The bond behaviour of reinforced concrete members at elevated temperatures

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Abstract

This paper presents a comprehensive parametric study on the bond behaviour of reinforced concrete members under fire conditions. The study identified some most important factors affecting the bond characteristics between concrete and reinforcement within reinforced concrete beams and slabs at elevated temperatures. These factors are: steel bar yielding, concrete cover, concrete compressive strength and concrete spalling. The results indicated that concrete cover has a great influence on the bond strength, by providing the confinement to the reinforcement for both the reinforced concrete beam, and the slab. The impact of concrete spalling on the beam is very significant, for both full bond and partial bond cases. The behaviour of the reinforced concrete frame under different fire scenarios was also investigated – assuming a two hours fire resistance rate. Those results indicated that isolated members behave differently, compared to those members within a building. Indicating that continuity of the members and its surrounding cooler structures significantly affects the behaviour of the members within the fire compartment.

Keywords: Bond behaviour; Concrete cover; Concrete spalling; Fire resistance; Concrete beam; Concrete slab.

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RESEARCH HIGHLIGHTS:

- Study the influences of the bond on the fire behaviour of concrete beam and slab.
- Identify the main factors affecting the bond strength of concrete members in fire.
- Studies the behaviours of 3D concrete frames, under different fire scenarios.

1. Introduction

For reinforced concrete structures, research on the fire resistance of structures started since the early 1970, and mainly focused on the behaviour of isolated members [1-4]. In current design codes, such as Eurocode 2 [5], more advanced computer modelling methods are allowed. However, the influences of concrete spalling and bond behaviour between concrete and steel reinforcement, having so far been ignored – in the majority of the computer software developed. In current design practice, the concrete structural design against fire is still based on simplistic methods developed from standard fire tests – which arguably do not accurately represent real building behaviour. This makes it very difficult, if not impossible, to accurately determine the actual level of safety achieved through current design codes, or whether an appropriate level of safety could be attained more efficiently by using a performance-based design alternative.

Concrete is a very common construction material. There is a widely-held perception among designers that reinforced concrete structures have good fire resistance, compared with steel structures. This is based on the assumption that concrete cross-sections retain their integrity under fire conditions. However, evidence has shown that spalling of concrete in accidental fires causes severe damage, to the concrete members [6-7].

Numerous models are currently being developed, to support the study of the behaviour of reinforced concrete members at ambient and elevated temperatures [8-12]. The majority of these models are based on the assumption of full interaction – between the concrete and reinforcing bars. But further research indicates that the assumption of full bond between concrete and rebar at elevated temperatures is unconservative [13-16]. The behaviour of the bond, between concrete and steel bars, has a considerable influence on the fire resistance of reinforced concrete members [17, 18]. The authors have developed numerical models [14, 15] for analysing the bond-slip between the concrete and reinforcing steel bars under fire conditions. In this research the validated bond model developed in [15], is used to predict bond behaviours of reinforced concrete beam and slab members, at elevated temperatures.

In this paper, we report on a new and comprehensive numerical investigation, of the impacts of the bond characteristics on the fire behaviours of reinforced concrete beams and slabs. The main objectives of the study were to:

- Comprehensively and parametrically study the influence of the bond characteristics, on the responses of reinforced concrete beam and slab members under fire conditions.
- Study the factors that affect the bond behaviour. In particular the reinforcing steel bars' yielding, thickness of concrete cover, concrete compressive strength, and concrete spalling.

- Study fire behaviours of a 3D reinforced concrete frame, under different fire scenarios, leading to observed differences of structural behaviours; between isolated structural members and the members within whole structures.

2. Modelling background

A Finite Element program VULCAN [15, 19-21] was used to simulate a reinforced concrete frame in fire (as shown in Fig. 1). In VULCAN, a reinforced concrete structure is modelled as an assembly of finite plain beam–column and slab elements, reinforcing steel bar elements and bond-link elements, as shown in Fig. 2. Each node from these elements is defined in a common reference plane. This reference plane is assumed to coincide with the mid-surface of slab elements, with its location fixed throughout the analysis [19-21]. Both materials and geometric non-linearities are taken into account in the VULCAN model. Sophisticated behaviours of structures under elevated temperatures – such as thermal expansion, degradation of stress–strain curves, failure of concrete segments due to cracking and crushing, failure of steel reinforcement by yielding or the bar's rupture – are all considered in this model [19-21].

The analysis of reinforced concrete members involved two phases: the first phase was to carry out the thermal analysis on the beam or slab – using the two-dimensional non-linear finite-element computer program developed by Huang et al. [22]. In this program, the thermal properties for concrete and steel given in Eurocode 2 [5] are adopted. The second phase of the analysis was a structural analysis for the beam or slab members using VULCAN software. The output data from the thermal analysis are used as the temperature input data for the structural analysis.

The cross-sections of beam–column elements are subdivided into a matrix of segments. Each segment can have different material properties and temperature profile [19]. The non-linear layered procedure has been adopted in VULCAN, for modelling plain concrete slabs. Both material and geometric non-linearities are considered in the model [20, 21]. The slab elements are subdivided into layers (as shown in Fig. 2), to consider the temperature distribution through the thickness of the slab.

The bond-link element developed by the authors [15] was used to model the bond behaviour between concrete and reinforcement under fire conditions, based on a partly cracked thick-wall cylinder theory. The smeared cracking approach is adopted, to consider the softening behaviour of concrete in tension. The model is able to consider a number of parameters: such as different concrete properties and covers; different steel bar diameters and geometries. The bond model developed in [15] has been comprehensively validated against tested data at both ambient temperature [23-27] and elevated temperatures [2, 28-31]. The details of the validation can be found in [15]. It is evident that model adopted in this paper is able to predict the bond-slip characteristic between the concrete and reinforcing steel bar at elevated temperatures.

Previous studies concluded that the behaviour of bond stress–slip depends on numerous factors such as: type of steel bar (ribbed or smooth bar), roughness of rebar (related to rib area), yielding of reinforcing steel bars, the concrete strength, position and orientation of the bar during casting, state of stress, the boundary conditions, the concrete cover [32-34]. Those factors are also considered in the current study.

Influence of concrete spalling, on the thermal and structural behaviour of reinforced concrete members, is also considered in this study. The effect of concrete spalling is represented by using void segments within the cross-section of beam and column, and void layers within the cross-section of slab [13]. It is assumed that the void segments and void layers have zero mechanical strength, stiffness and thermal resistance. After spalling the outer parts from the concrete, the inner-parts will be exposed directly to the fire. In which the fire boundary moves within the cross-section of the concrete member. When the spalling reaches the steel reinforcement, the bond strength will become zero in this region.

It is also well known that there are still no robust numerical models available – to accurately predict concrete spalling. Therefore, in this research a simplified method was developed based on the critical temperature of concrete for predicting concrete spalling and its impact on the global behaviour of reinforced concrete structures in fire. Previous research has indicated that concrete spalling is very likely to happen before 30 minutes of an ISO834 fire test. The critical spalling temperature of concrete ranges from 200ºC to 400ºC [9, 35]. So we assume the critical spalling temperature of concrete to be 350ºC. A critical temperature is used as a criterion for the concrete spalling, in which the segment or layer will be spalled when the temperature of this segment or layer reaches this 'critical temperature'. The spalling of concrete is assumed not to reach a point beyond the reinforcing cage of members [13].

In this study a generic reinforced concrete building was used. It was assumed that the building contains of ten floors with 4.5 m height of each story. Dimensions of each floor are 37.5 m x 37.5 m with five 7.5 m x 7.5 m bays in each direction as shown in Fig. 1. The building was subjected to ISO 834 standard fire. The building represents a commercial office building and designed based on Eurocode 2 [5]. The self-weight is 7.5 kN/m^2 for concrete density of 24 kN/m^3. Raised floor involves: 0.5 kN/m^2 for ceiling, 0.5 kN/m^2 for services and 1.0 kN/m^2 for partitions with impose load of 2.5 kN/m^2. The total design load on the slab at the fire limit state is 10.75 kN/m^2. The concrete compressive strength is 45 MPa with moisture content of 4% by weight and the yield strength of steel bar is 460 MPa. It was assumed that the building required two hours fire resistance with slab cover of 25 mm. The floor slab thickness is 250 mm based on Eurocode 2 [5]. The length of the beam is 7.5 m and exposed to fire from three sides as illustrated in Figs 3 and 4. The cross-section details of the beams are shown in Fig. 5, which have dimensions of 500 x 350 mm and concrete cover of 30 mm.

