarding the flexural behavior of UHPC-strengthened concrete T-beams, particularly concerning the influence of UHPC layer thickness on stiffness evolution, strain compatibility, and the mechanisms of composite interaction. To address these issues, a predictive model for flexural capacity was developed based on the conversion section method in accordance with Chinese design codes [13]. Three strengthened concrete T-beams with UHPC layer thicknesses of 0 mm, 30 mm, and 50 mm were designed and tested under four-point loading conditions, with systematic analyses of failure modes, load-deflection curves, and strain distribution characteristics. The accuracy of the proposed model was subsequently validated against experimental results, providing both theoretical insights and practical guidance for optimizing UHPC-enhanced strengthening strategies in bridge rehabilitations.

PREDICTION OF FLEXURAL STRENGTH OF UHPC-REINFORCED T-BEAMS

Basic Assumptions

Based on the existing conversion section method for calculating the flexural capacity of UHPC-strengthened concrete T-beams, the flexural capacity of a UHPC-strengthened reinforced concrete (RC) T-beam primarily depends on four components: the tensile reinforcement in the original beam, the high-strength steel mesh or reinforcement bars within the UHPC layer, the concrete in the compressive zone, and the tensile strength of the UHPC. This section adopts the transformed section method for analysis, based on existing literature and design codes [13, 14]. The following fundamental assumptions are made for calculating the flexural capacity:

- (1) During flexural loading of a UHPC-strengthened RC T-beam, the strain distribution along the depth of the NC and UHPC sections always satisfies the plane section assumption;

- (2) No relative slip occurs at the interface between the UHPC strengthening layer and the existing RC T-beam;
- (3) After cracking, the original concrete is considered no longer effective in carrying tensile stress. The strain hardening behavior of reinforcement and the contribution of compressive reinforcement in the flange of the original beam are neglected. Tensile force is entirely resisted by the tensile reinforcement in the original beam and the steel mesh or reinforcement bars in the UHPC layer;
- (4) The influence of bond-slip between reinforcement and concrete, self-weight of the beam, as well as shrinkage and creep of the UHPC layer and the original concrete, are neglected;
- (5) The strain hardening behavior of UHPC material after tensile cracking is taken into account.

Material Constitutive Model

In the calculation and analysis of the ultimate flexural capacity of UHPC-strengthened beams, the constitutive relationships of NC, steel reinforcement, and UHPC were selected in accordance with the Chinese design code [15]. The stress-strain curve of the steel reinforcement includes a yield plateau, while the UHPC material does not cease to bear load immediately after cracking. This is primarily attributed to the bridging effect of steel fibers, resulting in mechanical behavior that significantly differs from that of conventional concrete. The constitutive models of the three materials are shown in Figure 1.

The equation for the compressive constitutive model of NC is:

$$\sigma_N^c = \begin{cases} f_{cu} [1 - (1 - \frac{\varepsilon_N^c}{\varepsilon_0})^2], & \varepsilon_N^c \leq \varepsilon_0 \\ f_c, & \varepsilon_0 < \varepsilon_N^c \leq \varepsilon_{cu} \end{cases} \quad (2.1)$$

The equation for the tension constitutive model of steel reinforcement is:

$$\sigma_s = \begin{cases} E_s \varepsilon_s, & 0 \leq \varepsilon_s \leq \varepsilon_y \\ f_y, & \varepsilon_y < \varepsilon_s \leq \varepsilon_u \end{cases} \quad (2.2)$$

The equation for the tension constitutive model of UHPC is:

$$\sigma_u^t = \begin{cases} E_c \varepsilon_u^t, & 0 < \varepsilon_u^t \leq \varepsilon_{tu} \\ f_{tu}, & \varepsilon_{tu} < \varepsilon_u^t \leq \varepsilon_{tp} \end{cases} \quad (2.3)$$

Where, f_{cu} -The axial compressive strength of concrete; ε_0 -The concrete compressive strain corresponding to the stress f_{cu} , taken as 0.0015; ε_{cu} -The ultimate compressive strain of concrete, taken as 0.002; E_s -The elastic modulus of steel reinforcement; f_y -The yield stress of steel reinforcement; ε_y -The tensile strain of steel reinforcement corresponding to the stress f_y , taken as 0.002; ε_u -The ultimate tensile strain of steel reinforcement, taken as 0.075; E_c -The elastic modulus of UHPC; f_{tu} -The peak tensile stress of UHPC; ε_{tu} -The peak tensile strain of UHPC, taken as 0.001; ε_{tp} -The ultimate strain of UHPC, taken as 0.002.

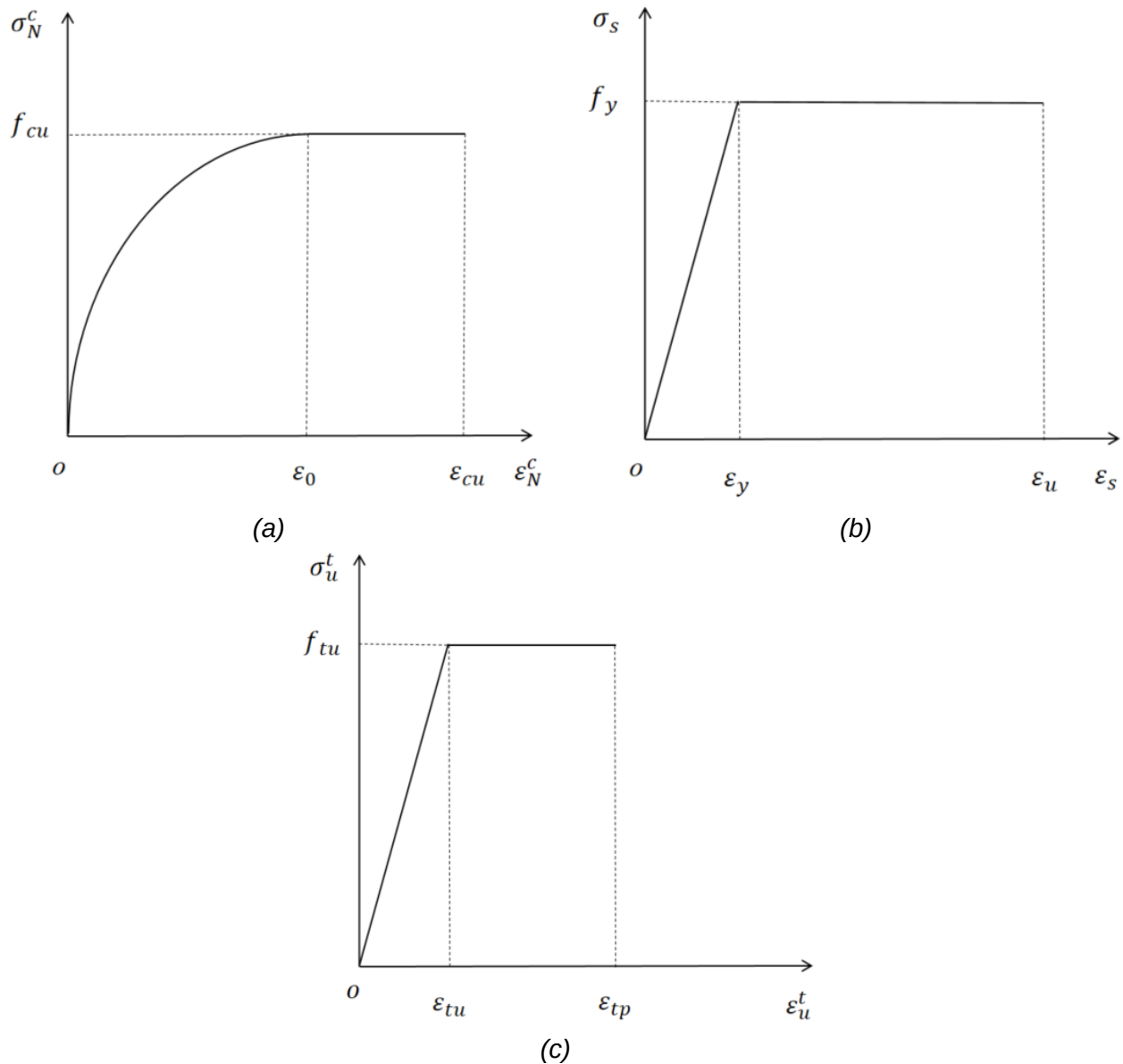


Fig. 1 - Constitutive models of the three materials: (a) NC compression; (b) Steel tension (c) UHPC tension

Analytical Formulation Proposed

The fundamental principle of the conversion section method involves combining the original T-beam with a layer of UHPC to create a new, effective section. In this analysis, the mechanical properties of the UHPC layer are translated into an equivalent NC section. The modulus conversion factor, denoted as n , is defined as the ratio of the modulus of elasticity of the UHPC (E_{uhpc}) to that of NC (E_c). This can be expressed as $n = E_{uhpc}/E_c$. Once the new effective section is established, the flexural capacity can be calculated based on the assumption of plane sections. Figure 2 shows the schematic cross-sectional diagrams and stress-strain distribution diagrams of the UHPC strengthened T-beam.

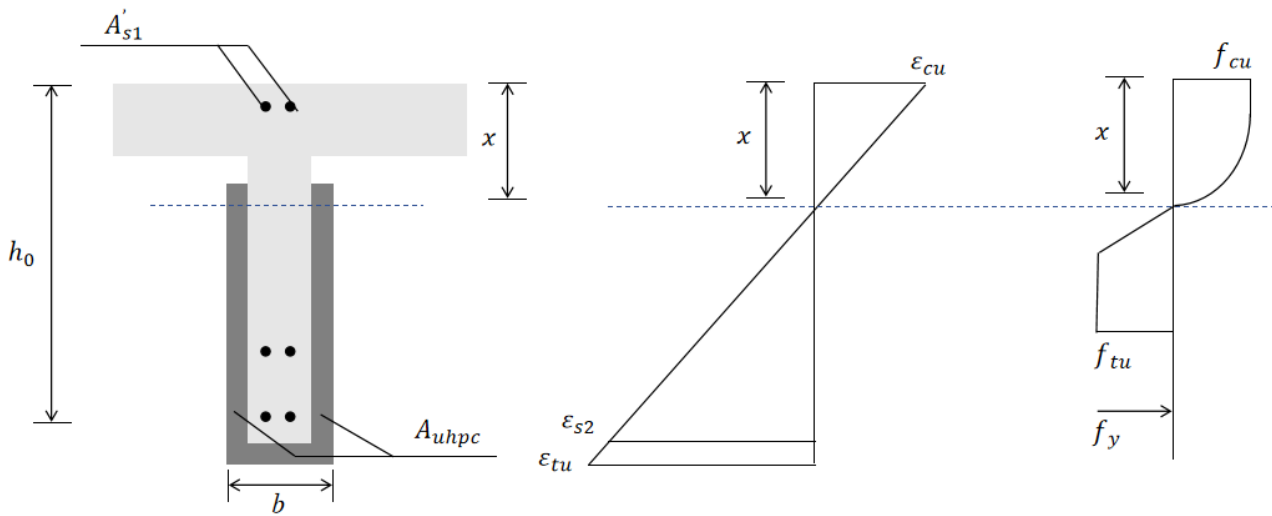


Fig. 2 - Stress-strain distribution across the cross-section of the T-beam specimens

According to the code [13], when flexural strengthening is applied to the tension zone of a T-beam, the flexural load-bearing capacity of the positive section should be calculated using the following formula:

$$M_d = f_{cd1}bx(h_0 - \frac{x}{2}) + f'_{sd1}A'_{s1}(h_0 - a'_{s1}) \quad (2.4)$$

The height of the concrete compression zone can be calculated using the following formula:

$$f_{cd1}bx = f_{sd1}A_{s1} - f'_{sd1}A'_{s1} + \sigma_{s2}A_{s2} \quad (2.5)$$

$$\sigma_{s2} = \epsilon_{s2}E_{s2} \leq f_{sd2} \quad (2.6)$$

The height of the concrete compression zone should also meet the following conditions:

$$2a'_{s1} \leq x \leq \xi_b h_{01} \quad (2.7)$$

Where, M_d -Design moment; f_{cd1} -Design value of axial compressive strength of original concrete components; f_{sd1} , f'_{sd1} -Design strengths for tensile and compressive, respectively of longitudinal steel bars in the original component; A_{s1} , A'_{s1} -The cross-sectional area of longitudinal ordinary steel bars in the tensile and compressive zones, respectively, of the original component; a'_{s1} -The distance from the longitudinal ordinary reinforcement in the compression zone to the extreme compression fiber of the section; A_{s2} -Add cross-sectional area of ordinary steel rebars; b -Section width of reinforced components; h_{01} -Effective height of original section; h_0 -Effective height of reinforced section; x -Height of concrete compression zone; σ_{s2} -Add tensile stress to longitudinal steel rebars; E_{s2} -Add elastic modulus of longitudinal ordinary steel bars; ϵ_{s2} -Add tensile strain of longitudinal steel rebars; f_{sd2} -Design value of tensile strength for newly added longitudinal steel rebars; ξ_b -Height of the compression zone at the relative boundary of the normal section.

