Numerical investigations on post-fire bond behaviour of reinforcement in concrete

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Abstract

Reinforced concrete (RC) structures may be subjected to accidental fire loads during their service life. In such a case, it is essential to have appropriate methods for the estimation of post-fire performance. One of the vital aspects for the performance of RC structures is adequate steel-to-concrete bond. The bond behaviour under ambient conditions is very well established. However, an appropriate model for the assessment of the post-fire bond capacity is still lacking. In order to provide more insight into the bond performance after fire, a 3D FE numerical study using a thermo-mechanical model with temperature dependent microplane model for concrete is performed. The effects of concrete cover, confinement through stirrups, fire duration and exposure type on post-fire bond performance are investigated using beam-end specimen. The results show a strong degradation of post-fire bond capacity, which is primarily attributed to the irreversible damage of concrete cover resulting from heating and cooling.

Keywords: Steel-to-concrete bond; Temperature dependent microplate model; Beam-end specimen; ISO 834-1 fire curve; Splitting failure

1   Introduction

Sufficient bond between concrete and steel is essential for the safe and reliable performance of RC structures/structural elements. Bond behaviour under ambient conditions is very well understood and described. For example, *fib* Model Code 2010 [1] provides an analytical bond stress-slip relationship (Fig.1) at room temperature. This model differentiates between the two possible failure modes: pull-out (full bond capacity attained) and concrete splitting (full bond capacity not attained). It should be noted that under ambient conditions full concrete pull-out strength cannot be achieved for the majority of cases found in practice (lap slice) i.e. concrete splitting generally governs the bond behaviour. In case of concrete splitting, the model considers the effects of different parameters such as concrete strength, concrete cover, bar diameter and spacing of transverse reinforcement (confinement). Similar provisions can be found in other codes, such as Eurocode 2 [2], ACI-318 [3] etc.

While the guidelines for bond assessment under ambient conditions are well-established, the recommendations for the bond assessment during or after a fire are rather crude. In general, bond degradation upon heating is compared to the degradation of either compressive or tensile concrete strength at elevated temperatures. For example, acc. to *fib*

Model Code 2010 [1] bond strength degradation can be assumed to correspond to that of tensile strength. Such relatively simplified provisions are based on experimental studies on bond pull-out behaviour using confined test conditions at elevated temperature (slow heating). These tests do not fully capture the realistic boundary conditions that involve fast heating due to fire, low concrete covers and unconfined test setup. Further research is needed to provide a solid basis for the assessment of bond after exposure to fire.
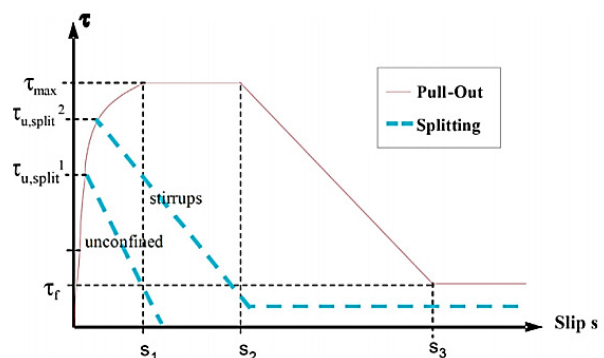

Figure 1. Analytic bond stress-slip relationship for monotonic loading.

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2   Literature review

Significant research has been performed so far to understand the behaviour of bond at elevated temperatures, mainly focusing on the pure bond failure using RILEM pull-out specimen [4]. In the vast majority of the available experimental studies the specimens were heated slowly to the target temperature and the residual bond capacity was measured either at elevated temperatures or after cooling to room temperature. In this paper, "usable bond capacity" refers to the highest bond stress obtained from any type of failure mode (concrete splitting failure or pure pull-out failure) under ambient conditions. The "residual (usable) bond capacity" refers to the maximum bond capacity obtained after exposure to elevated temperature or fire, independent of the failure mode.

Morley and Royles [5] found that there is an insignificant difference in the degradation of bond strength between specimens tested in hot (at high temperatures) and residual state (after cooling). Moreover, they showed that the presence of a mechanical load (applied before heating) does not significantly influence either hot or residual bond capacity. Several authors reported that the bond degradation after exposure to elevated temperature is roughly comparable to that of the concrete compressive strength, e.g. Hertz [6], Morley and Royles [5], Diederichs and Schneider [7], Bingöl and Gül [8], Lublóy and Balázs [9]. List is only exemplary and not exhaustive. Sager [10] was one of the first to clearly differentiate between pull-out and splitting failure after exposure to elevated temperatures.