3. Modelling isolated simply supported reinforced concrete beam

Fig. 3 shows an isolated simply supported reinforced concrete beam heated by ISO 834 standard fire [36]. The beam has the same dimensions and loading condition of the beams within the generic building (see Fig. 1). Hence, the span of the beam is 7.5 m and the beam has two layers of reinforcing steel bars (three bars at the bottom and two bars at the top), as shown in Fig. 5.

In the modelling, two steps should be followed in order to analyse the beam under fire condition. Firstly, the thermal analysis of the beam was conducted to calculate the temperature history within the cross-section of the beam. In this step, the cross-section of the beam was divided into 441 segments (21 x 21). Same segmentations were used within the cross-section of the beam for the structural analysis.

Spalling of concrete was taken into account in this investigation to study the influence of spalling on the thermal and structural behaviours of the beam. For assessing the influence of the bond on the behaviour of the beam, the beam with both of full or partial bond conditions between the concrete and reinforcement were modelled.

Previous research indicated that the bond-stress slip relationship of reinforced concrete members is affected by numerous factors such as steel bar roughness, thickness of concrete cover, concrete strength, position and orientation of steel bar during casting, boundary conditions, state of stress and yielding of reinforcing steel bars. Hence, the factors considered in this study were: yielding of reinforcing steel bars, concrete cover, concrete compressive strength and concrete spalling.

Details of the reinforced concrete beam are given in Figs 3 and 5. The uniform line load acting on the beam is 20.16 kN/m, which is the same as the beam within the frame shown in Fig. 1. The concrete compressive strength was 45 MPa and the yield stress of the steel bar was 460 MPa. The beam was modelled as an assembly of 5 three-node plain concrete beam elements and 25 three-node reinforced steel bar elements and 55 two-node bond link elements. Using the bond-link elements the beam can be modelled with full, partial or even zero interaction between concrete and reinforcing steel bars.

3.1 Yielding effect of reinforcing steel bars on the bond behaviour

The proposed model presented in [15] has been further improved in this paper to take into account the yielding effect of steel bar on the bond behaviour. Yielding of the steel bar within the plastic range has an effect on the bond strength similar to the effect of splitting of concrete cover. A reduction in the steel bar diameter can occur due to steel bar's yielding. This contraction leads to a reduction in the friction between concrete and rebar, and also affects the geometry of the bar ribs, which consequently reduces the bond stress [33].

In order to consider the yielding effect of steel bar on the bond behaviour, the bond stress τ should be modified by the factor Ω_y as follow [37]:

$$\tau_{\text{modified}} = \tau.\Omega_y \tag{1}$$

$$\Omega_y = 1.0 \qquad \textit{for} \quad \varepsilon_s \le \varepsilon_{sy} \tag{2}$$

$$\Omega_y = 1.0 - \left[0.85.\left(1 - e^{-5a^b}\right)\right] \qquad \textit{for} \quad \varepsilon_{sy} < \varepsilon_s \le \varepsilon_{su} \tag{3}$$

In which:

$$a = \frac{\varepsilon_s - \varepsilon_{sy}}{\varepsilon_{su} - \varepsilon_{sy}} \tag{4}$$

$$b = \left[2 - \frac{f_t}{f_y}\right]^2 \tag{5}$$

Where: ε_s is the steel strain; ε_{sy} is the yield strain of steel bar at the yield stress f_y; ε_{su} is the ultimate steel strain, f_t is the steel stress and f_y is the yield stress of the steel bar. The yielding effect of the steel bar on the bond stress-slip curve is shown in Fig. 6.

To demonstrate the influence of yielding effect of reinforcing steel bar on the bond behaviour, the isolated beam was modelled using the bond models with and without the yielding effect of reinforcing steel bar. Concrete compressive strength was 45 MPa, the steel strength was 460 MPa and the concrete cover was 30 mm.

Fig. 7 shows the influence of yielding steel bar on the central deflection of the beam under ISO 834 fire, together with the central deflection of the beam modelled with full bond interaction. As shown in the figure, the yielding effect of steel bar has some degrees of influence on the bond behaviour, but the influence is not significant. However, this demonstrated that the yielding effect of steel bar can reduce the bond strength between concrete and the steel bar.

3.2 The effect of concrete cover on the bond behaviour

Concrete cover is an important part for reinforced concrete members. Concrete cover provides the protection for reinforcing steel bars from the outside attacks such as chloride, which can result in corrosion of steel bars. Also, the concrete cover gives isolation for the reinforcement from the high temperatures under fire conditions. Moreover, concrete cover provides the confinement for the steel bars to generate the bond between concrete and reinforcement.

Same isolated beam with concrete compressive strength of 45 MPa and steel strength of 460 MPa was used in this investigation. In order to investigate the sensitivity of the bond-slip to the concrete cover of the beam at elevated temperatures, five values of concrete cover thicknesses (10, 20, 30, 40 and 50 mm) were adopted in this study. The cross-section of the beam was subdivided into segments of concrete and steel, as shown in Fig. 4. The same segmentation was used for both thermal and structural modelling of the beam. Also the beam was heated from three sides (see Fig. 4).

Figs 8 to 10 show the temperatures of the reinforcing steel bars within the cross-section of the beam (Bar 1, Bar 2 and Bar 3 as identified in Fig. 5) with different thickness of concrete covers. It is clear from the figures that the temperature of the steel bars decreases by increasing the concrete cover of the beam. Hence, the concrete cover can provide the protection to the reinforcing steel bars from the fire and reduce the effect of temperature on the steel bars and bond strength within the cross-section of reinforced concrete beams. It can be seen from the figures that after two hours (120 min) of exposing to ISO 834 standard time-temperature, the temperature for Bar-1 with cover 50 mm is less than that for cover 10 mm by about 47%, while for Bar-2 the temperature reduced by 49% and for Bar-3 the temperature reduced by 56%.

Influence of concrete cover on the bond behaviour at elevated temperatures is presented based on the response of the beam to the high temperatures. Generally, degradation of the bond strength results in extra central deflection of the beam due to the bond deterioration. Figs 11 to 15 give comparisons between the case of full bond (full interaction between the concrete and reinforcement) and partial bond between the concrete and steel bars for each thickness of concrete cover at elevated temperatures.

It can be seen from the figures that the concrete cover has considerable effect on the behaviour of the beam with full or partial bond. As shown in Fig. 11, for the case with the concrete cover of 10 mm, the behaviour of the two curves was identical until 40 min of fire exposure time. However, after that time a clear divergence between the two curves was observed. This different between the two curves is attributed to the degradation of the bond, in which the temperatures of Bar 1 and Bar 2 become more than 400^{o}C (see Fig. 8) and 350^{o}C (see Fig. 9), respectively. However, for the case with concrete cover of 50 mm, the effect of bond becomes insignificant and identical behaviour of both curves can be seen due to low temperatures of the steel bars and high interaction between the concrete and rebar as shown in Fig. 15.

It can be concluded from Figs 11 to 15 that, increasing the thickness of concrete cover has a great influence on the bond-slip, in which the degradation of the bond strength decreases with increasing the thickness of concrete cover. Hence, the concrete cover is the main factor that can provide the protection to the steel bars and gives the confinement to the reinforcement for the bond strength. Therefore, the designer should carefully choose the proper concrete cover.

Finally, it is important to explain the type of failure based on the outputs of the VULCAN software. The failure of the beam with the concrete cover of 50 mm occurred mainly due to concrete cracking. For concrete covers of 40 mm, 30 mm and 20 mm the failure of the beam occurred due to concrete cracking, yielding the steel bars as well as the bond failure within some elements. For concrete cover of 10 mm the failure of the beam firstly occurred due to concrete cracking, then yielding the steel bars followed by rupture of some steel bars, also the failure of the most bond-link elements between the reinforcement and concrete.

The results generated from this study further support the claims of previous researchers that the concrete cover has a great influence on the bond strength by providing the confinement to the reinforcement. Hence, reduce the concrete cover is not only leads to increase the temperature of the reinforcement during fire, but also decreases the concrete confinement which can play a significant factor on the bond strength [33].

Fig. 16 shows the influence of different concrete covers on the behaviour of the beam modelled with partial bond. It is evident that the concrete cover has a great influence on the behaviour of the beam. By considering the deflection criterion (span/20), the fire resistance for the beam is 77 min, 120 min, 167 min and 217 min for the concrete covers of 10 mm, 20 mm, 30 mm, 40 mm, respectively. And more than four hours (240 min) fire resistance can be achieved when the concrete cover is 50 mm. Hence, increasing the concrete cover from 10 mm to 40 mm can increase the fire resistance of the beam for more than two hours.

