

高海拔地区桥墩混凝土表层温度应力分析

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摘 要: 高海拔地区自然环境中的桥梁墩柱承受着大幅度的昼夜温度循环作用。为了预测其表层混凝土的开裂风险, 构建了混凝土表层温度应力分析模型。该模型综合考虑了桥梁墩柱内部温度分布的不均匀性和混凝土材料内部各组份间的热变形不协调性, 即整体各向同性和局部各向异性。在该模型中加载现场实测得到的高海拔地区桥梁墩柱内部各时刻温度场, 分别应用热弹性力学理论计算与平面应变问题有限元求解的办法, 分析了墩柱表层的三维热应力。结果表明, 墩柱表层开裂主要在上部结构架设之前。晴天时, 对温度作用下墩柱表面受力进行正交分解, 得到 1 d 中墩柱表面最大压应力约 12.52 MPa, 最大拉应力约 2.15 MPa。其中, 最大拉应力已接近混凝土在二轴受拉状态下的抗拉强度, 进一步叠加湿度应力后开裂风险较大。架设上部结构后, 墩柱混表面切向最大拉应力仍为 2.15 MPa, 竖向最大拉应力仅余 0.58 MPa, 均明显小于 C35 混凝土在该二轴受拉状态下的抗拉强度。采用热膨胀系数低的粗骨料虽可降低温度梯度导致的拉应力, 但会增大混凝土各组份热变形不协调产生的热应力, 导致墩柱混凝土表层最大拉应力略有上升。

关键词: 混凝土; 桥墩; 热应力; 温度场; 变形不协调; 热膨胀系数

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1 Introduction

The predominant characteristics of high-altitude (≥ 3500 m) climates include low air pressure, low humidity, and large diurnal temperature fluctuations^[1-3]. These complex climatic conditions often exert hidden yet long-lasting impacts on infrastructure, primarily manifesting as poor performance and severe surface cracking in concrete^[4-6]. Recent reports indicate that concrete surface deterioration is more severe in piers, particularly high piers, than in beams in high-altitude regions^[7-8]. Statistics further reveal that the sunlit surfaces of piers develop more cracks over time than shaded surfaces^[6,8-9]. Once moisture curing is terminated, the sunlit surface of pier concrete dries rapidly owing to intense solar radiation, increasing the risk of shrinkage cracking^[10-12]. Moreover, daily wide-range temperature cycles experienced by sunlit surfaces can significantly

enhance drying shrinkage at early ages, as observed by Wang *et al.*^[13]. Over time, the internal humidity of the piers decreases, eventually balancing with the surrounding environment. Consequently, thermal stress rather than humidity stress is the primary cause of surface cracking in the later stages. In practical engineering scenarios in high-altitude regions, many pier surface cracks only appear several months after erection, and cyclic thermal stress is identified as the main trigger for such cracking^[6,14].

The thermal stress in concrete structures has been investigated for decades but remains incompletely understood. Structural engineers typically regard concrete as an isotropic material and calculate the thermal stress using code-specified coefficients of thermal expansion (CTEs) along with temperature conditions and constraints^[6,14-16]. To lower the CTE of structural concrete, aggregates with a low CTE, such as limestone,

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are often recommended in design documents. However, researchers have not universally endorsed this approach. Because the CTE of hardened cement paste is more than twice that of many aggregates, reducing the CTE of coarse aggregates can further exacerbate the thermal deformation incompatibility between the coarse aggregate and mortar matrix^[17–19].

To analyze the deformation incompatibility between mortar and coarse aggregates, Hanaor and Christoforou^[17] developed a finite element model by dividing a two-dimensional plane into grids and assigning corresponding elastic moduli and CTEs to the components. Wang *et al.*^[18] conducted temperature cycling tests on concrete specimens and observed that microcracks initially formed at the interface between the hardened cement paste and coarse aggregate, which subsequently expanded to the specimen surface. Similarly, Al-Tayyib *et al.*^[19] studied deformation incompatibility in concrete under temperature cycles ranging from 20 °C to 80 °C and found that concrete using limestone as coarse aggregate exhibited a 30% reduction in both compressive and flexural strength after 90 cycles. Vanecanin^[20] discussed these findings and suggested that the low CTE of limestone negatively affected the resistance of concrete to large temperature variations. Zhang^[21] and Du^[22] recommended prioritizing quartzite and granite over limestone as parent rocks for coarse aggregates in environments with large temperature differences, citing improved thermal shock stability and deformation compatibility. Guo *et al.*^[23–24] studied the temperature response and damage to bridge concrete under strong radiation and wind conditions, highlighting the tensile stress caused by significant temperature changes owing to thermal deformation incompatibility at varying depths. However, it should be noted that the concrete units or specimens examined in these studies^[13–20] were not embedded in an overall structural system. Consequently, the deformations observed in these components may not fully align with the actual behavior of concrete in complete structures.

In the literature, researchers have employed various methods to simulate the temperature field inside high-altitude bridge piers. Ge^[6] measured the temperature field of a scaled column in a low-altitude region and

adjusted it based on temperature records from soil surfaces at high altitudes. Dong *et al.*^[14] simulated temperature fields within high-altitude piers by assuming specific boundary conditions. However, the spatiotemporal distributions of the temperature inside of piers have yet to be accurately measured. The lack of precise data regarding these temperature fields has made it challenging to determine the exact thermal stresses acting on highland pier surfaces, creating a significant obstacle in predicting surface cracking in these structures.

In this study, cracks appearing at ages of several months, that is, long-age cracks, were analyzed. In this case, the daily temperature is the dominant factor affecting cracking, whereas the effects of hydration heat and surface water evaporation vanish or are significantly weakened. A comprehensive thermoelastic mechanics model for pier concrete was developed. This model accounts for the uneven internal temperature distribution inside the concrete structure and thermal deformation incompatibility among the different components of the concrete material, integrating an overall isotropic framework with localized anisotropy. The real-time internal temperature fields of high-altitude concrete bridge piers, measured on-site, were incorporated into the model. By applying thermoelastic mechanics theory and finite element solutions for plane strain problems, the three-dimensional (3D) thermal stresses on the surface layer of high-altitude bridge piers were analyzed, and the cracking risks of these piers were evaluated.

2 Concrete properties and thermoelastic model

2.1 Mix proportions and concrete properties

Ordinary Portland cement P·O 42.5 was used as the cementitious material in the concrete. Overall, 80%–94% of the cement was composed of clinker (C₃S, C₂S, C₃A, and C₄AF) and gypsum, 6%–20% was composed of major mineral additions (GGBS, fly ash, and pozzolanic addition), and 0%–5% was composed of limestone. The physical and mechanical indices of the cement are listed in Table 1. The coarse aggregate was a hybrid mix primarily consisting of crushed diorite with sizes ranging from 5 mm to 20 mm. While the apparent density of diorite in the coarse aggregate was 2 830 kg/m³, the average apparent density of the hybrid coarse aggregate

was 2790 kg/m³. The fine aggregate primarily comprised river sand with an apparent density of 2640 kg/m³ and a fineness modulus of 2.81. The concrete aggregate grades are presented in Table 2. All raw materials were derived from a freeway construction project from Lhasa to Shigaste, which is a typical plateau engineering project.

C35 is the most common strength grade of pier concrete used in high-class highway projects in China. The mixture proportions of C35 concrete are listed in Table 3. This was also derived from the engineering archives of the Lhasa-Shigaste Freeway. The 28 d tensile strength of the concrete was 3.44 MPa. However, because the performance of the surface concrete is lower than that of the inner structural concrete, the tensile strength of the surface concrete is estimated to be 3.01 MPa, according to a previous study^[25]. Mortar sieved from fresh concrete, excluding the coarse aggregate, was molded into specimens for various tests. After prolonged exposure to the natural environment, the densities of the hardened concrete and mortar reduced to 2370 and 2120 kg/m³, respectively.

