

Article

Bond Behavior between High-Strength Rebar and Steel-Fiber-Reinforced Concrete under the Influence of the Fraction of Steel Fiber by Volume and High Temperature

Xiaodong Li ^{1,2,*} , Chengdong Lu ^{1,*} , Yifei Cui ^{1,2} , Lichen Zhou ^{1,3} and Li Zheng ¹

¹ School of Civil Engineering, Qingdao University of Technology, Qingdao 266525, China

² Engineering Research Center of Concrete Technology under Marine Environment, Ministry of Education, Qingdao 266525, China

³ China Construction Third Bureau First Engineering Co., Ltd., Wuhan 430040, China

* Correspondence: xiaodongli1976@163.com (X.L.); chengdonglu1998@163.com (C.L.)

Abstract: Steel-fiber-reinforced concrete (SFRC) is a composite material made by randomly distributing short steel fibers in normal concrete (NC). In this study, central pull-out tests of 32 specimens were performed to investigate the bond behavior between high-strength rebar and SFRC under the influence of the fraction of steel fiber by volume ($V_f = 0\%, 0.5\%, 1.0\%$ and 1.5%) and temperature ($T = 20, 200, 400$ and $600\text{ }^{\circ}\text{C}$). The results show that in NC specimens, splitting failure occurs below $400\text{ }^{\circ}\text{C}$, while split-pullout failure occurs above $600\text{ }^{\circ}\text{C}$. Split-pullout failure occurs in all SFRC specimens at each tested temperature. The bond strength between rebar and SFRC was found to decay significantly between 400 and $600\text{ }^{\circ}\text{C}$. The effect of V_f on the improvement in bond strength was more obvious between 400 and $600\text{ }^{\circ}\text{C}$ than between 20 and $400\text{ }^{\circ}\text{C}$. The positive contribution of steel fibers to bond behavior is the construction of a rigid skeleton with coarse aggregates that can play a bridging role and effectively retard the expansion of concrete cracks. This improves the bond strength between rebar and SFRC at high temperatures. The bond-slip curve can be divided into five stages, namely the initial micro-slide phase, slip phase, splitting failure phase, stress drop phase and residual pull-out phase. A model of the bond-slip relationship between rebar and SFRC considering temperature and V_f was developed by modifying the existing model of the bond-slip relationship between rebar and NC. The model calculation results agree well with those of testing.

Keywords: SFRC; high temperature; bond behavior; fraction of steel fiber by volume; bond-slip relationship



Citation: Li, X.; Lu, C.; Cui, Y.; Zhou, L.; Zheng, L. Bond Behavior between High-Strength Rebar and Steel-Fiber-Reinforced Concrete under the Influence of the Fraction of Steel Fiber by Volume and High Temperature. *Appl. Sci.* **2023**, *13*, 2399. <https://doi.org/10.3390/app13042399>

Academic Editor: Jong Wan Hu

Received: 22 December 2022

Revised: 2 February 2023

Accepted: 8 February 2023

Published: 13 February 2023



Copyright: © 2023 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) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

At present, new construction materials are being designed to meet the development needs of building disaster prevention and mitigation and these materials are experiencing rapid investment. Notably, steel-fiber-reinforced concrete (SFRC) is becoming increasingly popular in engineering projects [1,2]. It is used in various fields, including foundation, bridge and tunnel engineering [3–5]. Steel fibers have good tensile properties [6], which can compensate for the deficiencies of normal concrete (NC) with respect to tensile and shear properties [7], demonstrating the superiority of SFRC. As a result, SFRC members have greater strength [8], flexural toughness [9], impact resistance [10] and wear resistance [3] than NC members.

The bond behavior between rebar and concrete is the basis for the design of structural members [11,12]. Many scholars have used pull-out testing to study the behavior and influencing factors of the bond behavior between rebar and SFRC. Key indicators of the bond-slip characteristics between rebar and SFRC include the bond strength, bond-slip curve and bond stress distribution. Garcia-Taengua [13] investigated the effects of various factors on bond strength and toughness (concrete compressive strength, rebar diameter,

concrete cover thickness, fraction of steel fiber by volume, fiber length and aspect ratio). Pull-out tests revealed that the concrete compressive strength is the most critical factor of the bond strength, while the fraction of steel fiber by volume only has a minor effect. Chu [14,15] found that the volume fraction and aspect ratio of steel fiber affect the bond stress slip relationship, strength, stiffness and toughness. Chu then used this information to create a new rebar bond model that includes the finite initial bond stiffness, based on the existing rebar bond model, that provides bond strength, stiffness and toughness prediction equations for NC and SFRC. Gao [16] analyzed the bond stress distribution between rebar and SFRC for different volume fractions of steel fibers using pull-out tests of local bond specimens of SFRC. The midpoint stress interpolation method was proposed as part of a new bond stress prediction model. It was shown that the incorporation of steel fibers increases the inhomogeneity of bond stress between rebar and SFRC. As the volume fraction of steel fibers is increased, the slip near the loading end decreases and the corresponding bond stiffness greatly increases. Zhang [17] found that steel fibers have a dual effect on the bond strength. Steel fibers increase the concrete strength and decrease the mobility. The recommended steel fiber volume fraction was proposed to be approximately 0.25~0.75% in the structure.

The bond behavior between rebar and SFRC deteriorates at high temperature due to fire. Fakoor [18] proposed a new method for evaluating the effectiveness of steel fibers with respect to bond strength. It was noted that the post-peak behavior of the bond–slip curve could be considered an effect of reinforcing the bond strength between rebar and concrete. Varona's [19] study showed that incorporating mixed steel fibers with high aspect ratios increases the peak bond strength at a maximum test temperature of 825 °C. A reliable bond between rebar and SFRC is the basis for ensuring the safety of SFRC structural members under fire. Therefore, it is especially critical to study the bond–slip performance between rebar and SFRC at high temperatures. In this study, the bond failure process and bond–slip performance of rebar and SFRC at high temperatures are investigated. The effects of temperature and the fraction of steel fiber by volume on the bond strength between rebar and SFRC are analyzed by introducing a fiber reinforcement coefficient and a temperature reduction coefficient. Based on the impact parameters of temperature and steel fiber volume fraction, a bond–slip relationship model between rebar and SFRC at high temperature is established.

2. Experiment

2.1. Materials and Mix Proportions

Granite aggregates (from Qingdao Granite Quarry, Shandong Province, China) with particle sizes ranging from 5 to 20 mm and continuous grading, natural river sand (from Dagou River, Qingdao, Shandong Province, China) with a fineness modulus of 2.6, P.O 42.5R ordinary silicate cement (produced by Shanshui Cement Group Co., Jinan, Shandong Province, China), polycarboxylic acid high-performance water reducer (1% of the total cementitious material, produced by Shanghai Hengchuang Chemical Co., Shanghai, China) and hook-end steel fibers (produced by Lianzhu Steel Fiber Manufacturing Co. in Anping County, Hengshui, Hebei Province, China) were used in the tests. The concrete was designed with a C50 strength grade and a water cement ratio of 0.38. The concrete mix proportions and mechanical performance at room temperature are shown in Figure 1 and Table 1, respectively. The performance indexes of steel fibers are shown in Table 2. Hot-rolled ribbed rebar with a nominal diameter of 14 mm and a yield strength of at least 500 MPa (HRB500 [20]) was used for rebar (see Table 3).

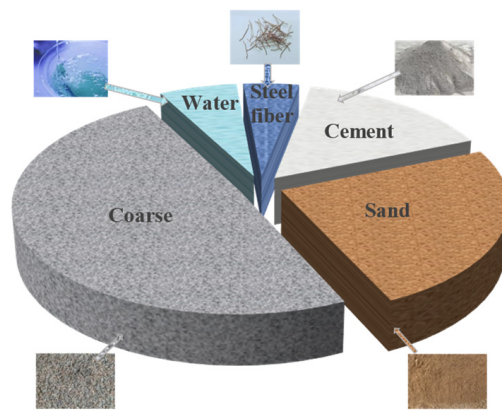


Figure 1. Mix proportions.

Table 1. Concrete mix proportions and mechanical performance at room temperature.

Group	V_f (%)	Coarse (%)	Sand (%)	Cement (%)	Water (%)	Steel Fiber (%)	28-Day Compressive Strength (MPa)	28-Day Splitting Tensile Strength (MPa)
SF0	0	52.31	25.83	15.61	6.25	0	53.5	3.3
SF0.5	0.5	51.20	25.28	15.68	6.27	1.57	54.5	4.1
SF1.0	1.0	50.13	24.75	15.74	6.30	3.07	62.5	5.3
SF1.5	1.5	49.11	24.25	15.81	6.32	4.52	67.0	6.7

Note: V_f is the fraction of steel fiber by volume. Materials in the table are the mass percentages.