For T-beams strengthened with a UHPC layer in the tensile zone, the cracking load under flexural loading should be calculated using the following formula:

$$M_{cr} = f_{cd1}b(h_0 - x)(\frac{h_0 - x}{8}) + \frac{1}{8}f'_{sd1}A'_{s1}(h_0 - x) + \frac{6}{7}\beta A_{uhpc}f_{uhpc}(h_0 - 0.8x) \quad (2.8)$$

For T-beams strengthened with a UHPC layer in the tensile zone, the bending capacity of the cross-section under bending rebars should be calculated according to the following formula:

$$M_u = f_{cd1}bx(h_0 - \frac{x}{2}) + f'_{sd1}A'_{s1}(h_0 - a'_{s1}) + \beta A_{uhpc}f_{uhpc}(h_0 - 0.8x) \quad (2.9)$$

Where, A_{uhpc} -Cross-sectional area of UHPC layer; f_{uhpc} -Tensile strength of UHPC; h_0 -Effective height of UHPC reinforced section; β -Transfer effect coefficient, taken as 0.07. Other symbols have the same meaning as the formula (2.4).

EXPERIMENTAL PROGRAM

Specimen Design and Preparations

Figure 3 shows the configuration and geometric dimensions of the T-beams, which have a length of 3,000 mm, a height of 520 mm, and a width of 640 mm. The web has a width of 120 mm and a height of 440 mm. The internal longitudinal reinforcement consists of 10 mm ($\Phi 10$) and 16 mm ($\Phi 16$) diameter steel rebars. The UHPC layer covers the entire web on both sides and the bottom of the T-beam. The interface between the UHPC and NC was enhanced by short steel rebars implanted after surface roughening treatment.

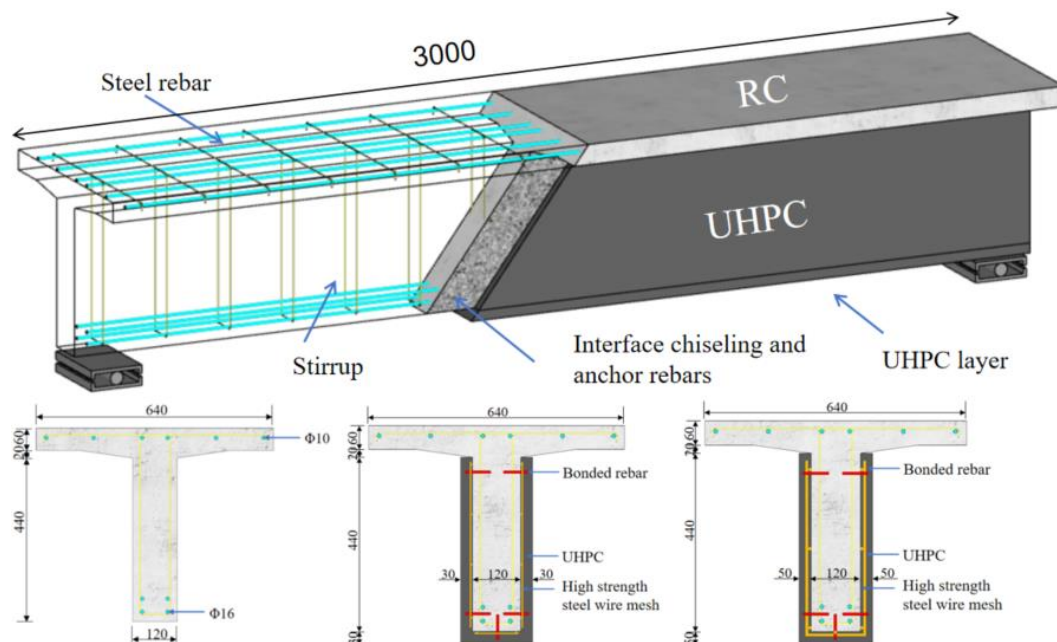


Fig. 3 - Configuration of UHPC-reinforced T-beam specimens

Table 1 provides details of the test T-beam specimens. A total of three T-beams were designed and fabricated, with UHPC layer thicknesses of 30 mm and 50 mm, respectively. Steel wire mesh and steel rebars were embedded within the UHPC layers. The T-beams are labeled as follows: "RC-TH0" refers to the unreinforced concrete T-beam, "UN-TH30" indicates the T-beam reinforced with a 30 mm-thick UHPC layer, and "UN-TH50" denotes the T-beam reinforced with a 50 mm-thick UHPC layer.

Tab. 1 - Information of UHPC reinforced T-beams

Specimen code	Reinforcing form	UHPC thickness	Reinforcement ratio	Interface treatment
RC-TH0	--	--	1.24%	--
UN-TH30	UHPC layer	30 mm		Chiseling + shear steel rebars
UN-TH50		50 mm		

Figure 4 shows the main process of T-beam specimen construction: (a) Fabricating the mold and steel rebars framework; (b) Pouring NC and curing for 28 days; (c) Surface roughening of the NC and embedding the steel rebars; (d) Pouring UHPC and curing for 28 days.

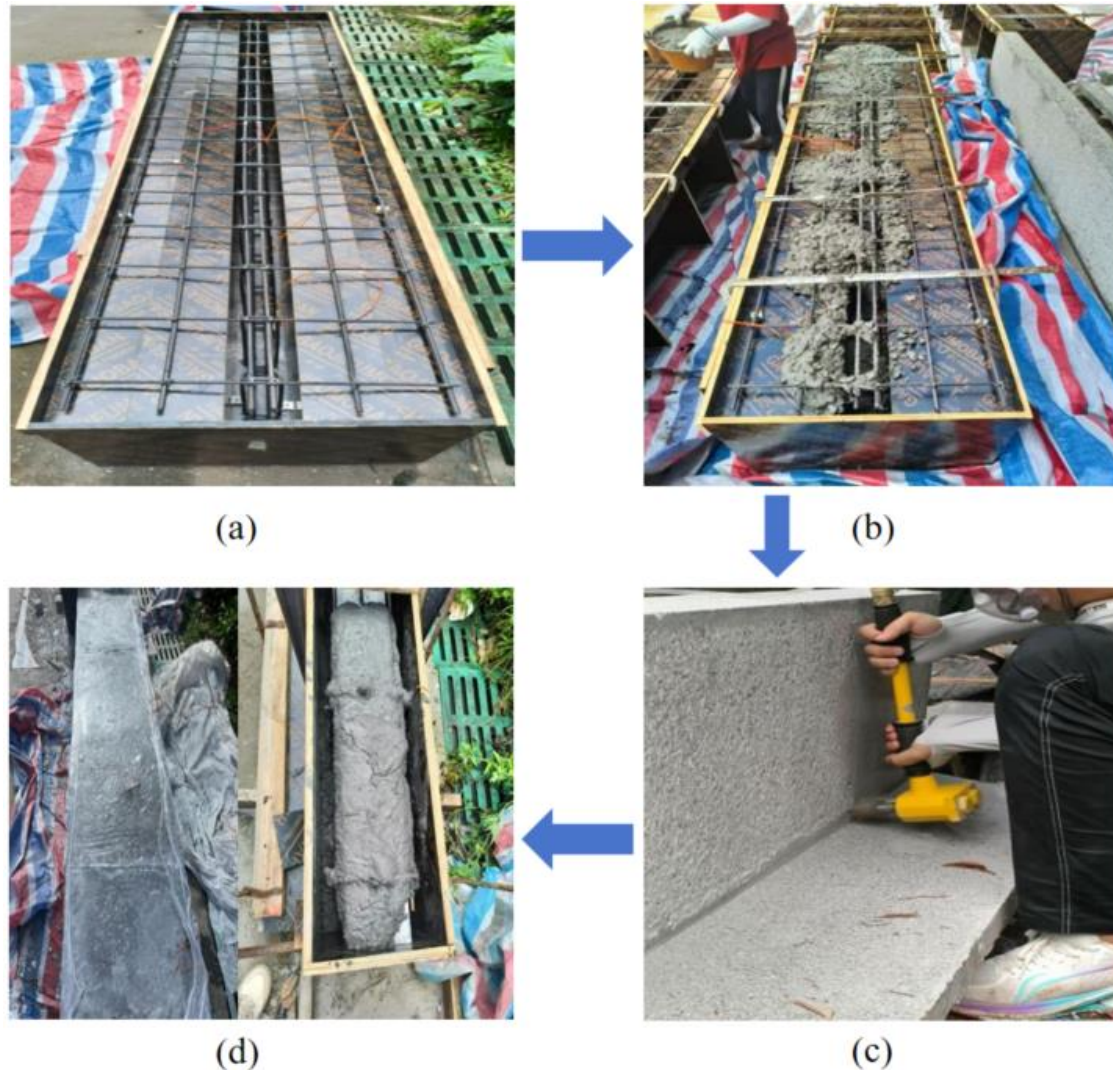


Fig. 4 - Fabrication of test beam specimens

Material Properties

The NC used in this experiment was C55 grade concrete with a target compressive strength of 55 MPa, while the UHPC used is a commercial pre-mixed dry material, with a steel fiber content of 2% and a target compressive strength of 120 MPa. The material properties of NC and UHPC were determined based on Chinese codes [15, 16]. The measured compressive strength and elastic modulus of UHPC cubes were 136.5 MPa and 46.7 GPa, respectively, while the compressive strength and elastic modulus of NC were 59.1 MPa and 36.1 GPa, respectively. The internal steel rebars used HRB400, with measured yield strength, ultimate tensile strength, and elastic modulus of 483 MPa, 594 MPa, and 200 GPa, respectively.

