Research Article

Effects of Sudden Temperature Drop on Stress at Rapidly Repaired Bonding Interface of Pavement

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The numerical simulations were employed to establish an edge-corner repair model with magnesium phosphate cement (MPC) concrete as the repair material and ordinary Portland cement concrete as the old pavement. After the simulation of repair construction by using MPC concrete with different coarse aggregates, the effect of sudden temperature drop during the stable stage of hydration reaction on the stress distribution at each bonding interface was analyzed. The numerical calculations indicate that the sudden temperature drop led to temperature-induced stress on the bonding interfaces. The stress distribution at each bonding interface was obtained and the maximum principal stress at each bonding interface was at the intersection angle of three bonding interfaces. The relationship between the temperature and stress at each bonding interface was found when different coarse aggregates were used to prepare the repairing material. Also, the effect of different coarse aggregates on the bonding interface of the repairing material was obtained when basalt was the coarse aggregate of old concrete. The stability of bonding surface from best to worst was as follows: basalt > limestone > granite > conglomerate > sandstone > quartzite.

1. Introduction

In recent years, owing to its high rigidity, good stability, and convenient construction, cement concrete has been extensively used in the runway projects of airports [1]. However, apart from these advantages, cement concrete has prominent disadvantages. As a relatively brittle material, cement concrete has relatively poor tensile deformation, and its flexural, tensile, and fatigue properties are also relatively low. Especially in areas with large variations in temperature and humidity in day and night, or within four seasons, the concrete pavement is more prone to damage due to continuous impact of natural environment and repeated load [2]. For a concrete pavement, the weakest areas are the edges and corners. Hence, it is necessary to repair the damaged areas in time, which can not only prevent further expansion of damage but also extend the working life of concrete pavement [3].

Magnesium phosphate cement (MPC) is a relatively novel type of rapid-repair material for the concrete pavement. It has several advantages such as short setting time, high strength, remarkable wear resistance, strong temperature adaptability, and excellent volume stability [4, 5]. In the early stage of repair with magnesium phosphate cement, the hydration reaction as neutralization reaction releases a large amount of heat and heat further accelerates the hydration reaction. Hence, in a short time, the repair concrete achieves decent early-stage mechanical properties [6, 7]. However, due to thermal expansion and contraction, microexpansion and deformation occur on the repair concrete in the early hydration process. All the interfaces of the repair concrete impose compressive stress on the old concrete, which is favorable for the interfacial bonding between the repair concrete and old concrete. When the repair construction is completed and most of the hydration heat has been released, the repair concrete slowly cools down until it reaches the ambient temperature.

At this time, the strength and bonding strength of the repair concrete continue to increase slowly. Hence, the interfacial stress between the repair concrete and old

concrete pavement is close to zero. If the ambient temperature is constant, the strength and bonding strength of the repair concrete continue to increase until they reach a certain level. Therefore, the bond strength becomes significantly greater than the temperature stress caused by the changes induced by the external environment, and the repair interface becomes stable. However, in practical applications, the temperature is not constant and some areas have larger temperature variation between day and night. The large temperature variation causes a certain tensile stress between the new and old concrete parts. When the tensile stress is greater than the current interfacial bonding strength, it causes damage of the bonding interface that is in the strength-growth period [8, 9]. Therefore, in the rapid-repair project under extreme weather conditions, it is essential to study the effect of temperature load on the stress distribution of the repaired bonding interface.

At present, the research on the effect of temperature on concrete is mostly focused on the cracking caused by the high internal temperature of the newly poured large-scale concrete [10], on the variations of the bonding interface between the two reinforced concretes due to temperature variation [11], on the bonding conditions between the concrete and the in-concrete rebar under high temperature [12], on the effect of high temperature on the bonding performance of concrete after repairing [13, 14], and so forth. However, there are few studies on the stress distribution of bonding interface between the new and old concretes under the conditions of drastic drop in temperature. Therefore, this paper discusses the repair of the ordinary Portland cement concrete pavement using the MPC concrete. When the hydration becomes stable and the strength continues to increase, the stress distribution is adopted as the research object and the effect of environmental temperature change on the stress distribution of each bonding interface is investigated. The study provides a theoretical basis for the applications of magnesium phosphate cement concrete in the repairing engineering under extreme climate conditions.