Table 2. Physical mechanical performance index of the steel fibers.

Fiber Type	Equivalent Diameter (mm)	Length (mm)	Aspect Ratio	Tensile Strength (MPa)
hook-end steel fiber	0.77	35	45	1150

Table 3. Mechanical performance of HRB500 rebar at different temperatures.

Temperature Exposure	Diameter (mm)	Cross-Sectional Area (mm ²)	Yield Strength (MPa)	Ultimate Strength (MPa)	Elastic Modulus (MPa)
20 °C	14.0	153.9	581	755	2.041×10^5
200 °C	14.0	153.9	503	707	1.922×10^5
400 °C	14.0	153.9	411	602	1.454×10^5
600 °C	14.0	153.9	241	373	0.793×10^5

2.2. Test Specimens

The specimens were designed as cubes with side lengths of 150 mm. A rebar anchorage length of $5d$ (d is the diameter of the rebar, i.e., anchorage length $l_a = 5 \times 14 \text{ mm} = 70 \text{ mm}$) was set in the center of the concrete specimen. Two PVC pipes 40 mm in length covering the outer surface of the rebar controlled the nonbonded zone. A thickened double-sided adhesive was used to fill the gap between the rebar and the PVC pipe. A total of 44 specimens were made, and the volume fraction of steel fiber (V_f) and high temperature were studied in terms of their influence on the bond behavior. Of the 44 manufactured specimens, 32 nonthermocouple specimens were used for central pull-out testing and 12 thermocouple specimens were used to test the temperature of the bond zone during the high-temperature testing. The 32 nonthermocouple specimens were divided into 16 groups, as shown in Table 4. The specimen size and photographs are shown in Figure 2.

Table 4. Specimen details.

Group	Specimen ID	Rebar Diameter (mm)	Fraction of Steel Fiber by Volume (V_f) (%)	Temperature Exposure (T) (°C)	Number
1	SF0T20- n	14	0	20	2
2	SF0.5T20- n	14	0.5	20	2
3	SF1T20- n	14	1	20	2
4	SF1.5T20- n	14	1.5	20	2
5	SF0T200- n	14	0	200	2
6	SF0.5T200- n	14	0.5	200	2
7	SF1T200- n	14	1	200	2
8	SF1.5T200- n	14	1.5	200	2
9	SF0T400- n	14	0	400	2
10	SF0.5T400- n	14	0.5	400	2
11	SF1T400- n	14	1	400	2
12	SF1.5T400- n	14	1.5	400	2
13	SF0T600- n	14	0	600	2
14	SF0.5T600- n	14	0.5	600	2
15	SF1T600- n	14	1	600	2
16	SF1.5T600- n	14	1.5	600	2

Note: SF1.5T200- n denotes that the specimen with the fraction of steel fiber by volume was 1.5% at 200 °C. n denotes the number of specimens, $n = 1, 2$.

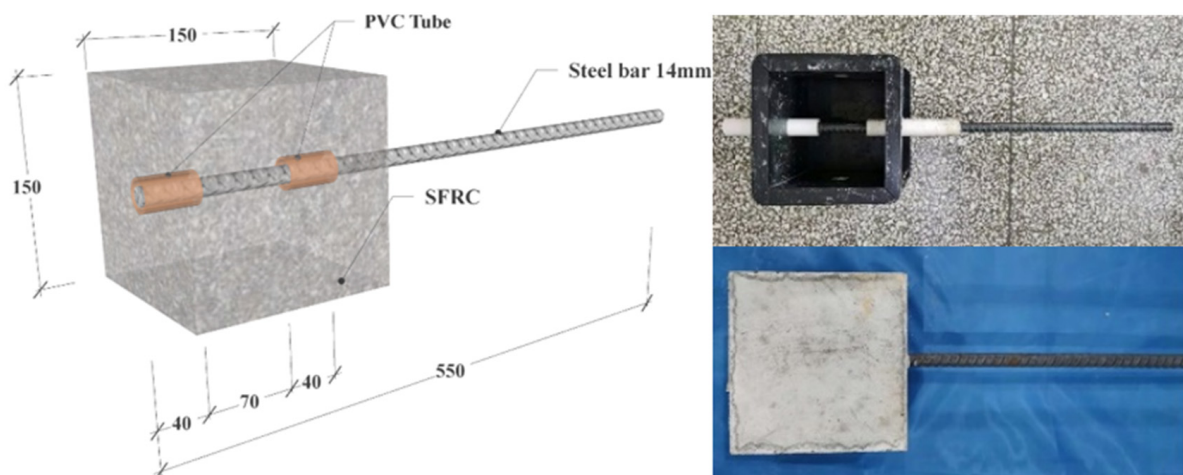


Figure 2. Specimen size and photograph (mm).

2.3. Test Programs

2.3.1. High-Temperature Acquisition System

The test heating equipment was a GWX-1100 high-temperature environmental test furnace (produced by FangRui Technology Co., LTD Changchun, Changchun, Jilin Province, China). The furnace has external dimensions of 550 mm × 500 mm × 600 mm, chamber dimensions of 300 mm × 220 mm × 350 mm, a maximum heating rate of 800 °C/h, a thermostatic control accuracy of ±2 °C, a temperature gradient of 3 °C and an operating power rating of 5 kW. The test temperature control system was the GWX-1100 Human Machine Interface (HMI) operating mode, which consisted of a temperature control system with an HMI. The heating power was adjustable, and the power output was limited at any time using the built-in touchscreen program. As a result, the temperature inside the furnace could be precisely set and maintained. The high-temperature test equipment and temperature control monitoring system are shown in Figure 3.

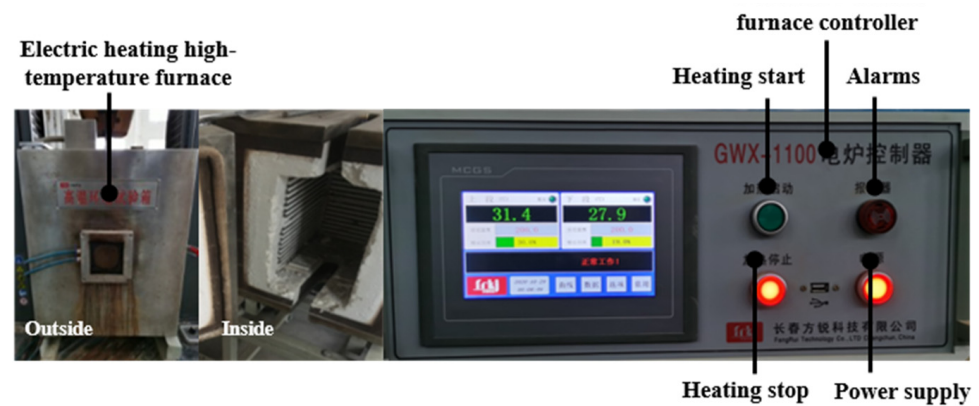


Figure 3. Diagram of the electric heating high-temperature furnace and electric furnace controller.

The temperature of the bond zone between rebar and SFRC was recorded, using a type-K thermocouple connected to an Agilent 34980A data acquisition instrument. The thermocouple was pre-built in the center of the bond zone between rebar and SFRC.

2.3.2. Central Pull-Out Test

The pull-out test loading device was an MTS E45.305 electronic universal testing machine with a test system consisting of a mainframe, controller and test software. The maximum force rating of the test machine was 300 kN, the loading rate range was 0.001 to 250 mm/min, the deformation measurement range was 0.2% to 100% FS and the positional resolution was 1.7×10^{-5} mm. The specimen could be loaded using force, displacement or strain control. The test was performed by clamping the bar in the upper end of the fixture and then applying tension by precisely controlling the position of the center beam electronically. The high-temperature furnace was moved to the loading position of the testing machine using a moving bracket for the central pull-out test at high temperatures. The loading equipment is shown in Figure 4. The pull-out test was fully auto-controlled by the test system. The specimen was loaded at a rate of 98 N/s at the start stage. After 80% of the ultimate load, the specimen was converted to displacement control at a rate of 0.005 mm/s.

