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Experimental Characteristics Impact on Bond Strength at Elevated Temperatures: A Literature Review

Ahmed Ali[1*], Éva Lublóy[1]

[1] Department of Construction Materials and Technologies, Faculty of Civil Engineering, Budapest University of Technology and Economics, Műegyetem rkp. 3., H-1111 Budapest, Hungary
[*] Corresponding author, e-mail: ahmedomar.civil@hotmail.com

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Abstract
Evaluating structure strength characteristics post-fire or when subjected to prolonged periods of elevated temperature is essential, as these conditions degrade concrete strength, specifically bond strength between rebar and concrete. Residual bond strength can be determined through direct pull-out or beam tests. Several factors influence bond strength at normal temperatures, including the properties of the concrete and the rebar, the specimens' characteristics, the concrete cover, and the stress state of the specimen during testing. The situation becomes more complex at higher temperatures as heating introduces additional variables. In this context, the bond is influenced by the heating characteristics of the specimen, such as the heating rate and cooling process. While researchers acknowledge these factors, the degree of their impact is still a subject of debate. This paper aims to review the existing literature on bond strength at elevated temperatures and to explore how various factors influence this strength under such conditions. The study gathers data to examine the bond-slip curve at high temperatures, the effects of experimental variables on bond strength, and the residual bond strength after exposure to elevated temperatures. Key experimental characteristics considered in this paper include the heating procedure, heating rate, heating duration, cooling regime, properties of the rebar, properties of the specimen, and the relationship between concrete strength and bond strength.

Keywords
bond, elevated temperature, pullout test, beam test, residual bond strength

1 Introduction

The phenomenon where force is transferred from reinforcement to concrete is known as "bond" [1, 2]. Researchers [3–6] generally employ the conventional definition of average bond stress, which is characterized as the rate of axial force transfer between a reinforcing bar and the surrounding concrete, quantified as the axial force (F) normalized by the nominal contact area over the embedment length. This definition is given in Eq. (1).

$$\tau_b = \frac{F}{\pi d_b l_b}, \quad (1)$$

where:

- τ_b: Bond stress
- F: Applied tensile force on the rebar,
- d_b: Diameter of the rebar
- l_b: Embedment length

This standard bond stress definition oversimplifies reality by treating load transfer as uniformly distributed across the nominal surface [7]. As it is well established that the bond stress distribution is not uniform [8, 9]. Moreover, conventional equations often neglect the effects of slip between the reinforcement and concrete [10]. Structural design codes typically assume a uniform bond stress distribution over the development length for practical calculation [11–14], despite experimental evidence showing non-uniform stress transfer, the last update of EN 1992-1-1 considers a non-linear bond characteristic for anchorage and lab calculations [15]. The impact of concrete grade is less significant than previously assumed in EC2.

In EC2, the bond strength is related to the concrete's tensile strength, which is conventionally estimated from the compressive strength f_{ck}. Previously, the bond was assumed to vary with $f_{ck}^{0.67}$; the current formulation reduces this

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dependence to approximately $f_{ck}^{0.45}$, reflecting the non-uniform stress distribution along practical lap lengths [16].

The real bond behavior in reinforced concrete progresses through three primary mechanisms: the bond mechanism comprises chemical adhesion, mechanical interlock, and friction [17, 18]. This behavior was introduced and described in the *fib* Model Code 2010 through a local bond stress–slip relationship [2], which captures the inherently non-uniform force transfer along the anchorage, and it continues to be adopted in the *fib* Model Code 2020 [19]. The corresponding formulation is presented in Eqs. (2)–(5) and illustrated in Fig 1.

$$\tau_b = \tau_{b\max}\left(\frac{s}{s_1}\right)^{\infty} \quad 0 \le s \le s_1 \tag{2}$$

$$\tau_b = \tau_{b\max} \quad s_1 < s \le s_2 \tag{3}$$

$$\tau_b = \tau_{b\max} - \left(\tau_{b\max} - \tau_{bf}\right)\left(\frac{s - s_2}{s_3 - s_2}\right) \quad s_2 \le s \le s_3 \tag{4}$$

$$\tau_b = \tau_{bf} \quad s_3 \le s\,, \tag{5}$$

where:

- τ_b: Bond stress.
- $\tau_{b\max}$: Maximum bond stress
- τ_{bf}: Residual bond stress.
- s, s_1, s_2, s_3: Characteristic slip values defining the bond stress variation.
- ∞: Exponent for the initial rise of bond stress, based on the bar's relative rib area.

The approximation of uniform distribution remains sufficiently accurate for bonded lengths that are relatively short, generally within the range of 3 to 7 times the bar diameter [20]. These short embedment lengths are recommended to ensure failure occurs by interfacial debonding rather than other modes, and to allow simplification of bond stress calculations by assuming a uniform average stress along the embedded length [21].

According to Eurocode 2, under standard fire exposure, structural elements are required to maintain either their load-bearing capacity, their separating function, their insulation, or a combination of these for a specified duration under fire exposure [22].As a passive fire safety measure, structural fire resistance ensures the integrity of a structure during high-temperature events, unlike active systems that detect or suppress fire [23]. Within this context, the bond strength between concrete and reinforcement becomes critical, as it directly influences the structural performance and stability of concrete elements [24]. At serviceability, bond greatly influences concrete behavior by enabling tension stiffening, where cracked concrete between reinforcing bars continues to carry tensile stress due to the bond's presence [25], which influences crack widths and deflection [26]. At the ultimate limit state, bond slip behavior plays a fundamental role in evaluating the anchorage capacity of reinforcement bars and establishing the minimum development and lap-splice lengths [27].

Bond causes the steel force and concrete to change along the rebar length, causing different strains in the two materials and causing relative displacement known as slip [1, 28]. The measured performance of bond strength is affected by concrete strength, aggregate types, and admixtures [29, 30]. In addition, testing methods also influence bond strength, which is generally conducted by four bond test methods: pull-out test, beam end test, beam anchorage test, and lap splice test [31]; these tests are shown schematically in Figs. 2 and 3.

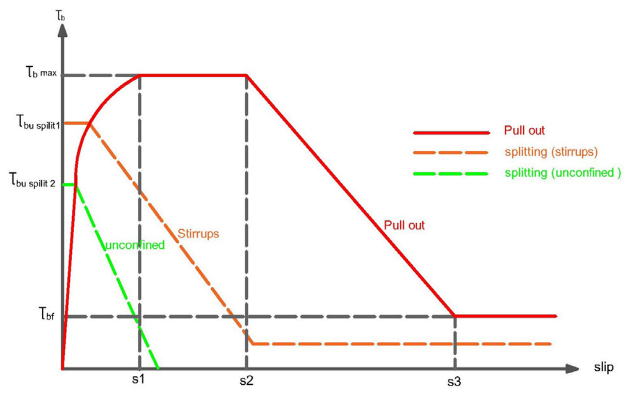

Fig. 1 Fib model for Bond-Slip curve [2]

