

Article

An Investigation of Cracks in PK-Section Concrete Beams at Early Ages

Zepeng Zhang ¹, Jia Wang ¹, Hongsheng Li ¹, Xuefei Shi ^{2,*}  and Bin Huang ^{2,*} 

¹ Guangdong Transportation Industrial Investment Co., Ltd., Guangzhou 510623, China; hongshengli812@126.com (H.L.)

² Department of Bridge Engineering, Tongji University, Shanghai 200092, China

* Correspondence: harperhuang177@gmail.com

Abstract

Early age cracking induced by cement hydration heat in a 37.6 m-wide PK-section concrete box girder was investigated through full-scale field testing and numerical simulation. Material properties, temperature, and strain were measured, and the obtained thermal and mechanical parameters were used to simulate temperature and stress distributions during cantilever casting. Results show that direct casting on the foundation cap led to extensive vertical cracking in diaphragms, where tensile stresses exceeded concrete strength, corresponding to a cracking index of approximately 1.8, with thermal-to-shrinkage stress ratios up to 3:1 in critical regions. Under cantilever construction conditions, significant transverse stress occurred only at the diaphragm bottom, reaching a cracking index of about 1.6, with a thermal-to-shrinkage ratio of 2:1. Reducing casting temperature lowered thermal stress by 0.1 MPa/°C, while adding 0.9 kg/m³ polypropylene fibers increased early-age tensile strength by 15%. Optimized mix design or the inclusion of mineral admixtures such as silica fume further reduced shrinkage. The combined application of these measures effectively mitigated early-age cracking risk, providing practical guidance for the construction of wide-box girders in subtropical climates.

Keywords: early-age cracking; concrete box girder; hydration heat; thermal stress; mitigation measures

1. Introduction

In the early age stage of concrete, the heat of hydration induces nonuniform temperature rises and drops within the structure. Meanwhile, shrinkage effects are more pronounced during this stage compared with the service phase. These two mechanisms—thermal deformation and shrinkage—lead to stress development under restraint, which can result in cracking once the stress exceeds the material tensile capacity, thus posing a threat to structural safety [1]. Traditional studies have mainly focused on massive concrete components with regular geometries, such as pile caps, piers, and anchorage blocks. In these structures, the internal temperature caused by hydration is relatively high, producing significant thermal stresses and consequent cracking [2]. However, box girder structures are geometrically complex, with nonuniform stiffness distribution, making it more difficult to predict and evaluate early-age cracking behavior under coupled thermal and shrinkage effects [3,4].

Accurate analysis of the concrete hydration temperature field is paramount for assessing early-age behavior, as thermal evolution directly governs stress development [5].



Academic Editor: Cedric Payan

Received: 15 December 2025

Revised: 9 January 2026

Accepted: 19 January 2026

Published: 22 January 2026

Copyright: © 2026 by the authors.

Licensee MDPI, Basel, Switzerland.

This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\)](https://creativecommons.org/licenses/by/4.0/) license.

The thermal behavior of concrete box girders is considerably more complex than that of traditional mass concrete structures; dimensional disparities among various plates create significant temperature gradients, and the inter-component heat transfer effects are non-negligible. Existing literature has largely concentrated on box girders with regular geometries, resulting in findings that may not adequately capture the behavior of more complex structures. Furthermore, although computational frameworks are mature, predictive accuracy is frequently hindered by uncertainties in thermal parameters. Calibrating these parameters against experimental data serves as an effective mitigation strategy, with full-scale models providing superior fidelity regarding thermal boundaries and heat transfer conditions compared to component-level models. Do et al. analyzed diaphragm temperature fields, identifying mass-concrete behavior in high-strength concrete (>45 MPa). However, their use of a localized FE model ignored the global heat transfer response of the box girder, weakening the simulation accuracy. Additionally, their work relied on adiabatic temperature rise tests without verifying actual structural thermal boundaries or conductivity [6]. Wu et al. performed measurements and simulations on a massive 0# segment using a full-scale model, validating the numerical results against field data. Nevertheless, the specific thermal boundary conditions were not elaborated, and the focus on a standard single-box, double-cell section limits the generalizability of their conclusions [7]. To resolve parameter uncertainty, Yang et al. introduced a BO-XGBoost model, establishing a non-linear mapping between peak temperatures and thermal properties. While effective for simple single-cell box girders, the model's robustness for complex geometries requires further validation [8]. PK-section concrete box girders, characterized by the intersection of mass concrete zones and multiple variable-thickness plates, differ distinctively from traditional designs. Currently, there is a paucity of targeted research on the hydration temperature field of such structures. Therefore, a combined approach utilizing full-scale experimentation and numerical simulation constitutes the most rigorous method to characterize the thermal behavior of PK-section girders.

The computation of early-age thermal stress is considerably more intricate than that for the service stage, being intrinsically linked to the evolution of the elastic modulus and creep effects. Since the early-age elastic modulus is time-dependent, determining its development curve is critical. Traditional studies often simplify the modulus as a function of time alone; however, this approach neglects the influence of the degree of cement hydration, leading to significant computational errors. Furthermore, early-age concrete exhibits pronounced creep characteristics, providing a stress relaxation effect that can exceed 50% [9,10]. Nevertheless, many studies either ignore creep or indiscriminately apply service-stage models without experimental calibration or consideration of hydration, resulting in substantial discrepancies between calculated and actual stresses [11]. Previous research has attempted to address these complexities with varying limitations. Choi et al. [12] confirmed that early-age thermal stress behavior differs substantially from the service stage and explored the relationship between hydration and material properties; however, their work was confined to bridge decks and did not address mass concrete structures. Do et al. [13] developed an Ansys subroutine to investigate the effect of casting temperature on box girder thermal stresses, yet their study failed to account for equivalent age or the time-varying nature of material properties. Han et al. [14] incorporated the equivalent age method into simulations, using empirical formulas for the evolution of modulus, strength, and creep. However, their model lacked calibration against experimental material data and validation with measured strains. Moreover, as their subjects were precast segmental girders with relatively simple boundary conditions, the observed stress evolution may differ from that of cantilever-cast box girders.

Concrete shrinkage has long been recognized as one of the primary causes of structural cracking [15]. Prior to setting, plastic shrinkage dominates while the concrete remains in a semi-fluid state with negligible strength. From 1 to 14 days after setting, autogenous shrinkage becomes the governing factor, whereas drying shrinkage takes precedence beyond 28 days [16]. Most existing studies rely on standard code-based shrinkage models; however, these models generally prioritize long-term accuracy for the service phase and fail to precisely capture early-age behavior [17]. In engineering practice, early-age shrinkage often exerts a more critical influence on cracking risk [18]. To accurately simulate these effects and minimize calculation errors, it is essential to integrate early-age shrinkage tests with appropriate mathematical models. Furthermore, the concept of equivalent age should be incorporated into shrinkage predictions. Since both shrinkage and thermal stresses originate from restrained free deformation, early-age creep provides a significant stress relaxation effect that must be accounted for in rigorous calculations. Neglecting these coupled effects, as is common in existing literature [14], may lead to considerable discrepancies in stress evaluation.

In this study, a full-scale experimental investigation was conducted on a 37.6 m-wide PK-section concrete box girder. The early-age temperature and strain developments were measured to validate computational approaches for temperature and stress prediction. The effects of thermal and shrinkage stresses on diaphragm cracking were analyzed, along with the influence of restraint conditions on stress evolution. The experimentally obtained parameters were then applied to a simulation of the actual cantilever construction process to assess cracking risk under realistic conditions, and corresponding optimization measures were proposed.

2. Experiment

2.1. PK-Section Concrete Box Girder Test

In this study, the engineering background of this study is a four-tower cable-stayed bridge, with both towers and main girders constructed from concrete, featuring a maximum span of 360 m, located in Qingyuan, Guangdong Province, China. The main girder is a PK-section concrete box girder with a width of 37.6 m, constructed using balanced cantilever segmental construction with form travelers. A full-scale 1:1 test model was established for the #4 girder segment near the tower. The model is 3 m in longitudinal length, with a top slab thickness of 0.3 m, bottom slab thickness of 0.9 m, inclined bottom slab thickness of 0.45 m, and a transverse diaphragm positioned at the longitudinal center with a thickness of 0.39 m. The cross-sectional configuration of the test model is shown in Figure 1.

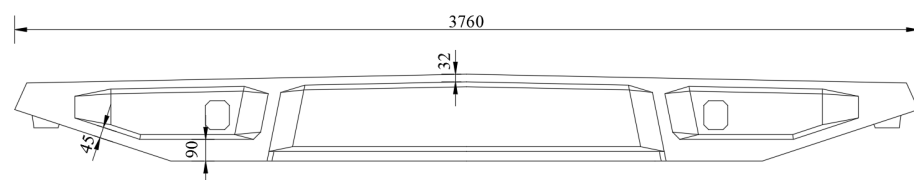


Figure 1. Cross-sectional view of the test model (Unit: cm).

The main girder was made of C55 concrete reinforced with steel bars. As the test site was located in an area with low subgrade bearing capacity, pine piles were employed to strengthen the foundation. A 0.2 m thick C20 concrete leveling layer was placed over the piles, followed by a 0.3 m thick C30 concrete cap. The test model was directly cast on the cap. Steel formwork was used instead of timber formwork to provide improved heat dissipation performance [19]. After casting, the concrete was cured with the formwork in place for three days before demolding. The full-scale test model is shown in Figure 2.



Figure 2. On-site photograph of the test beam.