3.3 The effect of concrete spalling on the behaviour of the beam

Spalling of concrete in fire has a significant effect on steel bars and bond strength, which influences the behaviour of reinforced concrete members. When the spalling of concrete occurs, the member will lose the concrete cover and the reinforcement will be exposed directly to the fire. Then, a great reduction in the strength and stiffness of steel bars can occur. Also, failure of the bond between concrete and rebar can happen due to losing the confinement of concrete to steel bar. The results from this study indicated that the influence of spalling for full-bonded member is significant, especially when the spalling occurs within the mid-span elements.

In this study the effect of concrete spalling on the bond behaviour was considered by modelling the beam using partial bond model. Same isolated beam has been used for this investigation. The beam has concrete compressive strength of 45 MPa, the steel bar yielding of 460 MPa and concrete cover of 40 mm. As shown in Fig. 3, the beam was modelled as an assembly of five plain concrete beam elements, 25 steel bar elements and 55 bond-link elements. In order to consider the effect of concrete spalling, void segments were used to represent the spalled concrete for the thermal and structural analysis. In this study, both full and partial bond were considered to investigate the effect of spalling on the behaviour of the beam. The time-temperature histories of the reinforcing steel bars for the cases of concrete spalling and no-spalling are illustrated in Fig. 17. It is clear from the figure that the spalling of concrete cover has a great influence on the thermal behaviour of the beam. For instant, Bar 1 reached to temperature about 500^{o}C after 120 min with no-spalling of concrete, but with spalling Bar 1 reached to this temperature after just 24 min.

Fig. 18 shows the influence of concrete spalling on the behaviour of the beam modelled as full or partial bond. It was assumed that all elements were subjected to concrete spalling. It can be seen that the impact of concrete spalling on the beam is very significant for both full bond and partial bond cases. However, the effect of spalling on the beam modelled with partial bond is greater than that modelled with full bond. Figs 19 to 22 give comparisons for the behaviour of the beam subjected to different concrete spalling and bond conditions.

Fig. 19 shows the comparison of the behaviour of the beam modelled as full bond or partial bond in which all elements were spalled. It is clear from the figure that a big different can be seen between full bond and partial bond cases. It is evident from Figs 20 to 22 that the effect of spalling becomes more significant when elements 1&5 were spalled (see Fig. 3 for element's positions) than the other elements. This can be justified as the bond stress at the end elements of the beam is higher than the bond stress at the middle elements, hence weaken the bond at the end elements has more influence on the response of the beam. It was observed from VULCAN output files that the failure of the beam, when elements 1&5 were spalled, occurred due to bond failure and even bar rupture. While for other cases, the failure of the beam was dominated by yielding the steel bars. This effect can be clearly observed from the Figs 19 to 22.

3.4 The effect of concrete strength on the bond

Concrete compressive strength is one of the factors that effect on the bond strength. The maximum bond strength $\tau_{\max}$ is related to concrete compressive strength as:

$$\tau_{\max} = a\sqrt{f_{ck}} \qquad (6)$$

where: f_{ck} is the concrete compressive strength and a is a constant based on the bond characteristic [32, 37]. Bond stress is resulted from the shear strength of the concrete in the front of steel bar ribs and the circumference tensile stress of the concrete surround the rebar. Therefore, degradation of the bond at elevated temperatures is directly linked to the concrete deterioration.

Same isolated beam with concrete cover of 40 mm and steel bar yielding strength of 460 MPa was used in this section to study the effect of concrete compressive strength on the bond behaviour. A range of concrete compressive strengths (20, 30, 40 and 45 MPa) were adopted in this investigation in order to examine the effect of concrete compressive strength on the bond characteristic between concrete and reinforcement. From the thermal analysis, the temperatures of the reinforcing steel bars are shown in Fig. 23.

The comparisons of the behaviour of the beam modelled with full bond and partial bond using different concrete compressive strength are presented in Figs 24 to 27. As shown from Figs 26 and 27, the concrete compressive strength has no significant effect on the bond behaviour when the compressive strength is more than 40 MPa; and the behaviour of the beam modelled with full or partial bond is similar. However, great influence can be observed when concrete compressive strength is less than 40 MPa, as shown in Figs 24 and 25. This behaviour can be attributed to the considerable degradation of the bond strength.

Fig. 28 shows the influence of concrete compressive strength on the behaviour of the beam modelled with full bond. It can be seen from the figure that there is no significant effect for concrete compressive strength on the behaviour of the beam modelled with full bond. However, as shown in Fig. 29, there is a considerable influence of concrete compressive strength on the behaviour of the beam modelled with partial bond, especially for lower concrete compressive strength.

4. Modelling isolated simply supported reinforced concrete slab

Figs 30 and 31 show an isolated simply supported reinforced concrete floor slab heated by ISO 834 standard fire. The slab has the same dimensions and loading condition of the slab floor within the generic building (see Fig. 1). Hence, the dimensions of the slab are 7.5 m x 7.5 m and the load condition is 10.75 kN/m^2. Concrete compressive strength is 45 MPa and yield stress of the reinforcing steel bar is 460 MPa. The slab has two orthogonal reinforcing steel bar layers with steel area of 646 mm^2/m for each layer.

As shown in Fig. 30, the slab was modelled as an assembly of 25 nine-node plain concrete slab elements, 50 three-node reinforced steel bar elements (25 elements in each direction) and 85 two-node bond link elements which connected the plain concrete slab elements to steel bar elements. Hence, the influence of the bond between concrete and reinforcing steel bars can be modelled as partial or full bond conditions. The cross-section of the slab was subdivided into 14 layers in order to perform the thermal and structural analysis.

4.1 The effect of concrete cover on the behaviour of slab

As mentioned before, concrete cover is an important part of reinforced concrete members, especially under fire conditions. Concrete cover provides a protection to fire exposure, as well as gives a confinement to the steel bars for anchorage. For considering the influence of concrete cover on the behaviour of the slab under fire condition, five thicknesses of concrete cover (10, 15, 20, 25 and 30 mm) were adopted in this study. Fig. 32 shows the temperature histories of the reinforcing steel bars for different thicknesses of concrete covers under ISO 834. It is evident that the temperatures of reinforcing steel bars are significantly affected by concrete covers.

The structural analyses were carried out for each thickness of concrete cover using both full bond and partial bond conditions. Figs 33 to 37 present the comparison of the central deflections of the slab modelled as full bond or partial bond using different thicknesses of concrete cover.

It can be seen from Fig. 37 that no big difference exists between the deflections of the slab modelled with full or partial bond, when the concrete cover was 30 mm. However, when the slab cover was decreased to 25 mm and 20 mm, a clear different can be observed between the cases of full and partial bond, as shown in Figs 36 and 35. This difference becomes greater when the concrete covers were 10 mm and 15 mm as illustrated in Figs 33 and 34, respectively. This behaviour is mainly due to the increasing of the bond temperature (see Fig. 32). Based on the output data from the modelling, the failure of the slab with 10 mm concrete cover and full bond was due to rupture of many steel bars. This rupture of the steel bars is attributed to the full bond between concrete and reinforcement in which high steel strain was concentrated at some locations within the slab. However, when the slab was modelled with partial bond, just few elements were ruptured. This behaviour is attributed to the relative slips between the bars and concrete in which the steel strains within the steel bars can be more uniformly distributed along the length of the steel bars.

Similar behaviour was observed for 15 mm concrete cover with few bars ruptured, which leads to more divergence between the full and partial bond cases. In the cases of 20 mm and 25 mm concrete covers, the slab failed mainly due to bond failure for partial bond, and also due to yielding of steel bars for full bond. For the slab with concrete cover of 30 mm, the failure of the slab with full bond was because of yielding of some steel bars while for partial bond just few bond elements failed. Fig. 38 shows the influence of different concrete covers on the central deflections of the slab modelled as partial bond.

4.2 The effect of concrete strength on the behaviour of slab

To investigate the influence of concrete strength on the behaviour of slab, same isolated slab with concrete cover of 30 mm and steel bars yielding strength of 460 MPa was used. Four values of concrete compressive strength (20, 30, 40 and 45 MPa) were adopted in this study. The temperature history of the reinforcing steel bars mesh within the slab is shown in Fig. 39.

The effect of concrete compressive strength on the response of the slab is shown in Figs 40 to 43. It can be seen from the figures that there was no great influence when the compressive strength was more than 40 MPa. However, the slab failed earlier when the compressive strength was less than 30 MPa. Generally, the effects of concrete compressive strengths on the behaviours of the slab were not significant, as shown in Fig. 44.