The interfacial transition zone (ITZ) between the mortar and the coarse aggregate is considered to be a layer of hardened cement paste with a higher water-to-cement (W/C) ratio than that of the matrix. The

W/C ratio of the ITZ was calculated as 0.61 using Equation (1)^[26]. Additionally, according to Garboczi and Bentz^[27], if the coarse aggregate ranges in size from 5 mm to 20 mm with a continuous gradation and occupies 38.8% of the concrete volume, the ITZ accounts for a volume fraction of 1.8%.

$$W / C_{ITZ} = 3.2553W / C - 1.70667(W / C)^2 - 0.4688 \quad (1)$$

Where, W/C represents the water-to-cement ratio of the matrix, while W/C_{ITZ} represents the water-to-cement ratio of the ITZ. Cement paste with a W/C ratio of 0.61 was also molded into specimens for further testing.

A series of mechanical and thermophysical tests were conducted on the diorite aggregate, ITZ cement paste, and mortar. For the coarse aggregate, hardened paste, and mortar, the CTEs were evaluated using 4 mm×4 mm×25 mm prism specimens with a 402 Expedit Supreme dilatometer. The thermal conductivities were measured on 10 mm×10 mm×5 mm specimens using an LFA467 conductometer. Finally, the elastic modulus and Poisson's ratio were measured on 20 mm×20 mm×40 mm prism specimens using a pressure machine and a strain gauge. The mechanical and thermophysical tests of the concrete were conducted in accordance with GB/T 50081–2019^[28]. Table 4 presents the results of these evaluations.

Table 1 Physical and mechanical indexes of the cement

Grade	Standard consistency water content /%	Setting time/min		Flexural strength/MPa		Compressive strength/MPa	
		Initial	Final	3 d	28 d	3 d	28 d
42.5	28	195	320	5.0	7.2	21.6	48.3

Table 2 Gradations of concrete aggregates (accumulated sieve residue)

Aggregate	Mesh size											w/%
	19/mm	16/mm	9.50/mm	4.750/mm	2.360/mm	1.180/mm	0.60/mm	0.300/mm	0.150/mm	0.075/mm	Bottom	
Sand				6.8	18.6	31.4	53.8	74.8	97.8	98.8	100	
Crushed stone	5.5	28.3	73.8	98.4	99.8							

w is mass fraction

Table 3 Mix proportion of C35 concrete

Cement	Water	Sand	Crushed stone	Chemical admixture	Air void	Sum
3050	1000	2640	2790	1100		

Table 4 Mechanical and thermo-physical properties of concrete and concrete components

Parameter	Density/(kg·m ⁻³)	Elastic modulus /GPa	Poisson's ratio	CTE/(10 ⁻⁶ ·K ⁻¹)	Thermal conductivity/ (W·m ⁻¹ ·K ⁻¹)	Specific heat capacity/ (J·kg ⁻¹ ·K ⁻¹)
Diorite aggregate	2 830	67.75	0.22	6.8	1.92	668
ITZ cement paste	1 390	16.08	0.20	17.6	0.58	1081
Mortar	2 120	29.73	0.22	11.8	1.41	977
Concrete	2 370	32.53	0.20	9.7	1.80	901
Concrete	2400	31.5	0.20	10.0	2.00	100

Note: There are suggested values from standards. Elastic modulus, Poisson's ratio, and CTE are from GB/T 50010-2010^[16]. Density, thermal conductivity, and specific heat capacity are from BS EN 12524-2000^[29].

2.2 Thermoelastic model

In structural design, concrete is typically considered as an isotropic material. However, in reality, it is a multi-phase composite. When the deformation of each phase at the interface is incompatible, interface stress is generated, which increases the risk of cracking. To better understand the interface behavior between mortar and coarse aggregates in concrete, a typical coarse aggregate was extracted from the concrete surface and ideally modeled as a spherical shape. It was assumed that the local concrete consists of a peeled aggregate surrounded by a layer of ITZ shell and a layer of mortar shell, forming a sphere-shell-shell composite structure.

For the entire concrete structure, the distribution density of the coarse aggregate was $9.35 \times 10^{-4} \text{ mm}^{-3}$, as calculated using Equation (2)^[27]. Based on the volume fraction and distribution density of the coarse aggregate in the concrete, the average particle volume of the coarse aggregate was determined to be 415 mm^3 using Equation (3). By applying the spherical volume formula (Equation (3)), the average particle radius of the coarse aggregate was calculated to be 4.627 mm .

$$N = \sum_{i=1}^M \frac{9v_{\text{ag}} X_i}{4\pi(R_{i+1}^3 - R_i^3)} \ln \left(\frac{R_{i+1}}{R_i} \right) \quad (2)$$

$$v_0 = \frac{v_{\text{ag}}}{N} = \frac{4\pi r_0^3}{3} \quad (3)$$

Where, N is the distribution density of the coarse aggregate in concrete structure, mm^{-3} ; v_{ag} is the volume fraction of the coarse aggregate in concrete structure, equals to 38.8%; R_i is the i -th sieve radius from bottom to top, mm; X_i is the i -th grader retained percentage of the coarse aggregate; v_0 is the average particle volume of the coarse aggregate, mm^3 ; r_0 is the average particle radius of the coarse aggregate, mm.

The typical aggregate is surrounded by an ITZ layer, with a width of $90 \mu\text{m}$ determined using a micro-hardness tester. The surface area and volume of the aggregate-ITZ composite spheres were calculated using Equations (4) and (5), respectively. The outer layer of the ITZ shell is the mortar shell, and its thickness was determined using Equation (6). By combining Equations (4)–(6), Equation (7) was derived to calculate the mortar shell thickness, which was found to be 2.300 mm .

$$s' = 4\pi(r_0 + h_{\text{ITZ}})^2 \quad (4)$$

$$v' = \frac{4\pi(r_0 + h_{\text{ITZ}})^3}{3} \quad (5)$$

$$s'h_{\text{mo}} = \frac{v_{\text{mo}}}{v_{\text{ag}} + v_{\text{ITZ}}} \times v' \quad (6)$$

$$h_{\text{mo}} = \frac{v_{\text{mo}}}{v_{\text{ag}} + v_{\text{ITZ}}} \times \frac{r_0 + h_{\text{ITZ}}}{3} \quad (7)$$

Where, h_{mo} is mortar shell thickness, mm; h_{ITZ} is ITZ shell thickness (ITZ width), equals to 0.090 mm ; r_0 is the average particle radius of the coarse aggregate, 4.627 mm ; s' is the surface area of the aggregate-ITZ composite sphere, mm^2 ; v' is the volume of the aggregate-ITZ composite sphere, mm^3 ; v_{mo} is the volume fraction of mortar in concrete, equals to 59.4%; v_{ag} is the volume fraction of the coarse aggregate in concrete structure, equals to 38.8%; v_{ITZ} is the volume fraction of the ITZ in concrete structure, equals to 1.8%.

Following these calculations, a model of the bridge pier concrete for surface-level analysis was constructed. The model consists of a concrete unit formed as a sphere-shell-shell composite, including the aggregate, ITZ, and mortar layers, embedded in the surface layer of a bridge pier. Outside this unit, the pier concrete was treated as an isotropic, homogeneous elastic material. A schematic of the model is shown in Fig. 1.