To investigate the early-age temperature and stress fields of the PK-section concrete box girder, six temperature sensors and four strain sensors were installed at the mid-span cross-section of the test model. The strain sensors were arranged to measure the lateral deformation. Data were recorded at intervals of 0.5 h, and the monitoring lasted for 14 days. The temperature sensors used were semiconductor-type sensors (Model JMT-36B, Changsha Jinma, Changsha, China) with an accuracy of ± 0.1 °C, while the strain sensors were embedded vibrating-wire strain gauges (Model JMZX-215HAT, Changsha Jinma, Changsha, China) featuring a measurement accuracy of $\pm 0.1\%$. The experimental scheme is illustrated in Figure 3, while the on-site layout of sensors is presented in Figure 4.

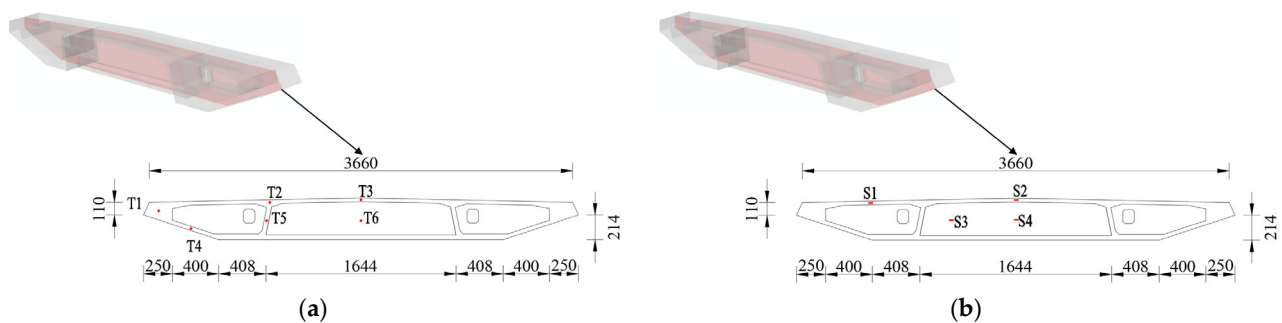


Figure 3. Layout of temperature and strain sensors (Unit: cm): (a) Layout of temperature sensors in the test model; (b) Layout of strain sensors in the test model.

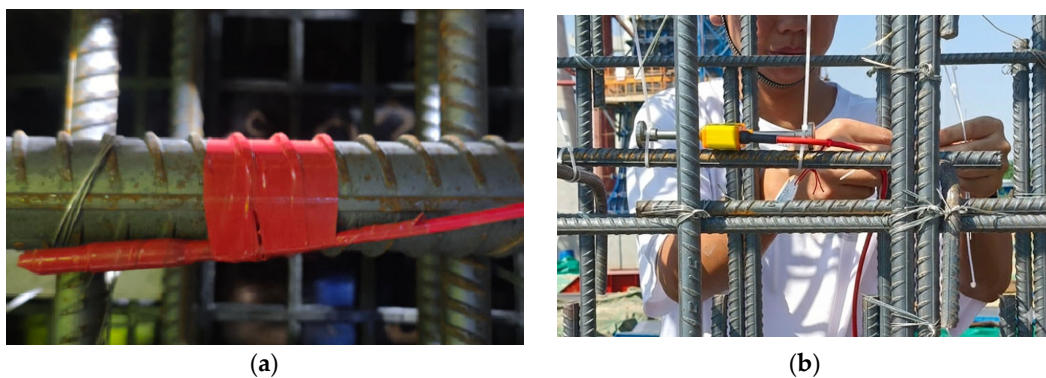


Figure 4. On-site installation of sensors: (a) Temperature sensors; (b) strain sensors.

During the experiment, the ambient air temperature was monitored simultaneously to provide realistic parameters for numerical simulation. The maximum recorded temperature was 40 °C, and the minimum was 25 °C. The variation in ambient temperature is shown

in Figure 5. After formwork removal, vertical cracks were observed on both sides of the transverse diaphragm, with a total of 18 cracks and a maximum length of 160 cm. The crack distribution and on-site photographs are shown in Figures 6 and 7.

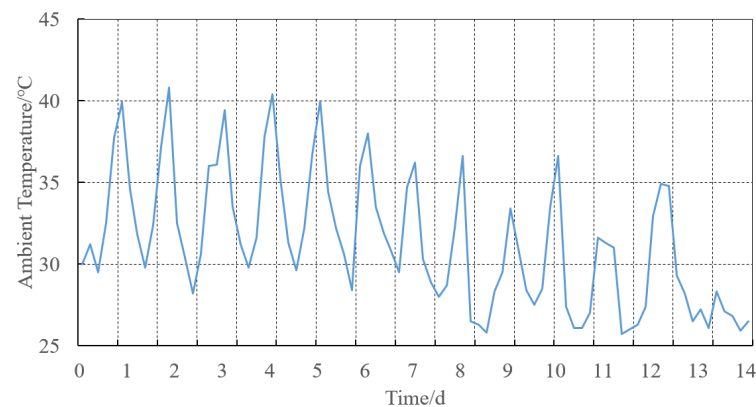


Figure 5. Variation in ambient air temperature during the test.

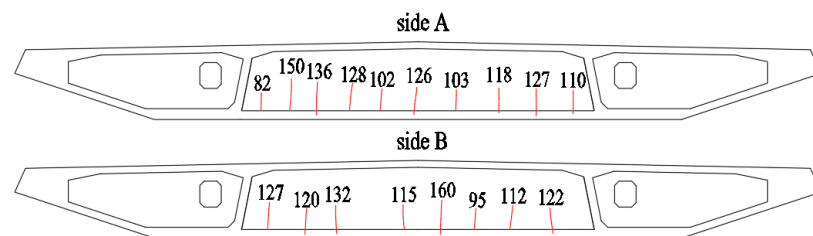


Figure 6. Distribution of vertical cracks on both sides of the transverse diaphragm.

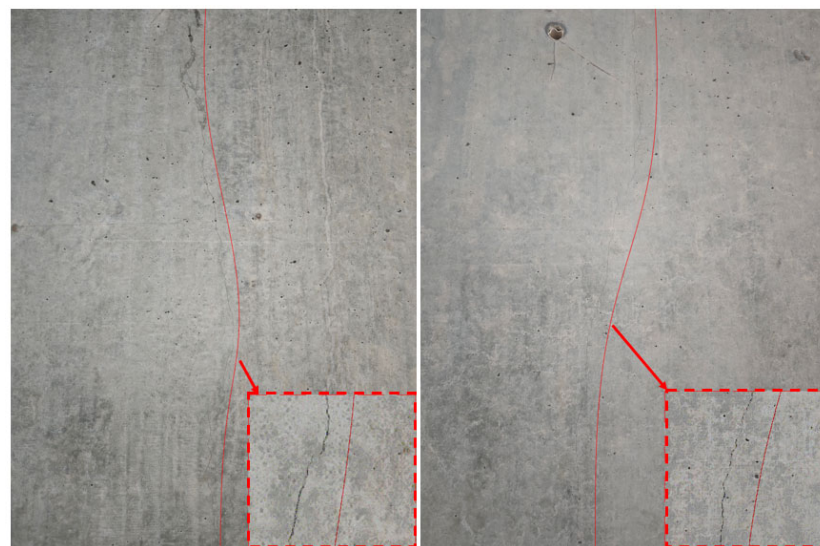


Figure 7. On-site photographs of cracks on the transverse diaphragm.

2.2. Material Tests

The calculation of early-age stress in concrete involves the time-dependent evolution of the elastic modulus, and the assessment of cracking risk is closely related to strength development. To provide accurate parameters for numerical simulation, material tests were conducted on the C55 concrete used in the main girder. The mix proportions of the concrete are presented in Table 1.

Table 1. Concrete mix proportions (kg/m³).

Material	Cement	Fly Ash	Slag	Fine Aggregate	Coarse Aggregate	Admixture	Water
Content	277	105	93	690	1163	4.9	142

Standard specimens of 15 × 15 × 15 cm and 15 × 15 × 30 cm were prepared to determine the early-age development curves of compressive strength, tensile strength, and elastic modulus [20]. The experiments were conducted in accordance with the Chinese standard [21].

Compressive strength tests were conducted at ages of 1, 2, 3, 5, 7, and 28 days, while elastic modulus tests were performed at 1, 2, 3, 5, 7, 14, and 28 days. To account for the inherent dispersion of concrete properties, five replicate specimens were tested for each group. The final result was determined by calculating the arithmetic mean of the three intermediate values, after excluding the maximum and minimum readings. The analysis indicates that the Coefficient of Variation (COV) for the valid data remained consistently below 10%, confirming the reliability of the experimental results.

Since the early-age development of concrete properties is significantly influenced by the degree of hydration, the calculation requires the use of equivalent age to account for differences in hydration. The specimens were cured at a constant temperature of 20 °C, and due to their small size, the concrete temperature remained nearly constant at 20 °C. Under these conditions, the equivalent age coincides with the actual elapsed time, and the measured results can be regarded as representing the relationship between material properties and equivalent age. The FIP Model Code formulates the dependency of material property development on equivalent age. Based on the authors' extensive analysis of substantial in situ data, it has been determined that these models exhibit remarkable consistency with the characteristics of concrete used in bridge engineering—a conclusion further substantiated by the fitting results presented herein. The data were then fitted using the material property models based on equivalent age as defined in the FIP guidelines [22]. The 28-day compressive strength, tensile strength, and elastic modulus of the main girder concrete were 71.5 MPa, 3.8 MPa, and 47.9 GPa, respectively. The development curves of these properties are presented in Figure 8. The fitted curves exhibit good agreement with the experimental data, with the calculated coefficient of determination (R²) exceeding 0.9.