5. Modelling of reinforced concrete frame structure in fire

After the studies on the behaviour of isolated reinforced concrete members at elevated temperatures, it is important to investigate behaviour of these members within a building. The structural members within a real building behave different compared to the isolated members. Fire in a building is normally localised in a number of areas called fire compartments – each surrounded by cooler regions. The surrounding cooler structures can provide a considerable restraint, to the members within each fire compartment. Using isolated member subjected to a standard fire curve gives the upper and lower limits of its behaviours, but does not give the actual behaviour of the member within a whole building.

Therefore, in this section a 3D reinforced concrete frame with floor slabs was modelled under different fire scenarios. It was assumed that concrete compressive strength was 45 MPa and the yield strength of the steel bar was 460 MPa. The details of the whole floor were shown in Fig. 1.

The same beam shown in Figs 3 and 5, and the same slab shown in Figs 30 and 31 were used. The concrete cover of the beam was 30 mm, while the concrete covers of the slab and column were 25 mm. Due to very high computational power demanded, the frame was modelled using only full bond between concrete and reinforcement.

As shown in Fig. 45, a quarter of the floor was modelled due to symmetry in this study. For the first case, it was assumed that the whole floor of the building was engulfed by ISO 834 standard fire. Fig. 46 shows the deflections at positions a, b, and c (see Fig. 45). It can be seen from the figure that the deflections at three positions of the floor slabs were approximately similar with little less deflection at position b. This is due to the continuity of the slabs and the restraint provided by some columns.

In order to investigate the influence of different locations of fire compartments on the structural behaviour of floor slabs. Six locations of fire compartments were investigated in this study. Fig. 45 gives the locations of the fire compartments and identify as FC-1, FC-2, FC-3, FC-4, FC-5 and FC-6. Only one fire compartment was exposed to ISO 834 standard fire while the rest of the structure was assumed to be at ambient temperature. Fig. 47 gives the central deflections of the floor slabs for different fire compartments. It can be seen from the figure that the lowest central deflection was the fire compartment FC-3 and highest central deflection was the fire compartment FC-1. The central deflections of the fire compartments FC-2 and FC-6 were similar. Also there were similar central deflections for the fire compartments FC-4 and FC-5.

It is clear here, that restraint provided by the cooler structures surrounding the fire compartment has a positive effect on the fire resistance of the structural members within the fire compartment. The compressive membrane action may play an important role to enhance the loading capacity of the floor slabs within the fire compartment. Figs 48 to 50 show the comparisons of the deflections at three locations for compartment fire and whole floor fire, together with the central deflection of isolated simply supported slab. It is evident from the figures that the deflections of the slabs subjected to whole floor fire were greater than the deflections of the slabs under compartment fires. This is because under whole floor fire only very limited restraint was provided by columns for the floor slabs. It is also clear that the behaviour of isolated simply supported slab was considerably different, with the slab within the whole structure.

6. Conclusions

In this paper, a parametric study has been conducted to investigate the effect of the bond characteristic between concrete and steel reinforcement on the behaviours of isolated simply supported reinforced concrete beams and slabs. Also, a 3D reinforced concrete frame has been modelled under different fire scenarios. Based on the results some conclusions can be drawn:

- The yielding effect of the steel bar has some degrees of influence on the bond behaviour of the reinforced concrete beam. This yielding effect can reduce the bond strength between the concrete and steel bars within a beam, but the influence is not significant.
- Concrete cover has a great influence on the bond strength, by providing the confinement to the reinforcement for both the reinforced concrete beam and slab. Hence, reduction of the concrete cover not only leads to increases in the temperature of the reinforcement during a fire, but also decreases the concrete confinement which can play a significant role on the bond strength.
- Impact of concrete spalling on the beam is very significant, for both full bond and partial bond cases. However, the effect of spalling on the partially bonded beam, is greater than the one modelled with a full bond.
- For the reinforced concrete beam, it is clear that a concrete compressive strength of more than 40 MPa, has no significant effect on the bond behaviour. This conclusion remains true, when the behaviour of the beam is modelled with full or partial bond. However, great influence on the bond behaviour can be observed, when the concrete compressive strength is less than 40 MPa. This behaviour can be attributed to the considerable degradation of bond strength. The influence of concrete compressive strength on the slab is less significant.
- The 3D modelling of concrete frame, indicated that the restraint provided from the cooler structures surrounding the fire compartment has a positive effect on the fire resistance of the structural members within the fire compartment. Compressive membrane action may play an important role to enhance the loading capacity of the floor slabs within the fire compartment. It is also clear that the behaviour of isolated simply supported slab is considerably different with the slab within the structures.

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References

[1] Bizri, H. Structural capacity of reinforced concrete columns subjected to fire induced thermal gradients. Structural Engineering Laboratory, Report No. 73-1, University of California, Berkeley, 1973.

[2] Lin T.D., Ellingwood B., Piet O. Flexural and shear behaviour of reinforced concrete beams during fire tests. Report No. NBS-GCR-87-536. Centre for Fire Research, National Bureau of Standards; 1987.

[3] Ellingwood B. and Lin T. D. Flexure and shear behaviour of concrete beams during fires. Journal of Structural Engineering, 1991; 117(2): 440-458.

[4] Ali, F.A., O'Connor, D. and Abu-Tair, A. Explosive spalling of high-strength concrete columns in fire", Magazine of Concrete Research, 2001; 53(3): 197-204.

[5] BS EN 1992-1-2, Eurocode 2: Design of concrete structures - Part 1-2: General rules-Structural fire design; 2004.

[6] Bailey C. Holistic behaviour of concrete buildings in fire. Eng. Build. 2002; 152: 199–212.

[7] Bailey C.G., Ellobody E. Whole-building behaviour of bonded post-tensioned concrete floor plates exposed to fire. Eng. Struct. 2009; 31: 1800–1810.

[8] Gillie, M. et al. Modelling of heated composite floor slabs with reference to Cardington experiments. Fire Safety Journal 2001; 36: 745-767.

[9] Kodur VKR, Wang T, and Cheng F. Predicting the fire resistance behavior of high strength concrete columns. Cement & Concrete Composites 2004; 26(2): 141–53.

[10] Lim, L. et al. Numerical modelling of two-way reinforced concrete slabs in fire. Engineering Structures 2004; 26: 1081-1091.

[11] Dwaikat, M.B. and Kodur, V.K.R. A numerical approach for modeling the fire induced restraint effects in reinforced concrete beams. Fire Safety Journal 2008; 43: 291–307.

[12] Badiger N.S., Malipatil K.M. Parametric study on reinforced concrete beam using ANSYS. Civ. Environ. Res. 2014; 6: 88–95.

[13] Huang Z. Modelling of reinforced concrete structures in fire. Proc. ICE - Eng. Comput. Mech. 2010; 163(EM1): 43–53.

[14] Huang Z. Modelling the bond between concrete and reinforcing steel in a fire. Eng. Struct. 2010; 32: 3660–3669.

[15] Khalaf J., Huang Z., Fan M. Analysis of bond-slip between concrete and steel bar in fire.

Comput. Struct. 2016; 162: 1–15.

[16] Kodur V.K.R., Agrawal A. Effect of temperature induced bond degradation on fire response of reinforced concrete beams. Eng. Struct. 2017; 142: 98–109.

[17] Haddad R.H., Al-Saleh R.J. and Al-Akhras N.M. Effect of elevated temperature on bond between steel reinforcement and fiber reinforced concrete. Fire Safety Journal 2008; 43: 334–343.

[18] Pothisiri T. and Panedpojaman P. Modeling of bonding between steel rebar and concrete at elevated temperatures. Construction and Building Materials 2012; 27:130-140.

[19] Huang Z., Burgess I.W., Plank R.J. Three-dimensional analysis of reinforced concrete beam-column structures in fire. J. Struct. Eng. 2009; 135: 1201–1212.

[20] Huang Z., Burgess I.W., Plank R.J. Modeling membrane action of concrete slabs in composite buildings in fire. I: Theoretical development. J. Struct. Eng. 2003; 129: 1093–1102.

[21] Huang Z., Burgess I.W., Plank R.J., Modeling membrane action of concrete slabs in composite buildings in fire. II: Validations. J. Struct. Eng. 2003; 129: 1103–1112.

[22] Huang Z., Platten A., Roberts J. Non-linear finite element model to predict temperature histories within reinforced concrete in fires, Build. Environ. 1996; 31: 109–118.

[23] Xiao J. and Falkner H. Bond behaviour between recycled aggregate concrete and steel rebars. Construction and Building Materials 2007; 21:395–40.

[24] John Robert Prince M. and Bhupinder S. Bond behaviour of deformed steel bars embedded in recycled aggregate concrete. Construction and Building Materials 2013; 49:852–862.