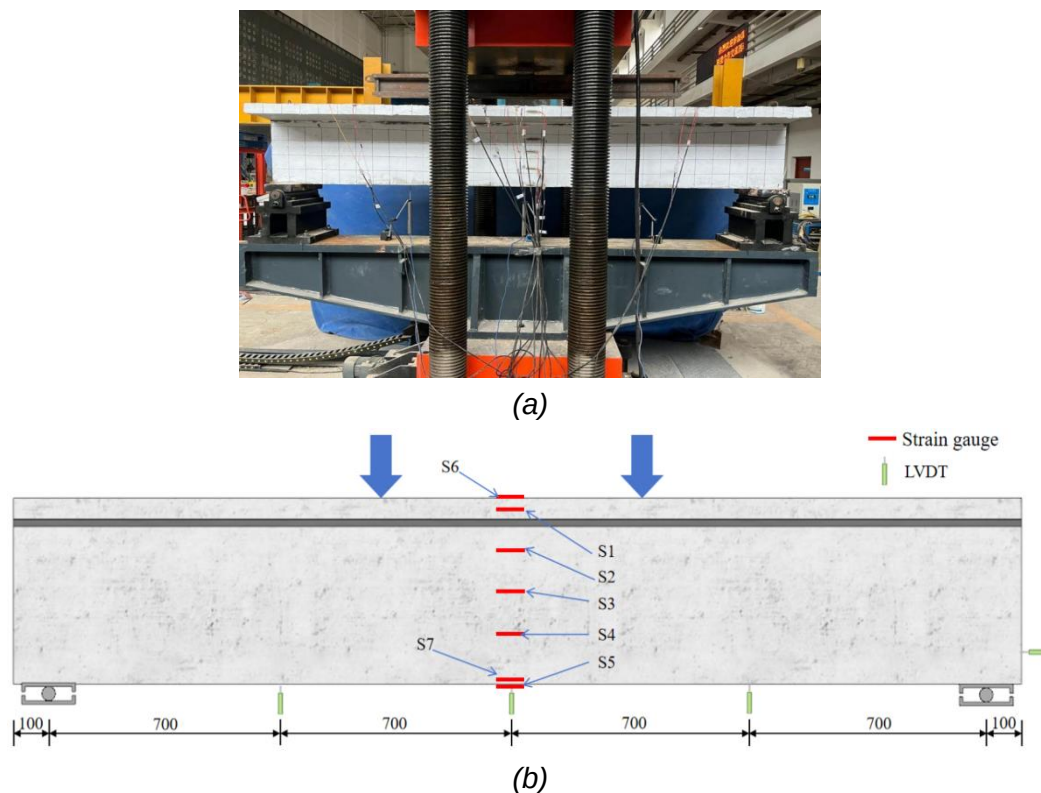
Test Step and Instrumentation

Figure 5(a) shows the loading schematic of the specimens. A 10,000-kN servo press was used to apply the load, with a distribution beam achieving a four-point bending load. The test

T-beams were installed on fixed-hinged supports and sliding-hinged supports. The loading procedure was divided into two steps: (i) Preloading stage: A load of 10 kN was applied to eliminate initial defects in the loading system and to check the operational status of the equipment. (ii) Formal loading stage: A monotonic incremental method was used, with an initial load increase of 50 kN per step, holding for 1 minute at each step. When cracks larger than 0.1 mm first appeared in the beam specimen, the loading was switched to displacement control (1 mm/min) until the T-beam failed.

During the testing process, the primary measured parameters included the deflection at various points of the beam, concrete strain, steel reinforcement strain, and interfacial slip between the UHPC and NC layers. Figure 5(b) shows the layout of strain gauges and displacement transducers on the test beam. The deflection data were obtained using vertically mounted LVDT displacement transducers, which were positioned at the L/4, mid-span, and 3L/4 sections along the bottom of the beam. Concrete and reinforcement strains were measured with electrical resistance strain gauges and collected via a strain acquisition system. At the mid-span section, multiple strain measurement points were arranged at the top and bottom surfaces of the concrete. To quantify the relative slip at the UHPC-NC interface, a horizontal displacement transducer was mounted on the UHPC surface at the right end of the beam, with an angle steel fixture installed on the NC.

Figures 5(c) and (d) show the test acquisition instruments used during the experiment. The load was measured by a pressure sensor connected to the test acquisition system. During the load-holding phases, crack widths were observed and crack propagation was recorded. Under each load level, the widths of multiple cracks were measured individually. The maximum crack width observed during the measurement process was defined as the representative crack width for that specific load level.





(c)

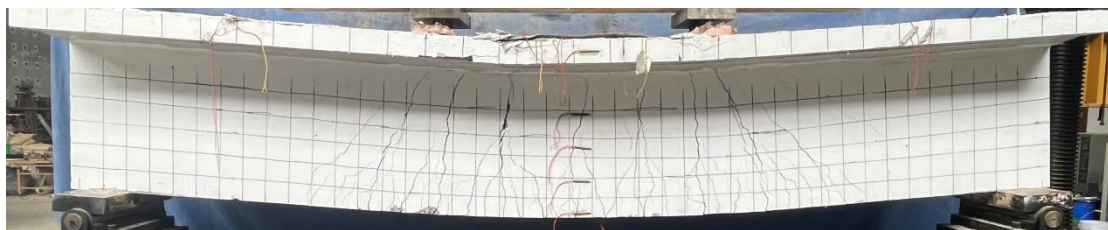


(d)

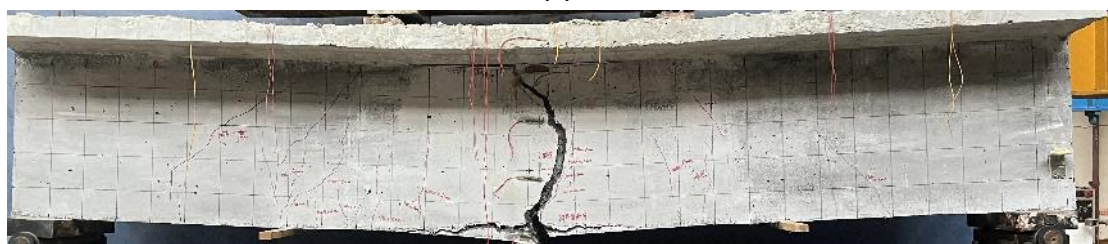
Fig 5 - Loading and strain gauge layout: (a) Loading diagram; (b) Strain gauge layout; (c) Test acquisition instrument; (d) Crack comprehensive tester.

Failure Pattern

Figure 6 illustrates the failure patterns observed in the tested T-beams. For the RC-TH0 specimen, cracks initially formed at the bottom of the T-beam and gradually extended along the sides to the top flange. As the tensile reinforcement bars at the bottom yielded, multiple cracks rapidly propagated, leading to an increase in the downward deflection of the beam, ultimately causing the concrete compression zone beneath the top loading point to collapse. In contrast, the beams reinforced with UHPC layers (e.g., UN-TH30 & UN-TH50) exhibited more concentrated cracking patterns. At the point of failure, the width of a primary crack increased rapidly while the widths of the other cracks widened to a lesser extent. This behavior suggests that the steel fibers in the UHPC provided a practical bridging effect, which helped to suppress both the formation and development of cracks during the failure process.



(a)



(b)



(c)

Fig. 6 - Failure pattern of test specimens: (a) RC-TH0; (b) UN-TH30; (c) UN-TH50

Load-Deflection Relationship

As shown in Figure 7(a), the load-deflection curves of the T-beams initially display a linear trend, indicating behavior within the elastic phase. As the applied load increases, the slope of the curves decreases, marking the transition into the elastic-plastic phase, characterized by a gradual reduction in stiffness. As the T-beams approach peak load, their mechanical responses begin to diverge; for RC-TH0, the load remains relatively stable while displacement increases significantly, indicating the yielding stage and the onset of ductile damage. In contrast, the loads for UN-TH30 and UN-TH50 drop rapidly after peaking. This behavior is attributed to the propagation of the primary crack in the UHPC, with the remaining NC working in conjunction with the steel reinforcements. These observations align with the observed failure modes. Figure 7(b) presents the relative slip at the NC-UHPC interface, showing that the slip for UN-TH50 is significantly smaller than that of UN-TH30 upon reaching peak load. This difference can be explained by the increased thickness of the UHPC layer, which enhances stiffness, delays structural deformation, and suppresses the progression of interfacial slip.

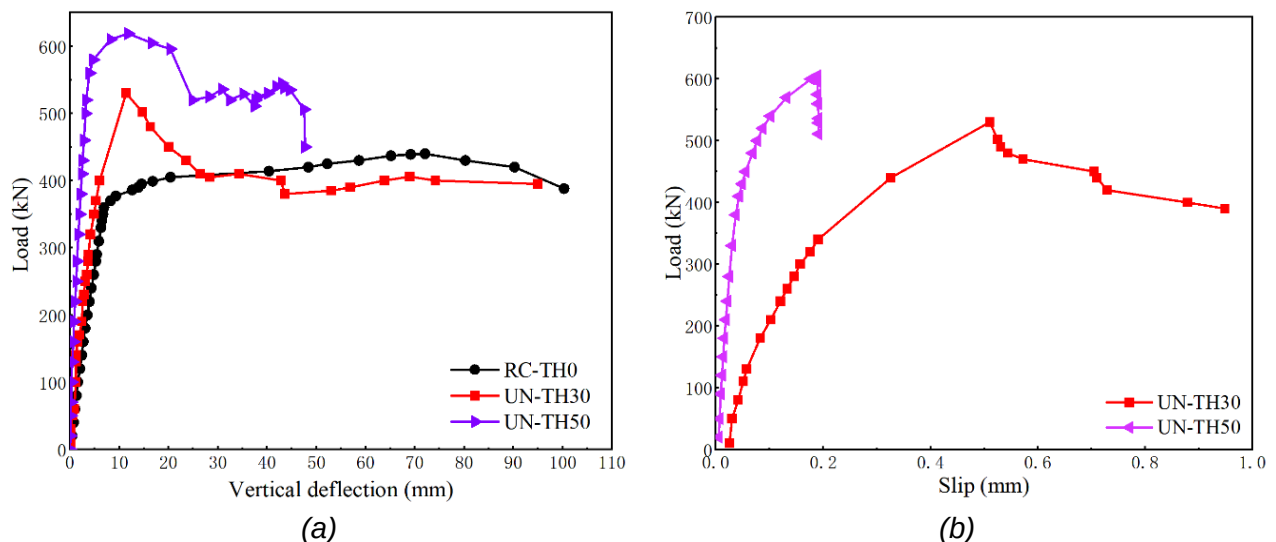


Fig. 7 - Deflection and interface slips: (a) Load-deflection curve; (b) Load-interface slip curve

Table 2 presents the load characteristics of the tested T-beams, where P_c represents the cracking load, P_y denotes the yield load, δ_y corresponds to the deflection at yield; P_u refers to the ultimate load, δ_u corresponds to the deflection at the ultimate load; and K is the ratio of the yield load to the corresponding deflection, which reflects the bending stiffness of the beam.

Tab. 2 - Summary of test results

Specimen code	P_c (kN)	P_y (kN)	P_u (kN)	δ_y (mm)	δ_u (mm)	K (kN/mm)
RC-TH0	60.4	360.2	440	6.91	72.05	52.12
UN-TH30	102.7	402.1	530.4	6.01	11.39	66.91
UN-TH50	205.9	561.3	619.3	4.02	11.80	139.62

Figure 8 presents the load characteristic values for each test specimen. The data indicate that incorporating UHPC at the bottom of the T-beam significantly enhances its structural performance, positively influencing the cracking, yielding, and ultimate load. Specifically, the most considerable improvement is observed in the cracking load, which can increase by as much as 241%. As the thickness of the UHPC is increased from 0 mm to 30 mm and subsequently to 50 mm, the yielding load of the beam demonstrates increments of 11.6% and 55.8%, respectively. Correspondingly, the ultimate load exhibits increases of 20.5% and 40.8%, respectively. These findings clearly illustrate that the reinforced UHPC layer effectively enhances T-beams' flexural performance.

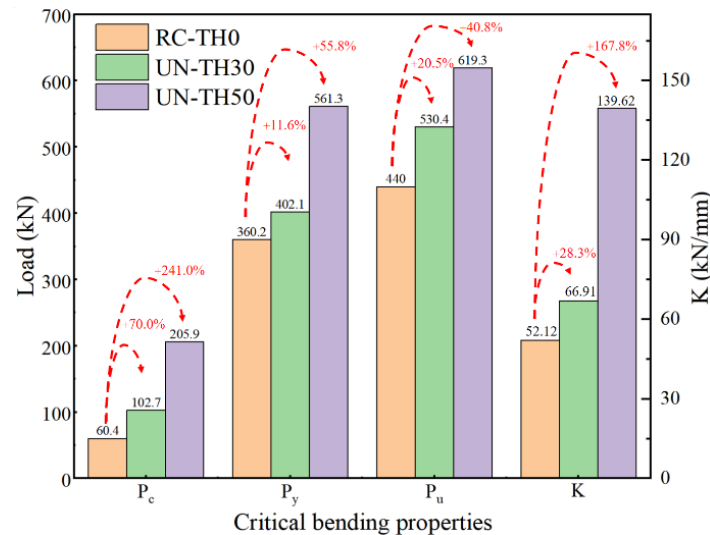


Fig. 8 - Critical mechanical properties

Strain History

Figure 9 illustrates the strain history of T-beams' mid-span reinforced with UHPC. The strain distribution along the height of the T-beam is linear as the load increases, indicating that the mid-span section adheres to the plane section assumption. The strain profiles demonstrate a clear linear relationship between strain and height at various load stages. At lower loads (e.g., $0.1P_u$), compressive strains dominate near the top of the beam, transitioning linearly to tensile strains toward the bottom. As the load increases to $0.9P_u$, the strain gradient becomes more pronounced, with maximum tensile strains exceeding $1,000 \mu\epsilon$ at the bottom of the T-beam, while compressive strains near the top approach $-200 \mu\epsilon$. It is observed that the neutral axis shifts slightly upward with higher loads, reflecting the gradual engagement of UHPC's tensile capacity. Both UHPC-reinforced T-beams exhibit minimal deviation from linearity, even near peak loads, underscoring the effectiveness of UHPC in maintaining sectional compatibility, which is crucial for accurately predicting the flexural behavior of strengthened T-beams under both service and ultimate loads.