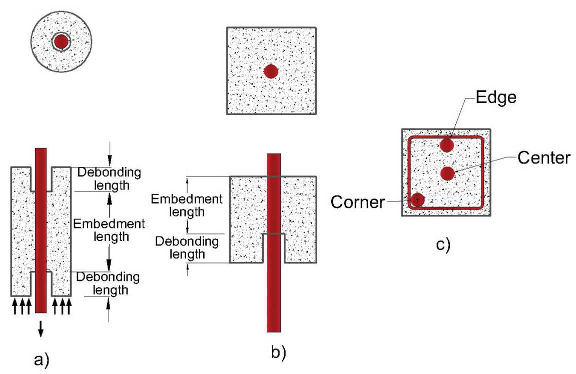

Fig. 2 Schematic drawing of pull-out samples, (a) cylinder specimen, (b) prism specimen, (c) possible rebar locations (prism)

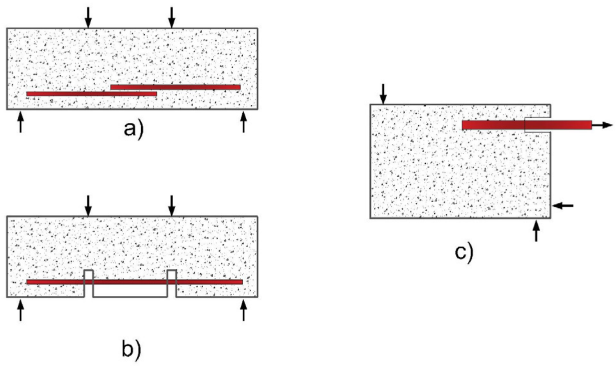

Fig. 3 Schematic of Beam samples: (a) splice beam, (b) beam anchorage specimen, (c) end beam sample

In addition to previous factors at elevated temperatures, other factors become substantial, such as soaking time [32], hating rate [33, 34], and testing regime in terms of loading, heating, and time sequence, which most researcher start their work unaware of their important roles [35], these factors are summarized in Fig. 4.

Elevated temperatures induce internal vapor pressure build-up [36] and thermal gradients in concrete [37], both of which contribute to changes in its physical behavior, including spalling and cracking. Strength reduction primarily results from microcracks, formed by pore-structure coarsening and the degradation of cement paste, particularly Calcium Silicate Hydrate C–S–H, which begins significantly above 600 °C [38], in addition, differential thermal expansion occurs between the aggregates and the cementitious matrix [39, 40], resulting in a weakened bond at the paste–aggregate interface [41–43]. which significantly reduces the member's load-carrying capacity and compromises its structural integrity [44].

As a result of elevated temperature, the bond between the concrete and reinforcing rebar or prestressed rebar is weakened [45–48], compromising the overall structural integrity. Temperatures disrupt bonding mechanisms through thermal stresses, leading to potential bond loss and rebar slippage. Leading reinforced concrete elements to failure when exposed to fire.

This review paper explores the mechanisms of bond degradation and studies the experimental factors that influence bond strength under thermal stress.

2 Bond-Slip curve at elevated temperature

Bond samples fail in two modes: pullout failure and splitting failure, as shown in Fig. 5. Pullout is characterized by shear occurring on the surface of the rebar, while splitting failure involves the loss of concrete cover and typically occurs with a small cover size and the absence of link reinforcement.

The *fib* Model Code curve for the Bond-Slip curve is displayed in Fig. 1. The model is a foundational element for empirical models developed by researchers. Scholars have proposed modifications to the *fib* Model Code to address the temperature influence, as shown in Table 1 [45, 49, 50].

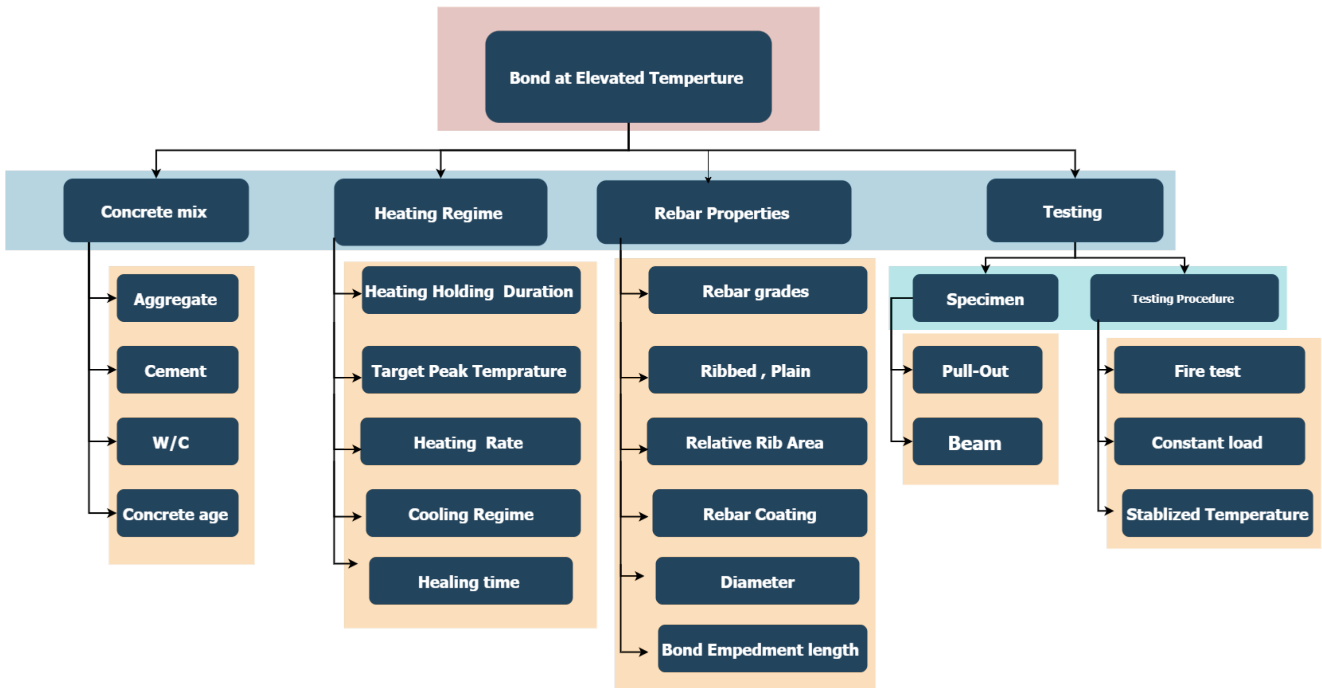

Fig. 4 Influencing factors on bond performance at elevated temperature

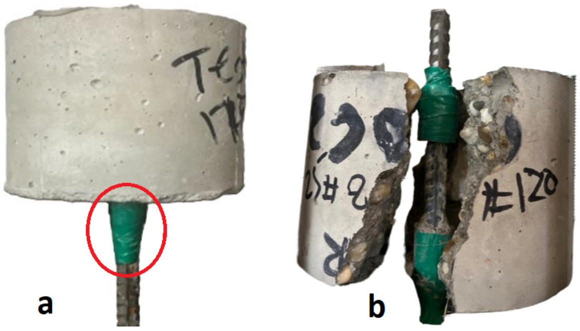

Fig. 5 Pullout cylinder: (a) pullout failure specimen, (b) splitting failure

Lublóy and Hlavicka [45] proposed modifications which depend on aggregate types and the temperature; Aslani and Samali [50] introduced modified formulas for the local bond stress–slip relationship depending on concrete strength, bond lengths, and cooling methods.

$$\frac{\tau_{b\max T}}{\tau_{20}} = 1.0538\left(\frac{f'_{cT}}{f'_c}\right) - 0.0255, \quad 30\ \text{mm} < l_b \le 100\ \text{mm} \tag{6}$$

$$\frac{\tau_{b\max T}}{\tau_{20}} = 0.5273\left(\frac{f'_{cT}}{f'_c}\right) + 04169, \quad 100\ \text{mm} < l_b \le 160\ \text{mm} \tag{7}$$

$$\frac{\tau_{b\max T}}{\tau_{20}} = 0.7905\left(\frac{f'_{cT}}{f'_c}\right) + 0.1862, \quad 30\ \text{mm} \le l_b \le 160\ \text{mm} \tag{8}$$

where:

- $\frac{\tau_{b\max T}}{\tau_{20}}$:Residual bond strength at temperature T
- $\frac{f'_{cT}}{f'_c}$: Residual compressive strength at temperature T
- l_b: Empedment length.