[25] Lee H. and Noguchi T. Evaluation of the bond properties between concrete and reinforcement as a function of the degree of reinforcement corrosion. Cement and Concrete Research 2002; 32:1313–1318.

[26] Viwathanatepa, S., Popov, E.P. and Bertero, V.V. Effects of generalized loadings on bond of reinforcing bars embedded in confined concrete blocks. *Report No EERC 79-22,* Earthquake Engineering Research Center, University of California, Berkeley; 1979.

[27] Burns N.H. and Siess C. P. Load-deformation characteristics of beam-column connections in reinforced concrete. Civil engineering Studies SRS No.234, University of Illinois, Urbana; 1962.

[28] Diederichs U. and Schneider U. Bond strength at high temperatures. Magazine of concrete research 1981; 33(115):75-84.

[29] Morley P.D. and Royles R. Response of the bond in reinforced concrete to high temperatures, Magazine of Concrete Research 1983; 35(123):67-74.

[30] Haddad R.H. and Shannis L. G. Post-fire behavior of bond between high strength pozzolanic concrete and reinforcing steel. Construction and Building Materials 2004; 18:425–435.

[31] Haddad R.H., Al-Saleh R.J. and Al-Akhras N.M., Effect of elevated temperature on bond between steel reinforcement and fiber reinforced concrete, Fire Safety Journal 2008; 43:334–343.

[32] C. euro-international du Béton, F. internationale de la Précontrainte, CEB-FIP Model Code 1990: Final Draft, CEB; 1991.

[33] du béton F. Bond of Reinforcement in Concrete: State-of-the-art report, International Federation for Structural Concrete; 2000.

[34] Wang H. An analytical study of bond strength associated with splitting of concrete cover. Eng. Struct. 2009; 31: 968–975.

[35] Yu X., Huang Z., Burgess I. W. and Plank R. J. Concrete spalling in fire: A review of the current state of knowledge. Research Report DCSE/05/F/04, Department of Civil & Structural Engineering, University of Sheffield, 2005.

[36] ISO-834, Fire resistance test, elements of building constructions. International Standard ISO 834; 1975.

[37] du béton F., Walraven J. Model Code 2010 - First complete draft - Volume 2: Model Code, International Federation for Structural Concrete (fib); 2010.

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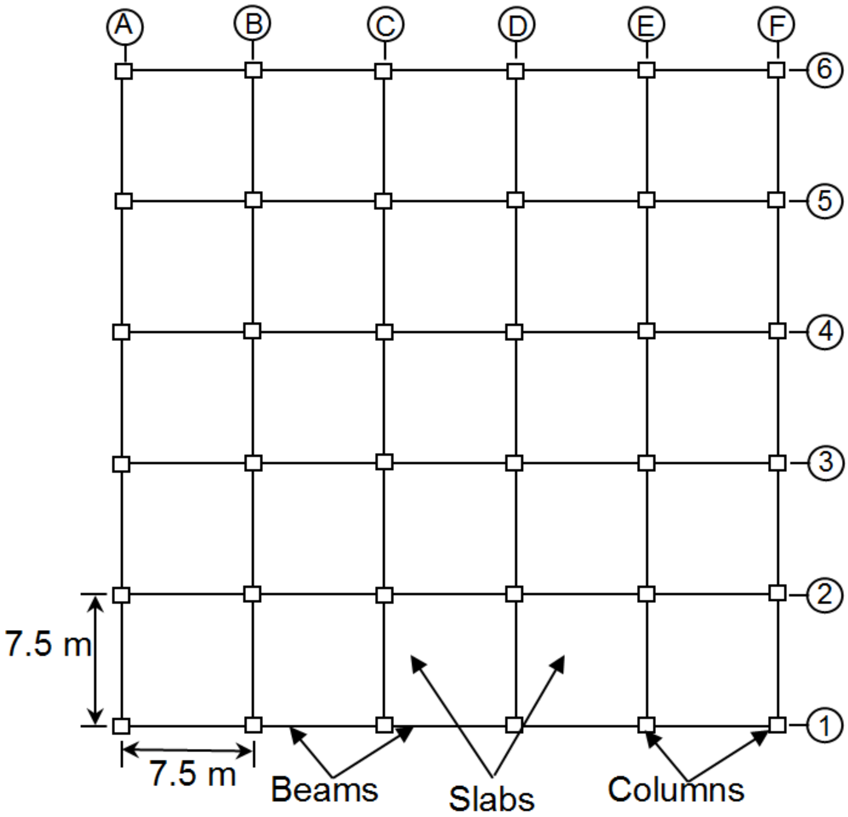

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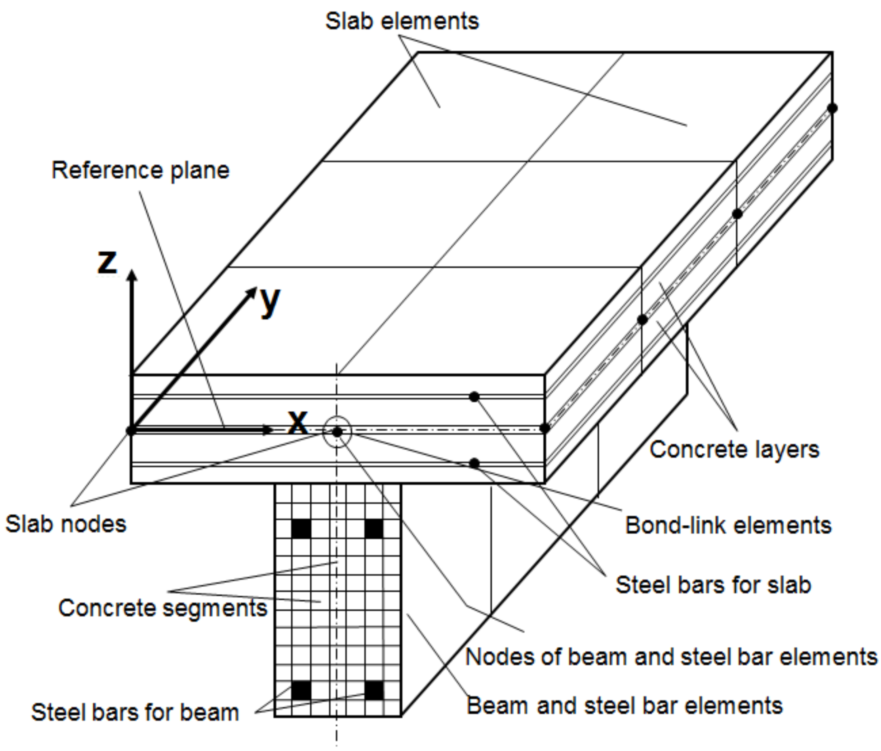

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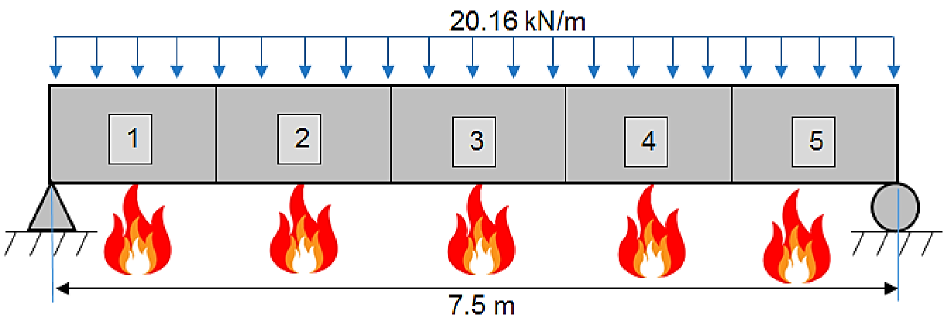