More recently, Shamseldein et el. [11] performed experimental investigations on residual bond strength after exposure to elevated temperature using beam-end specimens. Compared to the standard pull-out specimen, the chosen specimen geometry (beam-end specimen) and test setup (unconfined) are more representative of the real conditions in a beam. Specimens with a concrete cover of c_{min} = 50 mm were heated at a slow rate (2°C/min) up to 800°C and naturally cooled. The reported bond strength degradation corresponds very well to that observed on pull-out specimens (approx. 30% bond retention after exposure to 800°C). Even in case of splitting failure, the residual bond capacity is consistent with that obtained using pull-out specimens.

The experimental results obtained for slow heating regime are useful for the development of temperature dependent material model for pure bond, but do not provide sufficient basis to estimate the post-fire usable bond strength in a structural member.

Recently, Bošnjak et al. [12] investigated the bond behaviour of reinforcement in concrete after fire (standard ISO 834-1 fire [13]) using beam-end specimen. It is concluded that depending on the heating rate and type, thermally induced damage in concrete can lead to a change in failure mode (from pure bond failure to concrete splitting) and a very sharp decrease in the residual bond capacity. The obtained results highlight the importance of the heating rate for residual bond capacity.

The majority of the numerical work performed so far has been limited to phenomenological models, which directly impose the respective bond degradation with temperature. For example, Gao et al. [14] proposed a lower and upper bound of bond capacity at elevated temperature. Using this approach, a numerical study on RC beams subjected to fire was performed. It is reported that the inclusion of bond slightly improves the structural response of the beams under fire. Similarly, Huang [15] proposed a model for bond degradation and validated his approach on beam and pull-out tests. Both studies highlight the need to consider the bond characteristic in a discrete manner, rather than assuming perfect bond conditions. However, this approach does not offer much insight into failure mechanisms and bond capacity.

Sharma et al. [16] performed a numerical study using the 3D thermo-mechanical model with temperature dependent microplane model as constitutive law for concrete. The effect of different parameters influencing the degradation of bond strength in concrete exposed to standard ISO 834-1 fire [13] was investigated using beam-end specimen. It is reported that the effect of thermally induced cracks on post-fire bond capacity is more pronounced compared to that of material degradation due to elevated temperatures.

The work presented herein is a continuation of the same study which includes detailed investigation of several parameters influencing the post-fire bond performance. The analysis is performed for two heating types, namely one-sided and three-sided heating representing the exposure conditions of a slab and a beam under fire, respectively.

3   Numerical investigations

In this work, the residual bond capacity is numerically investigated considering following parameters (Table 1):

   a)   concrete cover
   b)   spacing of transverse reinforcement (confinement)
   c)   durations of fire and
   d)   types of fire exposure

The employed numerical approach was previously validated against several experiments [12]. In this paper, for brevity reasons only failure modes for analysis under ambient conditions and after one-sided exposure to fire are compared to the experimental data.

The numerical study is performed using beam-end specimen geometry. It was previously shown [12, 17] that this specimen is more suitable for bond investigation under fire than the standard pull-out specimen. The details of the specimen geometry are provided in the Fig.2. Standard ribbed steel rebar (grade BSt 500) with a diameter of 16 mm is used. The test rebar is provided with an initial debonded length of $7d_s$ (it is required to prevent concrete cone failure) followed by a bonded length of $8d_s$ (d_s is diameter of rebar), and a debonded length thereafter. The specimen is internally reinforced with four longitudinal rebar of 12 mm diameter to prevent concrete cone breakout. Closed stirrups of 8 mm diameter are provided at a spacing of 100 mm in the bonded zone. Minimum concrete cover (c_{min}) is varied from $1.5d_s$ =

24 mm) to $3.5d_s = 56$ mm (specimens A, B and C). Maximum concrete cover is kept constant at 142 mm. Furthermore, two different stirrup spacing in the bonded zone are considered, namely 100 mm and 200 mm (specimens B and D). Normal strength concrete confirming to the grade C16/20 according to Eurocode 2 [3] is considered.

Thermo-mechanical modelling

The constitutive law used to describe the behaviour of concrete is the temperature dependent microplane model proposed by Ožbolt et al. [18]. As the first step of coupling between mechanical properties of concrete and temperature, the temperature distribution over a solid structure of volume at a given time is calculated by satisfying the energy conservation law. The total strain of concrete at elevated temperatures is decomposed into three components:

$$\varepsilon_{total} = \varepsilon_m + \varepsilon_{fts} + \varepsilon_{lits} \qquad (1)$$

ε_{total} is the total strain, ε_m is the mechanical strain, ε_{fts} is the free thermal strain and ε_{lits} is the load induced thermal strain (a strain component that develops when concrete is heated while subjected to compressive stress).