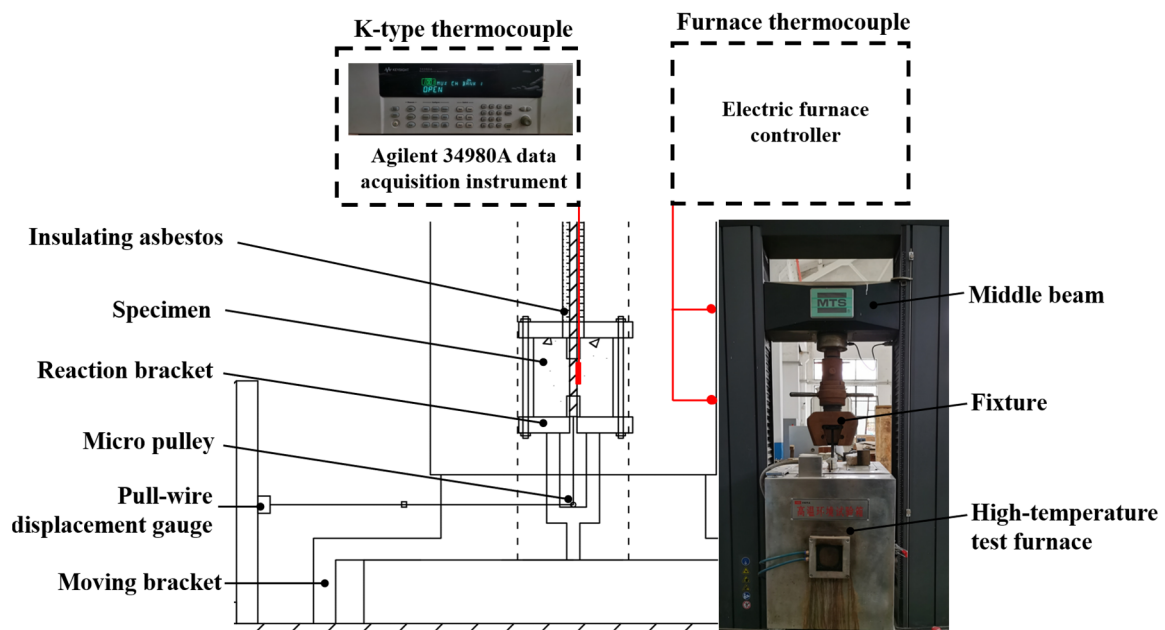


Figure 4. Loading device.

The test machine's TestSuite software recorded the external load in real time. The average bond stress (τ) between rebar and concrete was assumed to be uniform along the anchorage length and was obtained using Equation (1). The S_F was the slip at the free end, which was measured by the pull-wire displacement gauge.

$$\tau = \frac{P}{\pi d l_a} \quad (1)$$

where P is the tension force applied at the loaded end (kN); d is the diameter of the rebar (14 mm); l_a is the anchorage length of the rebar in concrete (70 mm).

3. Results and Analysis

3.1. Temperature–Time Curve

The temperature inside the furnace was controlled to increase at a rate of 8 °C/min at the start stage. Once the furnace temperature reached the target test temperature, it was kept constant by the furnace controller. The temperature–time curve of the furnace is shown in Figure 5.

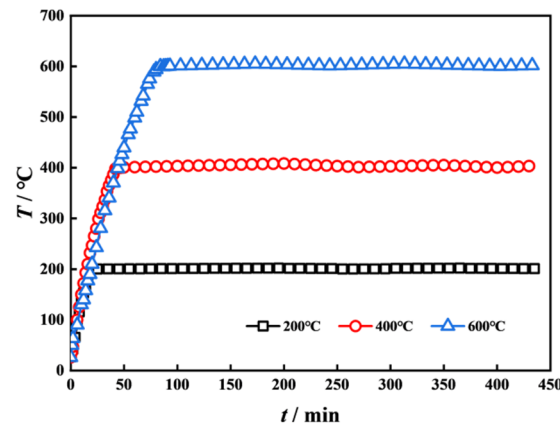


Figure 5. Temperature–time curve of the furnace.

The temperature–time curve of the bond zone between rebar and SFRC is shown in Figure 6. Because concrete is a poor conductor of heat, there was a significant temperature difference between the bond zone and the furnace at the start stage of the test, which gradually decreased with the increase in furnace temperature. The temperature of the bond zone between rebar and SFRC remained essentially constant after 6 h.

The following can be observed in Figure 6.

- (1) When the heating time reaches about 100 min, there is an apparent horizontal section of the temperature time curve at the target test temperature of 400 and 600 °C. The temperature of the horizontal section is approximately 140~180 °C. The reason for this is that the free water inside the concrete evaporates when the specimen's internal temperature reaches the boiling point of water. Some of the heat from the specimen is absorbed due to the evaporation of free water; thus, the temperature of the bond zone increases slowly. The temperature rate of the bond zone gradually increases after water evaporation in the specimen is complete.
- (2) The temperature–time curves for NC and SFRC are similar for each high-temperature environment. As V_f increases, the temperature between rebar and SFRC rises more rapidly and the temperature difference between the bond zone and furnace is gradually reduced.

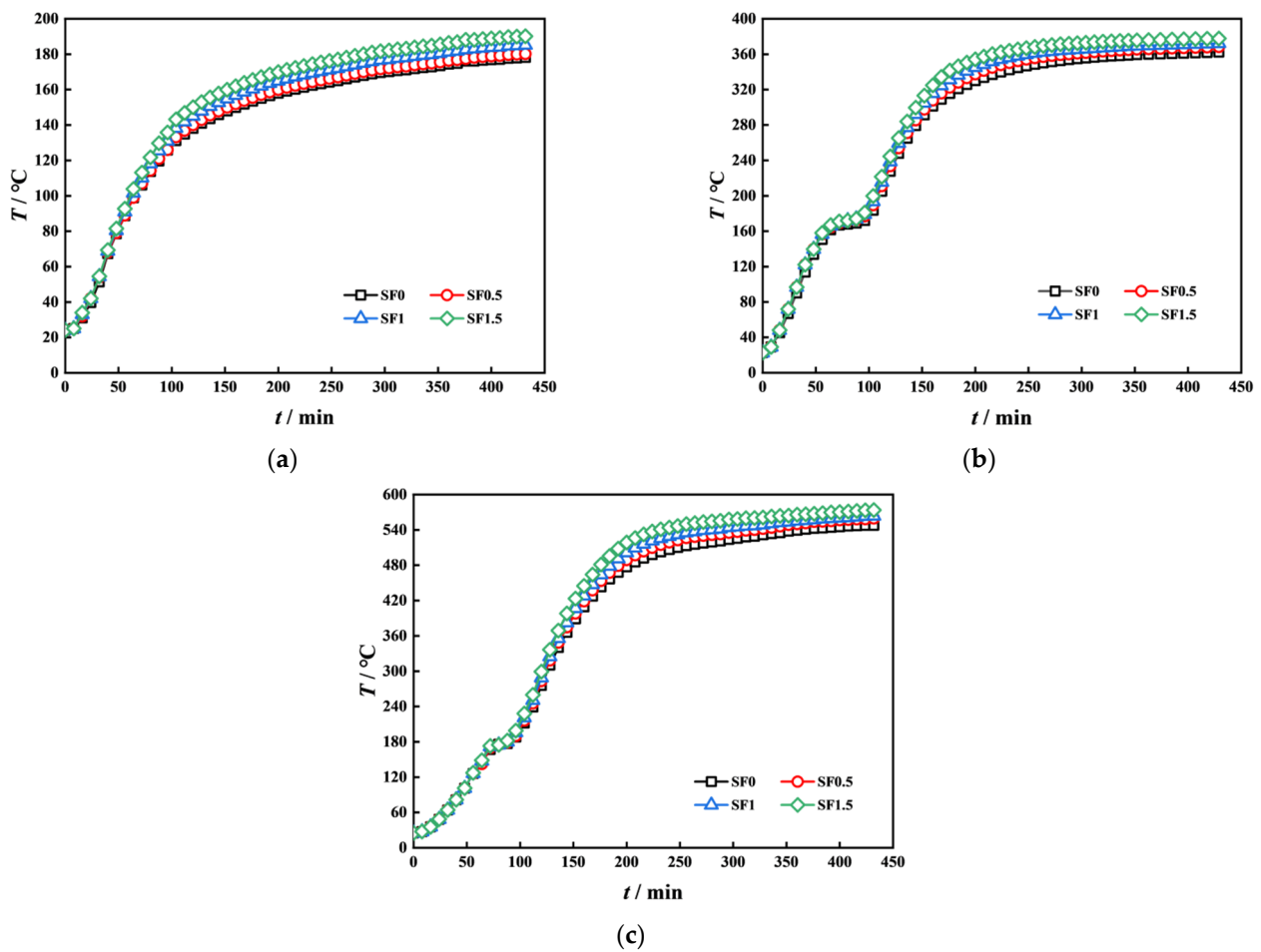


Figure 6. Temperature–time curve of the bone zone between rebar and SFRC at (a) 200 °C, (b) 400 °C, and (c) 600 °C.