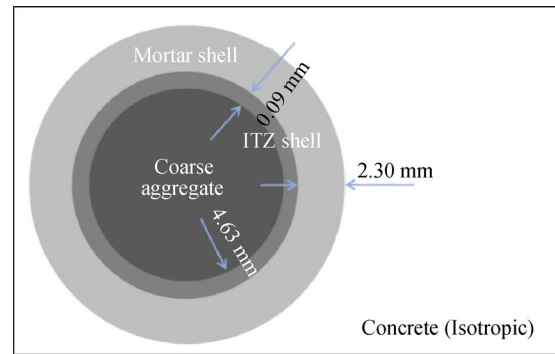


Fig. 1 Thermal-elastic model considering overall structure and local concrete components (sketch)

In high-altitude regions, the cover concrete thickness of bridge piers is generally greater than 45 mm , as specified by various codes including CCES 01–2018^[30], GB/T 50476–2019^[31], and TB 10005–2010^[32]. Consequently, the distance between the reinforcement and pier surface exceeds three times the diameter of the surface concrete unit, as shown in Fig. 1. Furthermore, because the coefficients of thermal expansion of the reinforcement and concrete are similar, the reinforcement can be considered as part of the isotropic concrete shown

in Fig. 1. As a result, the influence of the reinforcement on surface cracking is negligible.

3 Structural deadweight and spatiotemporal temperature distributions

3.1 Bridge structure and structural deadweight

In this study, we selected a bridge on the freeway from Lhasa to Shigatse as an example. According to the design documents, the bridge comprises two sections, each consisting of a prefabricated continuous beam with four box girders made of partially prestressed concrete. The supports for each section include two abutments and three pairs of piers with a span arrangement of 4 m×30 m. The target pier is 1.8 m in diameter and 6 m in height. The analysis focused on the stress field at the cross section located in the middle of the pier. The superficial stresses of the pier concrete before and after the erection of the upper structures were considered. However, structural deadweight was considered only after the upper structures were erected. Based on the bill of engineering quantities, the structural deadweight applied an even load of 1.57 MPa to each pier.

3.2 Spatiotemporal temperature distributions

A test pier was constructed on open ground near the

Yarlung Zangbo River at an altitude of 3800 m. The pier had a diameter of 1.8 m and height of 2 m. Temperature sensors were embedded in the cross-section at a height of 1 m, positioned along the south–north and east–west directions. The embedding depths (distances from the pier surface) were 0, 1.5, 3, 4.5, 6, 7.5, 9, 12, 15, 20, 25, 30, 35, 40, 50, 60, 70, 80 cm, and 90 cm. Temperature measurements inside the pier were initiated one year after construction to eliminate the effects of cement hydration heat and moisture evaporation. During the test, the temperature fields were recorded once an hour on sunny days from May 5–7 over a period of 72 h. The spatiotemporal temperature distribution within the pier is shown in Fig. 2. It should be noted that the study was conducted on sunny days of the dry season because these days exhibit the strongest solar radiation and lowest atmospheric counter radiation. The air and surface temperatures of the pier concrete exhibited the largest daily variations, resulting in significant thermal stresses. On rainy and cloudy days, the daily temperature variations were moderate. Furthermore, temperature variation in high-altitude regions is characterized by large daily differences and small annual differences. Therefore, seasonal temperature variations were not considered.

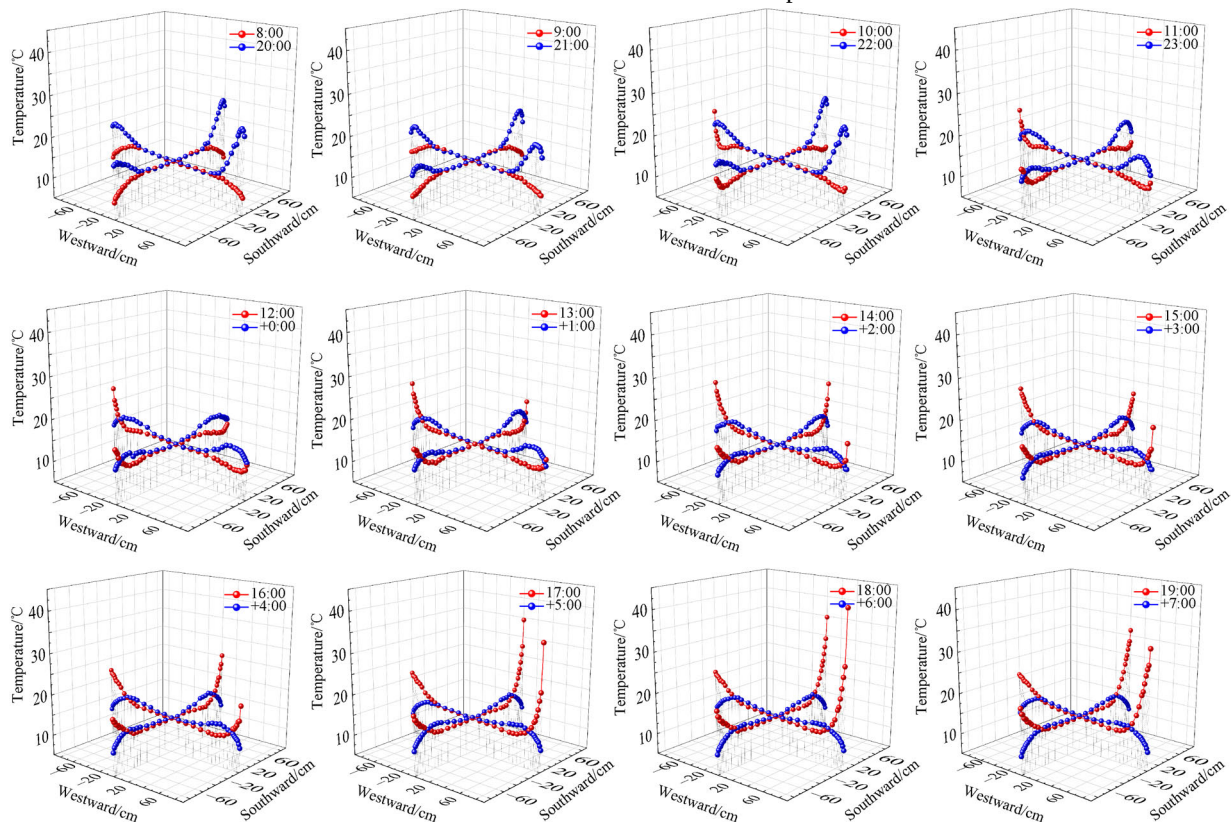


Fig. 2 Spatiotemporal temperature distributions inside the pier

When the surface temperature of the pier concrete was high and the internal temperature was low, compressive stress was generated on the surface owing to the temperature gradient. At 18:00, just before sunset, the temperature in the west quadrant of the pier reached its highest value in the entire temperature field during the day. During this heating phase, the temperature gradient near the pier surface reached its maximum value. The temperature difference between the pier surface and coolest point inside, as well as the temperature difference between the surface and center, were also the largest. Consequently, the highest compressive stress on the pier concrete surface occurred at 18:00.

Conversely, when the surface temperature of the pier concrete was low, and the internal temperature was high, tensile stress was generated on the surface. At 7:00, just before sunrise, the surface temperature across all the quadrants of the pier was at or near its lowest value during the day, with little variation between the quadrants. During this cooling phase, the temperature gradient near the pier surface reached its maximum value. The temperature difference between the pier surface and the hottest point inside, as well as the difference between the surface and the center, were the greatest. Consequently, the highest tensile stress on the pier concrete surface occurred at 7:00.