The empirical fitting equations for the material properties, derived from the experimental data, are shown in Equations (1)–(3).

$$\begin{aligned} f_c(t_r) &= \beta_{cc}(t_r)f_{c28} \\ \beta_{cc}(t_r) &= e^{\{0.25 \times [1 - (\frac{28}{t_r})^{0.5}]\}} \\ f_{c28} &= 71.5 \text{ MPa} \end{aligned} \quad (1)$$

$$\begin{aligned} E(t_r) &= \beta_E(t_r)E_{28} \\ \beta_E(t_r) &= [\beta_{cc}(t_r)]^{0.5} \\ E_{28} &= 47.9 \text{ GPa} \end{aligned} \quad (2)$$

$$f_t(t_r) = 0.88 \times 0.395 f_c(t_r)^{0.55} \quad (3)$$

In the equation, t_r denotes the equivalent age (unit: day); $f_c(t_r)$ represents the compressive strength of concrete at the equivalent age t_r (unit: MPa); $\beta_{cc}(t_r)$ is the strength development coefficient of concrete at the equivalent age t_r ; f_{c28} is the compressive strength of concrete corresponding to the equivalent age $t_r = 28$ d (unit: MPa). $E(t_r)$ denotes the elastic modulus of concrete at the equivalent age t_r (unit: MPa); $\beta_E(t_r)$ is the elastic modulus development coefficient of concrete at the equivalent age t_r ; E_{28} represents the elastic

modulus of concrete at the equivalent age $t_r = 28$ d (unit: MPa). $f_t(t_r)$ is the tensile strength of concrete at the equivalent age t_r (unit: MPa).

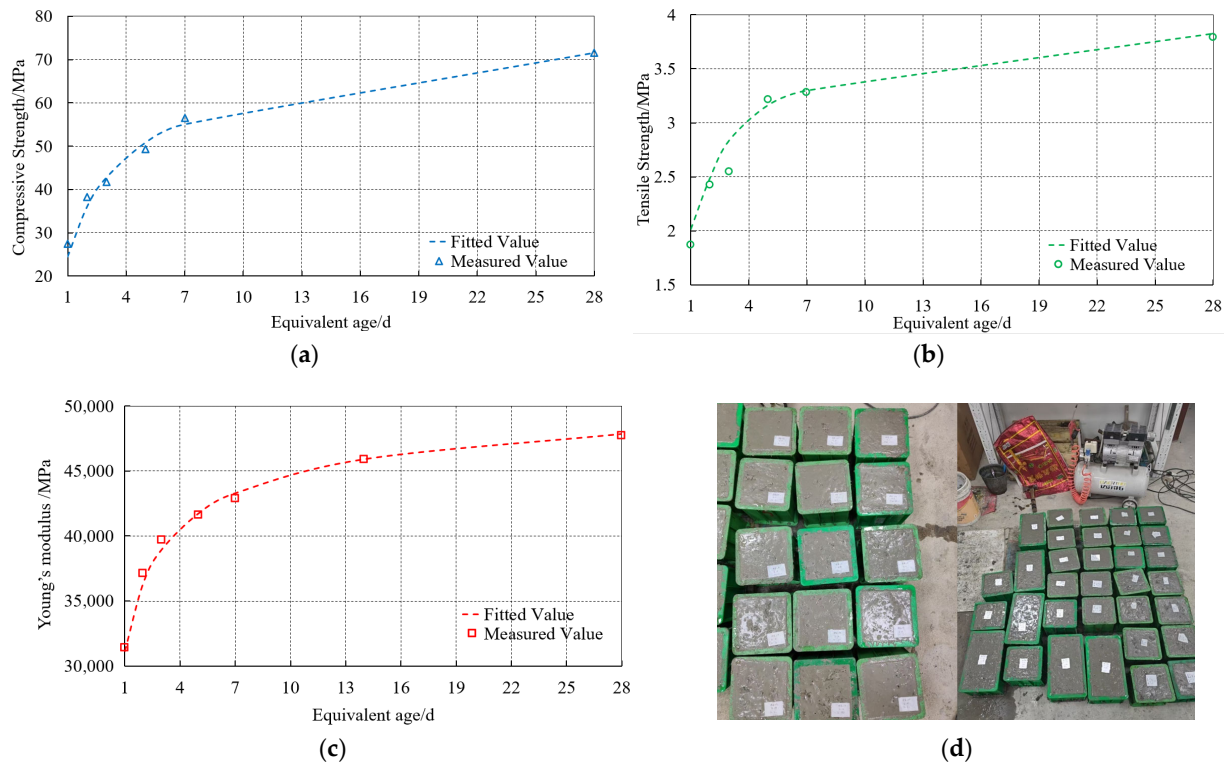


Figure 8. Experimental results of material properties: (a) Development curve of concrete compressive strength; (b) development curve of concrete tensile strength; (c) development curve of concrete elastic modulus; (d) preparation of concrete test specimens.

In practical construction, the temperature of concrete in different regions of the structure cannot be maintained constant at 20 °C. Consequently, the equivalent age deviates from the actual age. The equivalent age can be calculated from the temperature history using Equation (4), and the corresponding material properties are subsequently determined as functions of the equivalent age [22].

$$t_r = \sum_{i=1}^n \Delta t_i e^{\left[13.65 - \frac{4000}{273 + T(\Delta t_i)/T_0}\right]} \quad (4)$$

where Δt_i denotes the time increment (unit: days); $T(\Delta t_i)$ represents the temperature increment within each time step (unit: °C); and T_0 is assigned a value of 1 °C.

The equivalent age model, together with the regression equations for the compressive strength and elastic modulus of concrete, was implemented into the finite element software Abaqus through a user-defined subroutine. Based on the simulated early-age temperature field, the temperature-dependent material properties and stress distribution of each element were computed. In most existing studies, the early-age cracking risk was quantitatively evaluated using the Cracking Index (CI), as defined in Equation (5) [23]. Micro-cracking may typically initiate when the Cracking Index (CI) ranges from 0.6 to 0.8, whereas a CI exceeding 1.0 signifies the onset of macroscopic cracking. Although the CI method, grounded in elastic theory, provides a more generalized risk assessment compared to concrete damage models and is incapable of capturing crack propagation behavior, its superior computational efficiency renders it highly suitable for engineering risk estimation.

From a practical standpoint, considering constraints on construction costs and efficiency, the complete elimination of early-age micro-cracks is often unfeasible. Consequently, this study defines the critical hazardous state as $CI > 1.0$, focusing specifically on the prevention of macroscopic cracking. However, acknowledging the inherent dispersion of in situ concrete tensile strength—which directly affects the sensitivity of risk assessment—it is recommended to adopt a conservative approach in practice by reducing the acceptable CI threshold by 20% to ensure an adequate safety margin.

$$CI = \frac{\sigma_1}{f_t} \quad (5)$$

where CI denotes the cracking index; f_t represents the tensile strength; and σ_1 is the principal tensile stress.

2.3. Shrinkage Test

Many previous studies have indicated that concrete shrinkage is a significant cause of cracking. Under shrinkage, concrete undergoes stress-free deformation. However, when this deformation is restrained, tensile stresses develop in the concrete. From a mechanical perspective, the stress induced by shrinkage is analogous to the tensile stress generated during the cooling phase caused by hydration heat. Since the time-dependent evolution of concrete shrinkage is also influenced by the equivalent age rather than solely by the chronological age, it is necessary to determine the development of shrinkage as a function of equivalent age. The experimental methods were primarily based on the Chinese standard [21]. Under a constant temperature of 20 °C and 50% relative humidity, the shrinkage strain of concrete specimens (10 × 10 × 40 cm) was measured using a dial gauge from 12 h after casting (post-setting) to 28 days [24]. During this period, the concrete temperature remained essentially constant at 20 °C, so the chronological age coincided with the equivalent age.

The initial experimental protocol scheduled daily shrinkage measurements for the first 28 days. However, due to unavoidable on-site constraints, data for days 7–10, 13, 14, 16, 17, 20, 26, and 27 could not be recorded. Nevertheless, these discontinuities do not compromise the overall analysis of the shrinkage evolution trends. To ensure data robustness, three replicate groups were tested simultaneously, and the arithmetic mean was adopted as the final result. The Coefficient of Variation (COV) for all valid datasets remained consistently below 10%, confirming the reliability of the experimental data.

Among the numerous mathematical models describing concrete shrinkage, the Tazawa model is distinguished by its superior applicability to the early-age stage compared to those prioritizing long-term predictions. By incorporating early-age experimental data for fitting and calibration, this model significantly enhances data accuracy and mitigates computational errors. Consequently, this study adopts the Tazawa model, integrated with specific early-age shrinkage tests, to establish the mathematical framework [25]. The development of shrinkage strain of the main girder concrete as a function of equivalent age is shown in Figure 9. The fitting results demonstrate satisfactory agreement with the experimental data, yielding a coefficient of determination (R^2) exceeding 0.9. It is noted that during the 10–15 day interval, the experimental values are slightly lower than the fitted curves, with a maximum deviation of no more than 10%. While this discrepancy may lead to a minor overestimation of the calculated shrinkage stress in this phase, it represents a conservative approximation from a structural safety perspective and therefore does not compromise the overall applicability of the model.

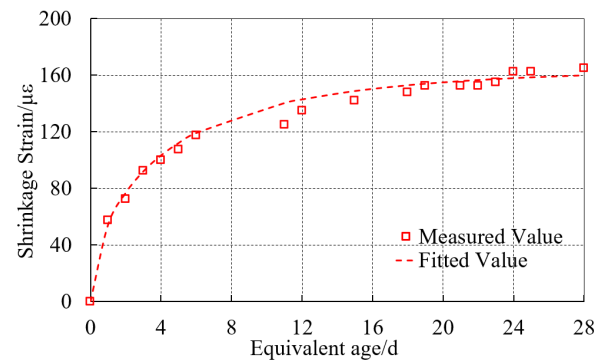


Figure 9. Concrete shrinkage development curve.