2. Theory

In the rapid-repair project, the cracking of repaired bonding interface occurs due to large external temperature variation. The fundamental reason is that different linear expansion coefficients of the two materials lead to the inconsistency in the deformation between the rapid-repair material and the old concrete material when the external environment temperature changes greatly [15]. Therefore, with regard to the repaired concrete pavement, it is necessary to study the linear expansion coefficients of rapid-repair materials and old concrete material. The linear expansion coefficient of concrete is primarily determined by that of the coarse aggregates. When the new and old concrete coarse aggregates are the same, the temperature variation leads to the consistent volume change of the two concretes, and hence there is no interfacial stress. When the two concrete coarse aggregates are different, the stress is induced at the interface with the variation of temperature. Therefore, to analyze the

bonding interface between the repair concrete and the old concrete, the temperature variation and the linear expansion coefficient of the old and new concretes need to be considered. The relationship between the linear expansion coefficients (α_l), the temperature (t), the length at $t°C$ (l_t), and the length at $0°C$ (l_0) is as follows:

$$\alpha_l = \frac{l_t - l_0}{l_0 t}, \tag{1}$$

Therefore, herein, ordinary Portland cement concrete is selected as the old concrete, basalt as coarse aggregate, and magnesium phosphate cement concrete as the fast-repair material. Different coarse aggregates were used to prepare the systems consisting of the old concrete pavement and repair concrete. The effect of temperature drop on the stress distribution of the repaired bonding interface was studied after the hydration heat release tended to stabilize.

3. Establishment of Repair Model

In the actual use of concrete pavement of airport, owing to various influencing factors in the early pouring and later use, the edges and corners are the weakest areas of the pavement. In other words, the edges and corners are most likely to be damaged under normal circumstances. Based on this phenomenon, the pavement corner was designed as the damaged position of the concrete pavement (see Figure 1). The dimension of the old concrete pavement was $5\,m \times 5\,m \times 0.3\,m$, while the dimension of the repair concrete was $0.7\,m \times 0.7\,m \times 0.07\,m$. There were three bonding interfaces between the repair concrete and the old concrete pavement: two vertical bonding interfaces A and B and one horizontal bonding interface C.

After the rapid-repair construction was completed, the hydration reaction tended to stabilize. In this stage, the established model primarily considered the impact of sudden temperature drop on the bonding interface between the new and old concretes. Based on the above requirements, the ABAQUS simulation software was selected for simulation and finite element calculation. This is because ABAQUS software has certain advantages for thermodynamic simulations, in particular the simulation of heat conduction, thermocouple analysis, and rock and soil mechanics analysis. Furthermore, ABAQUS has higher precision than ANSYS.

The model structure was mainly divided into two regions, namely, the concrete pavement and the repair concrete. Based on the overall model of the concrete pavement, three sections were set at its corners: two vertical sections and one horizontal section. The intersection of the three interfaces was replaced with repair concrete, as shown in structure in Figure 2. The white overall is the old concrete pavement, while the red frame indicates the location of the repair concrete.

The old concrete pavement material was 42.5 R ordinary Portland cement concrete, while the repair concrete was MPC concrete. Because the hydration and coagulation process of MPC concrete are extremely rapid, decent
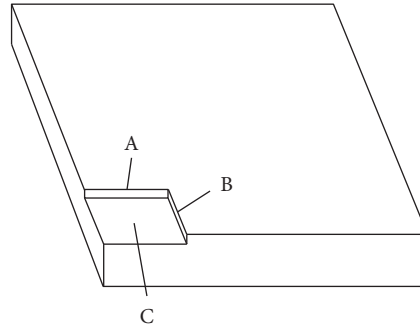
FIGURE 1: Schematic diagram of repairing corner of airport pavement.
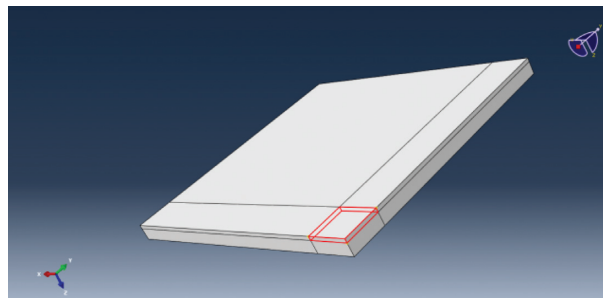

FIGURE 2: Model of repaired concrete.

strength was achieved at 1 d age under 20°C. The hydration stabilized, so the indexes in test at age of 1d were used as the experimental parameters of the material.