Figure 2 indicates that the concrete surface temperature of the pier varied between 9.6 °C (minimum) and 42.6 °C (maximum), yielding a daily thermal fluctuation of 33 °C. A preliminary investigation on surface temperatures of in-service bridge piers in Lhasa and Shigatse was conducted before this study, the results of which showed that the largest daily temperature differences were between 30 °C to 37 °C on sunny days during dry seasons. Therefore, the temperature differences observed in this study were representative of those experienced in real-world conditions.

4 Theoretical calculation of superficial stresses

4.1 3-D thermal stress caused by temperature fields

The thermal stress caused by an uneven temperature field was first calculated by treating the concrete as an isotropic material and ignoring the incompatibility among

its various components. Compared with the compressive strength, the tensile stress is of greater importance because most concrete cracks are initiated by tension. Therefore, the calculation process prioritizes the accuracy of the tensile stress. By contrast, the compressive stress can be slightly overestimated without adverse effects because the compressive strength of concrete typically provides a substantial safety margin compared with the actual structural loads in engineering practice. To ensure accuracy, the temperature field inside the pier concrete was assumed to be centrally symmetrical within the horizontal cross-section, meaning that the temperature distribution in any direction was identical to that in the west direction.

Unlike the thermal shock problems commonly encountered in mechanical engineering, the temperature changes in pier concrete exposed to natural environments are gradual and can be treated quasistatically. Under this quasistatic assumption, the temperature field T inside the pier concrete is only a function of the radius r of each point, presented as $T=T(r)$, and it is considered constant along the vertical direction. For a hollow cylinder with inner radius r_i and outer radius r_0 , the stress distribution in the cylindrical coordinate system is described by Equations (8)–(10), which are derived from the theory of elasticity^[11].

$$\sigma_r' = \frac{E\alpha}{1-\mu} \left[\frac{1}{r_0^2 - r_i^2} \left(1 - \frac{r_i^2}{r^2} \right) \times \int_{r_i}^{r_0} T r dr - \frac{1}{r^2} \int_{r_i}^{r_0} T r dr \right] \quad (8)$$

$$\sigma_\theta' = \frac{E\alpha}{1-\mu} \left[\frac{1}{r_0^2 - r_i^2} \left(1 + \frac{r_i^2}{r^2} \right) \times \int_{r_i}^{r_0} T r dr + \frac{1}{r^2} \int_{r_i}^{r_0} T r dr - T \right] \quad (9)$$

$$\sigma_z' = \frac{E\alpha}{1-\mu} \left[\frac{2}{r_0^2 - r_i^2} \times \int_{r_i}^{r_0} T r dr - T \right] \quad (10)$$

Where, σ_r' is the radial stress, MPa; σ_θ' is the tangential stress, MPa; σ_z' is the vertical stress, MPa; E is the elastic modulus of concrete, MPa; μ is the Poisson's ratio of concrete; α is CTE of concrete, °C⁻¹; R is the distance from the point to the axis, m; R_i is the inner radius, m; T is the temperature of the point, °C. For the surface of solid pier in this study ($r_i=0$, $r=r_0=90$ cm), T is the measured surface temperature T_0 . Therefore, in cylindrical coordinates, the radial stress σ_r' is 0, and the tangential stress σ_θ' and vertical stresses σ_z' are expressed

more simply as Equation (11). Using these equations, the superficial stresses on the pier concrete at various times were obtained, as listed in Table 5. When the pier concrete was treated as an isotropic material, the maximum tensile stress on the surface was 1.56 MPa,

whereas the maximum compressive stress was 10.47 MPa. These stresses occurred before sunrise and sunset, respectively, on sunny days.

$$\sigma'_\theta = \sigma'_z = \frac{E\alpha}{1-\mu} \left[\frac{2}{r_0^2} \times \int_0^{r_0} Trdr - T_0 \right] \quad (11)$$

Table 5 Superficial stresses on pier concrete at different times (isotropic assumption)

Time	$T_0 / ^\circ\text{C}$	$(\sigma'_\theta = \sigma'_z) / \text{MPa}$	Time	$T_0 / ^\circ\text{C}$	$(\sigma'_\theta = \sigma'_z) / \text{MPa}$	Time	$T_0 / ^\circ\text{C}$	$(\sigma'_\theta = \sigma'_z) / \text{MPa}$
8:00	9.3	1.09	16:00	20.9	-2.84	0:00	13.7	0.59
9:00	9.8	0.88	17:00	35.3	-8.05	1:00	13.0	0.84
10:00	11.4	0.26	18:00	42.6	-10.47	2:00	12.3	0.92
11:00	12.6	-0.18	19:00	33.6	-6.54	3:00	12.3	0.92
12:00	13.3	-0.43	20:00	23.7	-2.72	4:00	10.9	1.30
13:00	14.5	-0.81	21:00	18.6	-0.85	5:00	10.4	1.42
14:00	18.3	-2.19	22:00	14.9	0.35	6:00	10.0	1.48
15:00	21.9	-3.39	23:00	14.4	0.49	7:00	9.6	1.56

4.2 Superficial thermal stress caused by deformation incompatibility

Because of the differing CTEs of mortar and coarse aggregates, stress is generated at their interface whenever a temperature change occurs, whether caused by hydration or environmental effects after the concrete hardens. However, subtle defects and long-term creep cause the stress to relax over time. Consequently, the zero-force temperature of the interface was not considered to be the temperature during the initial or final setting of the concrete. In high-altitude regions, the daily temperature difference on the pier concrete surface was significant, whereas the daily average temperature changed gradually over the course of a month. Therefore,

in this study, the zero-force temperature of the interface between the mortar and concrete was considered as the daily average temperature. In the model depicted in Fig. 2, when the surface temperature deviated from the daily average, part of the thermal deformation expected in the mortar shell was constrained by the surrounding isotropic concrete. This constraint induces local stress in addition to the stresses analyzed in Section 4.1. Using a spherical coordinate system with the aggregate center shown in Fig. 1 as the origin, the stress at the tangent point between the mortar shell and the side of the pier was calculated using Equation (12) and is presented in Table 6.

$$\sigma''_\theta = \sigma''_\varphi = \frac{E_m \Delta T}{1 - \mu_m} (\alpha_c - \alpha_m) \quad (12)$$

Table 6 Superficial stresses on pier concrete at different times (ignoring temperature gradients)

Time	$\Delta T / ^\circ\text{C}$	$(\sigma''_\theta = \sigma''_\varphi) / \text{MPa}$	Time	$\Delta T / ^\circ\text{C}$	$(\sigma''_\theta = \sigma''_\varphi) / \text{MPa}$	Time	$\Delta T / ^\circ\text{C}$	$(\sigma''_\theta = \sigma''_\varphi) / \text{MPa}$
8:00	-7.7	0.62	16:00	3.9	-0.31	0:00	-3.3	0.26
9:00	-7.2	0.58	17:00	18.3	-1.46	1:00	-4.0	0.32
10:00	-5.6	0.45	18:00	25.6	-2.05	2:00	-4.7	0.38
11:00	-4.4	0.35	19:00	16.6	-1.33	3:00	-4.7	0.38
12:00	-3.7	0.30	20:00	6.7	-0.54	4:00	-6.1	0.49
13:00	-2.5	0.20	21:00	1.6	-0.13	5:00	-6.6	0.53
14:00	1.3	-0.10	22:00	-2.1	0.17	6:00	-7.0	0.56
15:00	4.9	-0.39	23:00	-2.6	0.21	7:00	-7.4	0.59