It should be noted that f'_{cT} dependent on the concrete strength (Normal or high) and type of aggregate (Siliceous aggregate), as proposed by Aslani and Samali [50].

Tang addressed the effects of high temperatures on the local bond-slip behavior between rebars and ultra-high-performance concrete [49]. In this modification $\tau_{b\max}$ is given by:

$$\tau_{b\max} = 25.97955 - 0.00844T + 0.039997 fc' + 0.137008 fcT', \tag{9}$$

where:

- $\tau_{b\max}$: Ultimate bond strength after exposure to high temperatures (MPa).
- f'_c: Compressive strength of the concrete (MPa).
- f'_{cT}: Residual compressive strength after exposure to high temperatures (MPa).
- T: Temperature (°C)

Das et al. [51] employed a fire test and modified the fib model by reduction factors for bond strength based on fire exposure duration and amplification factors for slip, as shown in Table 2, and given by the following equations:

$$\tau_{bmfi} = K_{fi}\tau_{bm,amb}, \tag{10}$$

$$S_{i,fi} = K_{si,fi}S_{i,amb}, \tag{11}$$

Table 2 Reduction and amplification factors according to Das et al. [51]

Equation's factors		
Exposure duration (min)	Values of K_{fi} [-]	Values of $S_{i,fi}$ [-]
0	1	1
15	0.5	1.25
30	0.4	1.5
60	0.33	2
90	0.3	2

Table 1 Modified *fib* Model Parameters at Elevated Temperatures

Variables	Tang [49]	Lublóy and Hlavicka [45]					Aslani and Samali [50]	
Aggregate type	quartz sand	quartz		quartz and expanded clay			-	-
Concrete type	HPC	HSC		HSC			HSC	NSC
Temperature (°C)	20-500	20-400	400-800	20-400	500-700	>800	100-800	
$\tau_{b\max T}$	Eq. (9)	$2.5f_{ck}^{0.5}$	$f_{ck}^{0.4}$	$2.0f_{ck}^{0.5}$	$f_{ck}^{0.4}$	0	Eqs. (6)–(8)	
τ_{bf}	$0.4\tau_{b\max T}$	$f_{ck}^{0.5}$	-	$f_{ck}^{0.5}$	-	-	$0.4\tau_{b\max T}$	$0.4\tau_{b\max T}$
s_1 mm	1.00	1.00		1.00			0.5	1
s_2 mm	3.00	3.00		3.00			2	3
∞	0.3	0.4		0.4			0.4	0.3
s_3 mm	clear distance spacing between ribs							

where:

- $\tau_{bm,amb}$, $S_{i,amb}$: Mean bond strength and slip values according to the fib Mode code.

The same formulation is also adopted in the *fib* Model Code 2020 for local bond-slip under fire [19].

In the fib Model Code 2020 [19], the bond stress–slip relationship is extended to account for elevated temperature effects, where it is explicitly formulated to describe bond behavior under such conditions, expressed as follows:

$$\tau_b = \tau_{b\max}\left(\frac{s}{s_{1,T}}\right)^a \quad 0 \le s \le s_{1,T} \tag{12}$$

$$\tau_b = \tau_{b\max,T} \quad s_{1,T} < s \le s_{2,T} \tag{13}$$

$$\tau_b = \tau_{b\max,T} - \left(\tau_{b\max,T} - \tau_{bf,T}\right)\left(\frac{s - s_{2,T}}{s_{3,T} - s_{2,T}}\right) \tag{14}$$

$$s_2 \le s \le s_3$$

$$\tau_b = \tau_{bf,T} \quad s_{3,T} \le s \tag{15}$$

$\tau_{bMax,T}$ for 20 °C< $T \le$ 740 °C is given by the following equation :

$$\tau_{bMax,T} = \tau_{bMax,20}\left(1 - \frac{T-20}{720}\right), \tag{16}$$

where:

- $\tau_{bMax,T}$: Maximum bond strength at T (°C)
- $\tau_{bMax,20}$: Maximum bond spitting or pullout strength at ambient temperature using the *fib* Model Code 2020 equation.

The values for $s_{1,T}$, $s_{2,T}$, and $s_{3,T}$ is given in the *fib* mode code as follows:

I. $s_{1,T}$ for:

- pullout failure

$$s_{1,T} = 6.94 s_{1,20}\left(\frac{740-T}{5020-T}\right) \quad \text{for} \quad 20°\text{C} < T \le 300°\text{C}, \tag{17}$$

$$s_{1,T} = 0.83 s_{1,20}\left(\frac{740-T}{864-T}\right) \quad \text{for} \quad 300°\text{C} < T \le 740°\text{C}, \tag{18}$$

- splitting failure

$$s_{1,T} = 1.11 s_{1,20}\left(\frac{740-T}{820-T}\right) \tag{19}$$

where:

$s_{1,20}$: Diplacment s_1 (mm) under ambient temperature according to *fib* model code 2020.

II. $s_{2,T}$, and $s_{3,T}$ for the condition of pullout failure:

$$s_{2,T} = 2 s_{1,T}$$

$$s_{3,T} = \text{Clear distance between ribs.}$$

III. $s_{2,T}$, and $s_{3,T}$ for the condition of splitting failure:

$$s_{2,T} = s_{1,T}$$

$s_{3,T}$ takes the following values:

$$s3,T = \text{half the clear distance between ribs with stirrups}$$

$$s_{3,T} = 1.2 s_{1,T} \text{ without stirrups.}$$

3 Experimental characteristics affect bond strength at elevated temperature

3.1 Specimen's characteristics

Bond Testing is classified into two main categories: pull-out and beam tests [52]. Pull-out and the end beam test are the most common in rebar-concrete bond testing [53]. The *fib* Model Code 2020 and ASTM A 944 set out the end beam test as a standard method for measuring bond strength [54, 55]. ASTM C234-91a and IS 2770-1 specify a pull-out test using prism specimens [56, 57], BS EN 10080 specifies a beam test or a pull-out test on a prism sample. Both ASTM C234-91a and BS EN 10080 are based on RILEM recommendations [57–59].