The elastic moduli of two concretes were obtained from the experiments, where the old cement concrete and repair concrete had a modulus of 32.5 GPa and 39.4 GPa, respectively. Because the concretes use the same fine aggregates and coarse aggregates except for cement, their thermal conductivity and specific heat capacity were relatively close. The common mean thermal conductivity and specific heat capacity of concrete as per the reference data are 1.28 W/m·K and 0.97 J/g·K, respectively. The test indicates that the masses of two concretes were similar when the concrete was small beam with dimensions of 100 mm × 100 mm × 400 mm. Therefore, the calculation was performed under the following parameters: the density was 2500 kg/m³; the coarse aggregate in old cement concrete was basalt; and the linear expansion coefficient was 0.86×10^{-5}°C. As shown in Table 1, the coarse aggregate of repair concrete was selected from different materials. "a" is linear expansion coefficient in Table 1.

The environmental temperature was set to drop from 20°C to 10°C, 0°C, −10°C, −20°C, and −30°C. The air convection coefficient on the surface of the concrete model was set to 15 W/(m²·K) and the initial ambient temperature was maintained at 20°C. Five temperature models and five stress models were established for calculations. To show the temperature and stress distributions on each bonding interface more clearly, only the modules of repair concrete were selected and the position of each bonding interface is shown in Figure 3. The longitudinal bonding interface A was

perpendicular to the x-axis, the longitudinal bonding interface B was perpendicular to the y-axis, and the horizontal bonding interface C was perpendicular to the z-axis.

4. Effect of Different Ambient Temperatures on Stress at Bonding Interfaces

Using quartzite as the repair material and the temperature to drop to 0°C as example, the stress distribution cloud chart of three interfaces of repair material is presented in Figure 4.

As shown in Figure 4, since $L_A = L_B$, (L_A and L_B are the lengths of the vertical bonding interfaces A and B) when the ambient temperature dropped, the stress on the bonding interface was symmetrically distributed, and maximum principal stresses reached the positions close to the intersection of the three bonding interfaces. Therefore, the most unfavorable position for pavement repair is at the intersection of the three bonding interfaces, where the stress is tensile stress. To understand the distribution of each axial stress when each bonding interface was subjected to maximum stress, stress analysis was performed on each bonding interface (see Figures 5 and 6).

As per Figure 5(a), the stresses on the vertical bonding interface A and along the x-axis direction were all tensile stresses. Closer to the intersection angle of the three bonding interfaces, the tensile stress was greater with the maximum value of 1.243 MPa. Figure 5(b) shows the shear stresses on the vertical bonding interface A and along the y-axis. The shear stress direction in the upper area was the positive direction of the y-axis, while that in the lower area was the negative direction of the y-axis. Figure 5(c) displays the shear

TABLE 1: Linear expansion coefficient of different coarse aggregates.

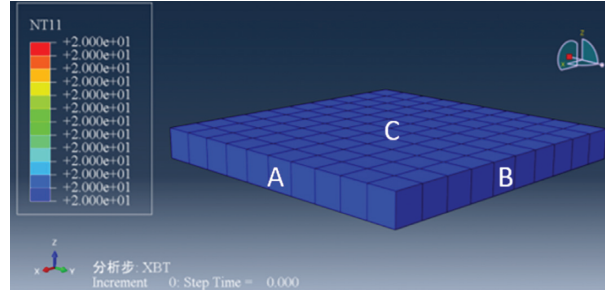| Coarse aggregate	Limestone	Basalt	Granite	Conglomerate	Sandstone	Quartzite
A (10^{-5}/°C)	0.68	0.86	0.95	1.08	1.17	1.19

FIGURE 3: Schematic diagram of positions of bonding interfaces.
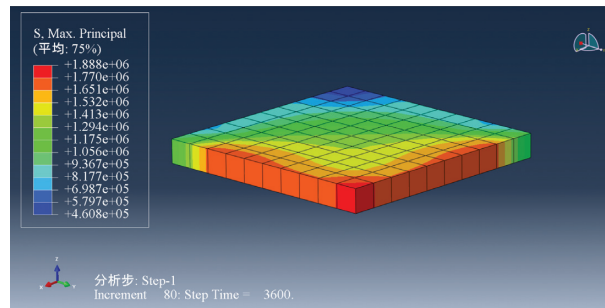

FIGURE 4: Cloud chart of maximum principal stress on three bonding interfaces.

stress along the z-axis direction and on vertical bonding interface A. The shear stress direction at each point on the surface was the positive direction of z-axis, with the magnitude increasing from top to bottom.