3.2. Failure Patterns

The failure pattern of specimens at different high temperatures are shown in Figure 7 and Table 5.

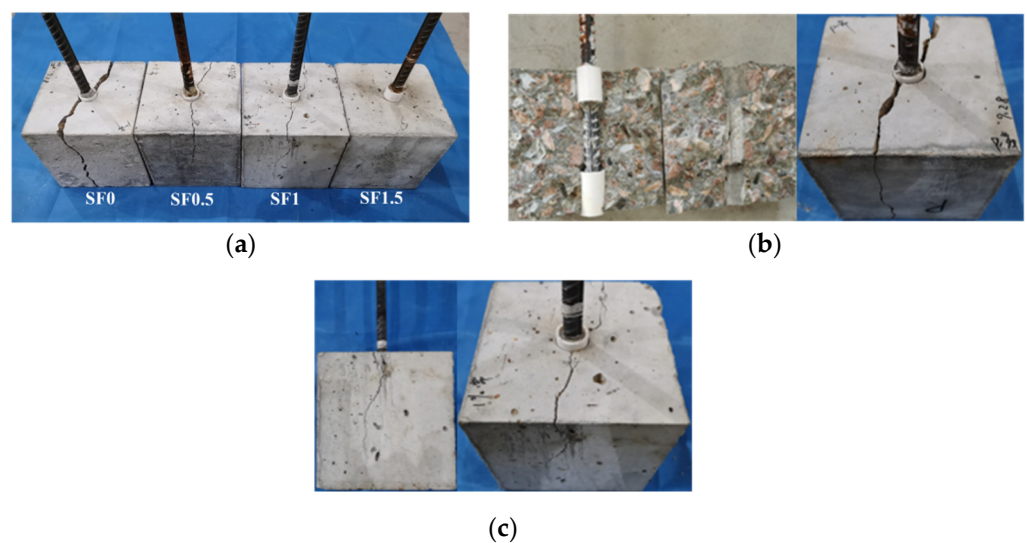


Figure 7. (a) Comparison of failure modes, (b) splitting failure pattern, and (c) split-pullout failure patterns of specimens.

Table 5. Experimental results of pull-out tests.

Specimen ID	P (kN)	τ_u (MPa)	s_u (mm)	β_T	β_V	Failure Patterns
SF0T20	74.155	24.08	0.38	1.000	1.000	Splitting
SF0.5T20	82.58	26.82	0.59	1.000	1.114	Split-pullout
SF1T20	90.02	29.235	0.805	1.000	1.214	Split-pullout
SF1.5T20	98.31	31.925	1.015	1.000	1.326	Split-pullout
SF0T200	62.92	20.43	0.445	0.848	1.000	Splitting
SF0.5T200	74.315	24.13	0.665	0.900	1.181	Split-pullout
SF1T200	82.37	26.75	0.91	0.915	1.309	Split-pullout
SF1.5T200	91.67	29.77	1.1	0.932	1.457	Split-pullout
SF0T400	39.50	12.825	0.625	0.533	1.000	Splitting
SF0.5T400	67.11	21.795	0.865	0.813	1.699	Split-pullout
SF1T400	74.375	24.155	1.08	0.826	1.883	Split-pullout
SF1.5T400	80.31	26.08	1.49	0.817	2.034	Split-pullout
SF0T600	24.09	7.82	0.735	0.325	1.000	Split-pullout
SF0.5T600	35.855	11.64	0.965	0.434	1.488	Split-pullout
SF1T600	46.56	15.12	1.37	0.517	1.934	Split-pullout
SF1.5T600	54.77	17.785	1.84	0.557	2.274	Split-pullout

It is shown in Figure 7 and Table 5 that the temperature significantly alters the failure patterns of NC specimens and has less of an effect on the failure patterns of SFRC specimens. In NC specimens, splitting failure occurred below 400 °C and split-pullout failure above 600 °C. In comparison, split-pullout failure occurred in all SFRC specimens at each temperature. The main longitudinal crack widths of the destructive SFRC specimens were significantly smaller than those of the NC specimens. The width and length of the cracks gradually decreased as V_f increased. The SFRC specimens had many small uneven cracks scattered in a radial pattern on both sides of the main crack.

The reason for the above two failure patterns can be explained as follows. The protruding rib of the rebar produces an oblique squeeze force on the concrete after the rebar is pulled. The horizontal component of the oblique extrusion force causes horizontal tensile and shear force in the concrete around the rebar. This results in internal oblique tapered cracks in the concrete. The radial component of the oblique squeeze force causes the concrete to produce circumferential tensile stress, which causes internal radial cracks in the concrete. The temperature reduces the tensile strength of the concrete in NC specimens, which allows radial cracks to extend over the surface of the specimen. Radial cracks link up between reinforcing ribs and splitting cracks form. The mechanical bite force is quickly lost and splitting failure occurs. SFRC specimens have higher tensile strength than concrete due to the incorporation of steel fibers. This retards the generation of cracks within the concrete. Moreover, the synergy between steel fibers and aggregates allows them to act as a rigid skeleton for bridging cracks [17,21,22]. Expansion of the internal oblique tapered cracks and radial cracks is prevented. Ultimately, split-pullout failure occurs in the SFRC specimens.

3.3. Bond–Slip Curves

Based on Equation (1) and S_F , the bond–slip curves of the specimens were plotted and are shown in Figure 8.

Scholars have observed that there are four characteristic points on the bond–slip curves of rebar and NC, namely internal cracking, splitting, limit and residual, which were used to classify five stages [23] in this study. According to the bond–slip curves and the failure patterns of the SFRC specimens, the bond–slip curves of SFRC at different high temperatures were divided into five stages by considering the effects of steel fibers and the temperature. A typical bond–slip curve of SFRC is shown in Figure 9. τ_u is the ultimate bond stress and s_u is the free-end slip corresponding to the ultimate bond strength in Figure 9.