The fitting results are expressed in Equation (6).

$$\begin{aligned}\varepsilon_s(t_r) &= 165 \cdot \beta_s(t_r) \\ \beta_s(t_r) &= 1 - e^{(-0.38 \times t_r^{0.63})}\end{aligned}\quad (6)$$

where $\varepsilon_s(t_r)$ is the concrete shrinkage strain at the equivalent age t_r (unit: $\mu\varepsilon$), and $\beta_s(t_r)$ is the shrinkage strain influence factor at t_r . The fitted shrinkage equations were implemented into the finite element software Abaqus through a user-defined subroutine. Based on the simulated early-age temperature field, the equivalent age and shrinkage strain of each element were computed, allowing for the assessment of the stresses induced by shrinkage.

3. Simulation

3.1. Finite Element Model (FEM)

The structural analysis was performed in Abaqus 2024 by combining the user sub-routines HETVAL, USDFLD, UMAT, and UEXPAN [26]. A finite element model was established, where the footing and the PK box girder were tied using a tie constraint, and the base of the footing was fully fixed. For the temperature analysis, DC3D8 elements were used. Based on the computed temperature field, the shrinkage and thermal stresses of the structure were subsequently calculated using C3D8R elements. The finite element model is shown in Figure 10.

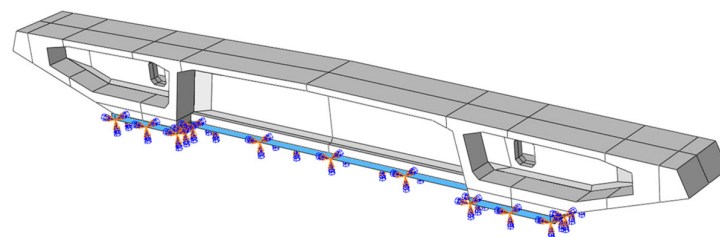


Figure 10. Test beam finite element model.

3.2. Parameter Fitting and Calculation Results

3.2.1. Simulation of Hydration Temperature

The computation of the early-age temperature field of concrete is treated as a transient thermal problem and is described by Equation (7) [27].

$$T = f(x, y, z, t) \quad (7)$$

where T is the temperature, x , y , and z are the spatial coordinates, and t is the time.

The transient temperature field was solved using the heat conduction equation based on Fourier's law, as expressed in Equation (8).

$$\frac{\partial T}{\partial t} = \frac{k}{\rho c} \left(\nabla^2 T + \frac{dQ}{kdt} \right) \quad (8)$$

where k is the thermal conductivity, ρ is the density, c is the specific heat capacity, and Q is the internal heat source.

The solution of the differential equation requires the specification of three types of boundary conditions. The first type corresponds to a prescribed surface temperature, the second type to a known heat flux at the concrete surface, and the third type to a known convective heat transfer at the surface. In addition, the initial temperature distribution must also be provided, as expressed in Equation (9).

$$\begin{aligned} T|_{\Gamma} &= g(x, y, z, t) \\ -k \frac{\partial T}{\partial \vec{n}} \Big|_{\Gamma} &= q_2 \\ -k \frac{\partial T}{\partial \vec{n}} \Big|_{\Gamma} &= h(T - T_f) \end{aligned} \quad (9)$$

where Γ represents the external boundary, q_2 is the surface heat flux density, h is the convective heat transfer coefficient, T_f is the fluid temperature, and $\vec{n}$ denotes the outward normal direction of the boundary.

The computation of the early-age temperature field of concrete is primarily influenced by the concrete thermal properties, the heat of hydration function, and the convective boundary conditions. These parameters are significantly affected by factors such as the concrete mix design, formwork configuration, and construction environment, making accurate prediction based on empirical methods challenging. In this study, the computed results were calibrated against measured temperature data to inversely determine multiple key parameters.

The concrete heat of hydration can be expressed using an exponential function, and the fitted result is given in Equation (10) [28]. The maximum accumulated heat generation was approximately 148,000 kJ. The heat evolution rate was observed to be rapid during the early stage, with a corresponding index of 3.4.

$$Q(\tau) = 148000 \left(1 - e^{-3.4\tau} \right) \quad (10)$$

where τ is the concrete age (unit: d), and $Q(\tau)$ is the heat of hydration of the concrete (unit: kJ/m³).

The fitted value of the thermal conductivity was 2.8 W/m·°C, the specific heat was 0.96 kJ/kg·°C, and the convective heat transfer coefficients are listed in Table 2.

Table 2. Convective Heat Transfer Parameters of Concrete (W/m²·°C).

Position	Deck Slab	Inclined Bottom Slab	Vertical Web	Transverse Diaphragm	Internal Slab
Before formwork removal	14	8	7	7	5
After formwork removal	14	14	12	10	8

The comparison between the measured and simulated temperature histories at typical monitoring points is shown in Figure 11. The results demonstrate excellent agreement, with temperature deviations within 2 °C, confirming the reliability of the calibrated thermal parameters. The results indicate that the initial casting temperature of the concrete was 28 °C, and The core region adjacent to the wind fairing exhibited the highest temperature,

reaching a peak of 70 °C approximately 1 day after casting, followed by a gradual cooling phase approaching the ambient temperature after about 12 days.

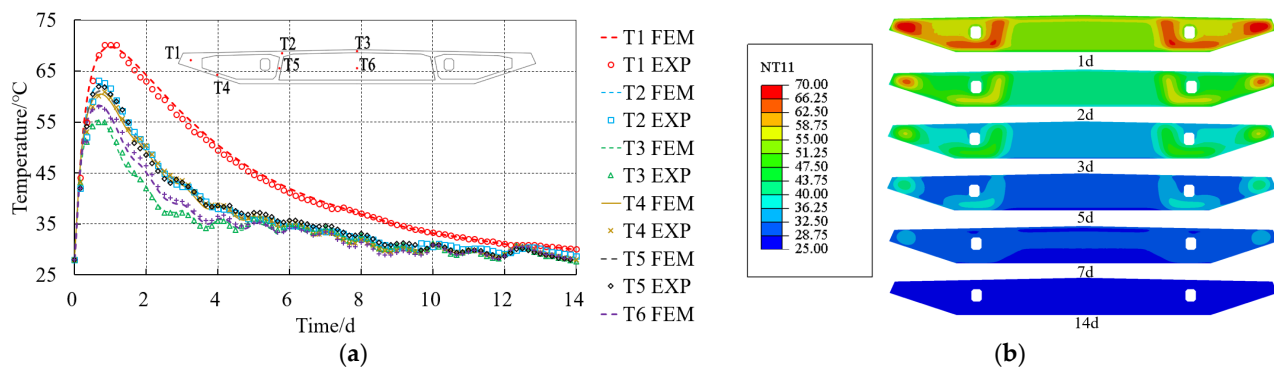


Figure 11. Experimental and simulated temperature results: (a) Temperature evolution curves; (b) Finite element simulation results of the temperature field (Unit: °C).

Due to the smaller section thicknesses of other structural components, their temperature peaks occurred slightly earlier than that of the core region. The maximum temperatures of the web, bottom slab, diaphragm, and top slab were approximately 63 °C, 60 °C, 58 °C, and 55 °C, respectively. These members exhibited faster cooling rates, with temperatures approaching the ambient condition within 4 days, after which minor fluctuations followed the environmental temperature variations.

Overall, pronounced temperature gradients were observed among different components of the PK-section concrete box girder, indicating significant thermal heterogeneity induced by the heat of hydration during the early-age stage.

3.2.2. Simulation of Stress–Strain

In addition to the factors discussed in Section 2, the calculation of early-age concrete stress should also account for the influence of creep on structural stress and strain. At early ages, the relaxation effect induced by creep can significantly reduce thermal stress.

Most commercial finite element software employs the Prony series to characterize viscoelastic behavior via parameter fitting. While computationally feasible, this approach faces significant challenges in simultaneously capturing the rapidly evolving elastic modulus of early-age concrete and is subject to inherent fitting errors. Consequently, although adequate for service-stage analysis, it yields insufficient accuracy for early-age simulations. To address these limitations, this study adopts the creep constitutive model based on the implicit integration method proposed by Zhu Bofang. This model is distinguished by its theoretical clarity, minimal parameter requirements, and high computational efficiency [29]. The creep parameters were determined by fitting the numerical results to the experimental data, as expressed in Equation (11).

$$C(t_r, \tau_r) = (0.23/E_{28}) \left(1 + 9.2\tau_r^{-0.45} \right) \left[1 - e^{-0.3(t_r - \tau_r)} \right] \quad (11)$$

where $C(t_r, \tau_r)$ denotes the creep strain per unit stress of concrete (unit: MPa^{-1}), τ_r represents the equivalent age at the onset of creep, and t_r denotes the equivalent age at the time of analysis.

Prior to initial setting, concrete exists in a viscoplastic state, with no development of internal strength, and a reliable bond between embedded sensors and the material cannot be established. In this study, the initial setting time was approximately 12 h; therefore, the strain data recorded during this period were uniformly corrected, and in the finite element simulations, the corresponding stress, strain, and strength were set to zero [30].

A comparison between the measured and simulated strain histories at representative monitoring points is shown in Figure 12. The results exhibit excellent agreement, indicating that the calibrated material parameters are reliable. Following formwork removal, the exposed structure was subjected to ambient conditions. Thermal expansion and contraction induced cyclic opening and closing of cracks, resulting in significant fluctuations in the experimental data after 3 days.