On the other hand, at elevated temperatures, there is a prevailing lack of standardization for evaluating thermal and mechanical properties of construction materials at elevated temperatures [60]. The pull-out test is cost-effective, but the ACI Committee 408 stated that it is insufficient for determining development lengths and recommended beam tests [29], because this test is not representative of the actual stress in structural elements, and beam samples replicate the stress condition of the actual structure more than pullout tests. Cairns and Abdalla [61] argue that the pull-out test gives insights into bond behavior related to splitting and emphasize the need for developing relations between the test and structural performance. In addition, the widespread application of pull-out emphasizes its significance in studying performance under unusual conditions such as elevated temperature. The specimen shape and size affect the measured bond strength. Scholars have utilized a prism [33, 62–65] and cylinder specimens [30, 45, 66–68]. While the pull-out test is typically conducted using cubic or prismatic specimens, most studies on bond strength at high temperatures have instead employed cylindrical specimens [69] since cylindrical specimens offer uniform stress and heating distributions [30, 45, 66]. End beam tests proposed by the *fib* Model Code 2020 and ASTM A944-22. Both tests are generally identical; the main difference is in bar location, the *fib* Model Code test examines both test conditions of casting, which gives economic and scientific advantages [54, 55], in addition to rebar location, a few scholars used beam tests [70–73] and fewer studies utilized pull-out and beam test methods to evaluate bond strength at elevated temperatures. Abuhishmeh et al. [37] tested pull-out and end-beam specimens. The end-beam test has a 49% higher bond strength after heat exposure. This highlights the need for more research on bond strength using beam tests and further comparisons between these two-specimen setups.

3.2 Specimens' s concrete cover

Specimens with larger concrete covers fail by pull-out, as the concrete in direct contact with the rib undergoes compressive stress; specimens with smaller concrete covers fail by tensile splitting [62, 63, 68]. This distinction in failure modes highlights the critical role of concrete cover in bond performance and underscores the need to consider cover thickness explicitly when assessing bond strength, particularly under elevated temperature, because reinforced concrete structures depend on the concrete cover to shield the steel from high temperatures during fire exposure [74].

Parametric studies [75] show that reinforced concrete with a low concrete cover to rebar diameter ratio c/d_b is more sensitive to bond strength loss at elevated temperatures, especially with low-strength concrete. A reduced concrete cover not only allows the reinforcement temperature to rise more quickly during a fire but also lessens the concrete's confinement, which is crucial for maintaining bond strength [76].

Morley and Royles [68] studied Pull-out specimens with concrete cover of 25, 32, 46, and 55 mm under stressed residual test conditions. However, only the 55 mm cover was used in the other three heating conditions. Specimens with larger cover depths followed the concrete compressive strength curve, exhibiting relatively large slips. In contrast, specimens with smaller cover depths experienced small slips and followed the tensile strength-temperature curve. According to Sharma et al. [62] larger covers maintain a higher bond strength at ambient and elevated temperatures. This effect diminishes at higher temperatures (500 °C and 700 °C). In Fig. 6, before 500 °C, larger cover gives a larger bond, and the deterioration is similar after reaching 500 °C. This is due to the decline in concrete strength and stiffness [62], which diminishes the effect of concrete cover confinement. They suggested the following equation for residual bond strength

$$\tau_{buT} = \tau_{bu20}\left(1 - \frac{T-20}{780}\right) \qquad (20)$$

where:

- τ_{buT}: Residual bond strength at T (°C)
- τ_{bu20}: Bond strength at (20 °C) using the *fib* Model Code 2010 equation.
- T: Temperature (°C)

Bošnjak et al. [77] used end beam test subjected to a fire test following the ISO-834 fire curve; with 15, 24, and 56 mm concrete covers, the influence on relative residual bond capacity is moderate for covers 56 and 24 mm. After 15 minutes of fire exposure (718 °C), the relative bond capacities were 55% and 45%, respectively. After 60 minutes (986 °C) the bond decreased to 32% and 25%. The enhancement was less than 10% for the concrete cover increased by a factor of 2.5.

3.3 Rebar reinforcement properties

Generally, studies such as [78–81] expose the rebar without insulation during the heating cycle, Ba et al. [65] wrapped the rebar to protect the interface from damage by the rebar's rapid temperature increase. Lee et al. [82] used concrete caps to replicate the conditions during a fire. When a rebar is covered, its temperature is lower than the surface. Future research should study two fire scenarios – before spalling (with cover) and after spalling (exposed rebar).

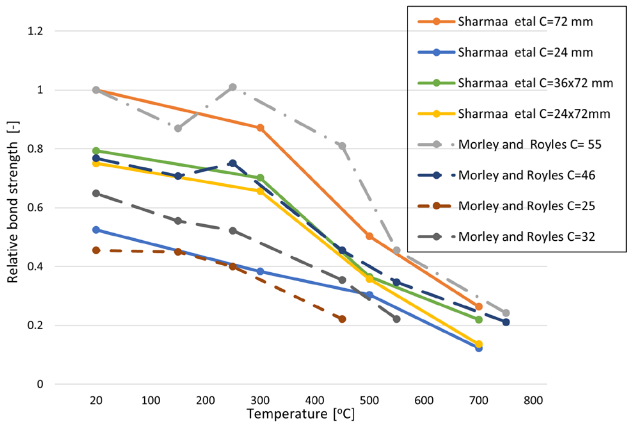

Fig. 6 RBS (Sharmaa et al. to reference bond strength with $C = 72$ mm and for Morely with $C = 55$ mm) as a function of temperature [62, 68]

At elevated temperatures, reinforcing steel typically loses yield strength but regains most of it upon cooling up to about 700 °C, apart from cold-worked steel, which may suffer permanent strength loss [24]. Abuhishmeh et al. [37] examined high-strength reinforcement steels (HSRS) and standard rebars. The results show minimal changes in all rebar after exposure to 350 °C. A reduction of more than 30% in yield and ultimate strength was observed for high-grade rebars, such as grade 690 (ASTM A1035); after 700 °C, only grade 420 rebar (ASTM A706) retained most of its ductility and strength. splitting failure at ambient after exposure to 500 °C occurred during the pullout test. The heat resulted in a 73% reduction in bond strength across all rebar types, with no significant differences between the various types of rebar. The end-beam tests showed an average bond strength reduction of 12.6%, while a 9% improvement in bond behavior was observed for ASTM A706 grade 420. For ASTM A1035 CS grade 690 and ASTM A706 grade 550, a slight reduction in bond strength, specifically 9% and 14%, respectively. Ergün et al. [67]examined plain rebar S220a and two types of deformed rebar, S420a and S500a. They observed a significant loss of bond strength in the temperature ranges of 200 °C–400 °C and 400 °C–600 °C.they suggested the following equations for residual bond strength, which depend on rebar grades.

- For S220a $T > 200$ °C

$$RBS_T = \left[0.618\left(\left(\frac{T}{1000}\right)^2\right) - 1.681\left(\frac{T}{1000}\right) + 1.036\right] RBS_{(T=20°C)}\ (R = 0.99) \tag{21}$$

- For S420a $T > 200$ °C

$$RBS_T = \left[\begin{array}{l} -0.821\left(\left(\frac{T}{1000}\right)^2\right) - 0.0268\left(\frac{T}{1000}\right) \\ +1.023 \end{array}\right] RBS_{(T=20°C)}\ (R = 0.98) \tag{22}$$

- For S500a $T > 200$ °C

$$RBS_T = \left[-0.629\left(\frac{T}{1000}\right)^2 - 0.352\left(\frac{T}{1000}\right) + 0.905\right], RBS_{(T=20°C)}\ (R = 0.96) \tag{23}$$

where

- T: Temperature (°C)
- $RBS_{(T=20\ °C)}$: Residual bond strength at 20 °C.
- RBS_T: Residual bond strength at T °C.