The temperature dependency of the microplane model is adopted such that the macroscopic properties of concrete (Young's modulus, compressive and tensile strength and fracture energy) are temperature dependent.

In a given time step during the incremental nonlinear finite element analysis, the material parameters, as well as the temperature distribution, are assumed to be constant, for

more details see [18]. Previous studies [16, 18, 21] have demonstrated the suitability of this modelling approach to realistically capture the behaviour of concrete under fire.

Numerical modelling approach

The numerical study is performed using the in-house developed 3D finite element software MASA. Concrete is modelled using 4-noded tetrahedral elements and test rebar is modelled using 8-noded hexahedral elements, see Fig.3. The material of the test rebar is considered as linear elastic to reduce computational time and avoid steel yielding. Other longitudinal corner rebars and stirrups are modelled using 2-node axial bar elements, for which a trilinear stress-strain law (elasticity modulus, yield strength, hardening modulus and ultimate strength) is defined. Arrangement of the test rebar, stirrups and other longitudinal rebars within concrete are shown in Fig.3c.

The bond between steel and concrete is modelled using 2-node bar elements that transfer compression and shear forces. The bond stress-slip law is shown in Fig.4. The values of bond strength of the elements in the bonded and deboned zone are considered as 12 N/mm² and 0.1 N/mm², respectively.

An overview of the used material properties is provided in Table 2. The temperature dependency of the mechanical and thermal material properties as well as free thermal expansion of the materials are assumed according to Eurocode 2 [19] and Eurocode 3 [20] for concrete and steel, respectively.

Table 1. Matrix for the numerical investigations on beam-end specimens.

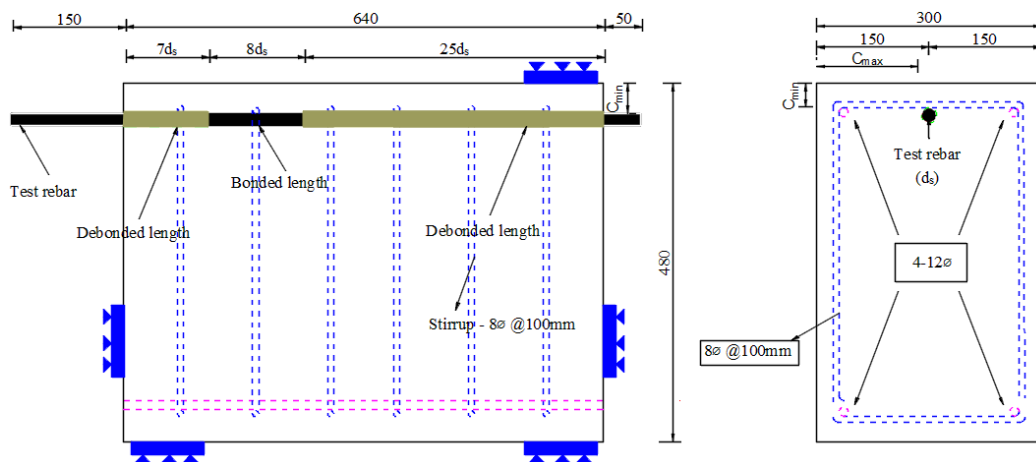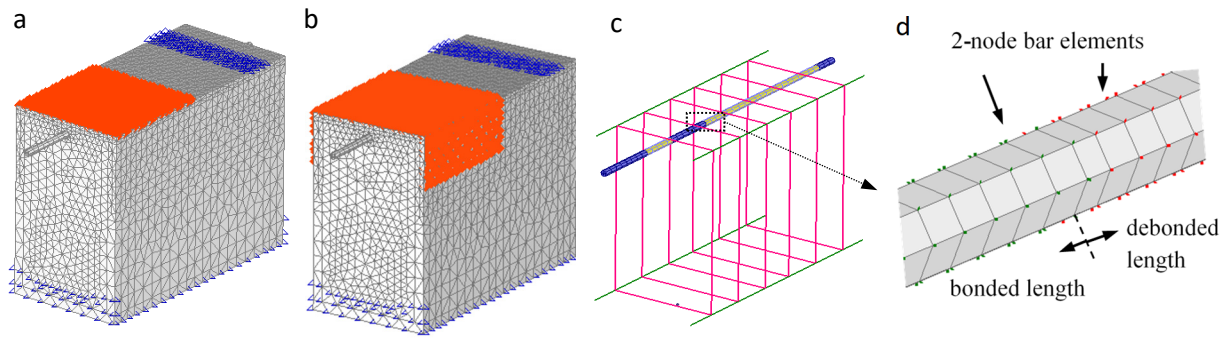| Investigated parameter: **Concrete cover**					
Specimen	Min. clear cover	Rebar diameter d_s	Stirrup spacing	Duration	Heating type
	[xd_s]	[mm]	[mm]	[min]	[-]
A	1.5	16	100	0,30,60	1-and 3-sided
B	2.5	16	100	0,30,60	1-and 3-sided
C	3.5	16	100	0,30,60	1-and 3-sided
Investigated parameter: **Spacing of transverse reinforcement**					
D	2.5	16	200	0,30,60	1-and 3-sided