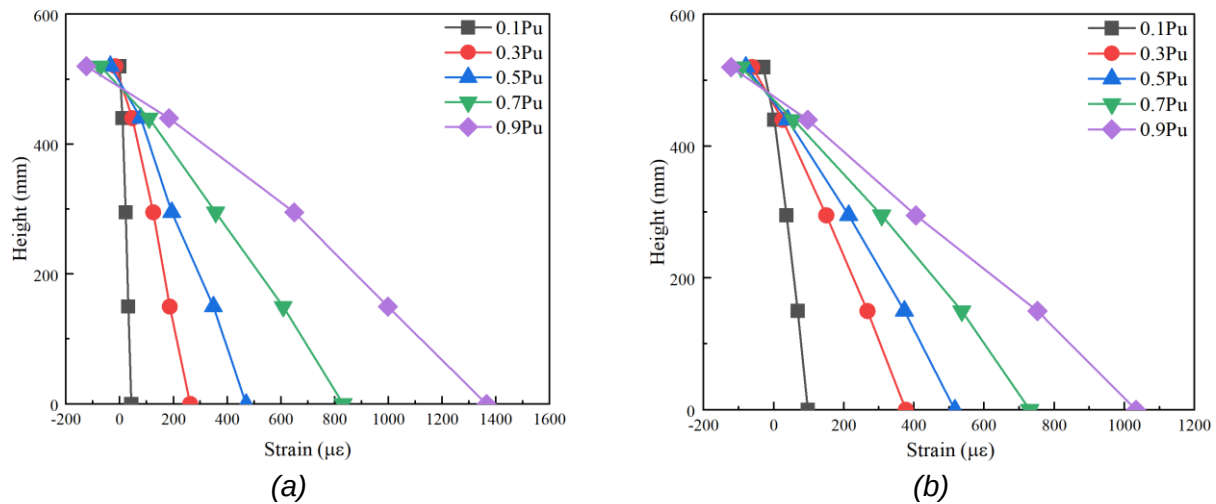


Fig. 9 - Strain distribution at mid-span section: (a) UN-TH30; (b) UN-TH50

Verification of Computational Model

Verification using Current Experimental Results

Table 3 compares the theoretical and experimental bending capacities of UHPC-reinforced T-beams. For ultimate loads, the experimental results exceed theoretical predictions by 6.3 to 9.5%, with all relative errors remaining below 10%, demonstrating the conservative reliability of the proposed model. This systematic underestimation in calculations ensures a built-in safety margin, validating the model's applicability for practical design while prioritizing structural safety under ultimate load conditions. For cracking loads, the errors between the experimental data and the theoretically calculated results range from 5.0% to 9.3%. However, the theoretical cracking load for beam UN-TH30 is higher than the experimental value, which may indicate insufficient reliability of the computational model. The underlying reason could be that significant residual tensile stresses existed in the original beam. These residual stresses, upon superposition with the stresses induced by the subsequently applied loads, resulted in higher actual tensile stresses in the UHPC layer. Furthermore, the prolonged storage time of the original beam may have led to a reduction in the elastic modulus of the concrete, while the UHPC retained a relatively high elastic modulus. The increased stiffness discrepancy between the two materials worsened deformation compatibility, generating additional bending stresses at the interface and accelerating crack initiation in the UHPC layer. Consequently, the experimentally measured cracking load of beam UN-TH30 was lower than predicted.

Tab. 3 - Comparison between theoretical and experimental values

	Specimen code	Theoretical value (kN)	Experimental value (kN)	Relative error (%)
Ultimate loads	RC-TH0	402.4	440.0	8.5
	UN-TH30	497.2	530.4	6.3
	UN-TH50	560.5	619.3	9.5
Cracking loads	RC-TH0	56.7	60.4	6.1
	UN-TH30	112.3	102.7	-9.3
	UN-TH50	195.6	205.9	5.0

Verification using Experimental Data in Literature

To further validate the accuracy of the computational model for predicting the flexural capacity of reinforced concrete T-beams strengthened with a thin UHPC layer, experimental data from twenty-seven groups of flexural tests on UHPC-strengthened T-beams in Reference [17-20] were selected for comparison. The comparison between the calculated ultimate flexural capacities and the experimental results is presented in Figure 10. As shown in Figure 10, the computational model accurately predicted the ultimate flexural capacity of UHPC-strengthened T-beams. The errors between the theoretical and experimental values are all within 10%, with the theoretical values are generally slightly lower than the experimental values. This indicates that the computational model not only provides precise predictions but also maintains a certain safety margin, which holds significant practical value for engineering applications.

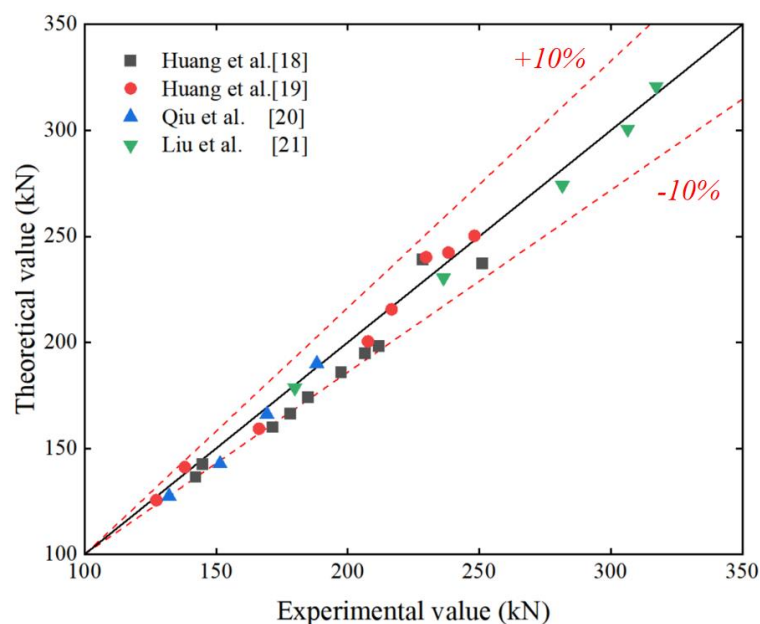


Fig. 10 - Comparison of theoretical and experimental values

CONCLUSION

This study systematically examined the flexural behavior of UHPC-reinforced T-beams through experimental testing and theoretical modeling. Three specimens with UHPC layers of varying thicknesses (0 mm, 30 mm, and 50 mm) were fabricated and subjected to four-point bending until failure. The failure mechanisms, load-displacement characteristics, and strain evolution were analyzed. A predictive model for flexural capacity was developed using the converted section method, integrating experimental observations with code-based principles. Key findings are summarized as follows:

- (1) UHPC layer significantly changes the failure mode of T-beams while enhancing their flexural performance. T-beams with 30 mm and 50 mm UHPC layers demonstrated 28.3% and 167.8% increases in flexural stiffness, respectively, compared to the unreinforced beam. Corresponding improvements in ultimate resistance reached 20.5% and 40.8%, highlighting the efficacy of UHPC in structural strengthening.

- (2) Strain distributions across the T-beam height under bending moment adhered closely to the flat section assumption. Linear strain gradients were consistently observed throughout the loading process, validating the applicability of classical beam theory to UHPC-reinforced systems.
- (3) The proposed predictive model, based on the converted section method, demonstrated conservative yet reliable estimates of flexural capacity. Experimental results exceeded theoretical predictions by up to 9.5%, confirming the model's inherent safety margin. This discrepancy underscores the model's suitability for engineering design applications, ensuring robustness while accounting for material and geometric uncertainties.
- (4) The existing methodology exclusively evaluates alterations in flexural bearing capacity subsequent to the application of UHPC thin-layer reinforcement, neglecting to consider additional factors such as potential degradation that may occur over extended periods of use. Future research efforts are needed to emphasize the investigation of the long-term performance characteristics of steel beams.

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PARAMETER ANALYSIS OF PRESTRESSED STEEL WIRE ROPE STRENGTHENED CONCRETE BEAMS

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ABSTRACT

This paper systematically analyzes the parameters of prestressed steel wire rope strengthened concrete beams using finite element software ANSYS, aiming to explore the influence of different reinforcement parameters on the flexural performance of the beams. Seven 2.4m concrete beam models with different reinforcement methods were established, including comparisons of factors such as the number of prestressed tendons, tensile control stress, and composite mortar protection. Through simulated loading tests, the deflection changes, stress distribution, and force characteristics of steel bars and steel strands were examined. The results indicate that prestressed reinforcement significantly improves the stress state of the beams: an increase in prestress can effectively reduce the tensile stress at the beam bottom and decrease the stress in the main reinforcement, but has no significant impact on structural stiffness. An increase in the cross-sectional area of prestressed tendons significantly enhances the beam's stiffness. Composite mortar protection, although increasing self-weight, reduces prestress loss and optimizes the stress distribution of the steel strands. In contrast, the steel plate bonding method fails to fully utilize its effectiveness under low loads and even increases the stress in the steel bars. Additionally, excessively high prestress values may lead to excessive tensile stress at the beam bottom, necessitating reasonable design in practical engineering. This study provides a theoretical basis and parameter optimization directions for the engineering application of prestressed steel wire rope strengthened concrete beams, verifying the effectiveness of this technology in enhancing bridge load-carrying capacity and extending service life.

KEYWORDS

Prestressed steel wire rope, Concrete beam strengthening, Finite element analysis, Parameter optimization, Flexural performance, Composite mortar

INTRODUCTION

Early-built bridges, designed with lower load standards, are still in service on most highways

and bridges, but they no longer meet the increasing traffic demands. Therefore, the task of reinforcing and repairing dilapidated bridges is heavy. The idea of complete reconstruction is neither practical nor scientific [1-3]. Practice has proven that adopting appropriate reinforcement techniques and widening measures can restore and enhance the load-carrying capacity and traffic capacity of old bridges, extending their service life to meet the needs of modern transportation [4].

The reinforcement and repair of old bridges are technically feasible and economically reasonable initiatives, but they also face many practical difficulties, such as difficulty in finding original bridge data, lack of funds, and mature technical support, making bridge reinforcement difficult to carry out [5-9]. Since reinforcing old bridges is like performing surgery on a critically ill patient, it is risky and not highly profitable. Additionally, each bridge's condition varies greatly, making generalization difficult, which discourages capable survey, design, and construction units from getting involved, resulting in a situation where bridge reinforcement and reconstruction are in demand but not widely undertaken [10-12].

There are many methods for bridge reinforcement, such as increasing the section area, bonding steel beams, and bonding composite fibers. However, these methods have significant limitations, often resulting in strain lag effects during the reinforcement design process, failing to fully utilize the properties of the reinforcement materials. Therefore, these methods are collectively referred to as passive reinforcement methods [13-16].

Adopting active reinforcement methods by applying prestress to the structure can effectively utilize the reinforcement materials and greatly improve the reinforcement effect, solving the strain lag effect caused by staged loading in passive reinforcement. Using composite mortar steel strand bonded prestressed reinforcement for bridge components can effectively address the strain lag effect [17-19]. Due to its convenient prestress tensioning, simple anchoring structure, and high structural durability, it has attracted attention in the engineering community. Currently, only experimental studies have been conducted on prestressed steel wire rope strengthened beams. Liu and Wu [20,21] conducted a large-scale bending test on concrete beams reinforced with prestressed steel wire ropes to study the effects of the steel wire rope reinforcement ratio, reinforcement method, and the unbonded length at the interface between the steel wire rope and polymer mortar on the bending resistance of concrete beams. The test results show that the proposed reinforcement technology can significantly improve the load-bearing capacity of reinforced concrete beams.