As $L_A = L_B$, the vertical bonding interface A and vertical bonding interface B were symmetrical in stress conditions and had the same dimensions; therefore, the vertical bonding interface B was not analyzed.

As per Figure 6(a), along the z-axis direction, the center and edge areas of horizontal bonding interface were both subjected to tensile stresses. Four corners of the horizontal bonding interface were subjected to higher tensile stresses and there was a circle of compression-stressed areas between the two tensile-stressed areas. Figure 6(b) shows that, along the x-axis, the stress on horizontal bonding interface was shear stress: the left half reflects the shear stress along the negative x-axis, while the right half reflects the shear stress along the positive x-axis. The two shear stresses acted on the horizontal bonding interface together. Figure 6(c) demonstrates that the stress along the y-axis of horizontal bonding interface was shear stress and the upper half was the shear stress along the positive direction of the y-axis, while the lower half was the shear stress along the negative direction of the y-axis. The two shear stresses acted together on the horizontal bonding interface. Therefore, when the temperature dropped sharply, the vertical and horizontal bonding

interfaces of MPC concrete were simultaneously subjected to the combined action of tensile stress and shear stress along different directions.

For maximum principal stress on the bonding interface of MPC concrete as the repair material with different coarse aggregates, the results under different ambient temperatures were calculated (see Figure 7). The positive values denote tensile stress, while negative values denote compressive stress.

As shown in Figure 7, after the hydration of MPC as repair concrete was completed, the set ambient temperature was 20°C. Hence, irrespective of which coarse aggregate was used to prepare the MPC concrete, the interfacial stresses were all 0 at 20°C. When the ambient temperature dropped sharply, the concrete shrank. Accordingly, stress occurred on each bonding interface because different coarse aggregates have different linear expansion coefficients. At this time, the stress was induced on the bonding interface between the old concrete and the MPC concrete prepared with different coarse aggregates. When the expansion coefficient of the coarse aggregate of old concrete was greater than that of the MPC concrete, that is, when limestone was used as the coarse aggregate of repairing concrete, the maximum principal stress at the intersection angle of the three bonding interfaces was the compressive stress and the compressive stress increases with the decrease in temperature. When the
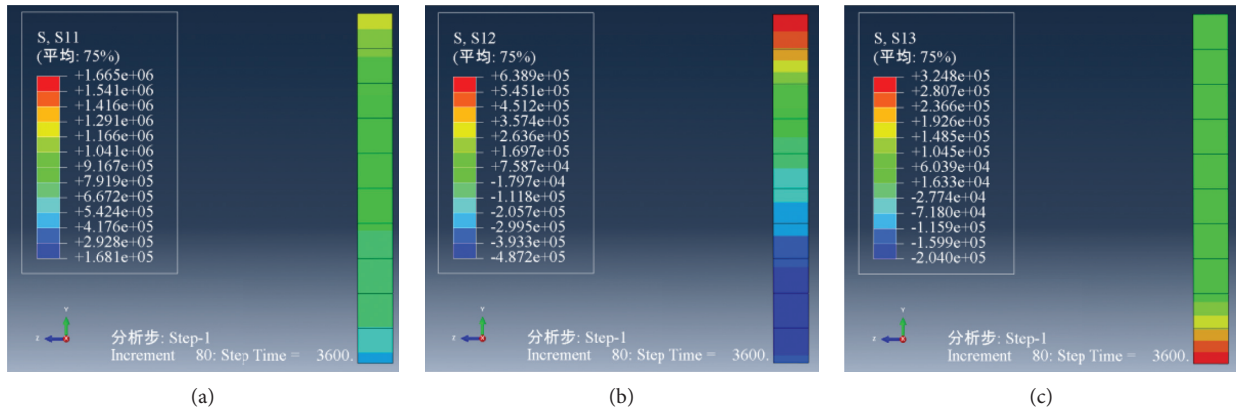
FIGURE 5: Cloud chart of stress on vertical bonding interface A. (a) Normal stress along x-axis (σ_x). (b) Shear stress along y-axis (τ_{xy}). (c) Shear stress along z-axis (τ_{xz}).
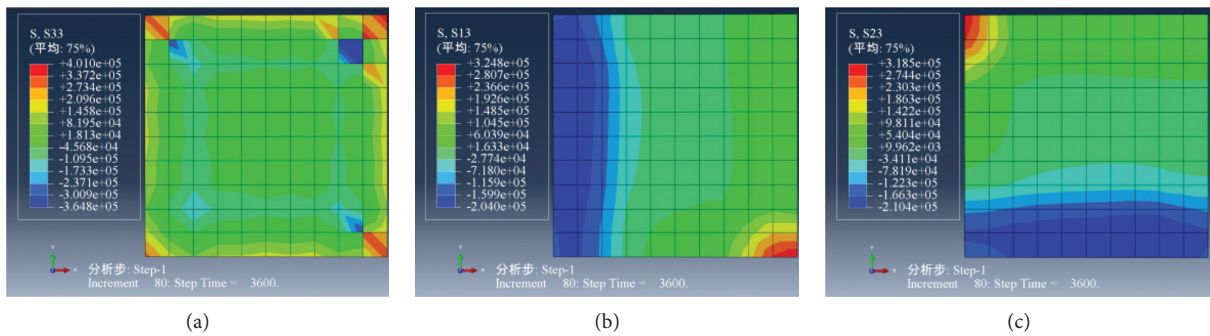