Figure 2. Beam-end specimen.

Figure 3. FE model: a) Full model for one-sided heating scenario b) Full model for three-sided heating scenario
c) Position of test rebar, other longitudinal rebars and stirrups d) Test rebar with the bond bar elements.

Table 2. Material properties.

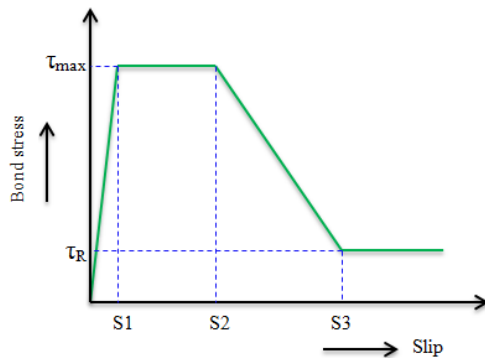| Material	Compressive strength	Tensile Strength	Elasticity modulus	Poisson's ratio	Fracture energy	Yield Strength
	[N/mm^2]	[N/mm^2]	[GPa]	[-]	[N/mm]	[N/mm^2]
Concrete	20	2.25	23.5	0.18	0.06	-
Test rebar	-	-	200	0.33	-	-
Longitudinal rebars	-	550	200	0.33	-	500
Stirrups	-	550	200	0.33	-	500

Figure 4. Idealized bond stress-slip relationship used as constitutive law for 2-node bar element.

The increase of air temperature of the concrete is defined by the ISO 834-1 fire curve [13]:

$$T = T_0 + 345 \log_{10}(8t+1) \qquad (2)$$

where,

T_0 - initial temperature (20°C)

T - actual temperature in °C

t - time in minutes

Two different exposure types are considered, namely one-sided heating and three-sided heating to simulate the exposure conditions in a slab and beam, respectively (Fig.3a and 3b). In the present study, following fire durations are considered: no fire (reference case), 30 min and 60 min. Upon exposing to fire for a specific duration, the specimen is cooled to room temperature. Subsequently, the rebar is pulled out in displacement control.

The upper surface of the specimen is constrained towards its rear end perpendicular to the loading direction to prevent any kind of rotation or uplift. The front surface is constrained far from the test rebar towards loading direction to avoid sliding. All the restrains are provided significantly far from the bonded zone to avoid any influence on the pull-out behaviour.

4 Results

From the numerical simulation applied load versus slip of rebar is obtained. The slip is monitored at the rear (unloaded) end of the rebar to exclude the elastic elongation of the rebar from the total displacement. The total applied load is then converted to average bond stress over the entire bonded length considering the uniform bond stress approach:

$$\tau_b = \frac{P}{\pi d_s l_b} \qquad (3)$$

where,

P - total applied tensile load on the rebar

d_s - diameter of test rebar and

l_b - length of the bonded zone ($8 \times d_s$)

One-sided heating

The results of the numerical simulations for specimen types A, B, C and D (see Table 1) for one-sided fire exposure are plotted in Fig.5. Under ambient conditions (reference case, no fire exposure) all investigated specimens exhibited concrete splitting, whereby the effective bond capacity decreased with decreasing minimum concrete cover (c_{min}). The same failure mode (concrete splitting) is also observed after exposure to fire.

In general, it is found that the fire exposure leads to a significant reduction of the residual bond capacity. This trend

appears to be similar for all the investigated specimens, almost irrespective of the minimum concrete cover (c_{min}). In case of specimen with relatively high concrete cover (specimen C with c_{min} = 56 mm), the residual bond strength after 30 min of fire is only approx. 55% of that under ambient condition. The reduction in bond capacity is slightly more pronounced (about 40% residual capacity) for the specimen with a concrete cover of 24 mm (specimen A with c_{min} = 24 mm). The difference in bond capacity for the two considered concrete covers is mainly resulting from the thermal degradation of concrete in the cover region (reduction of concrete strength with temperature). For smaller c_{min}, the temperature within the cover is higher than in case of larger covers, as visible from temperature profiles shown in Fig.6 (the position of the rebar axis for c_{min} = 24, 40 and 56 mm is marked). The material degradation due to elevated temperature is present in concrete independent of the heating type (slow or fast heating, fire etc.).