Although there have been studies on the scale-down test of reinforced beams, there are few studies on the finite element simulation of prestressed wire rope reinforced beams. This paper conducts finite element simulation and parameter analysis of prestressed steel wire rope strengthened beams based on experimental data.

FINITE ELEMENT MODEL ESTABLISHMENT

Element Selection

ANSYS software has comprehensive pre- processing and post-processing functions, as well as powerful graphics and data processing capabilities, making it convenient to simulate the stress conditions of various parts of the bridge and achieve simulation analysis of bridge design calculations. The reinforced concrete beam is modeled using solid modeling, with different elements selected for C30 concrete, composite mortar, HRB235 steel bars, and steel strands. ANSYS provides multiple element options, with SOLID65 (reinforced concrete solid element) selected for the concrete unit in this model, which has three translational degrees of freedom. The steel bar and steel strand units use LINK8 elements, a type of truss element that only bears axial force and not bending moment, similar to the stress conditions of steel bars and steel strands, with nodes also having three translational degrees of freedom. Although SOLID65 could be used for steel bars and steel strands as well, due to the thin thickness of the composite mortar (typically 3-5cm) and its application only

to the beam bottom to bear tensile stress, the SHELL41 element is used for more accurate simulation in this model.

Connection Method of Composite Mortar and Beam Body

In actual reinforcement, composite mortar is applied to the beam bottom after prestress is applied, bonding with the steel wire ropes into an integral whole. When the beam is subjected to load and deformation, the mortar and the beam bottom concrete have the same displacement (U_x , U_y , U_z). Therefore, the CFINTF (coincident node coupling) command is used to couple the degrees of freedom of the concrete beam bottom nodes and the mortar nodes, forcing them to have the same displacement to simulate the connection between the composite mortar and the beam body.

Parameter Selection

This model simulates a 2.4 m reinforced concrete rectangular beam, with basic parameters matching those of the beam in the indoor small-scale beam test. The parameters of concrete and steel bars are shown in Table 1.

Tab.1 - Material Parameters Table

Material Name	Elastic Modulus (MPa)	Poisson's Ratio	Equivalent Weight (kg/m ³)
Main Beam Concrete	3.00e ⁴	0.2	2500
Steel Bars	2.00 e ⁵	0.3	7800
Steel Wire Ropes	1.95 e ⁵	0.3	7800
Composite Mortar	3.00 e ⁵	0.3	2500

The main beam concrete is modeled using SOLID65 elements, while steel bars and steel wire ropes are modeled using LINK8 elements. After meshing the structure, the finite element models are shown in Figures 1 and 2.

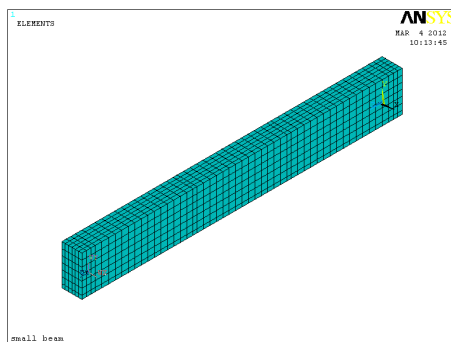


Fig. 1 - Concrete Meshing

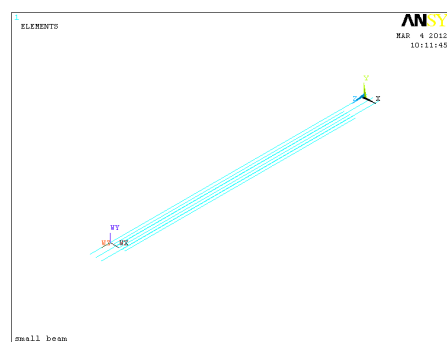


Fig.2 - Steel Bar Meshing

The composite mortar is modeled using SHELL41 elements, located at the beam bottom with a longitudinal range of 0.2 m-2.2 m and a thickness of 3cm, meshed similarly to the concrete. To simulate the bonding between the mortar and the beam bottom, the corresponding nodes of the beam bottom concrete and the composite mortar are coupled in the U_x , U_y , and U_z directions, as shown in Figures 3 and 4.

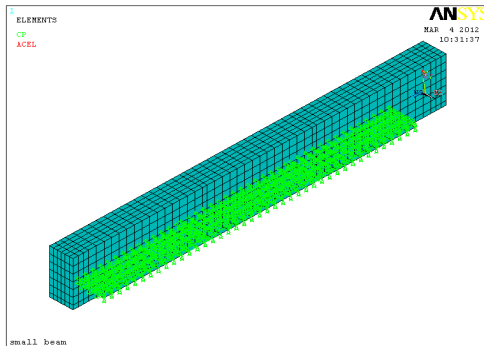


Fig. 3 - Mortar Simulation Reinforcement

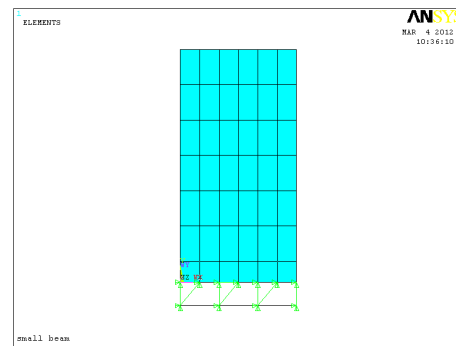


Fig. 4 - Mortar and Beam Bottom Connection Method

To achieve a pure bending state at the mid-span, a two-point loading method is used, applying two surface loads at the beam top. Since the structure is a simply supported beam, the supports are located 100.5 mm from the beam ends, with linear constraints applied.

This model analysis mainly investigates several influencing factors of the bonded prestressed reinforcement system: the cross-sectional area of prestressed steel bars, the contribution of composite mortar to beam stiffness, and the effect of tensile control stress on the reinforcement system. A total of seven models are established, as shown in Table 2.

Tab. 2 - Reinforcement Methods for Each Beam

Beam Number	Reinforcement Method	Prestress Description	Covering Layer
A1	Comparison Beam	Original Beam	No
B1	5 Steel Wire Ropes	Tensile Control Stress 835MPa	Mortar
B2	5 Steel Wire Ropes	Tensile Control Stress 835MPa,	No
C1	8 Steel Wire Ropes	Tensile Control Stress 835MPa	Mortar
C2	8 Steel Wire Ropes	Tensile Control Stress 1002MPa	Mortar

FLEXURAL PERFORMANCE ANALYSIS OF REINFORCED BEAMS

Model Deflection Analysis

The calculated values of this model serve as theoretical values for the small-scale beam reinforcement test in the past. Therefore, the same loading values as the test are applied to each beam at different levels, and the deflections of the beams are analyzed. The deflections of each beam under load are shown in Figures 5 and 6.

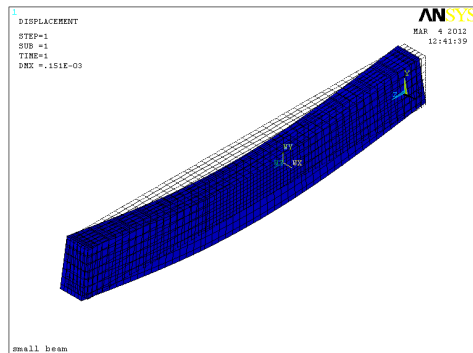


Fig.5 - Beam Deflection Under Load

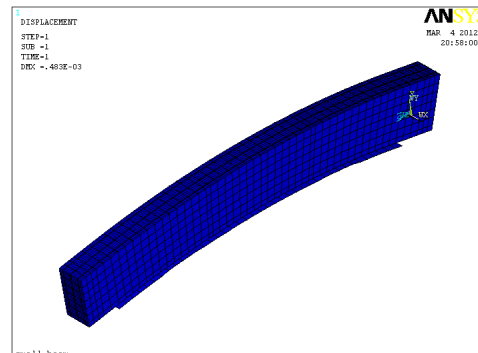


Fig.6 - Beam Camber Under Prestress

Tables 3-6 show the deflection values of each beam at various load levels (mm).

Tab.3 - Deflection Values of Each Beam at Various Load Levels Table 1 (mm)

Beam Number	Loading Value (kN)						
	0	3.3	6.6	9.9	13.2	14	14.8
	Deflection (mm)						
Beam A1	0.000	0.095	0.19	0.285	0.38	0.403	0.426
Beam B1	-0.193	0.077	0.154	0.234	0.311	0.33	0.349
Beam B2	-0.262	0.093	0.186	0.281	0.375	0.397	0.42
Beam C1	-0.320	0.077	0.154	0.23	0.32	0.33	0.35
Beam C2	-0.424	0.077	0.154	0.231	0.307	0.326	0.344
Beam C3	-0.493	0.077	0.18	0.253	0.285	0.293	0.3
Beam D1	0.077	0.152	0.226	0.301	0.376	0.394	0.412

Tab.4 - Deflection Values of Each Beam at Various Load Levels Table 2 (mm)

Beam Number	Loading Value (kN)						
	15.6	16.5	27.5	38.5	49.5	60.5	71.5
	Deflection (mm)						
Beam A1	0.449	0.475	0.792	1.109	1.426	1.743	2.06
Beam B1	0.443	0.468	0.78	1.092	1.404	1.716	2.028
Beam B2	0.443	0.468	0.78	1.092	1.404	1.716	2.028
Beam C1	0.37	0.39	0.642	0.899	1.156	1.413	1.669
Beam C2	0.363	0.38	0.643	0.9	1.156	1.413	1.67
Beam C3	0.308	0.316	0.781	1.082	1.386	1.691	1.996
Beam D1	0.43	0.45	0.699	0.948	1.197	1.445	1.694

Tab.5 - Deflection Values of Each Beam at Various Load Levels Table 3 (mm)

Beam Number	Loading Value (kN)						
	82.5	93.5	106	111.2	116	126	136
	Deflection (mm)						
Beam A1	2.377	2.693	3.054	3.342	/	/	/
Beam B1	1.943	2.202	2.496	2.732	2.967	3.203	3.438
Beam B2	2.34	2.652	3.007	3.29	3.574	3.857	4.141
Beam C1	1.926	2.183	2.475	2.708	2.942	3.175	3.408
Beam C2	1.926	2.183	2.475	2.708	2.942	3.175	3.409
Beam C3	2.301	2.607	2.954	3.232	3.363	3.51	3.788
Beam D1	1.943	2.192	2.474	2.7	2.809	/	/

Tab.6 - Deflection Values of Each Beam at Various Load Levels Table 4 (mm)

Beam Number	Loading Value (kN)						
	146	150	160	165.2	180	186	196.2
	Deflection (mm)						
Beam A1	/	/	/	/	/	/	/
Beam B1	3.758	/	/	/	/	/	/
Beam B2	/	/	/	/	/	/	/
Beam C1	3.502	3.735	3.857	/	/	/	/
Beam C2	3.502	3.735	3.857	4.202	4.342		
Beam C3	4.066	4.177	4.455	4.6	5.011	5.178	5.461
Beam D1	/	/	/	/	/	/	/

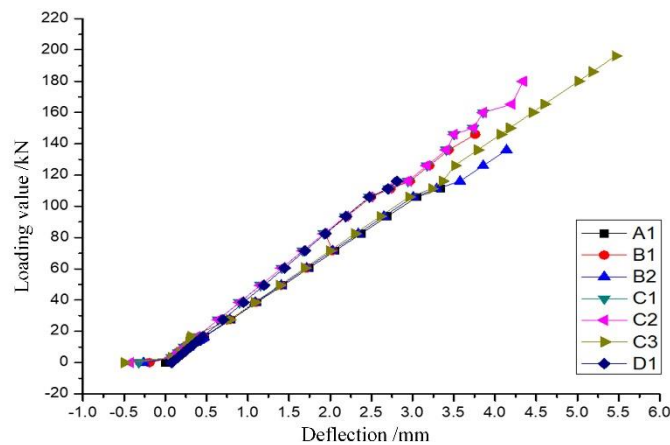


Fig.7 - Beam Deflection Values at Various Load Levels

Beam deflection values at various load levels Figure 7. From the deflection data, the following conclusions can be drawn:

- (1) The five beams (B, C) reinforced with prestressed steel wire ropes exhibit camber values (deflections under zero load) of 0.193 mm, 0.262 mm, 0.32 mm, 0.424 mm, and 0.493 mm, respectively, with the camber value proportional to the applied prestress value. The camber value of

beam B2 without mortar is 54% higher than that of beam B1, indicating that the application of mortar significantly affects the initial camber value of the reinforced beam.