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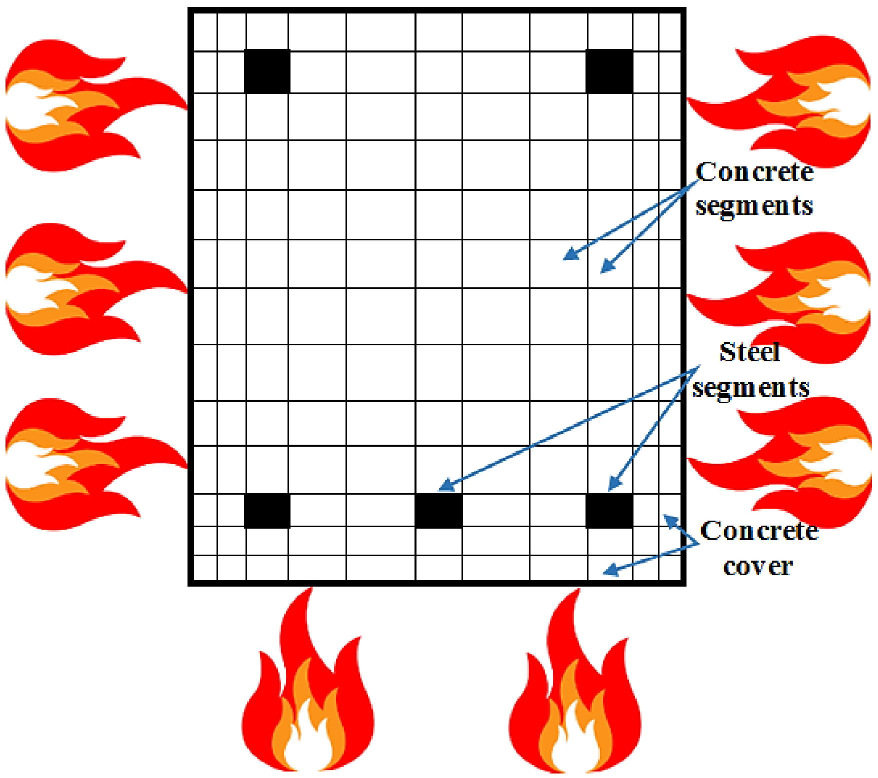

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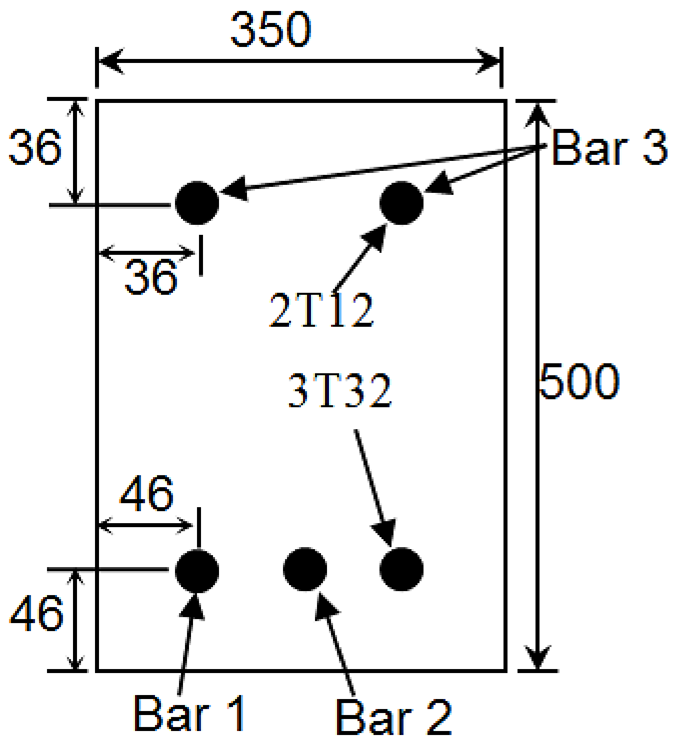

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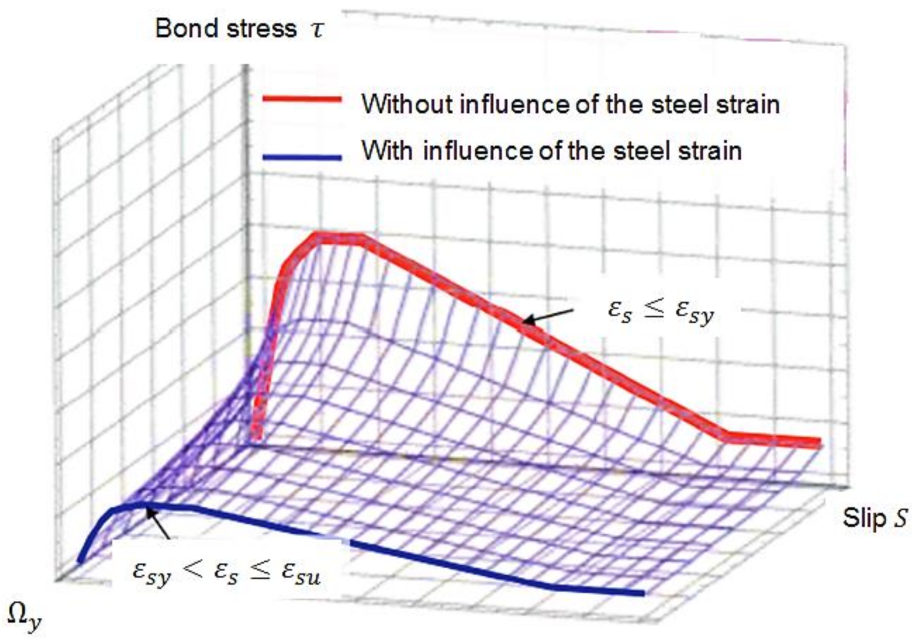

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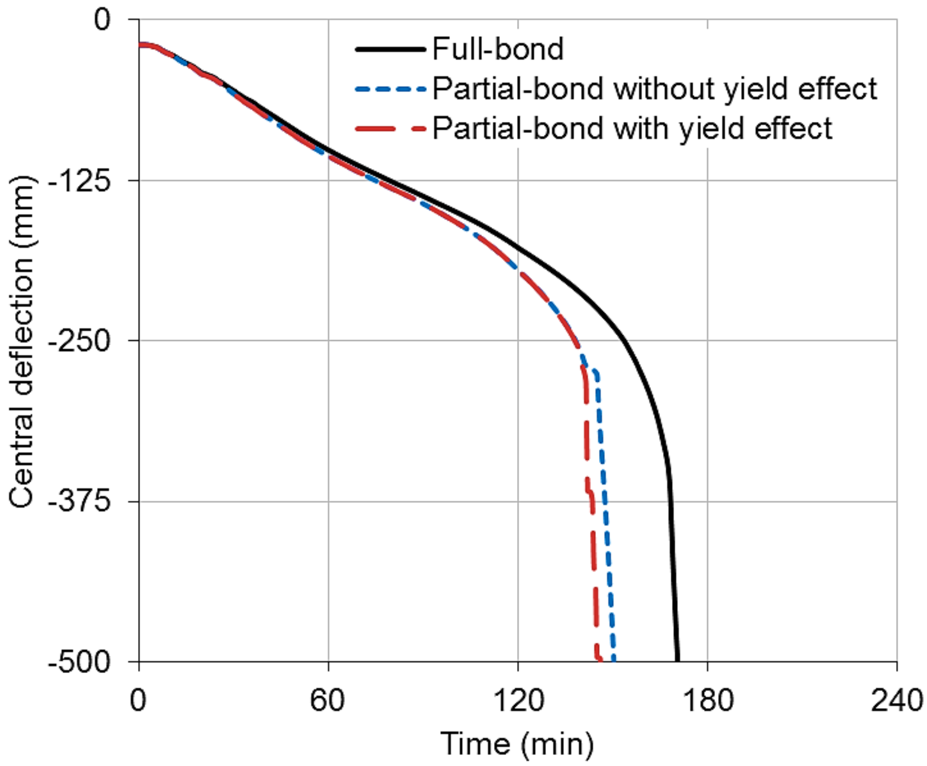

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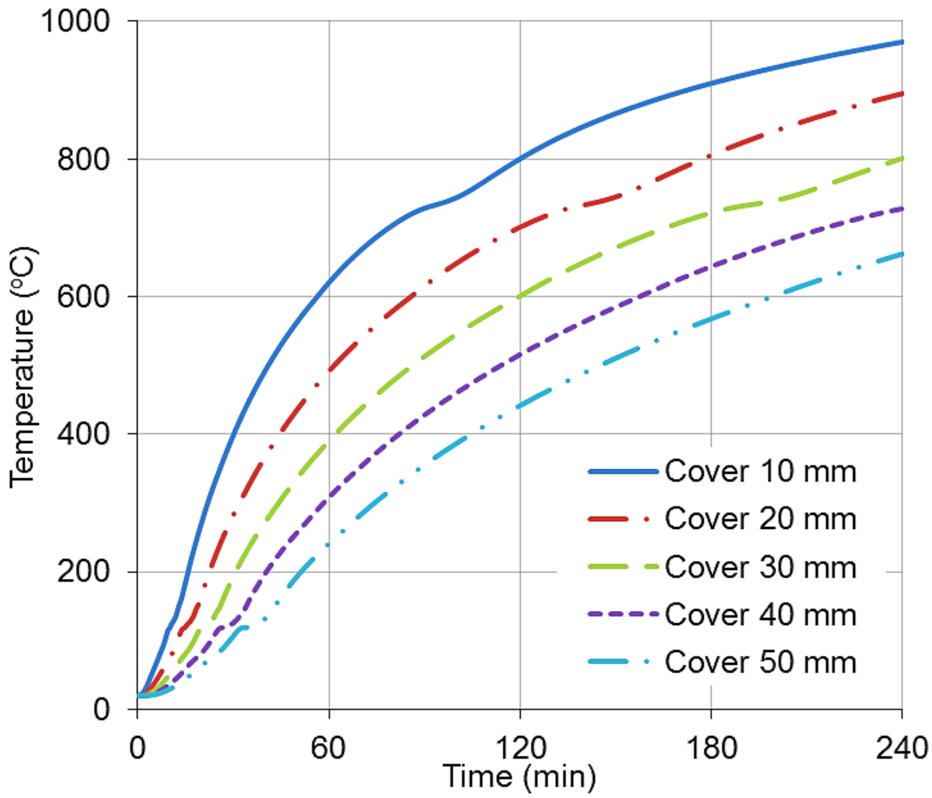