However, the reduction of concrete strength (with temperature) alone cannot account for the severe degradation of bond capacity after fire, which is observed even in case of relatively high cover c_{min} and for relatively short fire duration of 30 minutes.

The other cause of bond degradation, which can be more pronounced than material degradation itself, is related to the heating rate. In a fire, the air temperature typically increases very rapidly within a short period of time (e.g. ISO 834-1 standard fire). As the conductivity of concrete is low, a very high increase of surrounding air temperature leads to a relatively high temperature gradient inside the concrete cover, see Fig.6. This in turn results in a more or less pronounced thermally induced damage (cracks), depending on the severity of fire (heating rate). Cooling of the concrete specimens after fire can further aggravate the damage initiated during heating. It was previously shown by the Bošnjak and Sharma [17] that the level of concrete damage caused by the standard fire is substantially higher than that observed in slowly heated specimens. Thermally induced damage can lead to a significant reduction of the usable bond capacity since thermal cracks promote the formation of the longitudinal splitting crack.

Standard ISO 834-1 fire used in the present study exhibits a rapid temperature rise in the initial phase followed by a moderate temperature increase thereafter. Hence, the bond degradation is very severe even for relatively short fire duration (30 minutes). Upon further heating, bond capacity reduces at a significantly lower rate. After 60 min the retention of bond capacity lies between 25 and 40% for all the investigated cases. At this stage, the degradation is again mainly governed by the material degradation due to exposure to elevated temperatures. Due to prolonged exposure to fire, the average temperature in the concrete cover increases, leading to a reduced concrete resistance (strength).
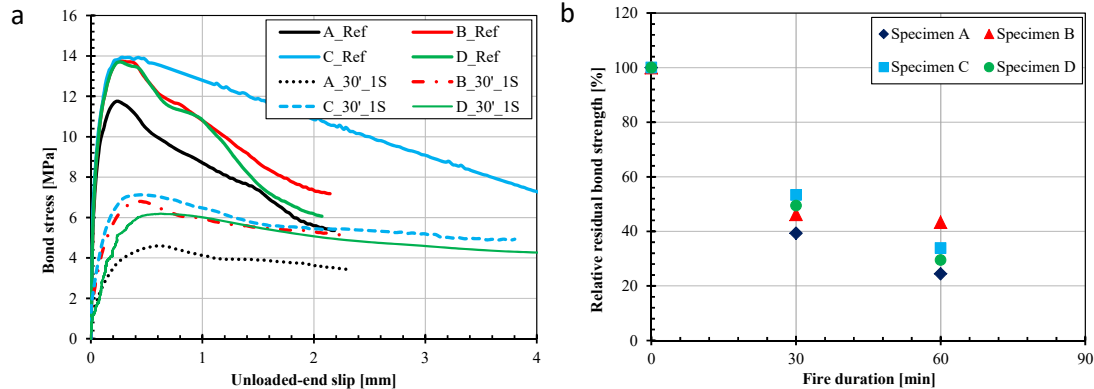

Figure 5. One-sided heating: a) Comparison of bond stress-slip curve obtained from reference tests and after heating (b) Relative residual bond stress for different fire durations.
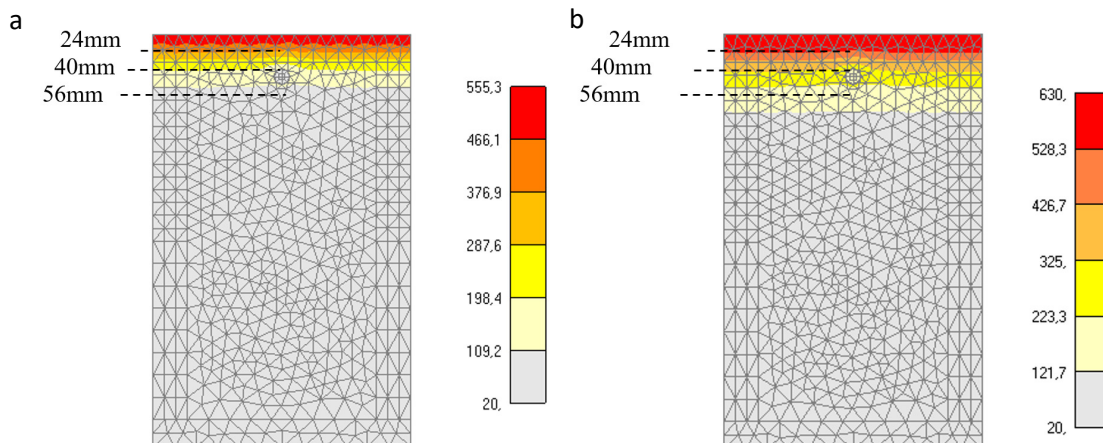

Figure 6. Temperature profile along the cross-section of a specimen with one-sided fire exposure a) after 30 min. and b) after 60 min. of fire.