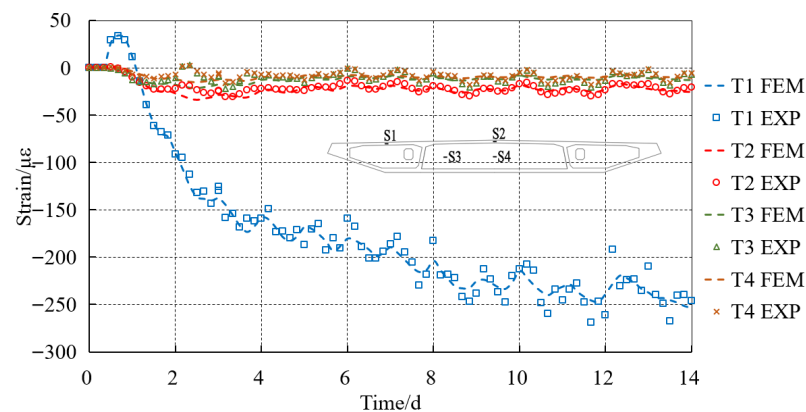


Figure 12. Strain development histories.

Both thermal stress and shrinkage-induced stress arise from restraint effects, and therefore, cannot be directly inferred from measured strain. To elucidate the specific contributions of temperature and shrinkage to the stress state of the diaphragms, the simulation parameters were first validated against experimental data. Subsequently, the effects of temperature and shrinkage were analyzed separately. Unlike previous studies, the computation of shrinkage-induced stress in this work explicitly accounts for the equivalent age variations caused by the hydration heat temperature field, the evolution of elastic modulus, and the stress relaxation due to creep. Thermal deformations were removed through a user-defined subroutine, enabling an accurate assessment of shrinkage-induced stresses.

The transverse stress and the Cracking Index (CI) distribution induced in the diaphragm are shown in Figure 13. The Abaqus subroutine framework permits the use of Solution-Dependent State Variables (SDVs) for tracking and displaying user-defined quantities. Accordingly, the Cracking Index (CI) from Equation (5) was mapped to SDV34 for post-processing. As illustrated in the simulation results, regions characterized by a CI greater than 1 are rendered in gray. Under the influence of hydration heat, the stress reached its peak at 3 days after casting, with the majority of the diaphragm exhibiting stresses greater than 4 MPa. After 3 days, the stress exhibited fluctuations. Two days after casting, the lower region of the diaphragm already shows $CI > 1$, and by 3 days, most of the diaphragm area exhibits $CI > 1$, highlighting a significant cracking risk. The calculated results are in good agreement with the observed cracking patterns in the experiment.

Under the influence of shrinkage, the stress reached its peak within 1–2 days after casting, with most regions of the diaphragm experiencing stresses of approximately 1.5 MPa, followed by slight fluctuations. Two days after casting, the majority of the diaphragm exhibits CI values below 60%, indicating a relatively low risk of cracking.

Representative points at the midspan of the diaphragm were selected to extract the evolution of temperature, stress, material strength and equivalent age, as shown in Figure 14. During the heating stage, the thermally induced stress is compressive. As cooling progresses, the stress gradually increases and transitions into tensile stress, reaching a peak of approximately 4.2 MPa at 3 days, while the corresponding tensile strength is only

3.3 MPa, indicating a critical risk of cracking. The tensile stress arises primarily due to the temperature drop.

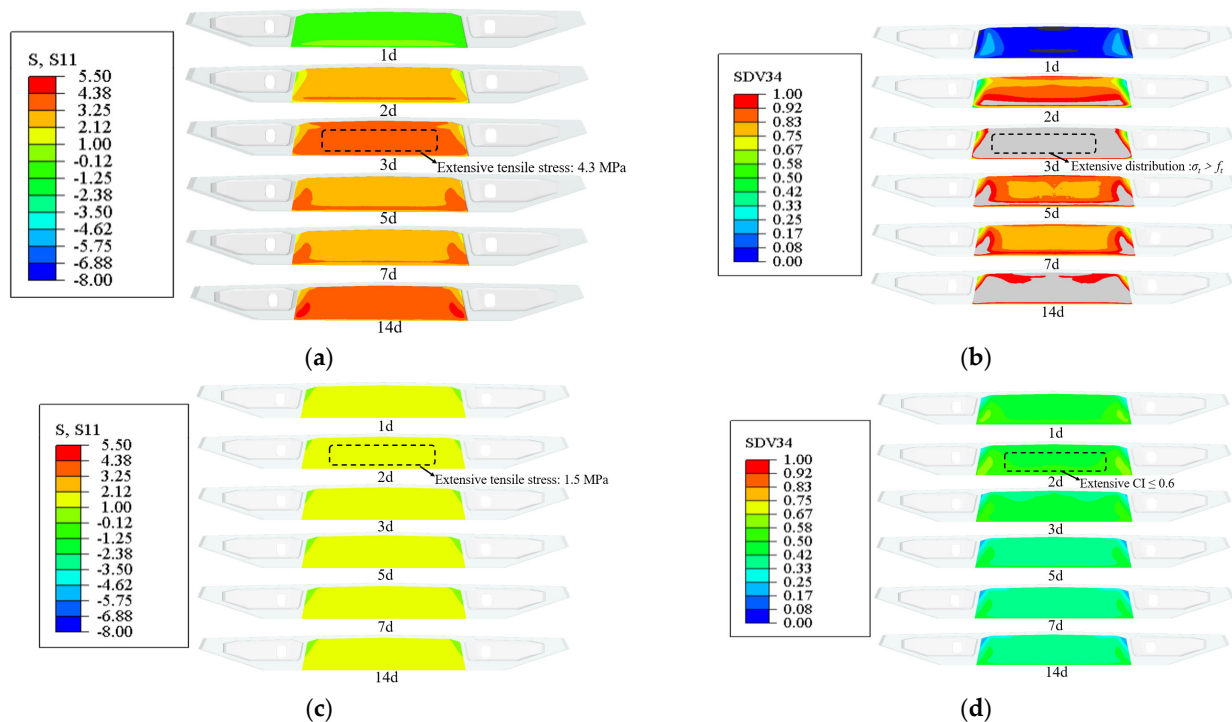


Figure 13. Finite element results: (a) transverse stress in the diaphragm induced by hydration temperature (Unit: MPa); (b) the Cracking Index (CI) for temperature-induced stress in the diaphragm; (c) transverse stress in the diaphragm induced by shrinkage (Unit: MPa); (d) the Cracking Index (CI) for shrinkage in the diaphragm.

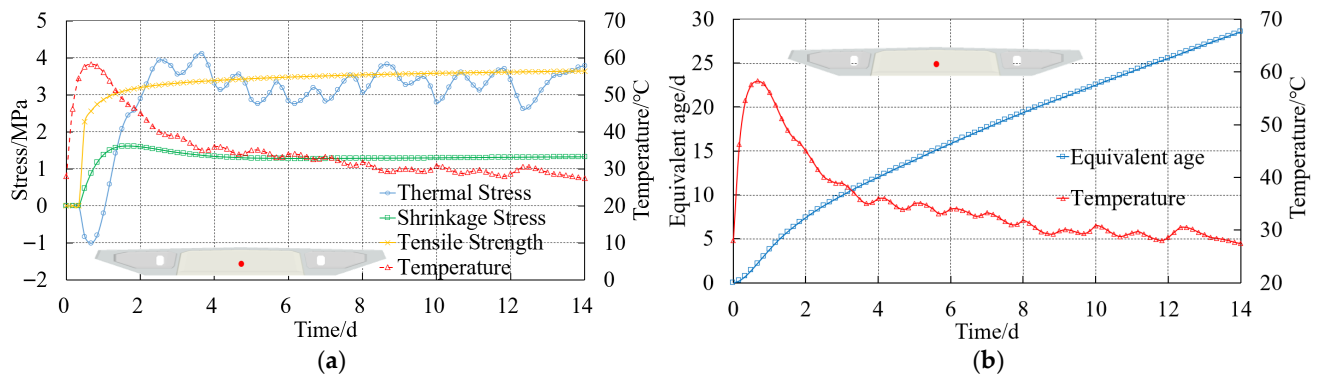


Figure 14. Finite element results: (a) Early-age performance development curves of the diaphragm; (b) development curve of equivalent age.

In contrast, the shrinkage-induced stress develops rapidly within 1–2 days, reaching a maximum of about 1.5 MPa. In conventional approaches, shrinkage is often converted to an equivalent temperature change. In this study, the 28-day maximum shrinkage strain is $165 \mu\epsilon$, and with a concrete coefficient of thermal expansion of $1.0 \times 10^{-5}/^{\circ}\text{C}$, the corresponding equivalent cooling is 16.5°C , whereas the maximum cooling induced by hydration heat reaches 29°C . The maximum shrinkage-induced stress would account for approximately 57% of the maximum thermally induced stress; however, the present numerical simulation indicates only 35%. The discrepancy is primarily attributed to the early-age development captured in the experimental shrinkage curves. Shrinkage progresses rapidly

during the initial phase; at an equivalent age of 7 days, roughly 80% of the 28-day shrinkage has already occurred. Concurrently, the early-age concrete temperature rises sharply within 1–2 days post-casting, accelerating the equivalent-age development to 7.4 days, such that most shrinkage is completed early. During this period, creep-induced stress relaxation is most pronounced, effectively reducing the peak shrinkage stress. In comparison, the cooling effect due to hydration heat occurs later and persists over a longer duration, during which creep relaxation is less significant. This explains why conventional methods tend to overestimate the contribution of shrinkage to early-age stress.

Quantitatively, the maximum temperature-induced stress reaches 4.2 MPa, whereas the maximum shrinkage-induced stress is only 1.5 MPa, yielding a ratio of approximately 3:1. These results clearly demonstrate that early-age cracking of the diaphragm is primarily governed by temperature-induced stress resulting from hydration heat, while shrinkage contributes to a lesser extent.