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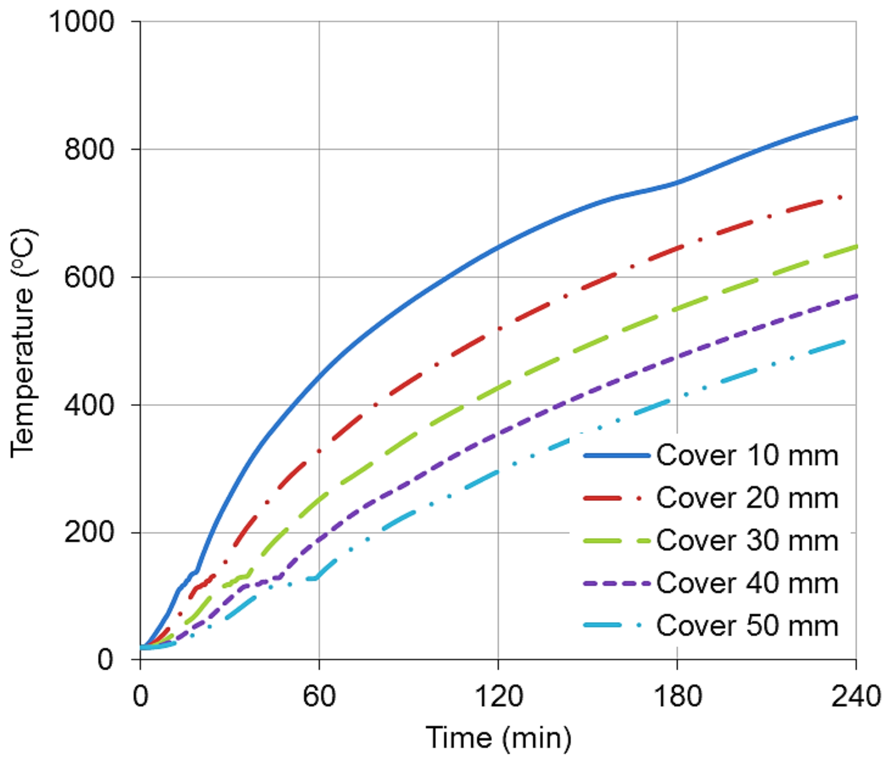

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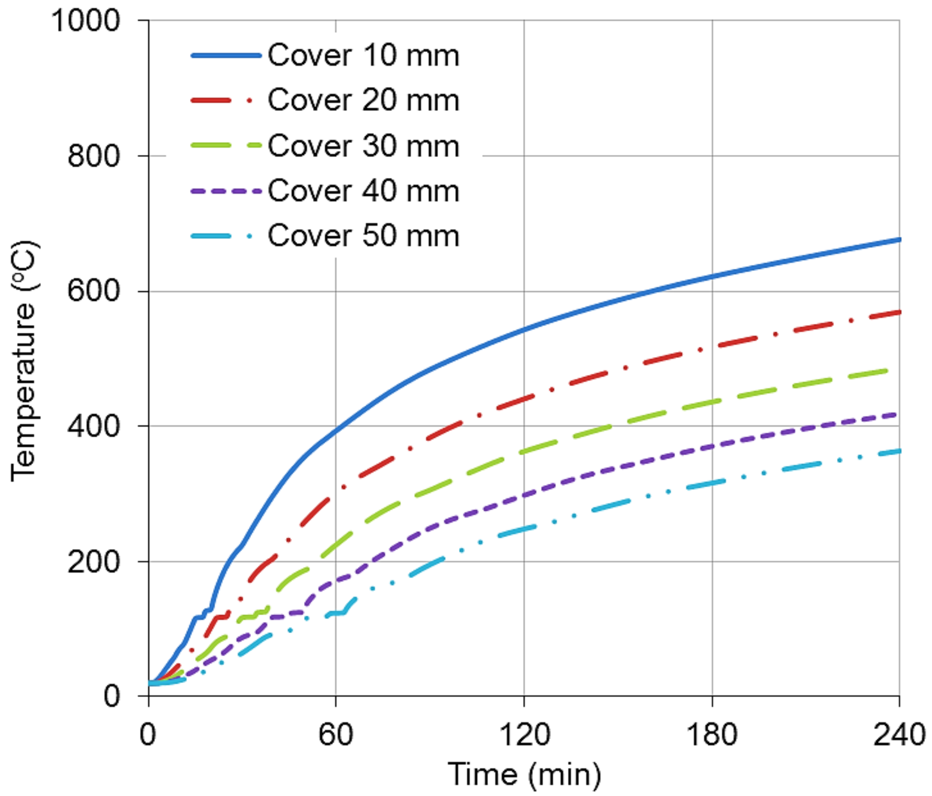

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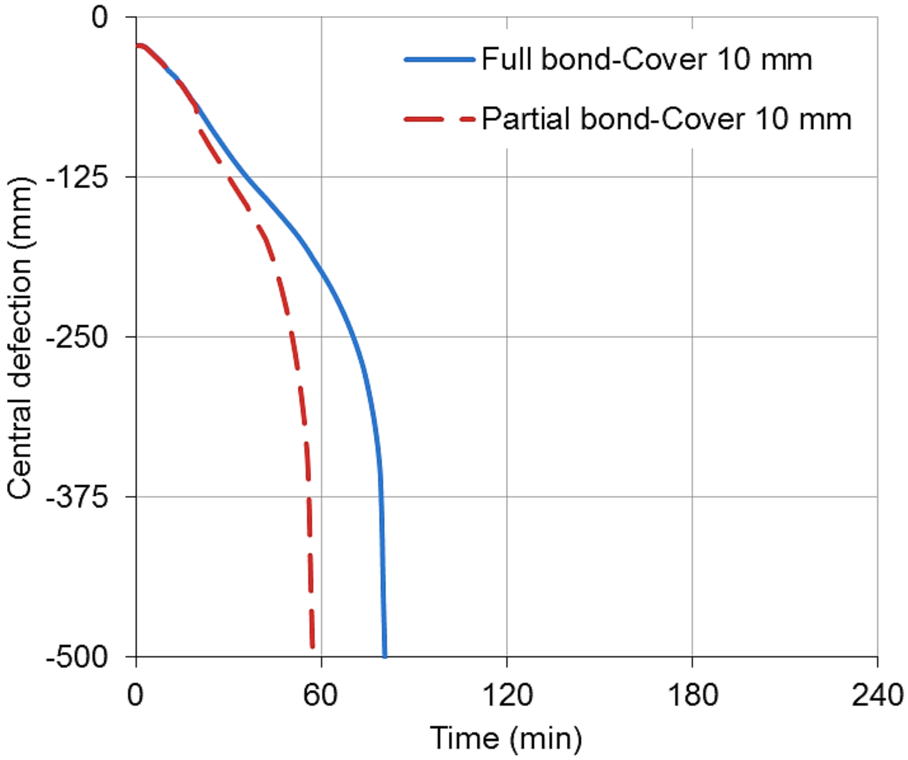