The effect of confinement by stirrups is investigated by varying the stirrup spacing in specimen types B and D. It can be observed that there is an insignificant difference in the residual bond capacity of the specimen with relatively small amount of transverse reinforcement (stirrup spacing of 200 mm in the bonded region, specimen D) compared to the specimen with more transverse reinforcement (stirrup spacing of 100 mm in the bonded zone, specimen B). In relative terms, the effect of stirrup spacing on the bond capacity after exposure to fire remains similar to that under ambient conditions. Even though concrete strength reduces after exposure to fire, the level of confinement by stirrups cannot be increased due to the thermally induced damage.

It is further found that the fire exposure results in a strong degradation of stiffness for all investigated cases. Stiffness is evaluated as secant stiffness at 50% of the peak stress (k_{50}) and at peak bond stress (k_{100}) for all the investigated fire durations. Relative stiffness is then calculated as the ratio between post-fire stiffness (e.g. k_{50}^{30min}, k_{100}^{30min}) and corresponding k_{50} stiffness of reference (unheated) specimen (e.g. k_{50}^{Ref}). The results are shown in Fig.7. As discussed above, the dominant effect governing the post-fire bond capacity is the thermally induced damage of the concrete cover. This is confirmed by observing the strong degradation of the stiffness after 30 minutes of fire. Thereafter, the stiffness decreases primarily owing to the increased material degradation (effect of elevated temperature).

Validation against experiments

The crack patterns obtained numerically and experimentally from the pull-out tests performed on specimens with a minimum concrete cover of 40 mm (c_{min}) are presented in Fig.8. The crack patterns (in terms of principal tensile strain) for the reference specimen and specimen with one-sided

heating for 30 min of fire are depicted. Red elements in the numerical models represent crack width larger than 0.1 mm, while the red and black lines of the experimental specimens represent thermal cracks and cracks due to mechanical pull-out loading, respectively.

A clear transverse crack can be observed at the end of bonded zone due to the tensile failure of concrete. A prominent longitudinal crack parallel to the test rebar can be observed for both the cases as failure occurs due to the splitting of concrete cover prior to bond pull-out. It can be observed that the numerically obtained failure modes are in very good with the experimental findings, thus indicating the suitability of the employed modelling approach for the analysis of bond capacity after fire exposure.
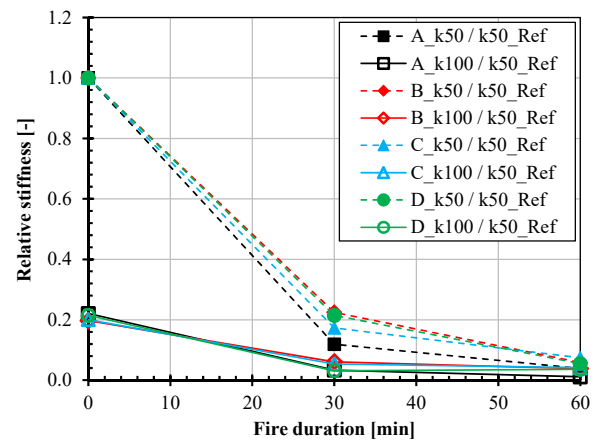