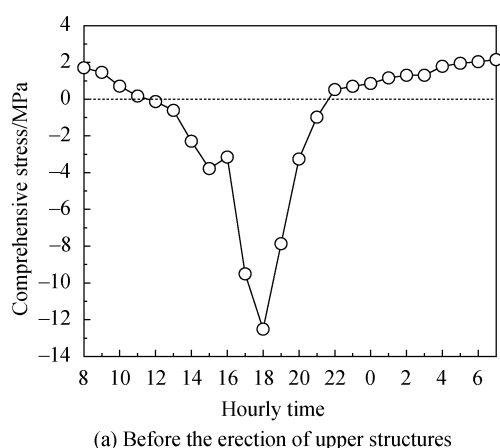
Where, σ''_θ is the azimuthal tangential stress in spherical coordinates, namely the tangential stress in cylindrical coordinates, MPa; σ''_φ is height-angle tangential stress in spherical coordinates, namely the vertical stress in

cylindrical coordinates, MPa; E_m is the elastic modulus of mortar, MPa; μ_m is the Poisson's ratio of mortar; α_m is the CTE of mortar, $^\circ\text{C}^{-1}$; α_c is the CTE of concrete, $^\circ\text{C}^{-1}$; ΔT is the difference between real-time surface temperature T_0

and daily average of T_0 , °C. According to Table 6, when the temperature gradient in the pier concrete is ignored, the maximum tensile stress on the surface in the tangential and vertical directions was 0.62 MPa and 0.59 MPa, respectively. Similarly, the maximum compressive stress on the surface was 2.05 MPa, which occurred before sunrise and sunset on sunny days.

4.3 Superficial stress considering temperature fields and deformation incompatibility

The thermal stress on the pier concrete surface was



obtained by superimposing the stresses caused by the uneven distribution of the internal temperature field and those caused by the incompatible thermal deformation among the different components in the surface concrete, as listed in Tables 5 and 6. In addition, after the erection of the upper structures, the vertical thermal stress must be combined with the uniform stress resulting from the structural deadweight (1.57 MPa), as described in Section 3.1. The comprehensive superficial stresses on the pier concrete surface at various times are shown in Fig. 3.

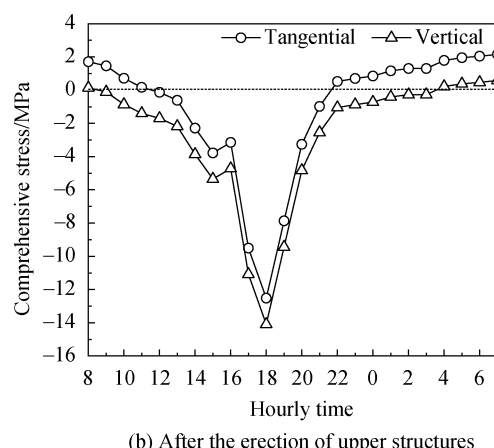


Fig. 3 Comprehensive superficial stresses on pier concrete surface at various times

Fig. 3a shows a combination of the results presented in Tables 5 and 6, which are presented as a visual line chart. Before the erection of the upper structures, the absolute values of the tangential and vertical stresses were the same; therefore, only one curve was observed. From 22:00 to 8:00, the pier concrete surface was in tension, whereas from 11:00 to 22:00, the pier concrete surface was in compression. The surface of the pier concrete was subjected to biaxial forces of equal magnitude with a maximum compressive stress of 12.52 MPa and maximum tensile stress of 2.15 MPa, respectively at 18:00 and 8:00. According to Appendix C of GB/T 50010-2010^[16], the tensile strength of concrete under biaxial tension is 0.79 times the uniaxial tensile strength. In this study, the maximum tensile stress in either direction (2.15 MPa) was close to the biaxial tensile strength of the concrete ($3.01 \times 0.79 = 2.38$ MPa). According to the fatigue defined using Equation (13) provided by Ge^[6], the concrete was predicted to crack after 21 d of temperature cycling.

$$S = 0.967 - 0.048 \lg N = 2.15 / 2.38 \quad (13)$$

Where, S is the stress level; N is the number of cycles. Moreover, if humidity-induced stress is added on top of this^[33], the tensile stress may approach or even exceed the concrete's tensile strength, thereby posing a significant risk of cracking.

In Fig. 3b, the tangential and vertical stresses no longer coincide because the upper structures have been erected. Compared with Fig. 3a, the curve of the tangential stress is unchanged, whereas the curve of the vertical stress is translated downwards by 1.57 MPa due to the structural deadweight. Therefore, the maximum tangential compressive stress remained 12.52 MPa, whereas the maximum vertical compressive stress increased to 14.09 MPa, both of which are well below the compressive strength of the C35 concrete. Additionally, the maximum tangential tensile stress was 2.15 MPa, and the maximum vertical tensile stress was 0.58 MPa. The ratio of the maximum vertical tensile stress to the uniaxial tensile strength was $0.58 / 3.01 = 0.19$. According to Appendix C of GB/T 50010-2010^[16], the tensile strength in the other direction is 1.02 times the uniaxial

tensile strength, namely $3.01 \times 1.02 = 3.07$ MPa, which is clearly higher than the tangential tensile stress of 2.15 MPa. According to Equation (13), the concrete is predicted to crack after 3.6×10^5 d of temperature cycling, corresponding to 1,000 years. Therefore, even when accounting for fatigue effects^[34–36], stresses are unlikely to cause cracking of the pier concrete surface.

Furthermore, the sunshade effect of the upper structures reduces the amplitude of the temperature cycles on the pier surface and smooths the temperature field inside the pier, further reducing the thermal stress on the concrete surface.

4.4 Superficial stress on concrete with a single-type coarse aggregate

In addition to concrete with a hybrid coarse aggregate primarily composed of diorite, concrete made with a single-type coarse aggregate was studied to evaluate the effect of the CTE of the aggregates on the thermal stress on the pier concrete surface. This analysis provides a basis for the selection of suitable coarse aggregates for bridge piers at high altitudes. The coarse aggregates considered included diorite, limestone, and basalt, and their mechanical and thermophysical properties are listed in Table 7.

Table 7 Mechanical and thermo-physical properties of aggregates

Aggregate	Apparent density/ ($\text{g} \cdot \text{cm}^{-3}$)	Elastic modulus / GPa	Bulk modulus / GPa	CTE/ ($10^{-6} \cdot \text{K}^{-1}$)	Thermal conductivity/ ($\text{W} \cdot \text{m}^{-1} \cdot \text{K}^{-1}$)	Specific heat capacity/ ($\text{J} \cdot \text{g}^{-1} \cdot \text{K}^{-1}$)	Thermal diffusivity/ ($\text{mm}^2 \cdot \text{s}^{-1}$)
Diorite	2.83	67.75	38.27	6.82	1.92	0.67	1.02
Limestone	2.67	63.90	38.04	4.80	2.45	0.84	1.09
Basalt	2.86	65.80	39.17	7.20	2.25	0.82	0.96

Because the three types of aggregates had similar elastic moduli and thermal diffusivities, the three groups of pier concrete with single-type aggregates also exhibited similar elastic moduli and temperature distributions. Therefore, the primary factor influencing the differences in the thermal stress was the CTE of the concrete. Based on the mixture proportions and the Kerner model^[37], the CTEs of concrete with diorite, limestone, and basalt were calculated to be 9.65×10^{-6} ,

8.72×10^{-6} , and $9.83 \times 10^{-6} \text{ K}^{-1}$. Using Equations (11) and (12), the maximum thermal stresses on the pier concrete surface were determined, as illustrated in Fig. 4. Each stress column in the figure consists of two parts. The left part represents the portion caused by the overall uneven temperature distribution, which was calculated using Equation (11). The right part is the portion caused by the local incompatible thermal deformation, calculated according to Equation (12).