The tests were conducted by Diederichs and Schneider and Ergün et al. [30, 57], and illustrated in Fig. 7, demonstrated that deformed and plain rebar exhibited the same temperature-bond relationships, but with improved performance of deformed rebar. It also indicated that corroded plain rebar performed better than new, as-rolled rebar [30], which was attributed to the increased roughness of the surface. This enhancement depends on the level of corrosion. Ba et al. found that below 400 °C, bond strength increases with corrosion up to a level of 0.05; after that, it decreases. At 600–700 °C, corrosion has little effect on bond strength [65]. Lee et al. found that bond strength generally decreases with temperature. However, at 200 °C, the coated rebar showed an improved bond due to epoxy melting, although the uncoated rebar outperformed the coated rebar at other temperatures [82].

According to Hertz [83] for a temperature rise to 500 °C, the rebar diameter had a relatively small impact on the bond strength degradation, as shown in Fig. 8. Ergün et al. [67] realized that the bond strength after

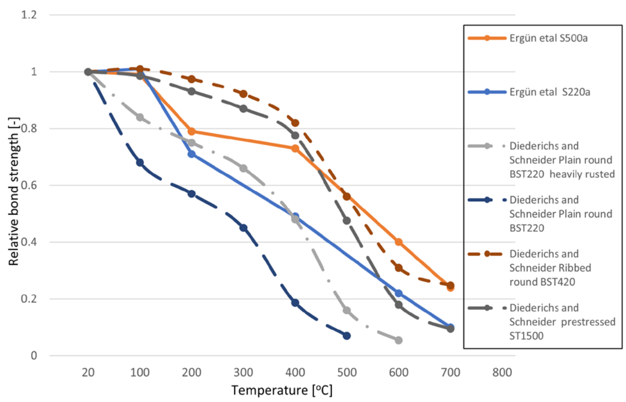

Fig. 7 Influence of rebar rib on residual bond strength at elevated temperature [30, 67]

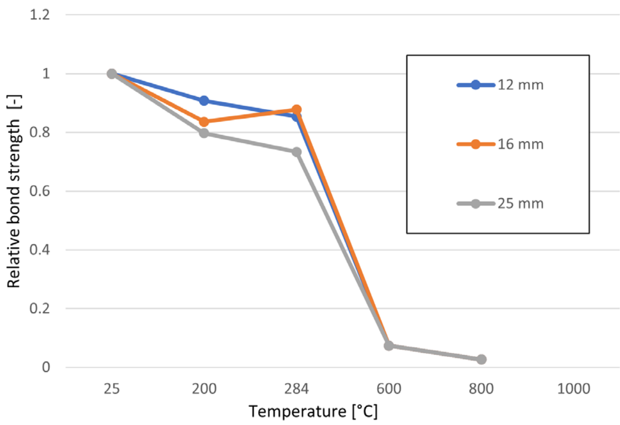

Fig. 8 Effect of diameter on relative bond strength of Danish deformed bar [83]

elevated temperatures decreases as the steel bar diameter increases; Das et al. [51] using a fire test, found the diameter increase from (16–25) mm enhances load capacity; the differences in relative residual bond capacity during shorter fire exposures are minimal, after prolonged fire exposure, the larger rebar diameter exhibits a slightly better residual bond capacity than the smaller one.

Diederichs and Schneider found the effect of the rebar's embedment length on the force-slip curve's profile insignificant, except for l = 110 mm [30]. Also, using a constant force procedure, Muciaccia and Consiglio tested specimens with four and eight-diameter embedment lengths. The results showed no significant difference in bond strength for specimens with centered bars that failed in pullout mode. When rebars were located at the edge and side, and a length of four bars in diameter, the uniform bond stress was approximately 30% to 40% higher. This difference diminished when bond strength was evaluated as a function of temperature [63]. Das et al. found that in the samples with 8, 12, and 16 times the diameter using a fire test and were positioned at the edge and the corner, the load-carrying capacity increased as the embedment length increased while the equivalent bond strength decreased; the residual bond capacity with a rebar at the edge was slightly higher than specimen where the rebar in the corner [64]. Ghazaly et al. tested end beam specimens at 600 °C and 800 °C with rebar located at the edge of a specimen; the bond strength suffered a slight decline when the bonded length was raised from five times the bar diameter to eight times the bar diameter in the reference and residual bond strength after elevated temperature [70]. Liu et al. [84] using two heating procedures, constant load, and constant temperature, shows the failure load of pull-out specimens under constant temperature increases with embedded length, as shown in Fig. 9; however, the ultimate bond stress decreases. Additionally, the embedded length does not affect the failure temperature and failure time of transient pull-out specimens, as shown in Fig. 10.

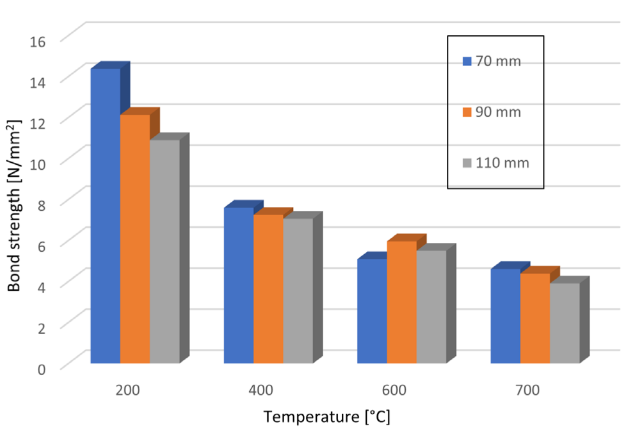

Fig. 9 Embedment length influence on bond using constant temperature [84]

Most studies focus on single rebar behavior at high temperatures, missing interactions in multi-rebar setups, where multiple rebars are present and subjected to thermal stress simultaneously. Das et al. [85] found a notable difference in bond capacity in which twin rebars have lower bond strength due to reduced concrete cover, but fire exposure slightly narrows this gap.

3.4 Influence of thermal processing conditions

Temperature distribution in fire-exposed concrete is highly non-uniform, influenced by fire growth rate, severity, cross-sectional shape, thermal mass, and the stage of fire development [86]. Determining the credible worst-case fire scenario for concrete members presents a significant challenge due to conflicting thermal effects. A high-temperature, short-duration fire may induce explosive spalling caused by rapid thermal gradients, whereas a low-temperature, long-duration fire can result in sustained heat penetration, leading to elevated average temperatures that degrade structural strength and stiffness over time [87].