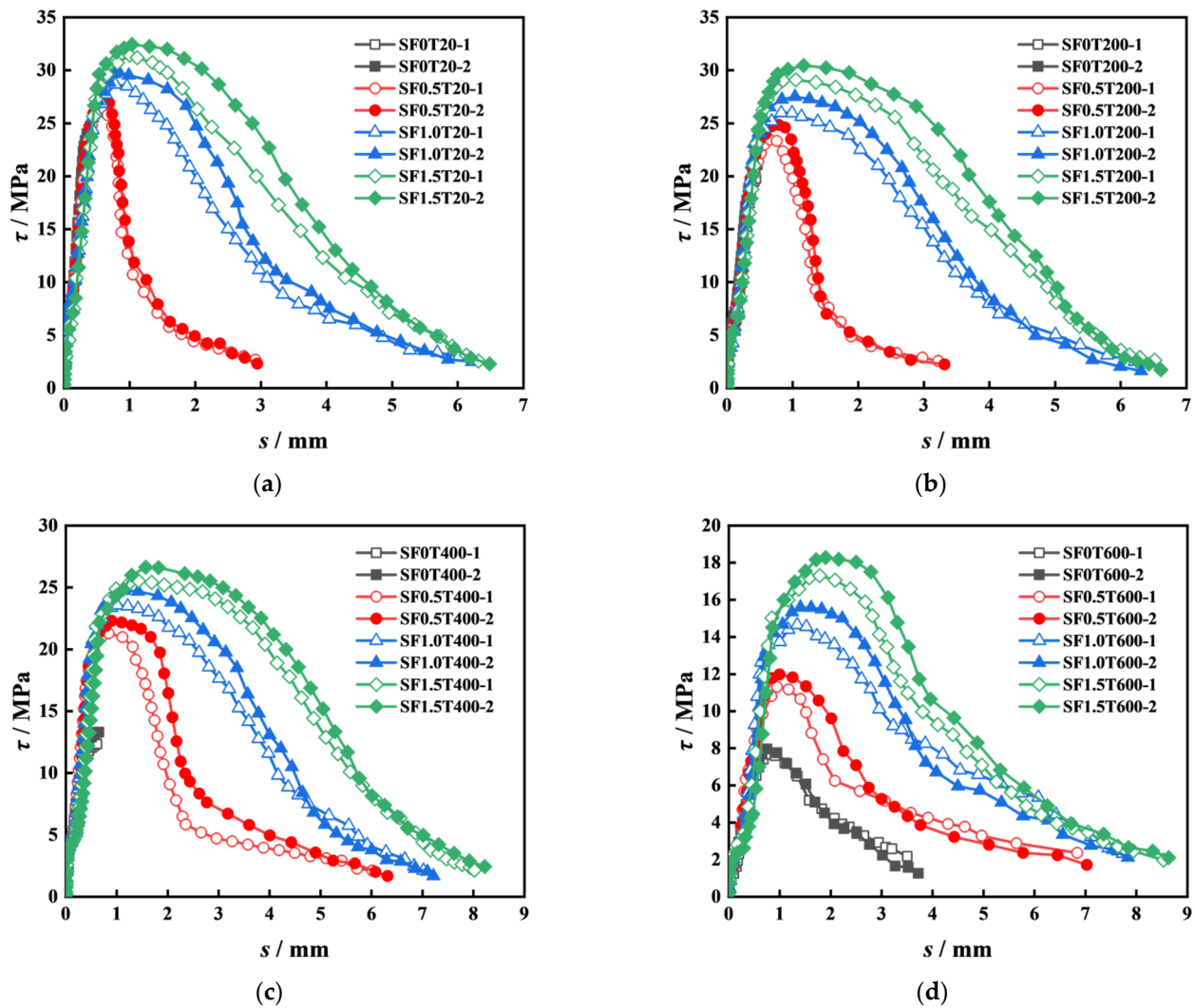


Figure 8. Bond-slip curves of specimens at (a) 20 °C, (b) 200 °C, (c) 400 °C, and (d) 600 °C.

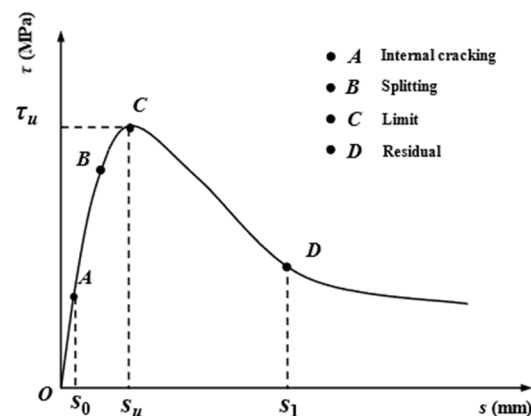


Figure 9. Bond-slip curves of SFRC.

(1) Initial micro-slide phase (OA)

The concrete around the rebar near the loaded end of the specimen is elastically deformed at the beginning of central pull-out test. A small relative slip between rebar and concrete appears. At this stage, the bond between rebar and concrete is dominated by the chemical bond force and supplemented by the mechanical bite force caused by the small

slip. The bond–slip curve at this stage is essentially linear. Steel fibers do not play a role in preventing cracking because the concrete around the rebar has not yet cracked.

(2) Slip phase (AB)

With the increase in pullout force, the chemical bond force disappears. The oblique squeezing force of the rebar rib on the concrete increases and its radial component causes the concrete around the rebar to produce radial cracks and relative slip from the loaded end. The bond force is mainly provided by the mechanical bite force of the rebar rib on the concrete and the frictional resistance between the surface of the rebar and the concrete. At this stage, the steel fibers, which are in the same direction as the main tensile stress, start to play a crack-arresting role. The development of cracks is thus greatly restricted. Therefore, the bond–slip curve enters a nonlinear growth phase.

(3) Splitting failure phase (BC)

As the oblique squeezing force increases, the ultimate tensile strength of concrete at the periphery of the rebar is reached. Cracks in the concrete appear along the direction perpendicular to the rebar ribs. The crack-bridging effect of steel fibers sharply increases, which effectively delays the development of cracks. There is a trend of slowly rising until the bond–slip curve reaches the peak point. As shown in Figure 8, the rising trend of curves becomes flatter with the increase in V_f before the peak point. The bond–slip curve reaches the peak of the curve at point C. Then, the concrete surface starts to produce obvious splitting cracks.

The temperature has a significant effect at this stage. The ultimate bond strength of SFRC specimens decreases significantly as the temperature rises. The bond–slip curve near the peak point flattens out with the increase in temperature at the same V_f . The incorporation of steel fibers greatly improves the ductility of the specimen at high temperature.

(4) Stress drop phase (CD)

When the ultimate bond strength is reached, the concrete in contact with the rebar rib is crushed and broken. The bond stress is provided by mechanical bite and frictional resistance. The contact surface between the rebar rib and the surrounding concrete gradually smooths out. The roughness of the contact surface decreases as the slip increases, which causes a reduction in mechanical bite force.

As a result, after the peak point, the bond stress between rebar and concrete gradually deteriorates with the increase in slippage. The bridging effect of the steel fibers diminishes with the increase in slippage. Cracks in the concrete around the rebar gradually become deeper and wider and rapidly extend to the boundaries of the concrete.

(5) Residual pull-out phase (after the D point)

At this stage, the bond–slip curve tends to flatten out. The specimen fails as the rebar is pulled out of the concrete. Because frictional resistance, aggregate bite and steel fiber remain between rebar and concrete, the bond–slip curves of SFRC specimens appear as longer horizontal segments. This indicates that the residual bond stress is maintained at high temperatures.

3.4. Bond Strength

The experimental results of pull-out tests are shown in Table 5.

To precisely understand the effect of V_f and high temperature on the bond strength, the steel fiber reinforcement factor (β_V) and temperature reduction factor (β_T) were introduced, as shown in Equations (2) and (3).

$$\beta_V = \frac{\tau_u^V}{\tau_u^0} \quad (2)$$

$$\beta_T = \frac{\tau_u^T}{\tau_u^{20}} \quad (3)$$

where τ_u^V is the ultimate bond stress when steel fiber is mixed with V_f ($V_f = 0.5\%$, 1% and 1.5%); τ_u^0 is the ultimate bond stress of the NC specimen. τ_u^T is the ultimate bond stress when the temperature of the bond zone is T ($T = 200, 400$ and $600\text{ }^\circ\text{C}$); τ_u^{20} is the ultimate bond stress at room temperature ($20\text{ }^\circ\text{C}$).

From Figure 10 and Table 5, it can be seen that both ultimate bond stress and peak slip increase as V_f increases. As the temperature rises, the ultimate bond stress decreases and the peak slip increases.

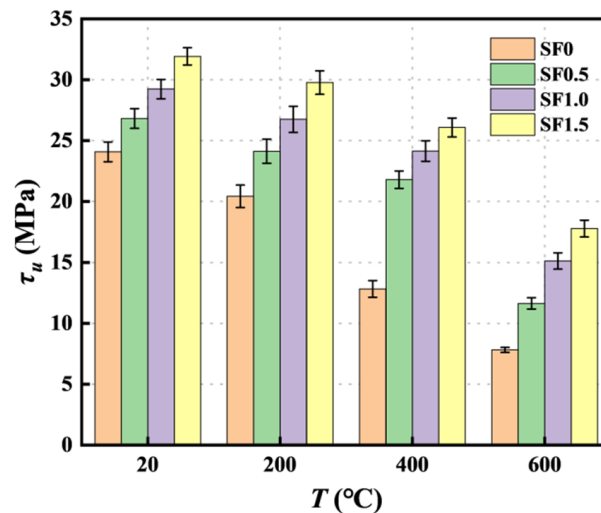


Figure 10. Ultimate bond stress at different high temperatures.

The effect of V_f on bond strength is shown in Figure 11. With the increase in V_f , β_V generally shows a linear growth trend. β_V is small at $20\text{ }^\circ\text{C}$ and $200\text{ }^\circ\text{C}$, i.e., less than 1.5 in both cases. The slope of the β_V – V_f curve is steeper between 400 and $600\text{ }^\circ\text{C}$, which indicates that the effect of V_f on the improvement in bond strength is more obvious between 400 and $600\text{ }^\circ\text{C}$.

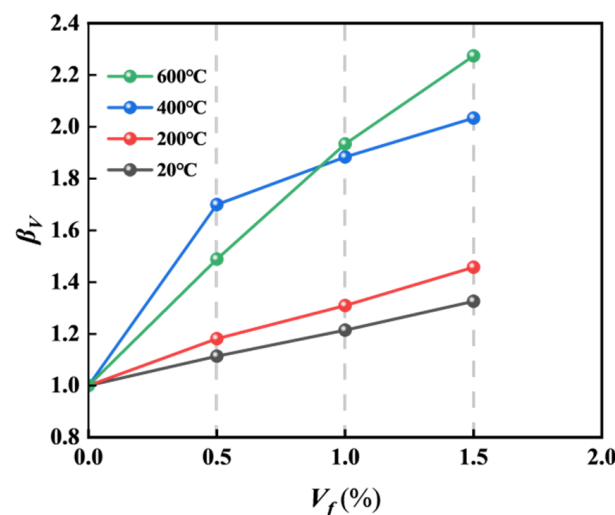


Figure 11. β_V – V_f curves.