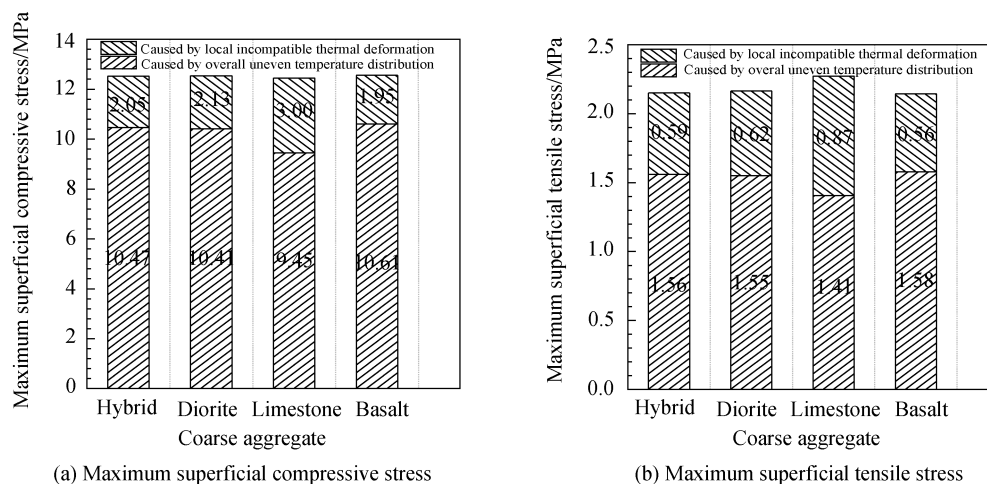


Fig. 4 Maximum superficial stresses on the surface of pier concrete with single-type coarse aggregate

From Fig. 4, it is evident that selecting coarse aggregates with a higher CTE, such as basalt, increases the thermal stress caused by the overall uneven

temperature distribution within the pier concrete while reducing the thermal stress caused by local incompatible thermal deformation. Conversely, the use of a coarse

aggregate with a lower CTE, such as limestone, has the opposite effect. As shown in Fig. 4a, although a higher CTE of the coarse aggregate slightly increased the maximum compressive stress, the differences among the three groups of concrete were minimal and could be ignored. Specifically, the maximum compressive stresses on the pier concrete surface were 12.54, 12.45 MPa, and 12.56 MPa when using diorite, limestone, and basalt, respectively. By contrast, Fig. 4b shows that a lower CTE of the coarse aggregate results in a greater maximum tensile stress on the pier concrete surface. For example, when using limestone, which has a low CTE, the maximum tensile stress on the pier concrete surface is 2.28 MPa, compared to 2.17 MPa when using diorite and 2.14 MPa when using basalt. Consequently, although using a coarse aggregate with a lower CTE had a negligible effect on the maximum compressive strength, it slightly increased the tensile stress on the surface, leading to a marginally higher risk of cracking. However, this conclusion is based on the assumption that the pier is a statically determinate structure modeled as a cantilever beam rotated by 90°.

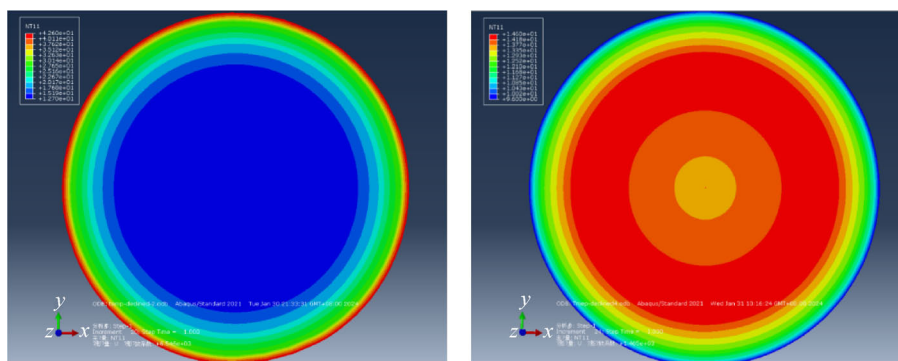
5 Finite element computation of integral stress fields

5.1 Model construction considering the overall structure and local material composition

From Equations (11) and (12), it is evident that the

tangential stress is numerically equal to the vertical stress before the erection of upper structures. Consequently, a 3-D problem can be simplified into a two-dimensional (2-D) problem. Based on the plane-section assumption, it is further assumed that the vertical deformation at each point within the cross section is fully constrained. The plane strain problem was solved using an ABAQUS finite element model, which comprehensively accounted for the thermal stresses caused by the overall uneven temperature distribution within the pier and the local incompatible thermal deformation among its concrete components.

In the initial state, with no external forces acting on the pier concrete surface, the temperature was set at 17 °C, which corresponds to the average temperature of the west-most surface, as indicated in Table 6. Based on this assumption, the temperature field was input as follows. First, the characteristic lines at different depths were identified, and the corresponding measured temperature loads were applied to these lines. Subsequently, the circular cross section of the pier was mapped onto a plane Cartesian coordinate system, with the center of the section located at the coordinate origin and the west quadrant point positioned along the positive X-axis. The temperature fields corresponding to the maximum stresses on the pier concrete surface are shown in Fig. 5.



(a) Temperature field at maximum compressive stress (b) Temperature field at maximum tensile stress
Fig. 5 Temperature fields at maximum superficial stresses in pier concrete

The interactions between the concrete components and the meshing of the pier concrete are illustrated in Fig. 6. Tie connections were established among the coarse aggregate, ITZ shell, mortar shell, and isotropic concrete (Fig. 2) to ensure that the interactions were fully

consolidated, thereby preventing damage or separation (Fig. 6a). After these bindings, the pier cross-section was discretized into a finite element mesh. The global mesh spacing was set to 8 mm, and an additional mesh refinement was applied to the surface composite unit for