4. Discussion

4.1. Analysis of Restraint Effects

Temperature and shrinkage-induced stresses fundamentally result from the restraint of free deformations. Such restraints can be classified as self-restraint and external restraint. Self-restraint arises from the mutual restriction among structural regions due to non-uniform thermal development. For example, in a mass concrete footing, the hotter core constrains the cooling-induced contraction of the cooler surface, or in a box girder, temperature differentials among slab components generate internal tensile interactions. External restraint refers to limitations imposed by supports or surrounding structures [31,32]. In the present study, the test beam is externally restrained by the supporting footing. To isolate and quantify the effect of external restraint on stress, a comparative analysis was conducted under identical conditions with the external constraint removed.

Temperature, stress, and strength development curves were extracted at the same location, as shown in Figure 15. The overall trend of stress evolution remained unchanged; however, the peak of temperature-induced stress decreased from 4.3 MPa to 1.5 MPa, while the peak of shrinkage-induced stress decreased from 1.5 MPa to 0.5 MPa. In the absence of external restraint, the reductions in temperature- and shrinkage-induced stresses were approximately 65% and 66%, respectively. The combined stress did not exceed the tensile strength, indicating a low risk of cracking. A comprehensive analysis suggests that external restraint was the primary cause of cracking observed in the test beams.

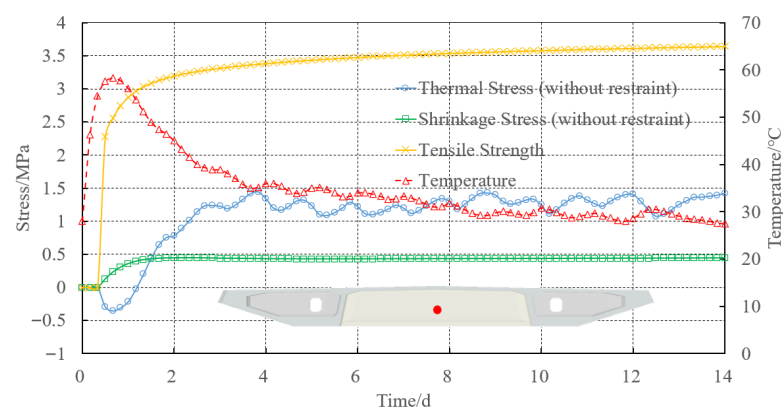


Figure 15. Finite element results under unconstrained boundary conditions.

To validate the analytical results, two additional tests under identical conditions were conducted, denoted as test beams #2 and #3. For test beam #2, the restraint at the bottom

was reduced by placing a smooth steel plate between the cap and the beam. Test beam #3 was cast on a support frame, further minimizing the restraint, as illustrated in Figure 16. After formwork removal, vertical cracks were still observed on the diaphragm of test beam #2, but the number of cracks decreased from 18 to 5 and the maximum crack length was reduced from 160 cm to 50 cm. No vertical cracks were observed on the diaphragm of test beam #3. As the bottom restraint was progressively reduced, the stress on the diaphragm decreased accordingly, resulting in a lower risk of cracking, which is consistent with the analytical predictions.

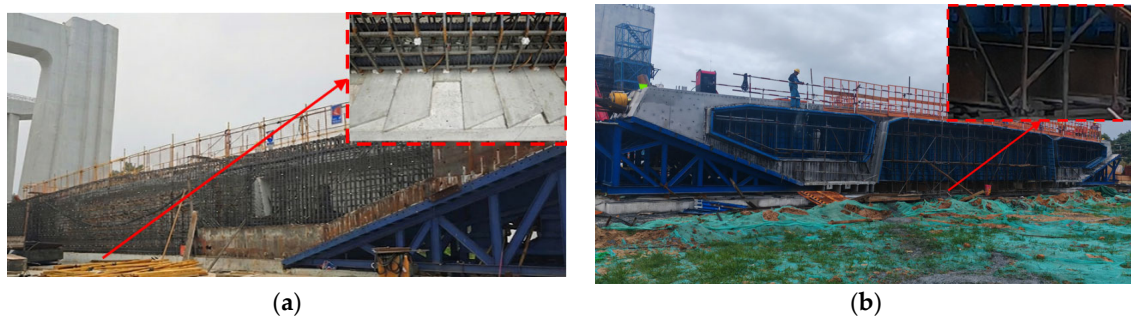


Figure 16. Test beams with reduced bottom restraint: (a) Test beam #2; (b) test beam #3.

4.2. Analysis of Construction Conditions of the Main Girder

The external restraint at the beam soffit was identified as the key factor causing cracking in the test beams. However, the boundary conditions of the main girder during actual construction differ significantly from test beams. To evaluate the cracking risk of diaphragms under real construction conditions, finite element (FE) analyses were performed using the thermal–mechanical parameters obtained from the experiments. Two representative segments, No. 4 and No. 7, were selected for analysis, which primarily differ in the bottom slab thickness. In segment No. 4, the inclined bottom slab is 45 cm thick and the flat bottom slab is 90 cm thick, whereas in segment No. 7, the inclined and flat bottom slabs are 35 cm and 50 cm thick, respectively.

A one-quarter 3D FE models were established, covering the cantilever from the 0# segment to either the No. 4 or No. 7 segment. Adjacent segments were connected using tie constraints to simulate the monolithic behavior, and the preceding segments provided realistic restraint stiffness. A temporary fixed support was applied beneath the 0# segment according to the construction drawings. The structural configuration and corresponding FE model of the main girder are shown in Figure 17.

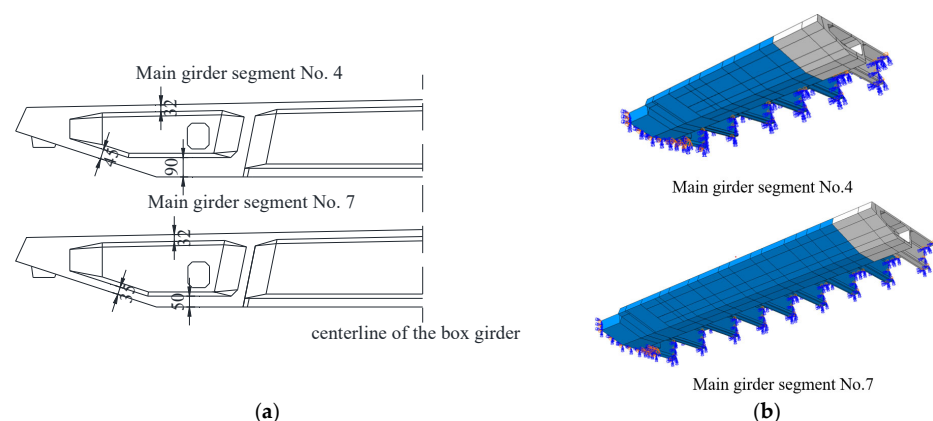


Figure 17. Structural configuration and FE modeling of the analyzed segments: (a) Structural configurations of the main girders (unit: cm); (b) finite element models.

The finite element simulation results of the 4# and 7# segments show good agreement. The temperature-induced stress and the Cracking Index (CI) in the diaphragm of the 4# segment are presented in Figure 18. During the temperature rise caused by hydration heat, compressive stress develops within the diaphragm. As cooling proceeds, tensile stress gradually increases, reaching a peak of 3.8 MPa at the bottom surface at 4 days, where the CI value exceeds 1.0, indicating a high cracking risk. Compared with the test beam, the temperature-induced stress in the diaphragm of the main girder under construction conditions is notably reduced, and no extensive cracking risk is observed. However, relatively high tensile stress still develops at the bottom region, suggesting a potential for localized vertical cracking.

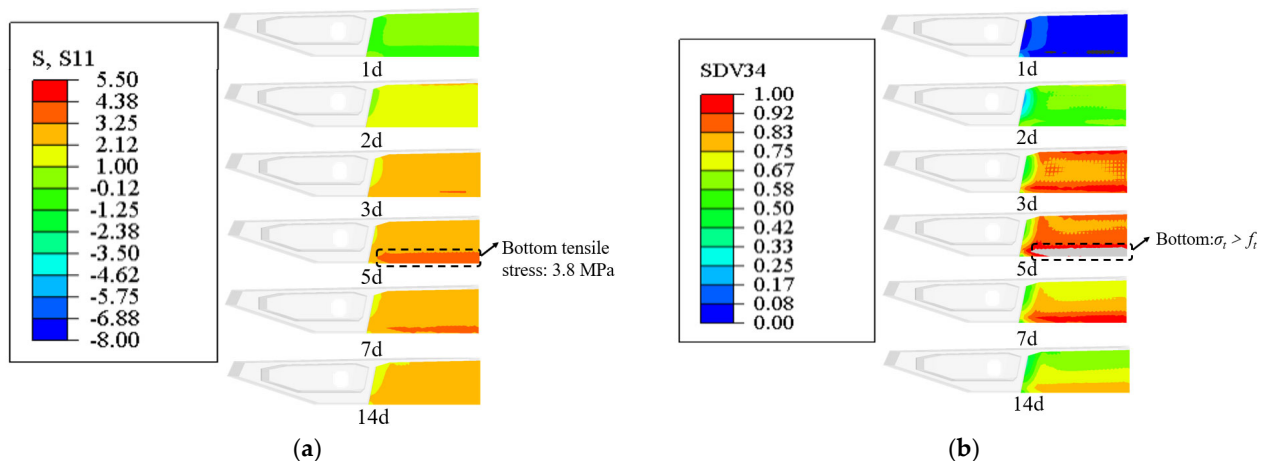


Figure 18. Finite element results: (a) transverse stress in the diaphragm induced by hydration temperature (Unit: MPa); (b) the Cracking Index (CI) for temperature-induced stress in the diaphragm.