β_V of the same V_f increases as the temperature rises. The main reason for this is that high temperatures significantly reduce the bond strength of NC.

The effect of temperature on bond strength is shown in Figure 12. It can be seen that there is less β_T decay of NC and SFRC between 20 and $200\text{ }^\circ\text{C}$. β_T of NC shows large decay between 200 and $400\text{ }^\circ\text{C}$. However, the degree of β_T decay of SFRC is consistent with that for temperature between 20 and $200\text{ }^\circ\text{C}$. The slope of the β_T – T curve for SFRC is greater

between 400 and 600 °C than between 200 and 400 °C. This indicates that the bond strength between rebar and SFRC experiences greater decay between 400 and 600 °C.

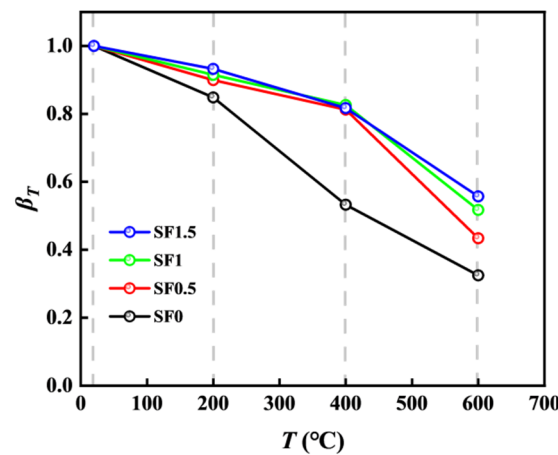


Figure 12. β_T – T curves.

At the same temperature, β_T increases as V_f is increased. This indicates that the increase in V_f improves the bond strength between rebar and SFRC.

The reasons for the effect of V_f and temperature on bond strength can be explained as follows.

- (1) The interaction of the steel fibers and the aggregate creates a rigid skeleton for bridging cracks [17,21,22]. As a result, the bridging action not only retards the expansion of concrete cracks but also moderates the degree of stress concentration at crack tips. This allows the scale and number of crack sources within the concrete matrix to be effectively controlled.
- (2) The difference in the thermal properties of rebar and SFRC at high temperatures destroys the bond behavior of rebar and SFRC. Firstly, there was a significant difference in the thermal properties of rebar and SFRC. During the heating process, the thermal conductivity of rebar was higher than that of SFRC. Therefore, the stress concentration generated by the temperature difference exists in the bond zone between rebar and SFRC. Secondly, while the thermal expansion of rebar and SFRC were of the same order of magnitude under temperature influence, the temperature expansion of rebar and SFRC above 400 °C varied considerably [24]. The differences in the expansion and deformation of the rebar and SFRC aggravated the extension of cracks in the concrete between the ribs in the radial direction of the rebar. This led to a decrease in the bond strength between rebar and SFRC as the temperature rose.
- (3) The continued high temperatures cause cumulative temperature damage to the matrix concrete, which results in the change in concrete strength properties [25]. At temperatures between 20 and 200 °C, hairline cracks and voids form inside the specimen due to the evaporation of free water inside the concrete. The water and water vapor in the crevice are pressurized due to temperature rise, which creates tension on the surrounding concrete. Stress is concentrated at the seam tip after the specimen is loaded. Stress concentration contributes to crack expansion and a slow decrease in bond strength. The free water inside the specimen evaporates when the temperature is between 200 and 300 °C. Coarse aggregates and cement paste within the concrete have unequal temperature expansion coefficients and the temperature deformation difference causes cracks to form at the aggregate interface. This causes a slight reduction in the tensile strength of the concrete. These factors of the reduction in bond strength are noticeable. However, after 400 °C, the temperature deformation difference between the aggregate and cement paste continues to grow and interface cracks develop and spread. Cement hydration products, such as $\text{Ca}(\text{OH})_2$, dehydrate and expand in volume, which causes

the cracks to expand. When the temperature rises above 600 °C, the quartz component of the unhydrated particles and aggregates in the cement decomposes and crystallizes, which results in the development of large-scale expansion. As a result, cumulative damage at high temperatures appears as follows: cracks and voids are formed inside the concrete by water evaporation; the difference between the thermal properties of coarse aggregates and cement paste produces deformation gaps and internal stresses and interface cracks are formed; coarse aggregates are ruptured by thermal expansion. Eventually, these microcracks in turn reduce the tensile strength of concrete more severely than the compressive strength of concrete, leading to the different failure patterns of the NC specimens at different temperatures. However, steel fibers in the heated specimens are able to control these cracks. With the increase in V_f , the ultimate bond strength of the SFRC specimens increases.

4. Bond–Slip Relationship Model at High Temperature

4.1. Establishment of the Bond–Slip Relationship Model

Many scholars have recently conducted research in relation to models of the bond–slip relationship between rebar and concrete. Haraji [26] used the rising section of the BPE model [27] to describe rebar and NC bond–slip curves, demonstrating that the predicated curves fit well with test results. Xu [28] described the bond–slip curve between rebar and fiber concrete by modifying the concrete stress–strain relationship model of Guo [29]. It was found that the modified model was in good agreement with the bond behavior between rebar and fiber concrete through the pullout test.

Based on the bond–slip curves obtained from the tests in this study, the bond behavior between rebar and SFRC was found to be similar to that of the BPE model and the modified Guo model after comparison. Therefore, the parameters in the model were modified on the basis of the bond–slip relationship between rebar and NC. As shown in Figure 13, the rising section of the BPE model and the falling section of the Guo model were extended to SFRC. A model of the bond–slip relationship between rebar and SFRC is now proposed in which temperature and V_f are considered.

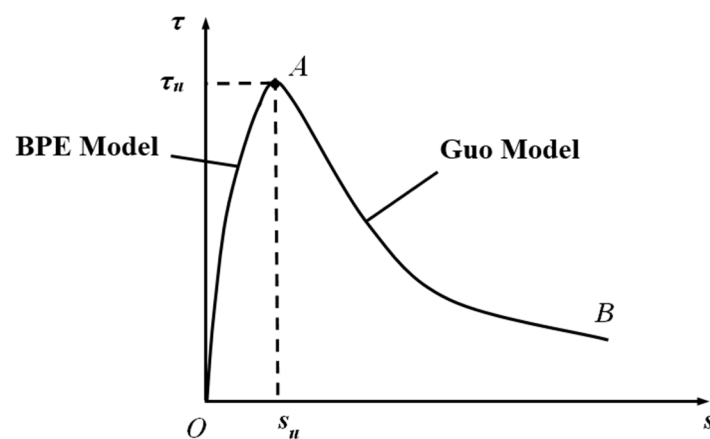


Figure 13. Bond–slip relationship model.

The bond–slip curve equation can be expressed in a dimensionless way. As shown in Figure 13, OA is the rising section and the curve has a nonlinear upward trend. AB is a falling segment and the curve shows a nonlinear decreasing trend. The formulas are as follows.

$$\text{Rising section } y = x^\eta \quad 0 \leq x \leq 1 \quad (4)$$

$$\text{Falling section } y = \frac{x}{\eta(x-1)^2 + x} \quad x \geq 1 \quad (5)$$

where φ and η are the fitting parameters determined from the experimental results, $0 \leq \varphi \leq 1$. The rising section ($0 \leq x \leq 1$) and the falling section ($x \geq 1$) are continuous at the peak point of the curve.

$$x = \frac{s}{s_u} \quad (6)$$

$$y = \frac{\tau}{\tau_u} \quad (7)$$

where τ_u is the ultimate bond stress of the curve. s_u is the slip corresponding to ultimate bond stress.

According to the characteristics of the curve, its mathematical expressions meet the requirements of the following geometric conditions.

- (1) $x = 0, y = 0$;
- (2) $0 \leq x < 1, d^2y/dx^2 > 0$,
slope of the curve (dy/dx) decreases monotonically, no inflection point;
- (3) $x = 1, y = 1, dy/dx = 0$;
- (4) $x \rightarrow \infty, dy/dx \rightarrow 0$;
- (5) $x \geq 0, 0 \leq y \leq 1$.

4.2. Parameter Solving and Model Validation

According to the test results, splitting failure of the NC specimens occurred below 400 °C. Therefore, the NC specimens were not fitted.