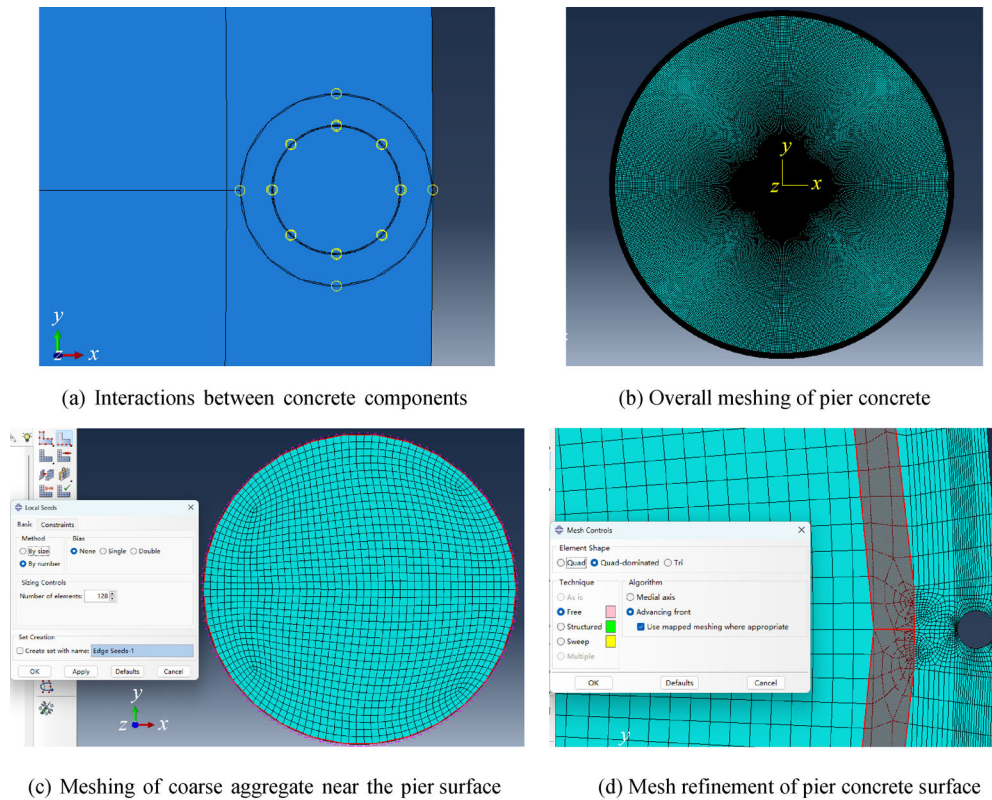


Fig. 6 Interactions between concrete components and the meshing of pier concrete

greater detail (Fig. 6b). The circumference of the pier section was divided into 128 elements. To improve the compatibility of this irregular geometry, the third layer of the temperature field employed a mixed mesh primarily consisting of quadrilateral elements with a few triangular elements. Quadrilateral elements were used exclusively in the other areas of the model. The mesh comprised 82 471 elements and 83 105 nodes. Among the elements, 82 419 were CPE4RT elements (four-node plane temperature-displacement coupled elements with reduced integration and hourglass control), whereas the remaining 52 were CPE3T elements (three-node plane temperature-displacement coupled elements).

5.2 Model solutions based on overall unevenness and local incompatibility

Based on the boundary conditions of thermoelastic mechanics, the radial stress on the pier concrete surface is zero, as dictated by Newton's second law. Under an uneven temperature field, only tangential and vertical stresses exist and their magnitudes are equal. Thus, for planar analysis, it is sufficient to focus solely on the tangential stress. It is important to note that the finite element simulations performed with ABAQUS use a Cartesian coordinate system, unlike the cylindrical

(Section 4.1) or spherical (Section 4.2) coordinate systems typically employed for axisymmetric or axisymmetric problems in elastic mechanics. For points in the quadrant along the positive X -axis, the X -axis in the Cartesian coordinate system corresponds to the radial direction in the cylindrical coordinate system, whereas the Y -axis corresponds to the tangential direction. Therefore, when analyzing the superficial stress, only the stress in the Y -axis direction must be computed, as shown in Figs. 7 and 8. Specifically, Fig. 7 shows the stress distribution when the compressive stress reaches its maximum value. Figure 8 shows the stress distribution when the tensile stress reaches its maximum.

The finite element simulation results indicated that the maximum compressive stress on the pier concrete surface was 11.72 MPa, whereas the maximum tensile stress was 2.10 MPa. These results are approximately consistent with the theoretical calculations presented in Section 4.3, which predicted a maximum compressive stress of 12.52 MPa and a maximum tensile stress of 2.15 MPa under temperature effects. This consistency provides mutual verification. Both the theoretical calculations and finite element simulation results demonstrated that diurnal temperature variations in

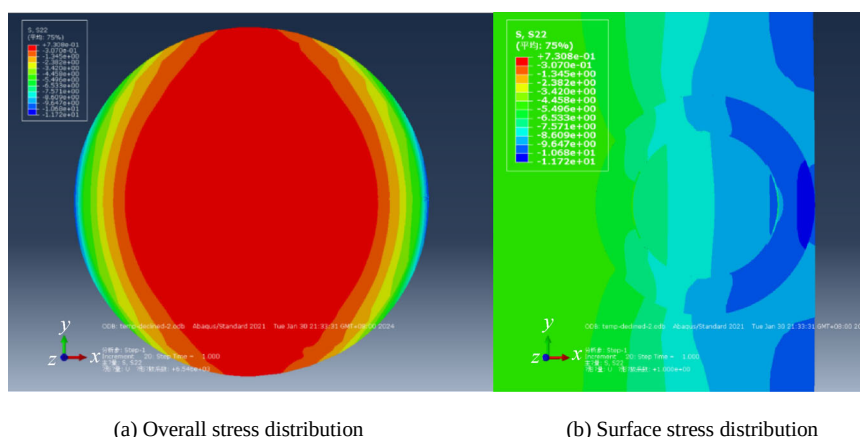


Fig. 7 Stress distribution along the direction of Y-axis at maximum compressive stress on pier concrete

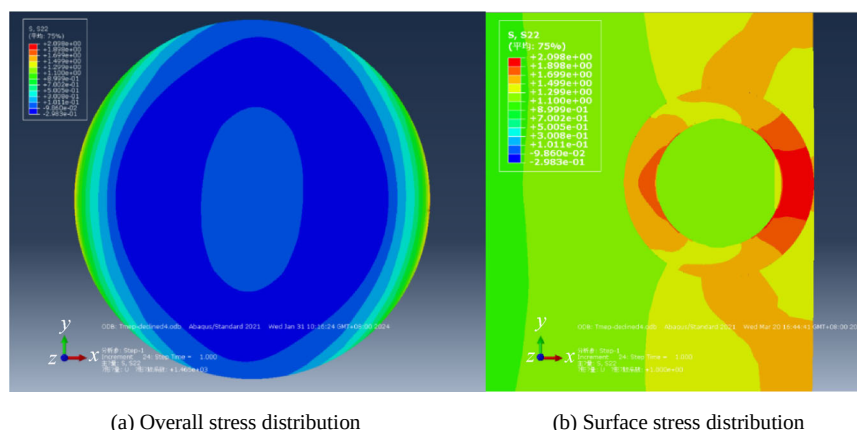


Fig. 8 Stress distribution along the direction of Y-axis at maximum tensile stress on pier concrete

high-altitude environments generate significant tensile stresses on the pier concrete surface, contributing to the prevalence of surface cracking.

6 Research limitations and prospects

This study innovatively analyzed the material behavior in mass concrete, assuming that the concrete is isotropic overall with localized anisotropy. Based on this assumption, a concrete unit with a single representative coarse aggregate was embedded in the surface layer of a bridge pier. Concrete is a complex multiphase material with significant internal structural inhomogeneity, such as aggregate distribution and porosity, which can affect the distribution of temperature-induced stress. Although it is impossible to analyze pier concrete comprising millions of coarse aggregates in detail, a larger system involving multiple coarse aggregates embedded in the surface layer of pier concrete will be investigated in the future. More accurate and reliable models and calculations may be obtained by considering the inhomogeneity of the internal structure of this system.

In addition, this study focused only on the long-term cracking risk, which is mainly affected by temperature variations. However, the risk of cracking at an early age is always high in engineering practices. At early ages, the cracking behavior is affected by both temperature and humidity, with complicated mechanisms. The effects of humidity will be further researched to achieve a more complete life-cycle cracking analysis of pier concrete.

7 Conclusions

This study utilized concrete from an actual engineering project in a high-altitude region as an example to calculate the thermophysical properties of this system and establish a thermoelastic model characterized by overall isotropy and local anisotropy. Using a solid pier as the object of analysis, temperature data were collected in four directions at various depths over 24 h on sunny days. By incorporating experimentally obtained temperature fields and the structural deadweight into the thermoelastic model, the superficial stresses of the pier were analyzed based on thermoelastic mechanics and

finite element computation. The main conclusions are as follows.