The temperature, stress, and strength development curves at the diaphragm bottom of segments 4# and 7# are shown in Figure 19. The peaks of temperature- and shrinkage-induced stresses are 3.8 MPa and 2.0 MPa, approximately in a 2:1 ratio. Two days after casting, the temperature-induced stress reaches 2.0 MPa, while the shrinkage-induced stress attains its peak of 2.0 MPa, resulting in a combined transverse tensile stress of 4.0 MPa, which exceeds the tensile strength of 3.2 MPa, indicating a high potential for vertical cracking at the bottom. By day 4, the temperature-induced stress peaks at 3.8 MPa, and the shrinkage-induced stress remains at 2.0 MPa, producing a total transverse tensile stress of 5.8 MPa, whereas the tensile strength is 3.5 MPa, suggesting that the risk of vertical cracking at the bottom is still significant. The temperature-induced stress generated by hydration heat is the primary factor influencing the cracking risk, while the external restraint imposed by the previously cast segments is the main cause of the high stress development in the diaphragm.

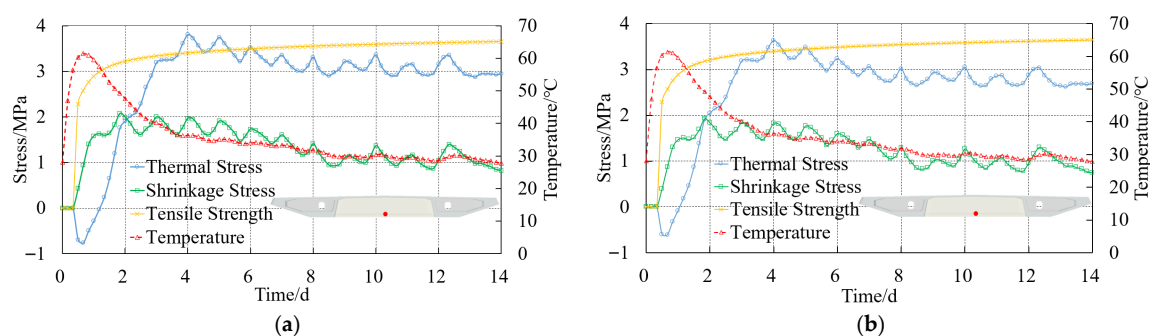


Figure 19. Finite element results: (a) Main girder segment No. 4; (b) main girder segment No. 7.

4.3. Construction Technique Optimization

To effectively mitigate the risk of diaphragm cracking during main girder construction, construction technique were optimized. Firstly, reducing the concrete casting temperature can decrease temperature-induced stress [33]. Taking segment 4# as an example, a finite element analysis was conducted by lowering the casting temperature from 28 °C to 20 °C, while keeping all other conditions unchanged.

The temperature and stress development curves at the diaphragm bottom are presented in Figure 20. The peak temperature-induced stress decreases from 3.8 MPa to 3.0 MPa, corresponding to a stress reduction efficiency of approximately 0.1 MPa/°C.

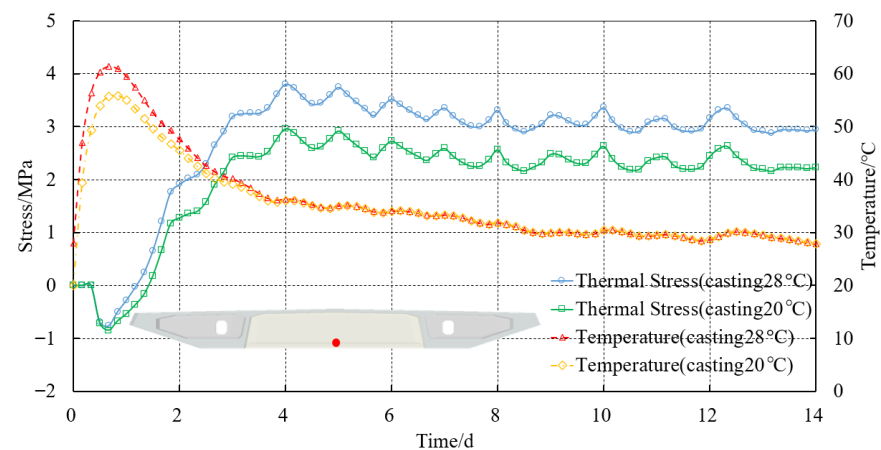


Figure 20. Finite element results under the optimized casting temperature condition.

Secondly, the incorporation of polypropylene (PP) fibers can enhance the tensile strength of concrete [34]. In this study, 0.9 kg/m³ of PP fibers was added to the main girder concrete. The addition of PP fibers resulted in an average increase of approximately 15% in the early-age tensile strength of concrete. The experimental tensile strength results are presented in Figure 21.

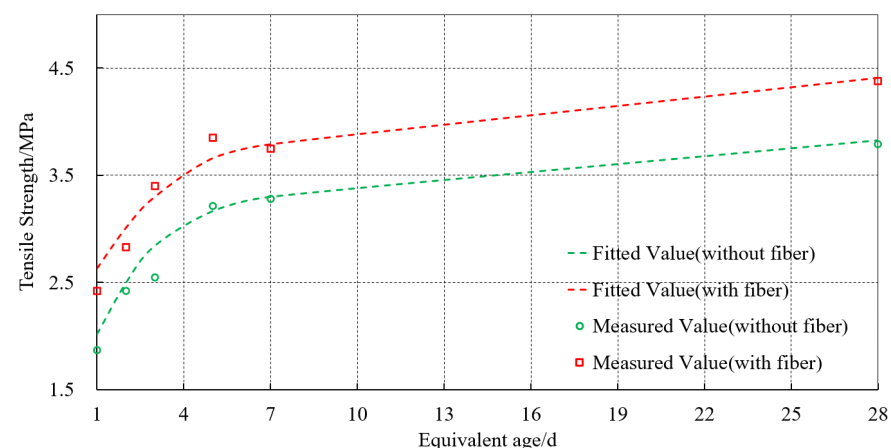


Figure 21. Comparison of tensile strength development.

Although temperature-induced stress is the primary factor contributing to cracking, the cracking risk associated with shrinkage-induced stress should not be neglected. Measures such as optimizing the concrete mix design and incorporating chemical admixtures can be employed to reduce early-age shrinkage of concrete [35].

5. Conclusions

Early-age concrete is significantly influenced by temperature-induced and shrinkage-induced stresses, which may lead to cracking. In this study, full-scale field tests were conducted on wide PK-section concrete box girders to investigate the formation of vertical cracks in the diaphragms, and finite element (FE) analyses were performed to identify the contributing factors. The influence of temperature- and shrinkage-induced stresses, as well as the primary causes of high stress development, were clarified. The parameters obtained from the experiments were further applied to simulate cantilever construction conditions, allowing for the assessment of early-age cracking risk in the main girder diaphragms and the development of optimized construction measures.

(1) Under the effect of hydration heat, the highest temperature in the core region adjacent to the wind fairing reached approximately 70 °C on day 1, while the maximum temperatures in the web, bottom slab, diaphragm, and top slab were 63 °C, 60 °C, 58 °C, and 55 °C, respectively, indicating significant temperature gradients among the components.

(2) In the finite element simulations, an equivalent age method was employed to account for the influence of the temperature field on material property development. Parameters such as elastic modulus, strength, and shrinkage were based on experimental data, whereas thermal parameters, including the heat generation function, convection coefficients, and creep model, were calibrated from the experiments. The simulation results demonstrated good agreement with the experimental measurements.

(3) In the full-scale test, 18 vertical cracks were observed in the diaphragm, with a maximum length of 160 cm. FE analysis showed that the temperature-induced stress in the diaphragm was 1.4 MPa on day 1 and peaked at 4.2 MPa on day 3, with high-stress regions distributed across the entire diaphragm. Shrinkage-induced stress reached 1.5 MPa on day 1, whereas the concrete tensile strength was 3.0 MPa on day 1 and 3.3 MPa on day 3. Temperature-induced stress was identified as the primary factor contributing to cracking.

(4) During the full-scale test, the girder was cast directly on the bearing platform, imposing significant restraint on the bottom deformation. When the bottom restraint was removed, the simulated temperature- and shrinkage-induced stresses were reduced by 66%. Complementary tests demonstrated that reducing bottom restraint decreased and ultimately eliminated vertical cracks in the diaphragms, confirming that external restraint from the bearing platform was the main cause of high stress development due to cooling and concrete shrinkage. In similar full-scale model tests, it is recommended to install a sliding layer or supports at the base to release foundation or support constraints, thereby preventing excessive stress in the diaphragms caused by over-constraint.

(5) Under cantilever construction conditions, the temperature-induced stress at the diaphragm bottom was 2.0 MPa on day 2 and reached a peak of 3.8 MPa on day 4, while stresses in other regions remained lower. The shrinkage-induced stress developed to a peak of 2.0 MPa on day 2. The tensile strength of concrete was 3.2 MPa on day 2 and 3.5 MPa on day 4. The high risk of vertical cracking at the diaphragm bottom was mainly due to temperature-induced stress from hydration heat, while external restraint from the previously cast segments was the primary factor contributing to elevated stress levels.

(6) Reducing the concrete casting temperature effectively decreased diaphragm temperature-induced stress, with an efficiency of approximately 0.1 MPa/°C. The addition of 0.9 kg/m³ polypropylene fibers increased early-age tensile strength by about 15%. Additionally, optimizing the concrete mix design or using chemical admixtures to reduce early-age shrinkage can mitigate shrinkage-induced stress. The combined implementation of these measures can effectively reduce the risk of cracking.

Author Contributions: Conceptualization, Investigation, Methodology, Writing—review & editing, Supervision, Z.Z.; Formal analysis, Funding acquisition, Writing—original draft, J.W.; Formal analysis, Resources, Validation, Visualization, Project, H.L.; Conceptualization, Investigation, Methodology, Writing—review & editing, administration, X.S.; Conceptualization, Software, Data curation, Writing - original draft, B.H. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Data Availability Statement: The datasets generated during and/or analyzed during the current study are not publicly available due to the inclusion of proprietary simulation parameters, industrial application-specific configurations, and third-party software licensing restrictions, but are available from the corresponding author on reasonable request.