(2) The load-deflection curves of all beams initially grow linearly and then decline in the later stage. Before cracking, the concrete beams behave elastically; with the growth of cracks, the beams enter the plastic stage.

(3) By comparing the C1~C3 curves, it is found that under each load level, the deflections produced by the beams are almost identical, indicating that the influence of the bonded prestressed reinforcement system on beam stiffness is related to the area of the steel strands and independent of the initial tensile control stress. The tensile force only changes the internal stress state of the concrete beam, reducing the tensile stress of the steel bars and the tensile stress at the bottom edge of the concrete; it has no significant impact on the stiffness of the structure. The steel wire ropes act as ordinary steel bars.

(4) According to the D1 curve, the steel plate bonding method increases the structural stiffness by 17.9%, while the bonded prestressed reinforcement method increases it by 19.8%-72.9%, showing a significant increase in stiffness.

Structural Stress Analysis

(1) Stress Analysis Under 6.6 kN Load

The strain cloud diagrams of the original and reinforced structures are shown in Figures 8-11.

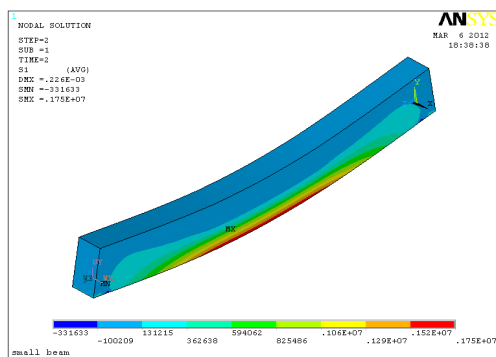


Fig.8 - Original Structure Concrete Strain Contour Diagram

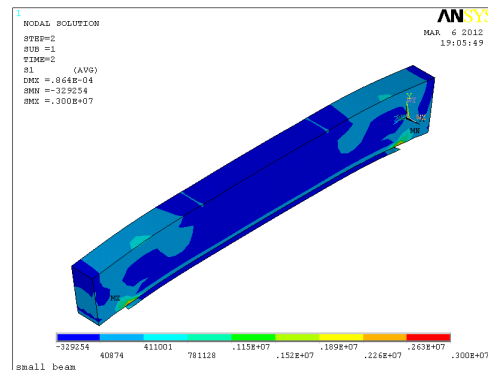


Fig.9 - B1 Concrete Strain Contour Diagram

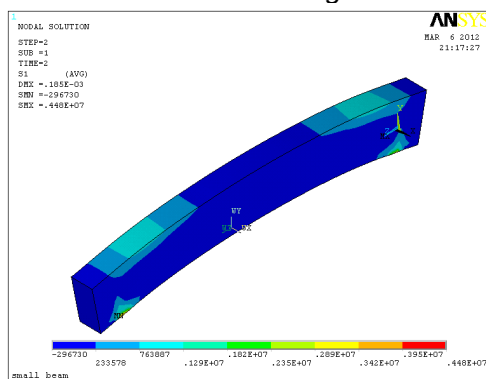


Fig. 10 - C1 Concrete Strain Contour Diagram

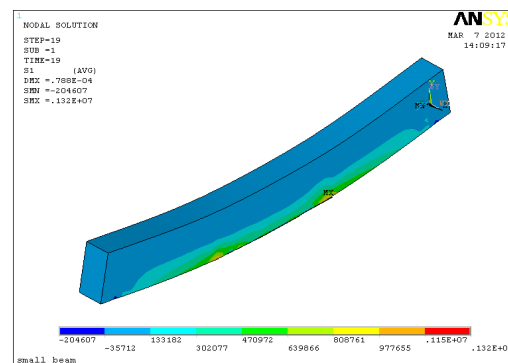


Fig. 11 - D1 Concrete Strain Contour Diagram

Under the action of the original structure and the reinforced structure, the main beam deflects

downward under load, generating corresponding tensile stress within the mid-span range of the beam bottom. The reinforced beam body exhibits smaller tensile stress compared to the original beam. In contrast, the actively reinforced beam body generates camber under small loads, with compressive stress at the beam bottom.

1) Concrete Stress Analysis at Mid-span Cross-section

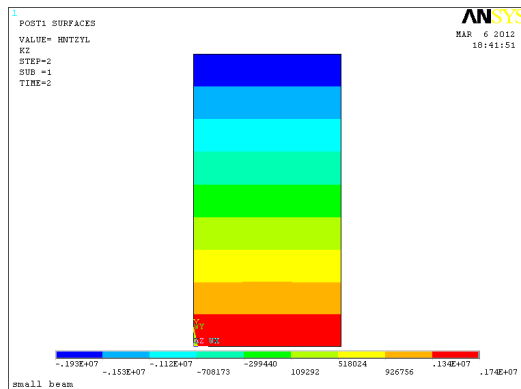


Fig.12 - A1 Mid-span Section Stress Contour Diagram

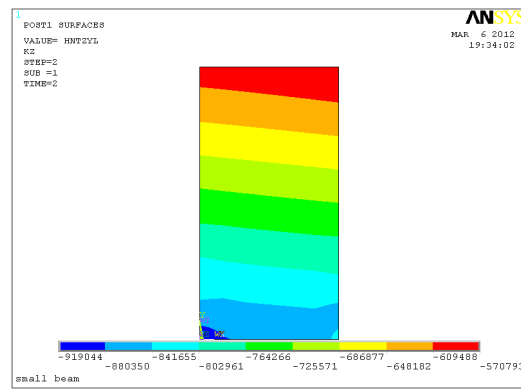


Fig.13 - B1 Mid-span Section Stress Contour Diagram

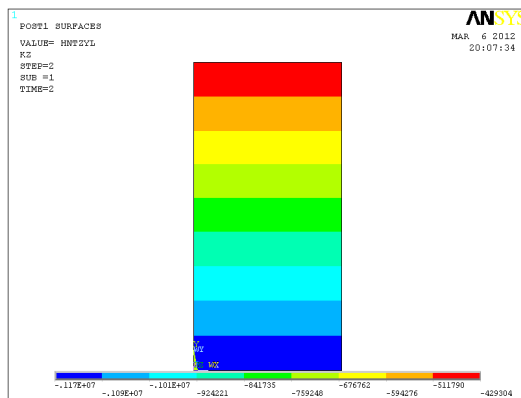


Fig.14 - B2 Mid-span Section Stress Contour Diagram

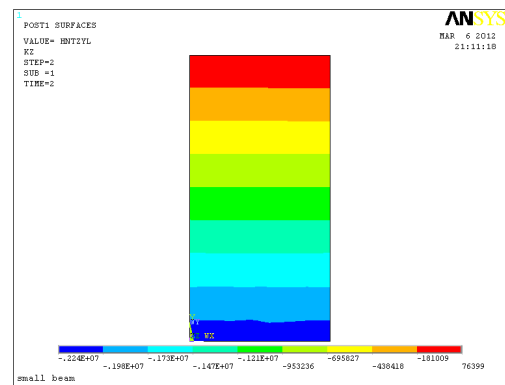


Fig.15 - C1 Mid-span Section Stress Contour Diagram

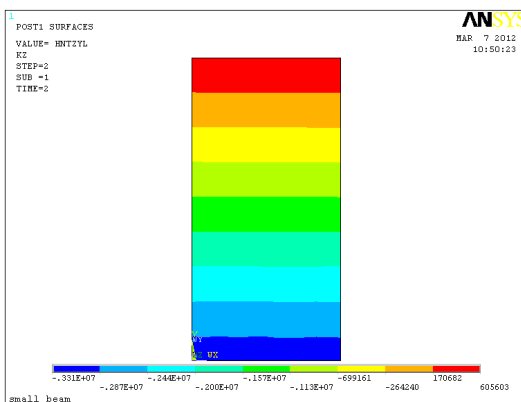


Fig.16 - C2 Mid-span Section Stress Contour Diagram

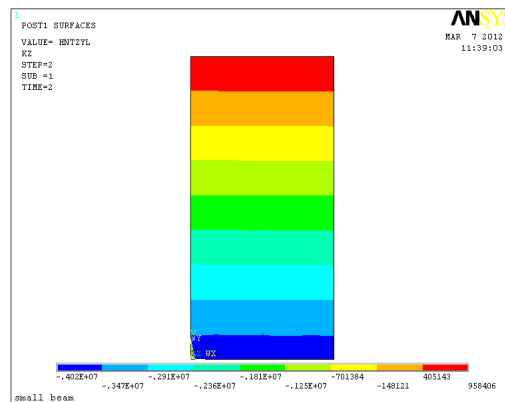


Fig.17 - C3 Mid-span Section Stress Contour Diagram

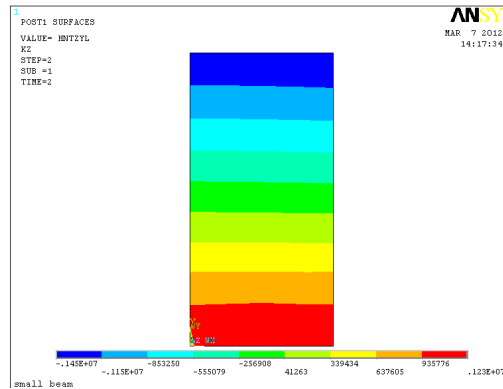


Fig.18 - D1 Mid-span Section Stress Contour Diagram

Tab.7 - Concrete Stress Data Analysis

Specimen Number	Top Edge Concrete Stress (MPa)	Bottom Edge Concrete Stress (MPa)	Top Edge Concrete Stress Ratio	Bottom Edge Concrete Stress Ratio
A1	-3.78	3.41	1	1
B1	-0.57	-0.92	0.15	-0.27
B2	-0.43	-1.17	0.11	-0.34
C1	0.08	-2.24	-0.02	-0.66
C2	0.61	-3.31	-0.16	-0.97
C3	0.96	-4.02	-0.25	-1.17
D1	-1.45	1.23	0.38	0.36

The stress cloud diagrams and data analysis of the mid-span cross-sections of each beam are shown in Figures 12-18 and Table 7. The passively reinforced beam (D1) generates tensile stress of 1.23MPa at the beam bottom, which is 36% of the original beam's stress, and compressive stress of -1.45 MPa at the beam top, which is 38% of the original beam's stress, indicating that the steel plate bonding method can reduce the concrete stress under external load. The actively reinforced beams (B, C) generate compressive stress at the beam bottom and tensile stress at the beam top, which is beneficial for the concrete's stress state and can delay crack development. The magnitude of the compressive stress at the beam bottom is proportional to the applied prestress value, increasing from -0.92 MPa to -4.02 MPa, showing better reinforcement effects compared to the steel plate bonding method.

2) Steel Bar Stress Analysis

Under external load, the main reinforcement bars within the structure bear tensile stress. When the tensile stress reaches the yield strength of the steel bars, the structure fails. Therefore, when studying the reinforcement system, reducing the tensile stress of the main reinforcement bars is a key condition for evaluating the load-carrying capacity improvement of the reinforcement system.

Tab.8 - Stress Data Analysis of Steel Bars and Steel Strands

Specimen Number	Midspan Steel Bar Stress (MPa)	Midspan Steel Wire Ropes Stress (MPa)	Mid-span Steel Bar Stress Ratio	Mid-span Steel Wire Ropes Stress Ratio
A1	9.42	/	1	/
B1	-9.32	-829	-0.99	/
B2	-17	827	-1.81	/
C1	-18.5	821	-1.96	/
C2	-24.6	1060	-2.61	/
C3	-28.8	1230	-3.06	/
D1	12.2	/	1.3	/

The stress data analysis of steel bars and steel wire ropes is shown in Table 8. The tensile stress of the original beam is 9.42 MPa. The actively reinforced method can significantly reduce the tensile stress of the main reinforcement bars, with an average tensile stress of -14.1 MPa for beam B, a reduction of 23.52 MPa, and -24 MPa for beam C, a reduction of 33.42 MPa. The reduction in tensile stress is proportional to the applied prestress value. In contrast, the passively reinforced beam D1 has a tensile stress of 12.2 MPa for the main reinforcement bars under a 6.6 kN load, 130% of the original beam's stress, indicating that under this load, the steel plate bonding method does not exert its reinforcement effect but increases the structure's self-weight, imposing additional load on the steel bars.