As a result, the choice of heating protocol – such as maintaining a stabilized high temperature or applying a constant thermal load – has a marked influence on the bond strength between reinforcing steel and concrete following exposure to elevated temperature [30, 63]. In the first method, specimens are heated to a target temperature and tested at laboratory temperature. The selected target temperature steps should accurately capture the changes in concrete properties as they are heated. As concrete is exposed to elevated temperatures, it undergoes a progressive series

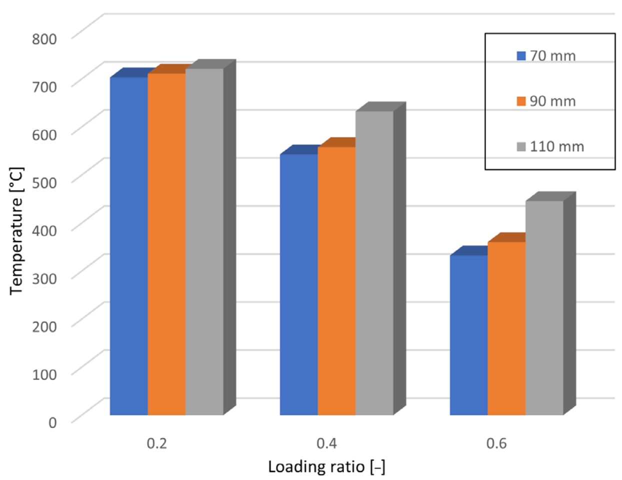

Fig. 10 Embedment length influence on bond using constant load temperature [84]

of mineralogical transformations. These changes can be examined petrographically to determine the maximum temperature the concrete has experienced and to estimate the depth of thermal damage [88]. At the beginning, concrete is largely unaffected up to around 100 °C, experiencing only thermal expansion, but above this temperature, it begins to lose free water as moisture evaporates from its capillary pores [89]. Free water within concrete evaporates between 100 °C and 150 °C when no pore pressure is present, whereas chemically bound water remains stable until temperatures approach approximately 450 °C [90], significant loss of strength commences at 300 °C [91]. Between roughly 400–550 °C, $Ca(OH)_2$ decomposes into CaO and H_2O, leading to pore expansion, microcracking, and considerable weakening of the cement matrix, which undermines the structural integrity of concrete even after cooling. For practical applications, a temperature of approximately 600 °C is generally regarded as the upper limit for maintaining the structural integrity of concrete produced with Portland cement [92–94]. Between 800 °C and 1200 °C, the calcareous components of both the aggregate and the cement paste undergo complete breakdown – through chemical dissociation and intense thermal loading – resulting in a light grey discolouration of the concrete accompanied by extensive micro-cracking, and start to melt after 1200 °C [94]. While elevated temperatures weaken concrete, the cooling phase – especially within the first few days – can worsen damage as calcium oxide absorbs moisture from the air, expands, and further propagates existing cracks [95]; water, which reacts with calcium oxide (CaO) in the heated concrete to form calcium hydroxide ($Ca(OH)_2$). This reaction causes expansion, leading to cracking and further deterioration of the concrete surface. The most critical changes involve the dehydration of calcium hydroxide and the following by potential for subsequent rehydration under appropriate conditions [96]. This general behavior of concrete with elevated temperature is illustrated in Fig. 11.

This procedure includes various stress scenarios, depending on whether the specimen is under stress during the heating cycle and its condition (either hot or cold) during testing, which results in four distinct testing scenarios. In the second stage, specimens are exposed to a continuous load while heated until failure occurs. Fig. 12 summarizes heating procedures for bond strength testing. The procedures aim to replicate structural behavior during and after a fire, considering stress before, during, and after the event. They assess load-bearing capacity during the fire and residual strength afterward.

Morley and Royle's experiments using stabilized temperature procedures indicate that specimens subjected to stress during the heating cycle demonstrate slightly greater strength than those not stressed, as shown in Fig. 13 [68]. This is attributed to the confinement provided by the loading, which prevents crack formation during heating. This may explain why many researchers use this test, as it simplifies the process and provides more conservative bond strength estimates.

Muciaccia and Consiglio [63] analyzed the two procedures; again, the findings revealed that the reduction in bond strength is more pronounced during the constant load procedure. Additionally, the positioning of the rebar significantly affects bond strength in the constant load procedure, unlike what was observed in the stabilized temperature procedures, as shown in Fig. 14.

Both tests for residual bond strength, conducted by Muciaccia and Consiglio [63] Morley and Royle's [68] at a constant temperature without stress during the heating cycle, and subsequently tested for residual strength, show similar behavior when the rebar is located in the center of the specimen. The results also reveal a notable difference in residual bond strength up to approximately 450 °C. Beyond this temperature, the difference in bond strength using different procedures diminishes, with some specimens failing before reaching 700 °C.

The heating rate is a key variable in bond tests, with slow rates usually between 2 °C/min and 10 °C/min [97]. A rapid heating rate, following ISO 834 fire curve [98], can simulate real fire conditions and be compared with other fire tests . Alternatively, a slow rate will eliminate