FIGURE 6: Cloud chart of stress on horizontal bonding interface. (a) Normal stress along z-axis (σ_z). (b) Shear stress along x-axis (τ_{zx}). (c) Shear stress along y-axis (τ_{zy}).
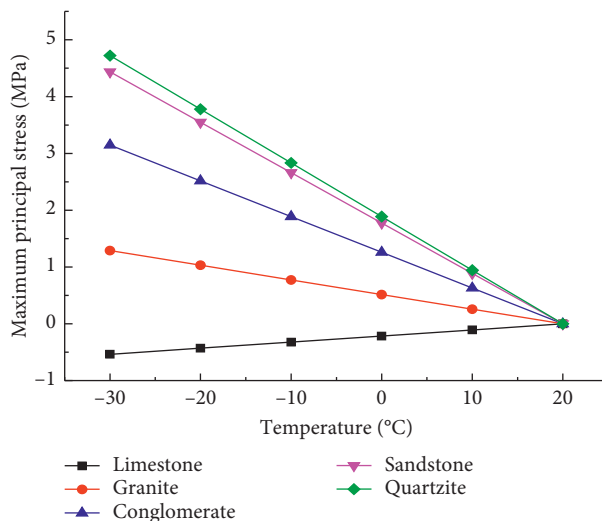

FIGURE 7: Relationship between maximum principal stress and temperatures on bonding interface of repair concrete with different coarse aggregates.

expansion coefficient of the coarse aggregate of old concrete was less than that of the MPC concrete, the maximum principal stress at the intersection of the three bonding interfaces was the tensile stress, which increases with the decrease in temperature. As per the calculation results, when the coarse aggregate of old pavement concrete was basalt, the influencing sequence of coarse aggregate in MPC concrete on the bonding interface from best to worst was as follows:

basalt > limestone > granite > conglomerate > sandstone > quartzite.

5. Conclusions

Herein, MPC repair materials with different coarse aggregates were prepared, which can be used in the rapid-repair project of airport pavement. After the repair was completed and when the hydration reaction entered a stable stage, the effect of sudden temperature drop on the stress distribution of each bonding interface was investigated. The conclusions are as follows:

(1) When the repair material was different from the coarse aggregates of old concrete, the sudden temperature drop causes a temperature stress on the repaired bonding interface. The magnitude of the temperature stress corresponds to the magnitude of temperature drop and the type of coarse aggregates.

(2) As the ambient temperature drops, the stress of the repaired bonding interface is symmetrically distributed and the maximum principal stress of each bonding interface was located at the intersection angle of the three bonding interfaces.

(3) When the linear expansion coefficient of the old concrete coarse aggregate was greater than that of the MPC as repair concrete, the maximum principal stress was located at the intersection angle of the three bonding interfaces. This stress is the compressive stress and its magnitude increases with the decrease in temperature. When the expansion coefficient of the coarse aggregate in old concrete was smaller than that in the MPC as repair concrete, the maximum principal stress was located at the intersection angle of the three bonding interfaces. The maximum principal stress is tensile stress and its magnitude increases with the decrease in temperature.