3) Stress Analysis of Steel Wire Ropes

The steel wire ropes, applied at the beam bottom, bear the bending moment induced by external loads together with the reinforced concrete. The steel wire ropes only bear tensile stress, and the stress values are shown in Figures 19-23. The tensile stress of the steel wire ropes increases under external load, and the stress is higher at the mid-span than at the beam ends, indicating that the deformation of the steel wire ropes is related to the position of the applied external load (and thus the structural deformation).

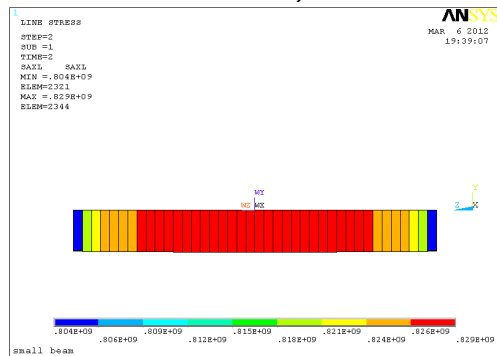


Fig.19 - B1 Steel Strand Strain Contour Diagram

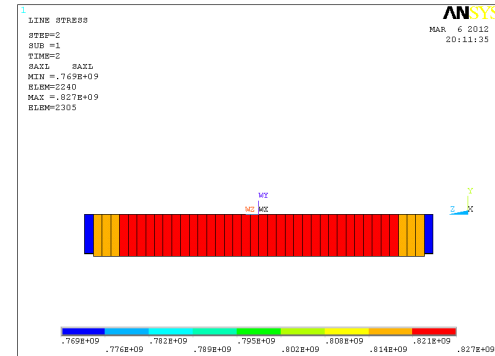


Fig.20 - B2 Steel Strand Strain Contour Diagram

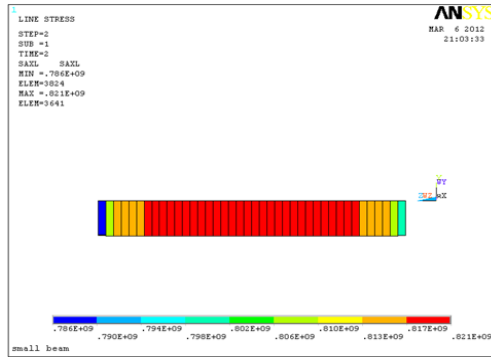


Fig.21 - C1 Steel Strand Strain Contour Diagram

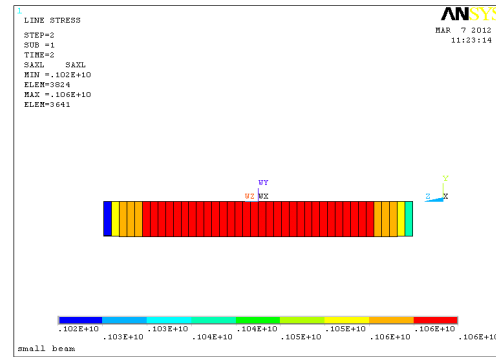


Fig.22 - C2 Steel Strand Strain Contour Diagram

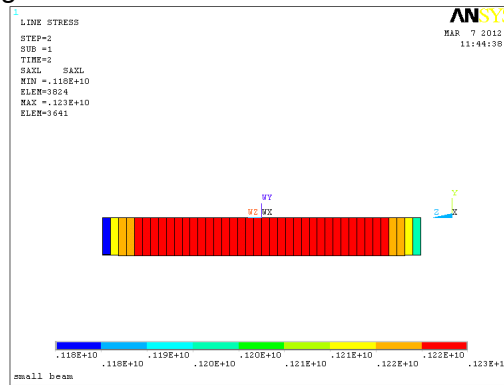


Fig.23 - C3 Steel Strand Strain Contour Diagram

(2) Stress Analysis Under 14 kN Load

A concentrated load of 14 kN is applied to each of the seven beams, generating a pure bending state at the mid-span. By analyzing the stress changes in the mid-span cross-section concrete, steel bars, and steel wire ropes, the stress state of the beams is obtained.

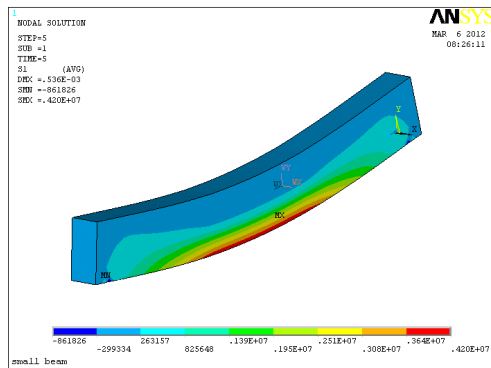


Fig.24 - Original Structure Concrete Strain Contour Diagram

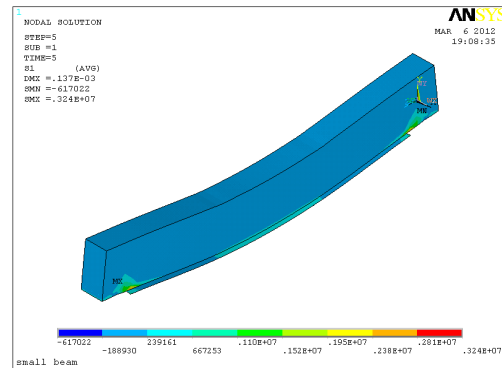


Fig.25 - B1 Concrete Strain Contour Diagram

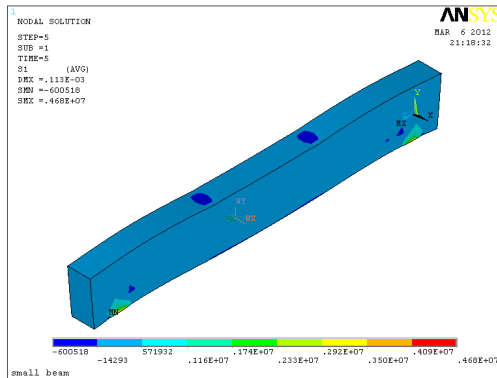


Fig.26 - C1 Concrete Strain Contour Diagram

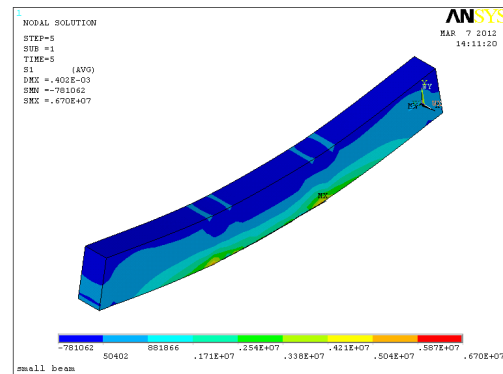


Fig.27 - D1 Concrete Strain Contour Diagram

Strain cloud diagrams of original and reinforced structures are shown in Figures 24-27. Under the action of the load, the original beam and the steel plate-reinforced beam deflect downward, generating corresponding tensile stress within the mid-span range of the beam bottom. The reinforced beam body exhibits smaller tensile stress compared to the original beam. In contrast, the actively reinforced beam body generates tensile stress at the beam bottom when the load exceeds the stress-relieving load, while the beam body does not exceed the stress-relieving load, generating compressive stress at the beam bottom.

1) Concrete Stress Analysis at Mid-span Cross-section

Tab. 9 - Concrete Stress Data Analysis

Specimen Number	Mid-span Steel Bar Stress (MPa)	Mid-span Steel Strand Stress (MPa)	Mid-span Steel Bar Stress Ratio	Mid-span Steel Wire Ropes Stress Ratio
A1	-4.61	4.17	1	1
B1	-2.2	0.35	0.48	0.08
B2	-2.26	0.46	0.49	0.11
C1	-1.54	-0.90	0.33	-0.21
C2	-1.01	-2.04	0.22	-0.49
C3	-0.66	-2.76	0.14	-0.66
D1	-2.51	2.14	0.54	0.36

The concrete stress data analysis is shown in Table 9. The stress ratio is the ratio of the stress of each beam to that of beam A1. The passively reinforced beam (D1) generates tensile stress of 2.14 MPa at the beam bottom under external load, 56.6% of the original beam's stress, and compressive stress of -2.51 MPa at the beam top, 66.4% of the original beam's stress, indicating that the steel plate bonding method can reduce the concrete stress under external load. The actively reinforced beam B generates tensile stress at the beam bottom under load, with an average value of 0.45 MPa, indicating that the concrete of beams B1 and B2 begins to bear tensile stress. The beam body of beam C is under compressive strain at the beam bottom, with an average compressive stress of -1.9 MPa, indicating that the stress state of the concrete in the bonded reinforcement system is related to the prestress loading value. The higher the prestress, the less likely the beam bottom is to generate tensile stress.

2) Steel Bar Stress Analysis

Steel bar strain is shown in Figures 28-34.