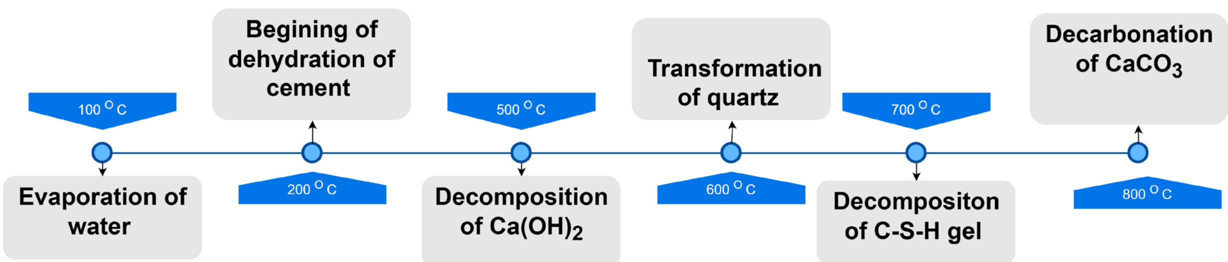

Fig. 11 Effect of elevated temperature on concrete

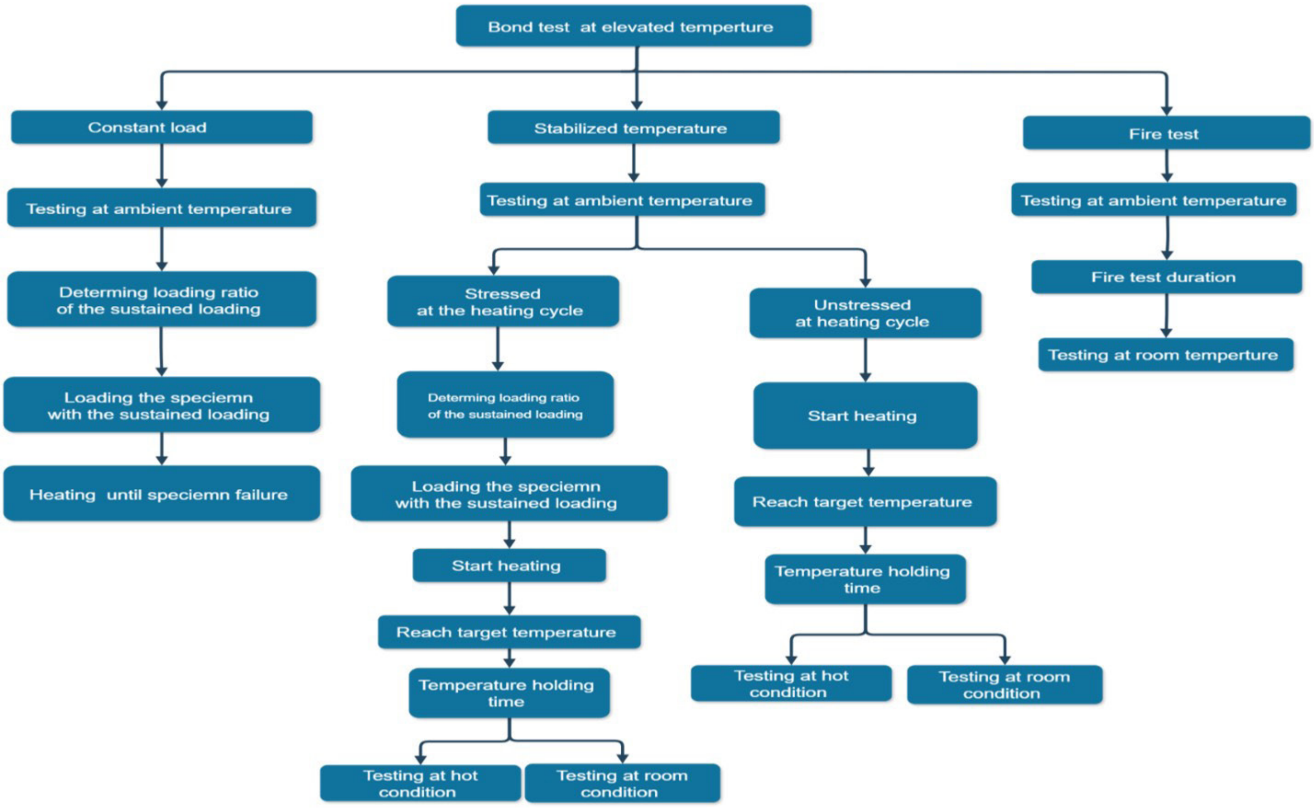

Fig. 12 Bond testing procedure at elevated temperature

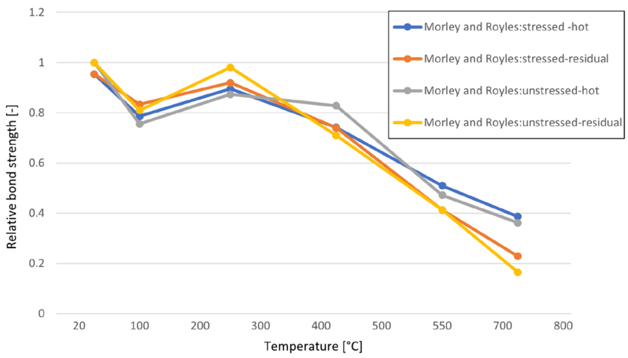

Fig. 13 Bond strength response to elevated temperatures using the stabilized temperature method according to Morley and Royle's [68]

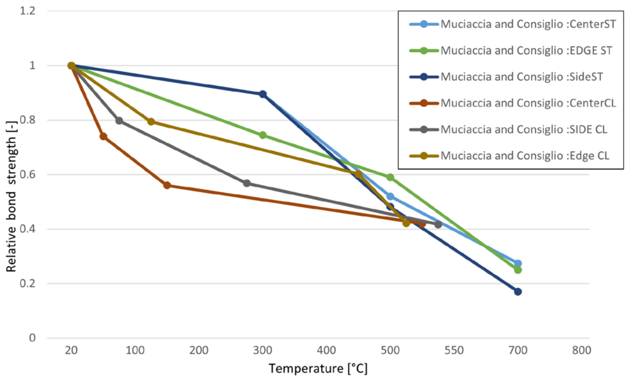

Fig. 14 Response to elevated temperatures using constant load and stabilized temperature according to Muciaccia and Consiglio [63]

the resulting stress from the different movements, isolating temperature effects on bond strength alone [24]. Banoth and Agarwal, using the slow rate of 2 °C/min and a fast rate by following ISO 834 standards, found that a more rapid heating rate leads to faster degradation of bond strength, as shown in Fig. 15 [99]. Lee et al. also found similar results, as shown in Fig. 15, in samples tested by applying 2 °C/min and 15 °C/min, representing slow and fast heating rates, respectively [82].

These results are attributed to stress caused by movement through a significant temperature gradient, as shown in Fig. 16. When a fast temperature is used, there is a considerable difference between the furnace temperature and the bond interface and vice versa. Tajik et al. [100]

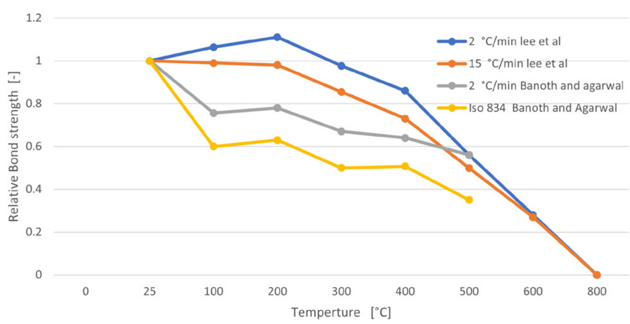

Fig. 15 Relation between heating rate and residual bond [82, 99]

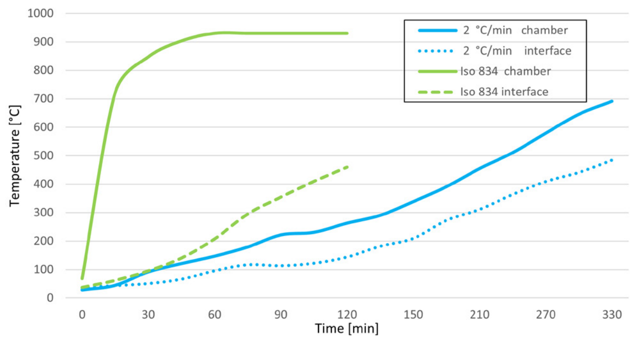

Fig. 16 Relation between thermal loading and thermal response [99]

employed two heating protocols at a rate of 12 °C/min: the uniform protocol, with the furnace set 50 °C above the target temperature to achieve even heating, and the gradient protocol, in which the furnace was limited to 450 °C, inducing a thermal gradient. The uniform method yielded slightly higher bond strength, likely due to reduced thermal damage to the concrete cover.

Soaking time ensures uniform heating after reaching the target temperature and simulates real fire exposure. Ahmed et al. [101] found that increasing soaking time reduces the residual bond strength, showing a difference of about 16% in residual bond strength between 60 and 120 min. exposure time at higher temperatures (400–600 °C). Chiang et al. [102] investigated pull-out specimens by exposing the specimens to temperatures ranging from 240 °C to 550 °C for durations between 0.5 and 3 hours. For all investigated temperatures, the relative bond strength decreases as the exposure time increases, as shown in Fig. 17.