(4) When the coarse aggregate of the old concrete pavement was basalt, the influencing sequence of coarse aggregate of MPC concrete as repair concrete on the bonding interface from best to worst was as follows: basalt > limestone > granite > conglomerate > sandstone > quartz rock.

Therefore, in the repair project, as much as possible, coarse aggregate in MPC concrete should be selected as close to that in the old concrete. In the absence of consistent coarse aggregates, the one with less stress under temperature drop should be selected for the preparation of repair concrete and the bonding strength of repair concrete should be higher than the maximum shear stress on the horizontal bonding interface and the maximum tensile stress on the vertical bonding interface. The above requirements for material and strength can ensure that the repair material would not crack under temperature drop.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

References

[1] T. Guo, Y. Xie, and X. Weng, "Evaluation of the bond strength of a novel concrete for rapid patch repair of pavements," *Construction and Building Materials*, vol. 186, pp. 790–800, 2018.

[2] A. Bieliatynskyi, E. Krayushkina, and A. Skrypchenko, "Modern technologies and materials for cement concrete pavement's repair," *Procedia Engineering*, vol. 134, pp. 344–347, 2016.

[3] T. Guo, X. Weng, C. Liu, Q. Yu, C. Zhang, and Y. Li, "Evaluation of the bonding and fatigue properties of an innovative rapid repair structure for concrete pavement," *Construction and Building Materials*, vol. 235, Article ID 117484, 2020.

[4] B. Xu, H. Ma, and Z. Li, "Influence of magnesia-to-phosphate molar ratio on microstructures, mechanical properties and thermal conductivity of magnesium potassium phosphate cement paste with large water-to-solid ratio," *Cement and Concrete Research*, vol. 68, pp. 1–9, 2015.

[5] J. Li, W. Zhang, and Y. Cao, "Laboratory evaluation of magnesium phosphate cement paste and mortar for rapid repair of cement concrete pavement," *Construction and Building Materials*, vol. 58, pp. 122–128, 2014.

[6] Y. Xie, X. Lin, X. Pan, and T. Ji, "Preliminary investigation of the hydration mechanism of MgO-SiO2-K2HPO4 cement," *Construction and Building Materials*, vol. 235, Article ID 117471, 2020.

[7] Y. Jianming, W. Luming, and Z. Jie, "Experimental study on the deformation characteristics of magnesium potassium phosphate cement paste at early hydration ages," *Cement and Concrete Composites*, vol. 103, pp. 175–182, 2019.

[8] Y. Jung, W. Lin, H. Hao, and Y.-H. Cho, "Interface behavior of partial depth repair for airport concrete pavement subjected to differential volume change," *Construction and Building Materials*, vol. 143, pp. 515–521, 2017.

[9] X. Jia, J. Li, P. Wang, J. Qian, and M. Tang, "Preparation and mechanical properties of magnesium phosphate cement for rapid construction repair in ice and snow," *Construction and Building Materials*, vol. 229, p. 116927, 2019.

[10] Y. Li, L. Nie, and B. Wang, "A numerical simulation of the temperature cracking propagation process when pouring mass concrete," *Automation in Construction*, vol. 37, pp. 203–210, 2014.

[11] A. Borosnyói, "Influence of service temperature and strain rate on the bond performance of CFRP reinforcement in concrete," *Composite Structures*, vol. 127, pp. 18–27, 2015.

[12] V. Carvelli, M. A. Pisani, and C. Poggi, "High temperature effects on concrete members reinforced with GFRP rebars," *Composites Part B: Engineering*, vol. 54, pp. 125–132, 2013.

[13] A. Sharma, J. Bošnjak, and S. Bessert, "Experimental investigations on residual bond performance in concrete subjected to elevated temperature," *Engineering Structures*, vol. 187, pp. 384–395, 2019.

[14] A. Albidah, A. Abadel, F. Alrshoudi, A. Altheeb, H. Abbas, and Y. Al-Salloum, "Bond strength between concrete substrate and metakaolin geopolymer repair mortars at ambient and elevated temperatures," *Journal of Materials Research and Technology*, vol. 9, no. 5, pp. 10732–10745, 2020.

[15] B. P. Gautam, D. K. Panesar, S. A. Sheikh, and F. J. Vecchio, "Effect of coarse aggregate grading on the ASR expansion and damage of concrete," *Cement and Concrete Research*, vol. 95, pp. 75–83, 2017.