1) During the experimental period, the lowest and highest temperatures on the bridge pier in the high-altitude region were 9.6 °C and 42.6 °C, respectively. These occurred before sunrise and sunset on sunny days, respectively, corresponding to the local maximum temperature gradients during the surface heating and cooling stages, as well as the maximum temperature difference between the surface and center during these stages.

2) The thermal stress in the pier concrete arose from the uneven distribution of the temperature field within the structure and the incompatible thermal deformation between the mortar and coarse aggregate in the concrete. When modeling, the concrete components in the surface layer should be locally distinguished, whereas the main structure can be treated as isotropic.

3) Before the erection of the upper structures, the maximum compressive stress on the pier concrete surface owing to temperature effects was 12.52 MPa, and the maximum tensile stress was 2.15 MPa. In both cases, the stress magnitudes were identical in the tangential and vertical directions. The maximum tensile stress was close to the tensile strength of C35 concrete under this biaxial stress state.

4) After the erection of the upper structures, the maximum tangential tensile stress on the pier concrete surface remained at 2.15 MPa, whereas the maximum vertical tensile stress decreased to 0.58 MPa. Both were significantly lower than the tensile strength of the C35 concrete in the biaxial tensile state. Therefore, the cracking risk of pier concrete in high-altitude regions primarily occurs before the erection of the upper structures.

5) The use of coarse aggregates with a lower CTE reduced the thermal stress caused by the uneven temperature distribution inside the pier; however, it could not offset the increase in thermal stress caused by incompatible thermal deformation among the concrete components. The use of limestone as a coarse aggregate slightly increased the maximum tensile stress on the pier concrete surface.

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Analysis of Temperature-Induced Surface Stress in Concrete Bridge Piers in High-Altitude Regions

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Extended Abstract

Introduction The predominant characteristics of high-altitude climates include low air pressure, low humidity, and large diurnal temperature fluctuations. In practical engineering scenarios in high-altitude regions, many pier surface cracks only appear several months after erection, and cyclic thermal stress is identified as the main trigger for such cracking. The thermal stress in concrete structures has been investigated for decades but remains incompletely understood. Structural engineers typically regard concrete as an isotropic material and calculate the thermal stress using code-specified coefficients of thermal expansion (CTEs) along with temperature conditions and constraints. Because the CTE of hardened cement paste is more than twice that of many aggregates, reducing the CTE of coarse aggregates can further exacerbate the thermal deformation incompatibility between the coarse aggregate and mortar matrix. In this paper, a comprehensive thermal-elastic mechanics model for pier concrete was developed to analysis the temperature-induced surface stress.

Methods A series of mechanical and thermophysical tests were conducted on the diorite aggregate, ITZ cement paste, and mortar, and concrete. A test pier was constructed on open ground near the Yarlung Zangbo River at an altitude of 3800 m. The pier had a diameter of 1.8 m and height of 2 m. Temperature sensors were embedded in the cross-section at a height of 1 m, positioned along the south–north and east–west directions. The embedding depths (distances from the pier surface) were 0, 1.5, 3, 4.5, 6, 7.5, 9, 12, 15, 20, 25, 30, 35, 40, 50, 60, 70, 80 cm, and 90 cm.

A model of the bridge pier concrete for surface-level analysis was constructed. The model consists of a concrete unit formed as a sphere-shell-shell composite, including the aggregate, ITZ, and mortar layers, embedded in the surface layer of a bridge pier. Outside this unit, the pier concrete was treated as an isotropic, homogeneous elastic material. The real-time internal temperature fields of

high-altitude concrete bridge piers, measured on-site, were incorporated into the model. By applying thermoelastic mechanics theory and finite element solutions for plane strain problems, the three-dimensional thermal stresses on the surface layer of high-altitude bridge piers were analyzed.

Results and discussion During the experimental period, the lowest and highest temperatures on the bridge pier in the high-altitude region were 9.6 °C and 42.6 °C, respectively. These occurred before sunrise and sunset on sunny days, respectively, corresponding to the local maximum temperature gradients during the surface heating and cooling stages, as well as the maximum temperature difference between the surface and center during these stages. The thermal stress on the pier concrete surface was obtained by superimposing the stresses caused by the uneven distribution of the internal temperature field and those caused by the incompatible thermal deformation among the different components in the surface concrete

Before the erection of the upper structures, the absolute values of the tangential and vertical stresses were the same; therefore, only one curve was observed. From 22:00 to 8:00, the pier concrete surface was in tension, whereas from 11:00 to 22:00, the pier concrete surface was in compression. The surface of the pier concrete was subjected to biaxial forces of equal magnitude with a maximum compressive stress of 12.52 MPa and maximum tensile stress of 2.15 MPa, respectively at 18:00 and 8:00. According to the fatigue equation, the concrete was predicted to crack after 21 d of temperature cycling. Moreover, if humidity-induced stress is added on top of this, the tensile stress may approach or even exceed the concrete's tensile strength, thereby posing a significant risk of cracking.

After the erection of upper structures, the tangential and vertical stresses no longer coincide because the upper structures have been erected. The curve of the tangential stress is unchanged, whereas the curve of the vertical stress is translated downwards by 1.57 MPa due to the structural deadweight. Therefore, the maximum tangential compressive stress remained 12.52 MPa, whereas the maximum vertical compressive stress increased to 14.09 MPa. Additionally, the maximum tangential tensile stress was 2.15 MPa, and the maximum vertical tensile stress was 0.58 MPa. According to Appendix C of GB/T 50010 and the fatigue equation, stresses are unlikely to cause cracking of the pier concrete surface.

Although a higher CTE of the coarse aggregate slightly increased the maximum compressive stress, the differences among the three groups of concrete were minimal and could be ignored. Specifically, the maximum compressive stresses on the pier concrete surface were 12.54, 12.45 MPa, and 12.56 MPa when using diorite, limestone, and basalt, respectively. By contrast, a lower CTE of the coarse aggregate results in a greater maximum tensile stress on the pier concrete surface. For example, when using limestone, which has a low CTE, the maximum tensile stress on the pier concrete surface is 2.28 MPa, compared to 2.17 MPa when using diorite and 2.14 MPa when using basalt.

The finite element simulation results indicated that the maximum compressive stress on the pier concrete surface was 11.72 MPa, whereas the maximum tensile stress was 2.10 MPa. These results are approximately consistent with the theoretical calculations. This consistency provides mutual verification.

Conclusions Surface cracking in pier concrete occurs predominantly before the erection of upper structures. Under sunny conditions, the orthogonal decomposition of the superficial stress revealed that the maximum compressive stress during the day was approximately 12.52 MPa, whereas the maximum tensile stress was approximately 2.15 MPa. This tensile stress approached the tensile strength of the C35 concrete under biaxial tension. The risk of cracking increased significantly when humidity-induced stress was considered. After the erection of upper structures, the maximum tangential tensile stress on the pier surface remained at 2.15 MPa while the maximum vertical stress decreased to 0.58 MPa, both of which are well below the tensile strength of C35 concrete under biaxial tension. Although the use of coarse aggregates with a lower coefficient of thermal expansion reduced the tensile stress induced by temperature gradients, it increased the stress owing to material deformation incompatibility, leading to a slight increase in the maximum tensile stress on the pier concrete surface.

Keywords concrete; bridge pier; thermal stress; temperature field; deformation incompatibility; coefficient of thermal expansion