The relationships between parameters φ and η and T and V_f are used in the following equations.

$$\varphi = \ln(1.210 - 0.021V_f) + e^{\frac{1}{0.168V_f - 1.246} - 0.024T' + 0.007T'^2} \quad (8)$$

$$\frac{\eta V_f^T}{\eta V_f^{20}} = \ln(\lambda + \gamma V_f) + e^{-2.255 - 0.236T' + 0.020T'^2} \quad (9)$$

$$\begin{cases} \lambda = 3.163 - 0.710e^{0.274T'} \\ \gamma = -0.049(1 - e^{0.717T'}) \end{cases} \quad (10)$$

where V_f is the fraction of volume by steel fiber, $V_f = 0.5, 1.0, 1.5$; T' is the temperature influence coefficient, $T' = T/100$; T is the maximum test temperature, $T = 20, 200, 400$ and 600 °C; η_v^T is the fitting parameter of the falling section; λ and γ are temperature-dependent coefficients.

The parameters φ and η obtained by fitting are shown in Table 6.

Table 6. Fitted model parameters.

Specimen ID	BPE Model					Guo Model		
	φ_T	R^2	φ_C	φ_C/φ_T	η_T	R^2	η_C	η_C/η_T
SF0.5T20	0.595	0.967	0.603	1.014	2.805	0.975	2.756	0.983
SF1T20	0.556	0.951	0.568	1.021	0.662	0.941	0.651	0.984
SF1.5T20	0.525	0.920	0.530	1.008	0.707	0.925	0.697	0.986
SF0.5T200	0.617	0.944	0.597	0.967	2.030	0.946	2.161	1.064
SF1T200	0.560	0.925	0.562	1.002	0.490	0.870	0.535	1.093
SF1.5T200	0.500	0.906	0.524	1.049	0.564	0.820	0.598	1.060
SF0.5T400	0.563	0.902	0.612	1.088	1.244	0.925	1.191	0.957
SF1T400	0.578	0.905	0.576	0.997	0.507	0.868	0.445	0.878
SF1.5T400	0.541	0.871	0.537	0.993	0.646	0.821	0.616	0.953
SF0.5T600	0.638	0.960	0.653	1.025	0.794	0.975	0.823	1.037
SF1T600	0.614	0.878	0.615	1.001	0.706	0.973	0.774	1.096
SF1.5T600	0.605	0.918	0.573	0.947	1.321	0.965	1.153	0.873

Note: φ_T, η_T denote the test values, respectively; φ_C, η_C denote the calculated values, respectively; R^2 denotes the correlation coefficient.

The average ratio of calculated values to test values of parameter φ is 1.009 and the coefficient of variation is 3.57%, as shown in Table 6. The average ratio of calculated values to test values of parameter η is 0.997 and the coefficient of variation is 7.57%. The calculated results are in good agreement with the test results.

Based on Equations (4)–(10), the fitting curves were plotted as shown in Figure 14. The results show that the bond–slip relationship equation established in this paper is in good agreement with the test results.

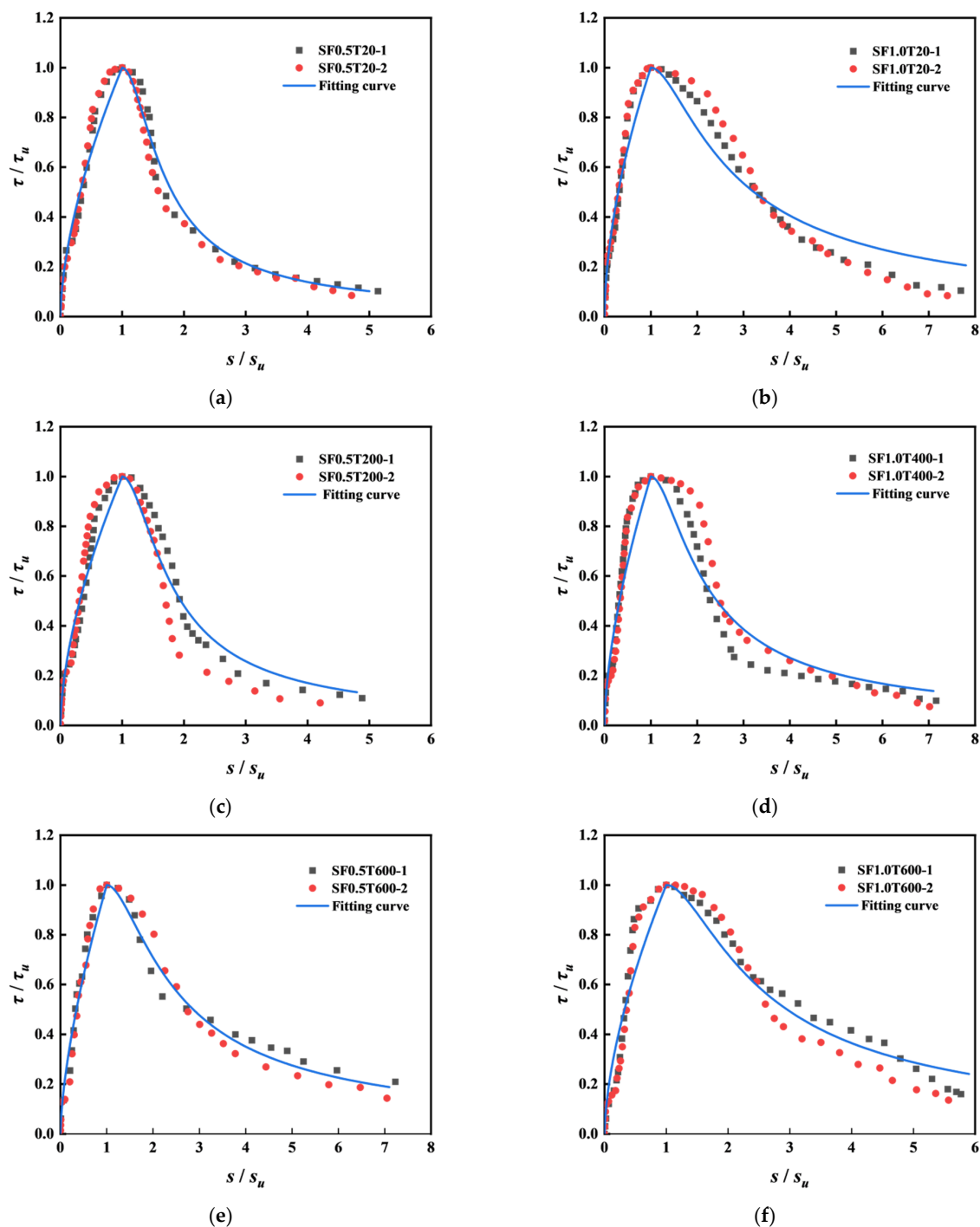


Figure 14. Cont.

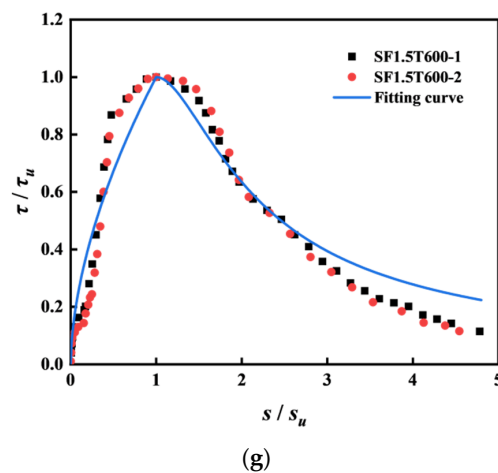


Figure 14. Partial test curves and fitted curves of (a) SF0.5T20 group, (b) SF1.0T20 group, (c) SF0.5T200 group, (d) SF1.0T400 group, (e) SF0.5T600 group, (f) SF1.0T600 group, and (g) SF1.5T600 group.

5. Conclusions

Central pull-out tests were carried out to investigate the bond behavior of high-strength rebar and SFRC at high temperatures in this study. The conclusions based on the experiment are as follows.

- (1) A steel fiber reinforcement factor (β_V) and temperature reduction factor (β_T) were introduced to analyze the effect of the fraction of steel fiber by volume (V_f) and temperature on bond strength. The effect of V_f on the improvement in bond strength is more obvious between 400 and 600 °C. The bond strength between rebar and SFRC decays significantly between 400 and 600 °C.
- (2) The interaction between the steel fibers and the aggregate creates a rigid skeleton for bridging cracks. The bridging action retards the expansion of concrete cracks. This improves the bond strength between rebar and SFRC.
- (3) The bond–slip curve between rebar and SFRC can be divided into five stages, namely the initial micro-slide phase, slip phase, splitting failure phase, stress drop phase and residual pull-out phase. At the splitting failure phase, the rising trend of curves becomes flatter with the increase in V_f before the peak point and the bond–slip curve near the peak point flattens out with the increase in temperature at the same V_f . At the residual pull-out phase, the bond–slip curves of SFRC specimens appear as longer horizontal segments. This indicates that residual bond stress is kept at high temperatures.
- (4) In NC specimens, splitting failure occurred below 400 °C and split-pullout failure occurred above 600 °C. In comparison, split-pullout failure occurred in all SFRC specimens at each temperature. The crack width after the failure was smaller for the SFRC specimen than for the NC specimen.
- (5) A model of the bond–slip relationship between rebar and SFRC considering the temperature and the fraction of steel fiber by volume was proposed by improving the existing model of the bond–slip relationship between rebar and concrete. The results show that the bond–slip relationship model established in this paper is in good agreement with the test results.