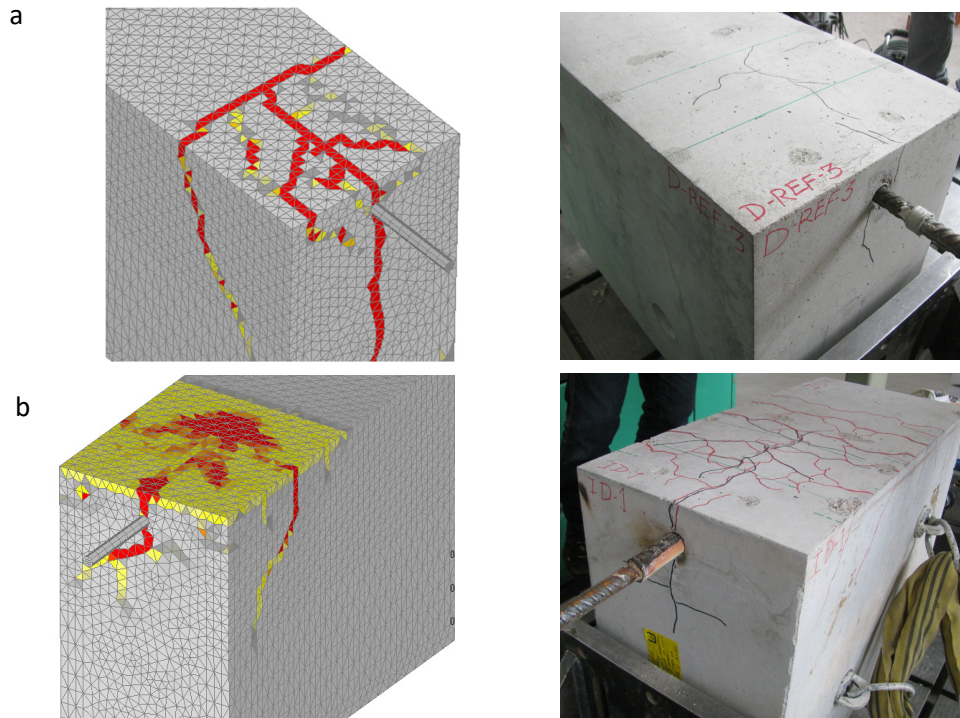**Figure 7.** Relative stiffness reduction for one-sided fire exposure.

Figure 8. Comparison of numerically and experimentally obtained crack pattern for a) reference specimen b) specimen with one-sided heating (specimen D) after 30 min of fire.
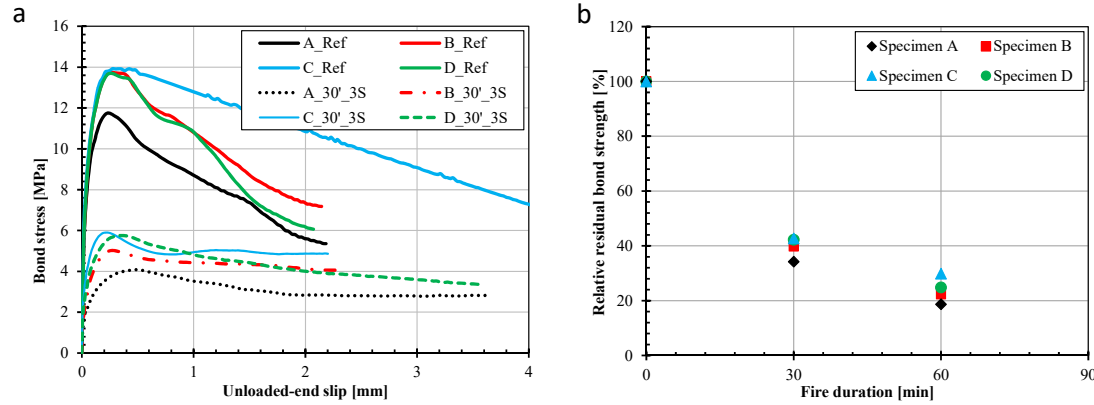
Figure 9. Three-sided heating: (a) Comparison of bond stress-slip curve obtained from reference tests and after heating (b) Relative usable residual bond stress for different fire durations.

Three-sided heating

The results obtained from the numerical simulations of the pull-out tests performed on beam-end specimens (Specimen type A, B, C and D, see Table 1) exposed to three-sided heating are plotted in Fig.9. In Fig.9a, a comparative study of bond stress vs. unloaded-end slip curve between reference tests and post-fire residual tests is presented, whereas in Fig.9b, relative usable residual bond stress of different specimens after exposing to fire for different durations are depicted. The reduction of relative stiffness with fire is shown in Fig.10.

It is interesting to note that though three sides of the specimens are exposed to fire, there are insignificant differences in terms of degradation of bond strength and secant stiffness as compared to the case of one-sided heating. In the present study c_{max} is not varied, but due to the change in c_{min}, the ratio c_{max}/c_{min} ranged from 2.5 to 5.9. The damage of the concrete cover on the two sides with larger cover (c_{max}) has only a minor effect on post-fire bond capacity. The behaviour is dominated by the damage in the smaller concrete cover (thickness of c_{min}). It can be observed from the temperature profiles in Fig.11 that three-sided exposure to fire does not contribute towards a greater thermal gradient in concrete. Due to relatively high side cover (c_{max}), the maximum temperature around the test rebar is only slightly increased compared to that for one-sided heating. For the fire duration of 60 minutes, the temperature of the concrete

cover is somewhat higher than for one-sided heating, resulting in slightly lower bond strength.