Conflicts of Interest: Zepeng Zhang, Jia Wang and Hongsheng Li was employed by Guangdong Transportation Industrial Investment Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

1. Qu, Z.; Zhang, Y.; Liu, Z.; Si, R.; Wu, J. A review on early-age cracking of concrete: Causes and control. *Case Stud. Constr. Mater.* **2024**, *21*, e03848. [\[CrossRef\]](#)
2. Sheng, X.; Xiao, S.; Zheng, W.; Sun, H.; Yang, Y.; Ma, K. Experimental and finite element investigations on hydration heat and early cracks in massive concrete piers. *Case Stud. Constr. Mater.* **2023**, *18*, e01926. [\[CrossRef\]](#)
3. Liu, Y.; Liu, J. Review on temperature action and effect of steel–concrete composite girder bridge. *J. Traffic Transp. Eng. (Engl. Ed.)* **2020**, *20*, 42–59. [\[CrossRef\]](#)
4. Yu, M.; Yao, X.; Xie, K.; Hao, T.; Wang, X. Analysis Method for the Pouring Stage of Concrete-Filled Steel Tube (CFST) Arch Bridges Considering Time-Varying Heat of Hydration and Elastic Modulus. *Buildings* **2025**, *15*, 1711. [\[CrossRef\]](#)
5. Kuryłowicz-Cudowska, A.; Wilde, K. FEM and experimental investigations of concrete temperature field in the massive stemwall of the bridge abutment. *Constr. Build. Mater.* **2022**, *347*, 128565. [\[CrossRef\]](#)
6. Do, T.A.; Verdugo, D.; Tia, M.; Hoang, T.T. Effect of volume-to-surface area ratio and heat of hydration on early-age thermal behavior of precast concrete segmental box girders. *Case Stud. Therm. Eng.* **2021**, *28*, 101448. [\[CrossRef\]](#)
7. Wu, H.; Liu, J. Investigations of the Temperature Field and Cracking Risk in Early Age Massive Concrete in the #0 Segment of a Box Girder Bridge. *KSCE J. Civ. Eng.* **2023**, *27*, 3971–3989. [\[CrossRef\]](#)
8. Yang, T.; Wang, X.; Li, Q.; Xu, A.; Ma, X. Inversion of Thermal Parameters and Temperature Field Prediction for Concrete Box Girders Based on BO-XGBoost. *Buildings* **2025**, *15*, 3408. [\[CrossRef\]](#)
9. Massoud, S.; Lumantarna, E.; Mendis, P.; Zhong, A. Thermal Stresses of Concrete at Early Ages. *J. Mater. Civ. Eng.* **2019**, *31*, 04019056. [\[CrossRef\]](#)
10. Liu, J.; Liu, Y.; Zhang, Z. Numerical Simulation on Thermomechanical Coupling Behavior of Early-Age Concrete in the Large-Scale Steel–Concrete Connecting Segment of a Hybrid-Girder Cable-Stayed Bridge. *J. Bridge Eng.* **2020**, *25*, 05020009. [\[CrossRef\]](#)
11. Bažant, Z.P. Constitutive Equation for Concrete Creep and Shrinkage Based on Thermodynamics of Multiphase Systems. *Matér. Constr.* **1970**, *3*, 3–36. [\[CrossRef\]](#)
12. Choi, S.; Cha, S.W.; Oh, B.H. Thermo-hygro-mechanical behavior of early-age concrete deck in composite bridge under environmental loadings. Part 2: Strain and stress. *Mater. Struct.* **2011**, *44*, 1347–1367. [\[CrossRef\]](#)
13. Do, T.A.; Tia, M.; Nguyen, T.H.; Hoang, T.T.; Tran, T.D. Assessment of temperature evolution and early-age thermal cracking risk in segmental high-strength concrete box girder diaphragms. *KSCE J. Civ. Eng.* **2022**, *26*, 166–182. [\[CrossRef\]](#)
14. Han, S.; Liu, Y.; Lyu, Y.; Liu, J.; Zhang, N. Numerical simulation investigation on hydration heat temperature and early cracking risk of concrete box girder in cold regions. *J. Traffic Transp. Eng. (Engl. Ed.)* **2023**, *10*, 697–720. [\[CrossRef\]](#)
15. Gribniak, V.; Kaklauskas, G.; Bacinskas, D. State-of-the-Art Review of Shrinkage Effect on Cracking and Deformations of Concrete Bridge Elements. *Baltic J. Road Bridge Eng.* **2007**, *2*, 183–193.
16. Kwak, H.-G.; Ha, S.; Weiss, W.J. Experimental and Numerical Quantification of Plastic Settlement in Fresh Cementitious Systems. *J. Mater. Civ. Eng.* **2010**, *22*, 951–966. [\[CrossRef\]](#)
17. Ghourchian, S.; Wyrzykowski, M.; Plamondon, M.; Lura, P. On the Mechanism of Plastic Shrinkage Cracking in Fresh Cementitious Materials. *Cem. Concr. Res.* **2019**, *115*, 251–263. [\[CrossRef\]](#)
18. Elzokra, A.A.E.; Al Hourri, A.; Habib, A.; Habib, M.; Malkawi, A.B. Shrinkage Behavior of Conventional and Nonconventional Concrete: A Review. *Civ. Eng. J.* **2020**, *6*, 1839–1851. [\[CrossRef\]](#)

19. Du, W.; Qian, C.; Xu, Y.; Wei, M.; Peng, B.; Xie, Y. Effects of Ambient Temperature, Formwork Type, and Demolding Time on the Thermal Deformation of Sidewall Concrete in Underground Engineering: Experiment, Simulation, and Engineering Practice. *Constr. Build. Mater.* **2022**, *353*, 129083. [[CrossRef](#)]
20. Yang, Q.; Sun, Y.; Peng, X. Experimental Study on Mechanical Properties of Concrete at Super-Early Age. *Materials* **2022**, *15*, 7582. [[CrossRef](#)]
21. Ministry of Transport of the People's Republic of China. *Testing Methods of Cement and Concrete for Highway Engineering (JTG 3420-2020)*; China Communications Press: Beijing, China, 2020. (In Chinese)
22. FIB—Fédération Internationale du Béton. *Fib Model Code for Concrete Structures 2010*; International Federation for Structural Concrete: Lausanne, Switzerland, 2013.
23. Bobko, C.P.; Edwards, A.J.; Seracino, R.; Zia, P. Thermal Cracking of Mass Concrete Bridge Footings in Coastal Environments. *J. Perform. Constr. Facil.* **2015**, *29*, 04014171. [[CrossRef](#)]
24. Kurup, D.S.; Mohan, M.K.; Van Tittelboom, K.; De Schutter, G.; Santhanam, M.; Rahul, A.V. Early-age Shrinkage Assessment of Cementitious Materials: A Critical Review. *Cem. Concr. Compos.* **2024**, *145*, 105343. [[CrossRef](#)]
25. Tazawa, E.; Miyazawa, S.; Kasai, T. Chemical Shrinkage and Autogenous Shrinkage of Hydrating Cement Paste. *Cem. Concr. Res.* **1995**, *25*, 288–292. [[CrossRef](#)]
26. Zhao, D.; Zhu, L.; Shen, Z.; Miu, Z.; Liu, X. Effect of Hydration Heat of Super-Large-Section Tunnel-Lining Concrete on Structural Stress and Its Mitigation Measures. *Tunn. Undergr. Space Technol.* **2025**, *157*, 106324. [[CrossRef](#)]
27. Cai, Y.; Wang, F.; Zhao, Z.; Lyu, Z.; Wang, Y.; Zou, P. Early-Hydration Heat and Influencing Factor Analysis of Large-Volume Concrete Box Girder Based on Equivalent Age. *Structures* **2023**, *50*, 1699–1713. [[CrossRef](#)]
28. Huang, Y.; Liu, G.; Huang, S.; Rao, R.; Hu, C. Experimental and Finite Element Investigations on the Temperature Field of a Massive Bridge Pier Caused by the Hydration Heat of Concrete. *Constr. Build. Mater.* **2018**, *192*, 240–252. [[CrossRef](#)]
29. Zhu, B.F. *Temperature Stresses and Temperature Control in Mass Concrete*, 2nd ed.; China Water Conservancy and Hydropower Press: Beijing, China, 2012. (In Chinese)
30. Zhang, L.; Mo, D.; Wang, L.; Wu, X. A method for directly measuring thermal stress in concrete under temperature gradients. *Constr. Build. Mater.* **2025**, *482*, 141676. [[CrossRef](#)]
31. Schmidt, C.; Chudoba, R.; Jin, C.; Hegger, J. Experimental investigations on early crack development in fully restrained reinforced concrete members with active zero-displacement control. *Materials* **2023**, *56*, 115. [[CrossRef](#)]
32. Safiuddin, M.; Kaish, A.B.M.A.; Woon, C.-O.; Raman, S.N. Early-Age Cracking in Concrete: Causes, Consequences, Remedial Measures, and Recommendations. *Appl. Sci.* **2018**, *8*, 1730. [[CrossRef](#)]
33. Le, Q.X.; Torero, J.L.; Dao, V.T.N. Stress–strain–temperature relationship for concrete. *Fire Saf. J.* **2021**, *120*, 103126. [[CrossRef](#)]
34. Latifi, M.R.; Biricik, Ö.; Mardani Aghabaglou, A. Effect of the addition of polypropylene fiber on concrete properties. *J. Adhes. Sci. Technol.* **2022**, *36*, 345–369. [[CrossRef](#)]
35. Lu, T.; Li, Z.; Huang, H. Effect of Supplementary Materials on the Autogenous Shrinkage of Cement Paste. *Materials* **2020**, *13*, 3367. [[CrossRef](#)]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.