Author Contributions: Conceptualization, X.L.; methodology, X.L.; software, C.L.; validation, X.L., C.L. and L.Z. (Lichen Zhou); formal analysis, C.L.; investigation, L.Z. (Li Zheng); resources, X.L. and L.Z. (Lichen Zhou); data curation, L.Z. (Lichen Zhou) and C.L.; writing – original draft, C.L.; writing – review & editing, X.L., Y.C. and C.L.; supervision, X.L. and C.L.; funding acquisition, Y.C. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by the National Natural Science Foundation of China [grant number 52108223, 51922052, 51778309].

Data Availability Statement: Not applicable.

Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

- Vijayan, D.S.; Sivasuriyan, A.; Parthiban, D.; Jakimiuk, A.; Bayat, H.; Podlasek, A.; Vaverkova, M.D.; Koda, E. A Comprehensive Analysis of the Use of SFRC in Structures and Its Current State of Development in the Construction Industry. *Materials* **2022**, *15*, 7012. [\[CrossRef\]](#) [\[PubMed\]](#)
- Amin, M.N.; Ahmad, W.; Khan, K.; Ahmad, A. Steel Fiber-Reinforced Concrete: A Systematic Review of the Research Progress and Knowledge Mapping. *Materials* **2022**, *15*, 6155. [\[CrossRef\]](#) [\[PubMed\]](#)
- Xiang, D.; Liu, S.; Li, Y.; Liu, Y. Improvement of flexural and cyclic performance of bridge deck slabs by utilizing steel fiber reinforced concrete (SFRC). *Constr. Build. Mater.* **2022**, *329*, 127184. [\[CrossRef\]](#)
- Kim, M.S.; Lee, S.S. Design study of steel fibre reinforced concrete shaft lining for swelling ground in Toronto, Canada. *Appl. Sci.* **2021**, *11*, 3490. [\[CrossRef\]](#)
- Wang, X.L.; Fan, F.F.; Lai, J.X.; Xie, Y.L. Steel fiber reinforced concrete: A review of its material properties and usage in tunnel lining. *Structures* **2021**, *34*, 1080–1098. [\[CrossRef\]](#)
- Yixin, C.; Jianye, Z.; Jicheng, M.; Shunli, Z.; Yongsheng, L.; Zhixuan, Z. Tensile strength and fracture toughness of steel fiber reinforced concrete measured from small notched beams. *Case Stud. Constr. Mater.* **2022**, *17*, e1401. [\[CrossRef\]](#)
- Singh Negi, B.; Jain, K. Shear resistant mechanisms in steel fiber reinforced concrete beams: An analytical investigation. *Structures* **2022**, *39*, 607–619. [\[CrossRef\]](#)
- Wang, Y.; Jin, H.; Demartino, C.; Chen, W.; Yu, Y. Mechanical properties of SFRC: Database construction and model prediction. *Case Stud. Constr. Mater.* **2022**, *17*, e1484. [\[CrossRef\]](#)
- Koksal, F.; Rao, K.S.; Babayev, Z.; Kaya, M. Effect of Steel Fibres on Flexural Toughness of Concrete and RC Beams. *Arab. J. Sci. Eng.* **2022**, *47*, 4375–4384. [\[CrossRef\]](#)
- Hao, Y.F.; Xiao, D.D.; Hao, H.; Li, J.; Li, J. Experimental study of reinforced concrete beams reinforced with hybrid spiral-hooked end steel fibres under static and impact loads. *Adv. Struct. Eng.* **2022**, *25*, 3019–3030. [\[CrossRef\]](#)
- Cui, Y.; Qu, S.; Bao, J.; Zhang, P. Bond performance of steel bar and fly ash-based geopolymer concrete in beam end tests. *Polymers* **2022**, *14*, 2012. [\[CrossRef\]](#)
- Cui, Y.; Zhang, P.; Bao, J. Bond stress between steel-reinforced bars and fly ash-based geopolymer concrete. *Adv. Mater. Sci. Eng.* **2020**, *2020*, 1–11. [\[CrossRef\]](#)
- Garcia-Taengua, E.; Marti-Vargas, J.R.; Serna, P. Bond of reinforcing bars to steel fiber reinforced concrete. *Constr. Build. Mater.* **2016**, *105*, 275–284. [\[CrossRef\]](#)
- Chu, S.H.; Kwan, A.K.H. Crack mitigation utilizing enhanced bond of rebars in SFRC. *Structures* **2021**, *33*, 4141–4147. [\[CrossRef\]](#)
- Chu, S.H.; Kwan, A.K.H. A new bond model for reinforcing bars in steel fibre reinforced concrete. *Cem. Concr. Compos.* **2019**, *104*, 103405. [\[CrossRef\]](#)
- Gao, D.; Huang, Y.; Chen, G.; Yang, L. Bond stress distribution analysis between steel bar and steel fiber reinforced concrete using midpoint stress interpolation method. *Constr. Build. Mater.* **2020**, *260*, 119866. [\[CrossRef\]](#)
- Zhang, X.H.; Zhang, W.; Cao, C.D.; Xu, F.; Yang, C.Q. Positive effects of aligned steel fiber on bond behavior between steel rebar and concrete. *Cement Concrete Comp.* **2020**, *114*, 103828. [\[CrossRef\]](#)
- Fakoor, M.; Nematzadeh, M. A new post-peak behavior assessment approach for effect of steel fibers on bond stress-slip relationship of concrete and steel bar after exposure to high temperatures. *Constr. Build. Mater.* **2021**, *278*, 122340. [\[CrossRef\]](#)
- Varona, F.B.; Baeza, F.J.; Bru, D.; Ivorra, S. Evolution of the bond strength between reinforcing steel and fibre reinforced concrete after high temperature exposure. *Constr. Build. Mater.* **2018**, *176*, 359–370. [\[CrossRef\]](#)
- GBT 1499.2-2018; Steel for the Reinforcement of Concrete—Part 2: Hot Rolled Ribbed Bars. Standardization Administration of the People's Republic of China: Beijing, China, 2018.
- Zhang, W.; Lee, D.; Lee, C.; Zhang, X.H.; Ikechukwu, O. Bond performance of SFRC considering random distributions of aggregates and steel fibers. *Constr. Build. Mater.* **2021**, *291*, 1–14. [\[CrossRef\]](#)
- Larsen, I.L.; Thorstensen, R.T. The influence of steel fibres on compressive and tensile strength of ultra high performance concrete: A review. *Constr. Build. Mater.* **2020**, *256*, 119459. [\[CrossRef\]](#)
- Sozen, M.A. *Principles of Reinforced Concrete Design*; CRC Press: Boca Raton, FL, USA, 2014.
- Pothisiri, T.; Panedpojaman, P. Modeling of bonding between steel rebar and concrete at elevated temperatures. *Constr. Build. Mater.* **2012**, *27*, 130–140. [\[CrossRef\]](#)
- Guo, Z.H.; Shi, X.D. *Behaviour of Reinforced Concrete at Elevated Temperature and Its Calculation*; Tsinghua University Press: Beijing, China, 2003.
- Harajli, M.H. Development/splice strength of reinforcing bars embedded in plain and fiber reinforced concrete. *ACI Mater. J.* **1994**, *91*, 511–520.

27. Rossetti, V.A.; Galeota, D.; Giammatteo, M.M. Local bond stress-slip relationships of glass fibre reinforced plastic bars embedded in concrete. *Mater. Struct.* **1995**, *28*, 340–344. [[CrossRef](#)]
28. Xu, L.H.; Chen, P.; Huang, L.; Zhang, A.L.; Li, B. Experimental research on bond properties between hybrid steel-polypropylenefiber reinforced concrete and deformed bar. *China Civ. Eng. J.* **2015**, *48*, 15–22.
29. Guo, Z.H. *Strength and Deformation of Concrete-Test Basis and Principal Relationship*; Tsinghua University Press: Beijing, China, 1997.

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.