Thermally induced cracks appear on the side surfaces (picture omitted for brevity) as well as on the top surface; however, their relative depth (in comparison to c_{max}) is insufficient to cause a marked reduction in bond capacity.

Similarly as in case of one-sided heating, relative bond degradation is almost independent of the concrete cover c_{min} or stirrup spacing.
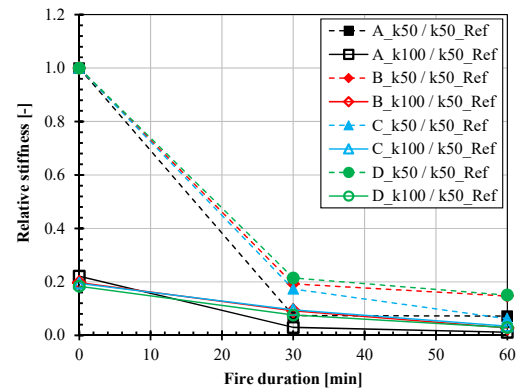

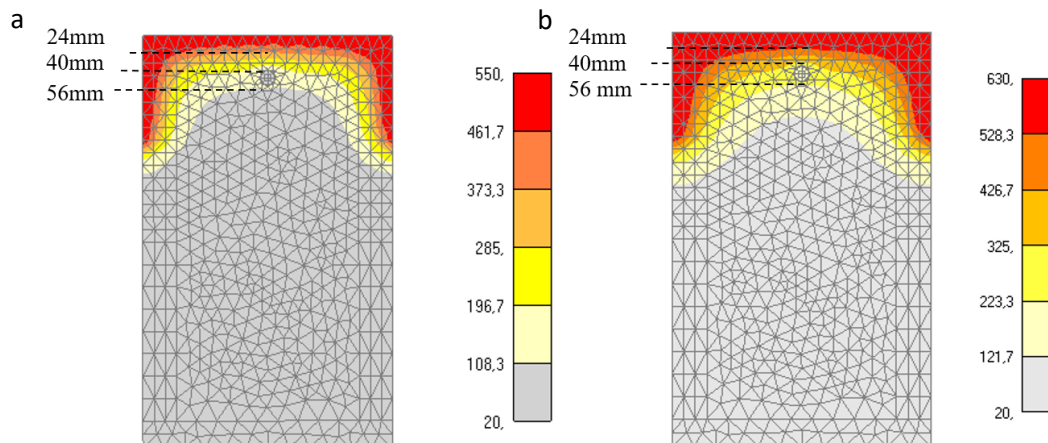**Figure 10.** Relative stiffness reduction for three-sided fire exposure.

Figure 11. Temperature profile along the cross-section of a specimen with three-sided fire exposure a) after 30 min. and b) after 60 min. of fire.

5   Conclusions

In the work presented herein, numerical investigations are performed on beam-end specimens after exposure to ISO 834-1 fire. The objective of the study was to understand the effect of parameters such as fire duration, concrete cover and confinement level on the post-fire bond capacity. Based on the obtained results, following conclusions can be drawn:

- Exposure to standard fire results in a strong degradation of post-fire bond capacity and stiffness, even for relatively short fire duration of 30 minutes.
- The degradation occurs mainly due to high thermally induced damage (cracks) triggered by temperature gradients in the surface layer of concrete. These cracks facilitate the formation of the longitudinal splitting crack. The severity of thermal cracking is directly linked to the heating rate, hence, this phenomenon is very pronounced in case of fire (as opposed to slow heating typically used in experiments).
- The effect of material degradation due to elevated temperature is shown to have a secondary role in the post-fire bond degradation. Since the material degradation can take place only after concrete attains certain temperature, this phenomenon contributes more towards bond degradation in case of longer fire durations and very small concrete covers.
- The effect of thermal cracking on relative bond degradation can outweigh possible effects of other parameters generally affecting bond. It is found that the relative bond reduction is similar for the investigated range of the cover-to-diameter ratio (c_{min} = 1.5 to 3.5 d_s).
- The effect of confinement through stirrups in case of fire-damaged specimens is found to be similar to that of non-damaged (reference) specimens.
- The difference in residual bond strength for one- and three-sided heating is relatively small. This is attributed to a relatively high cover ratio (c_{max}/c_{min}) i.e. the behavior is dominated by the damage in smaller concrete cover c_{min}. (c_{max}/c_{min} ranged from 2.5 to 5.9).

Acknowledgments

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