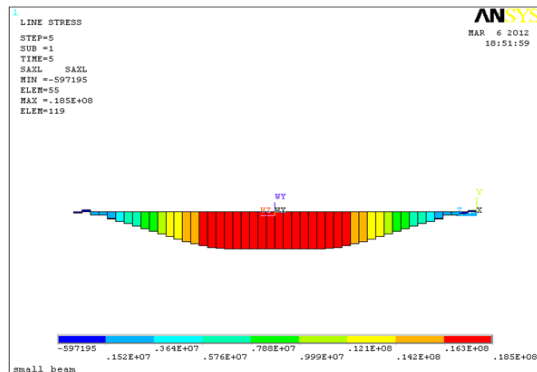


Fig.28 - Original Structure Steel Bar Strain Contour Diagram

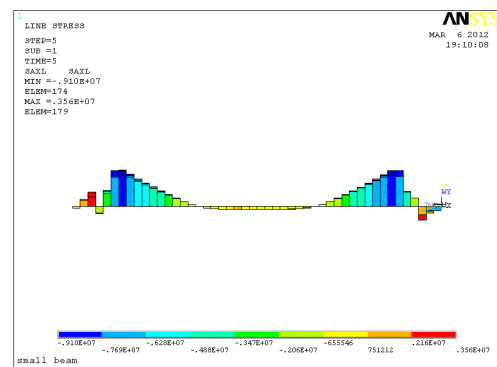


Fig.29 - B1 Steel Bar Strain Contour Diagram

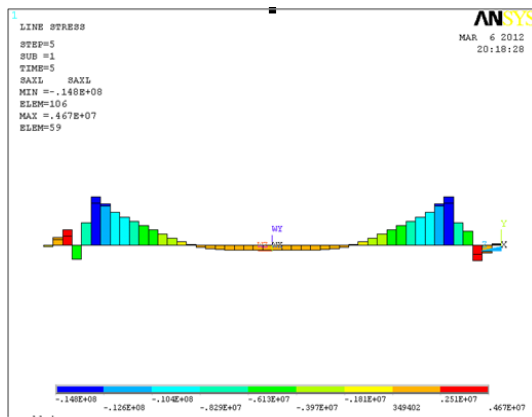


Fig.30 - B2 Steel Bar Strain Contour Diagram

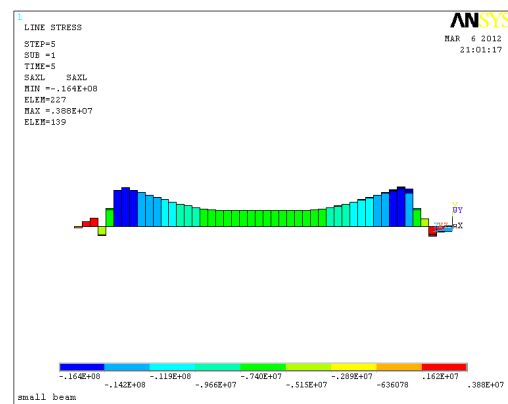


Fig.31 - C1 Steel Bar Strain Contour Diagram

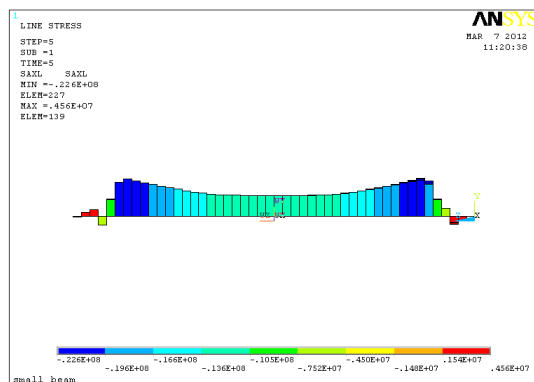


Fig.32 - C2 Steel Bar Strain Contour Diagram

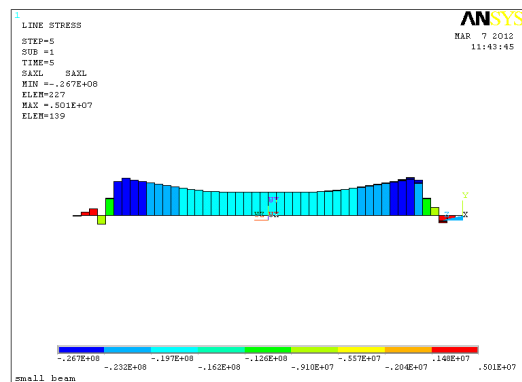


Fig.33 - C3 Steel Bar Strain Contour Diagram

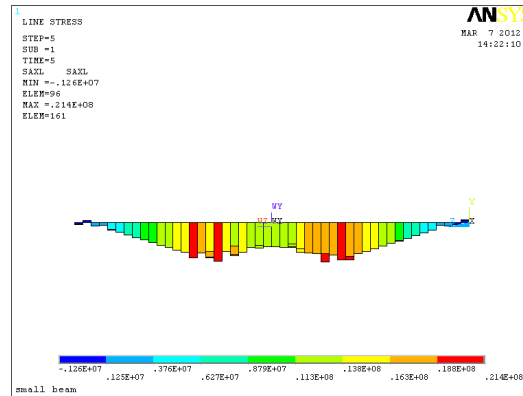


Fig.34 - D1 Steel Bar Strain Contour Diagram

Tab.10 - Stress Data Analysis of Steel Bars and Steel Wire Ropes

Specimen Number	Mid-span Steel Bar Stress (MPa)	Mid-span Steel Wire Ropes Stress (MPa)	Mid-span Steel Bar Stress Ratio	Mid-span Steel Strand Stress Ratio
A1	18.5	/	1	/
B1	-9.32	-837	-0.5	/
B2	-14.8	838	-0.8	/
C1	-16.4	829	-0.89	/
C2	-22.6	1070	-1.22	/
C3	-26.7	1240	-1.44	/
D1	21.4	/	1.16	/

The stress data analysis of steel bars and steel wire ropes is shown in Table 10. The tensile stress of the original beam is 18.5 MPa. The actively reinforced method can significantly reduce the tensile stress of the main reinforcement bars, with an average tensile stress of -12.1 MPa for beam B, a reduction of 30.6 MPa, and -21.9 MPa for beam C, a reduction of 40.4 MPa. The reduction in tensile stress is proportional to the applied prestress value. In contrast, the passively reinforced beam D1 has a tensile stress of 21.4 MPa for the main reinforcement bars, higher than the original beam, indicating that under a 14 kN load, the steel plate bonding method does not exert its reinforcement effect but increases the structure's self-weight, imposing additional load on the steel bars.

3) Stress Analysis of Steel Strands

Strain cloud diagrams of steel wire ropes are shown in Figures 35-39.

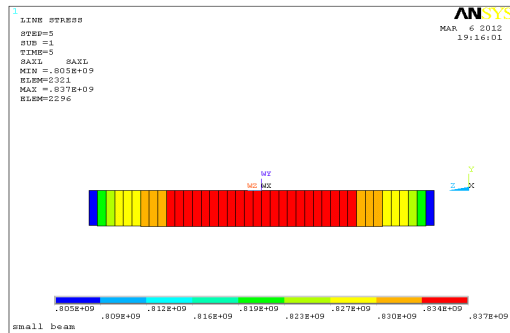


Fig.35 - B1 Steel Strand Strain Contour Diagram

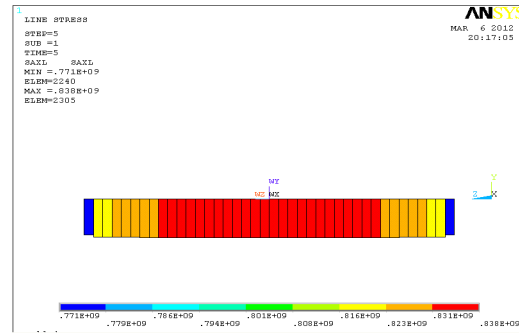


Fig.36 - B2 Steel Strand Strain Contour Diagram

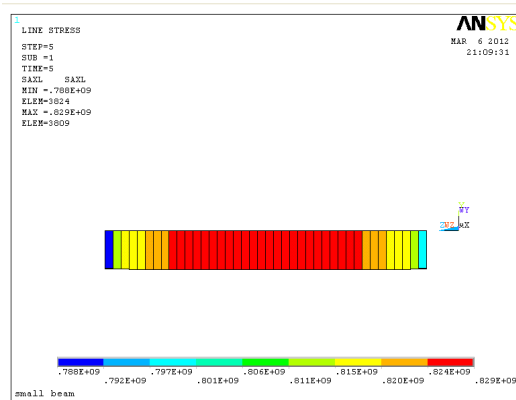


Fig.37 - C1 Steel Wire Ropes Strain Contour Diagram

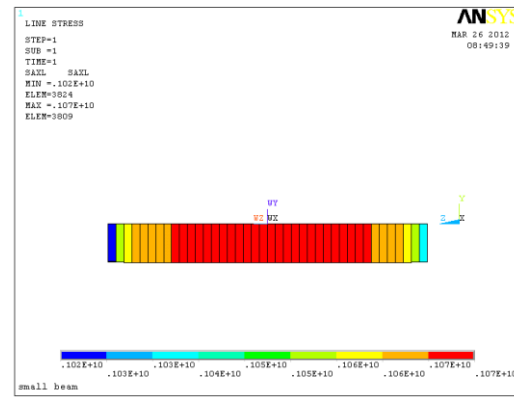


Fig.38 - C2 Steel Wire Ropes Strain Contour Diagram

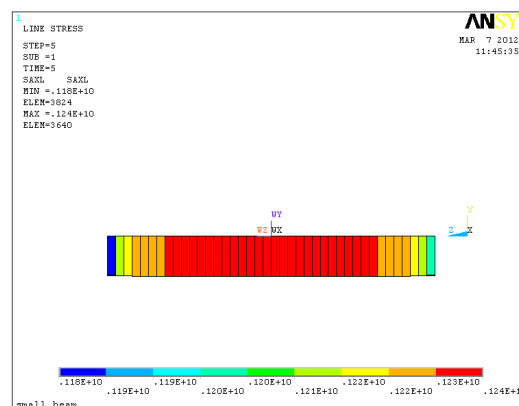


Fig.39 - C3 Steel Wire Ropes Strain Contour Diagram

Strain cloud diagrams of steel wire ropes are shown in Figures 35-39. Under external load, the tensile stress of the steel strands increases. The stress is higher at the mid-span than at the beam ends, indicating that the deformation of the steel wire ropes is related to the position of the applied external load (and thus the structural deformation).

CONSTRUCTION AND ECONOMIC ANALYSIS

1) Construction convenience analysis

The construction process is relatively simple: The construction process of prestressed steel wire rope reinforcement for Bridges is not complicated. It mainly includes the production of end anchors, the fixation of end anchor seats, the cutting of prestressed steel wire ropes and the production of extruded anchor heads, as well as the tensioning and anchoring of prestressed steel wire ropes. Compared with some traditional reinforcement methods such as the section enlargement reinforcement method, it does not require a large amount of concrete pouring and formwork erection work, and the construction process is relatively simplified.

The equipment requirements are not high: When tensioning prestressed steel wire ropes, since the tensile force of a single steel wire rope is relatively small, there is no need for large-scale tensioning equipment. Ordinary small tensioning tools can meet the requirements. This makes the preparation and operation of construction equipment more convenient, reducing the investment and usage difficulty of construction equipment.

Low requirements for on-site environment: This reinforcement method can achieve no wet operation or local wet operation, and has relatively low requirements for the environmental conditions of the construction site. In some bridge reinforcement scenarios with limited space or complex environments, such as Bridges in urban centers or those with small space under the bridge, prestressed steel wire rope reinforcement is easier to implement and can adapt to different construction site conditions.

Fast construction speed: The external prestressed reinforcement technology requires simple equipment, less human input, and a short construction period. Prestressed steel wire rope reinforcement is a type of external prestressed reinforcement. Its construction process is relatively efficient and can complete the bridge reinforcement work in a short time, reducing the impact on traffic.

2) Economy analysis

Low material cost: Prestressed steel wire ropes typically use high-strength, low-relaxation, and high-ductility steel wire ropes, and their material prices are relatively lower than those of some high-performance fiber composite materials and other reinforcing materials. Moreover, the diameter of steel wire ropes is generally small, and the amount of materials required is also relatively less, further reducing material costs.

Low construction cost: Due to the simple construction process, low equipment requirements, less labor input and short construction period, the construction cost of prestressed steel wire rope reinforced bridges is relatively low. Compared with methods such as reinforcing by pasting steel plates, it does not require a large amount of structural adhesives and also reduces the rework costs caused by problems such as bonding damage.

Low maintenance cost: Prestressed steel wire rope reinforcement basically does not increase the self-weight of the structure, does not reduce the usable space under the bridge, and the surface of the steel wire rope can be protected by applying protective mortar and other methods, which has good durability and is less likely to cause corrosion and other problems, thereby reducing the maintenance cost of the bridge in the subsequent use process.

High comprehensive economic benefits: Although prestressed steel wire rope reinforcement of bridges may require certain investment in the initial stage, in the long run, its relatively low construction cost, maintenance cost, and the ability to effectively enhance the load-bearing capacity and service life of Bridges make its comprehensive economic benefits quite significant, providing an economical and effective solution for the reinforcement and maintenance of bridges.

CONCLUSIONS

By establishing seven ANSYS small-scale beam models and analyzing the deformation and stress changes of each beam under different loads, the following conclusions are drawn:

- (1) Through structural stress analysis, it is known that the prestress value applied by the prestressed wire ropes (tested at 50%-70% of the tensile control stress) can effectively improve the stress distribution of the structure, generating compressive stress at the beam bottom interface and tensile stress at the beam top interface, reducing the tensile stress of the main steel bars and increasing the maximum load-carrying capacity of the structure. However, the prestress value does not affect the stiffness of the structure.
- (2) The cross-sectional area of the prestressed steel wire ropes can significantly change the structural stiffness. That is, under the same prestress level, the more prestressed steel wire ropes (lower tensile control stress), the greater the increase in structural stiffness, but the load-carrying capacity remains the same. The bonded prestressed reinforcement method increases it by 19.8%-72.9%, showing a significant increase in stiffness.
- (3) When the prestressed steel wire ropes do not reach the ultimate tensile strength under ultimate conditions, the prestress level directly affects the load-carrying capacity of the structure. The higher the prestress level, the more significant the improvement in load-carrying capacity. Excessive prestress will cause excessive camber of the beam. Therefore, it is not advisable to adopt excessively high prestress values in actual reinforcement.
- (4) The steel plate bonding method can increase the stiffness of the structure, but under small loads, the structure bears additional loads, and the steel bar stress values are higher than before reinforcement under the same load conditions. The reinforcement material does not exert its intended effect, only when loaded to a certain value does the structural stiffness improve to some extent.

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