According to a study by Liu et al. [103], the effects of various temperatures (ranging from 120 °C to 350 °C) and heating durations (from 3 to 24 hours) were examined. The study found that bond strength gradually decreases, as shown in Fig. 18, while peak slip gradually increases with longer heating durations, as shown in Fig. 19; observations indicate that the bond performance

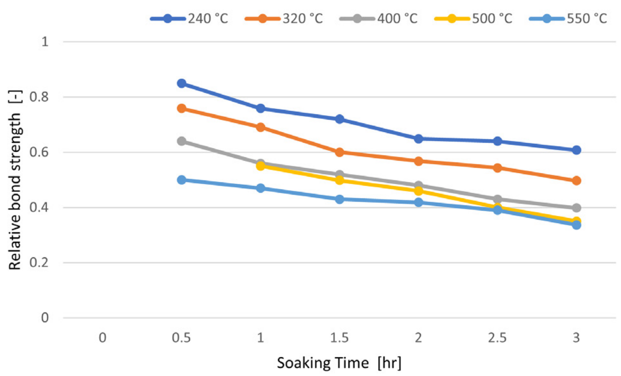

Fig. 17 Relation between Relative bond strength and soaking time [102]

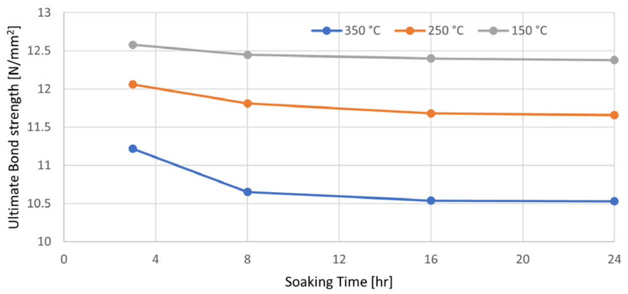

Fig. 18 Relation between ultimate bond strength and soaking time [103]

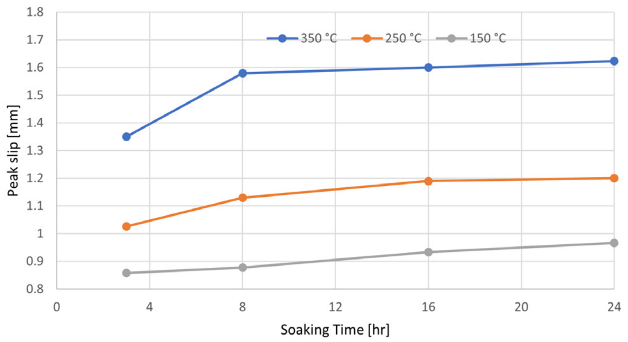

Fig. 19 Relation between peak slip and soaking [103]

stabilizes after 24 hours of exposure to elevated temperatures. Specifically, specimens exposed to temperatures of 150 °C, 250 °C, and 350 °C for 24 hours showed a decrease in bond strength of 1.6%, 3.2%, and 6.2%, respectively. Additionally, the peak slip increased by 12.5%, 17.2%, and 20.1% at the same temperatures.

Ghajari and Yousefpour [72] conducted experiments using end beam test specimens subjected to two different soaking times: one for 2 hours and the other for 24 hours. The specimens were exposed to maximum furnace temperatures of 400 °C, 600 °C, and 700 °C in the first heating regime and 400 °C in the second regime. The maximum core temperatures measured for the 2-hour specimens were 260 °C, 470 °C, and 580 °C, respectively. The highest temperatures at the rebar-concrete interface within the bonded regions for these specimens were calculated to be 310 °C, 487 °C, and 590 °C, respectively. A uniform temperature distribution was achieved for the Series B specimens, which were exposed to the furnace 24 hours.

Researchers examined different cooling methods for heated specimens, including air- and water-cooling techniques. Ahmed et al. [101] studied the effects of two cooling techniques: slow cooling in air and rapid cooling in water. Their findings indicated that the cooling method had no significant impact on residual bond strength. Bingöl and Gül studied both procedures in specimens with different embedment lengths (6, 10, 16) cm [104]. For the shortest length sample, no vital difference was observed in residual bond strength as shown for both methods in Fig. 20, generally because of the small length, which prevented the development of full bond strength. This effect of small embedment length was also observed by Botte and Caspeele, who tested the effect of quenching on water and spraying for 5 minutes [105]. The deterioration was more pronounced for other lengths (10, 16) cm, particularly for the C20 grade specimen tested by Bingöl and Gül and cooled with water. This is due to the thermal effects of rapid cooling in water compared to the gradual cooling that happens when the specimen is cooled in air [104]. Lee et al. also tested pull-out specimens using both methods and found that the cooling methods do not significantly affect samples with uncoated rebar, as shown in Fig. 20. However, a notable difference was observed in the coated rebar samples: those cooled with water exhibited higher strength at 400 °C and 600 °C temperatures. Despite this increase in bond strength, no clear explanation for this phenomenon was identified [82]. It is important to note that ACI Committee 408 states that if surface properties, such as epoxy coatings, are critical, the results of pull-out tests may not be applicable for practical design purposes [29]. Consequently, epoxy-reinforced bars should be tested using beam specimens to evaluate their performance correctly.

4 Conclusion

This work reviews the available literature about bond strength at elevated temperatures. It can be concluded that

1. Further beam tests are needed to better understand residual bond strength at high temperatures. Pull-out tests show faster degradation than beams, but both methods require more study for a complete evaluation.
2. Researchers often use procedures like stabilized temperature, constant load, and fire tests. Most studies focus on constant temperature and residual bond strength, but more comparisons with other methods and deeper exploration of fire tests are needed.
3. A larger cover gives a larger bond strength in ambient temperature and elevated temperatures, but the deterioration of the residual bond strength with temperature is comparable after 500 °C.
4. Faster heating rates result in more rapid bond strength degradation. This heating rate varies depending on whether the goal is to study fire behavior or the bond characteristics.
5. As the soaking time increases, bond strength tends to deteriorate. Studies using pullout specimens indicate that a soaking time of 24 hours stabilizes the reduction in bond strength at temperatures up to 350 °C. Another study using end beam specimens suggests that this soaking duration ensures uniform heat distribution.
6. The cooling regime does not affect the residual bond strength of uncoated rebar, but it affects coated rebar with epoxy. Further research should be conducted.
7. Mild and deformed rebars exhibit the same shape for residual bond strength temperature curves but with more rapid deterioration for plain rebars.
8. Rebar diameters have little to no influence on residual bond strength.
9. Rusted plain rebar performs better than freshly rolled rebar; this depends on the corrosion level and for corrosion levels less than 0.05 and temperatures less than 400 °C.
10. Most research has focused on the bond performance of individual rebars, leaving a gap in understanding the combined behavior of multiple rebars in a reinforced concrete member after fire exposure, as their collective response may differ significantly from individual behaviors.

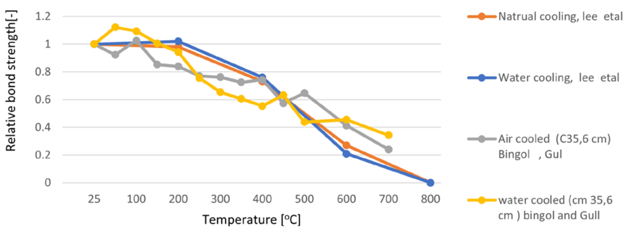

Fig. 20 Impact of cooling methods on bond strength at elevated temperatures [82, 104]

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