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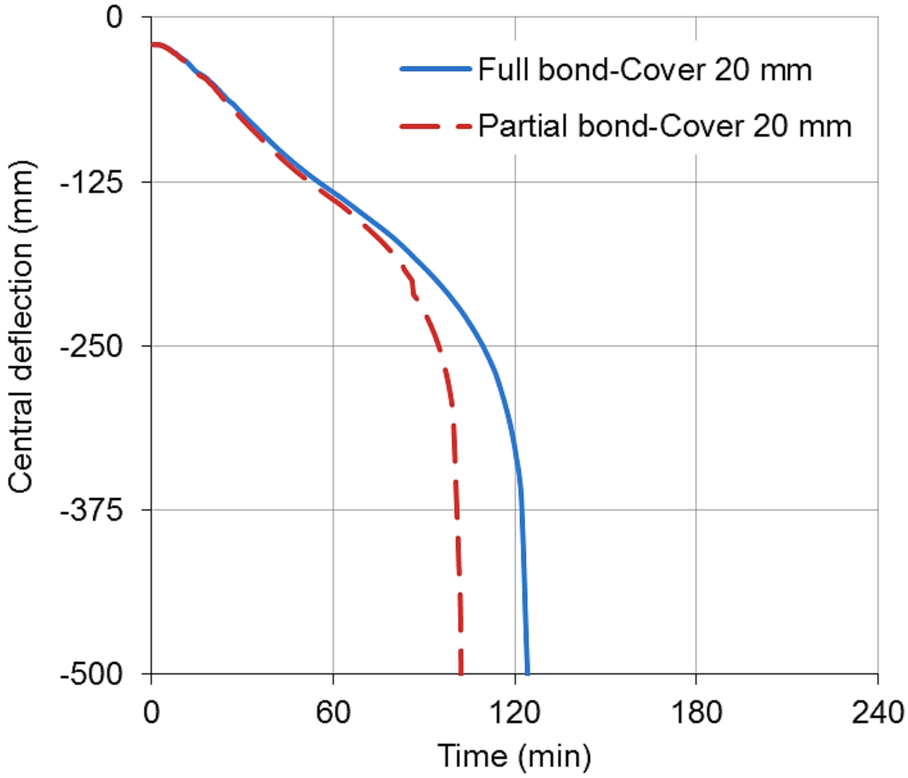

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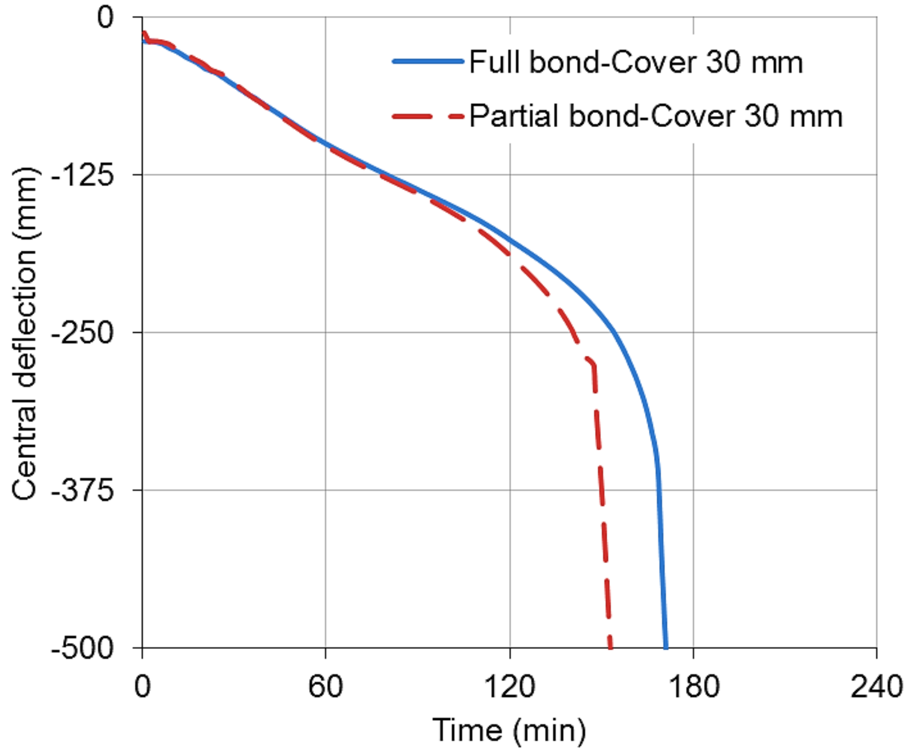

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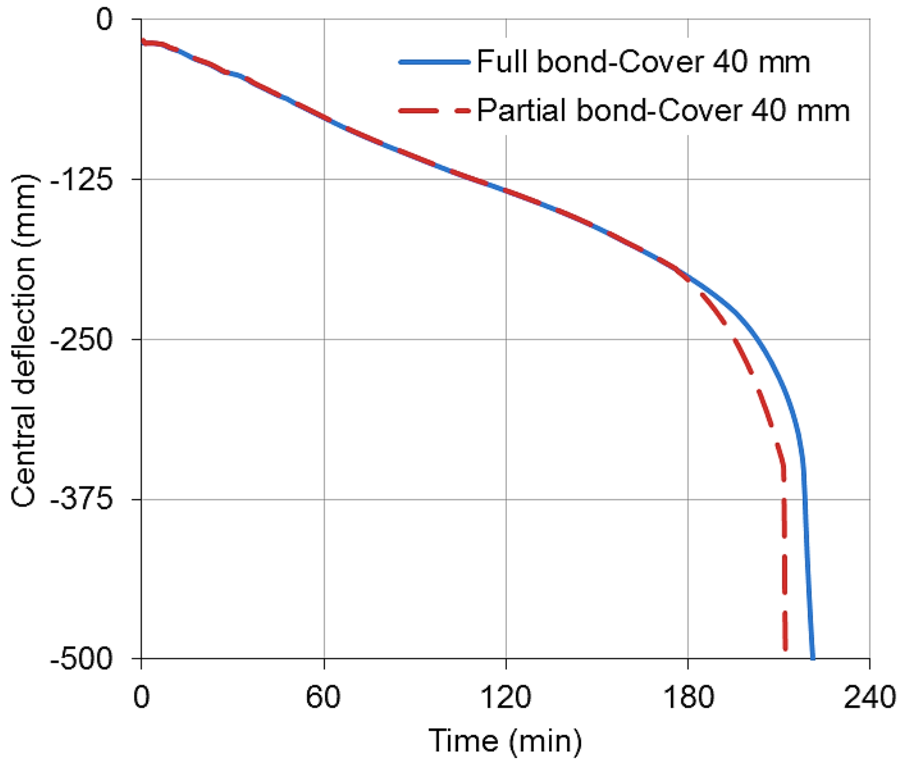

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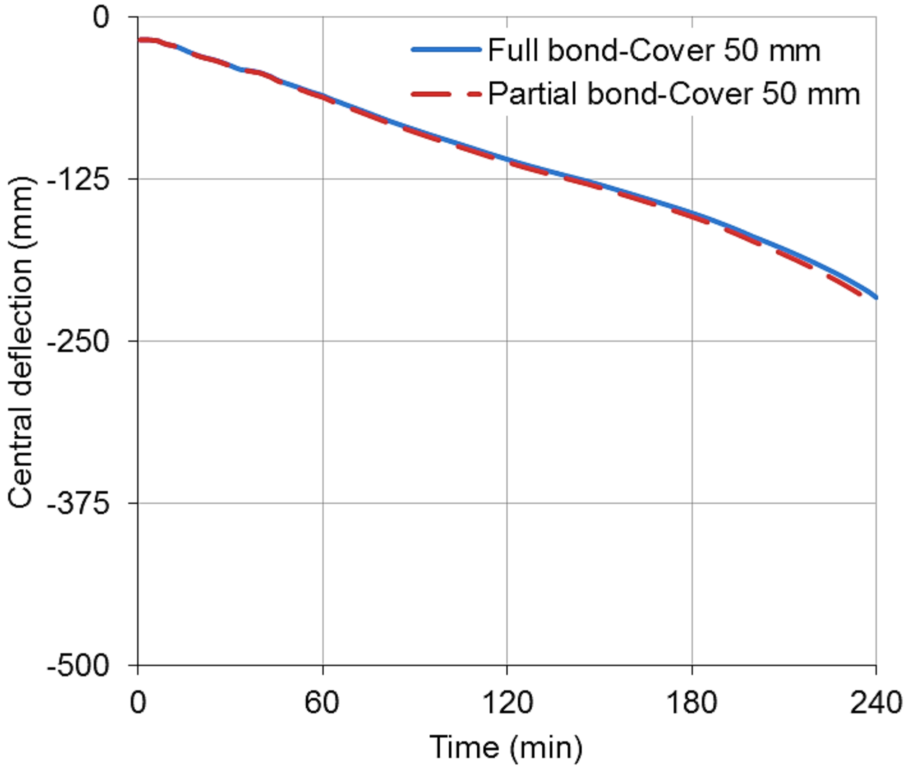

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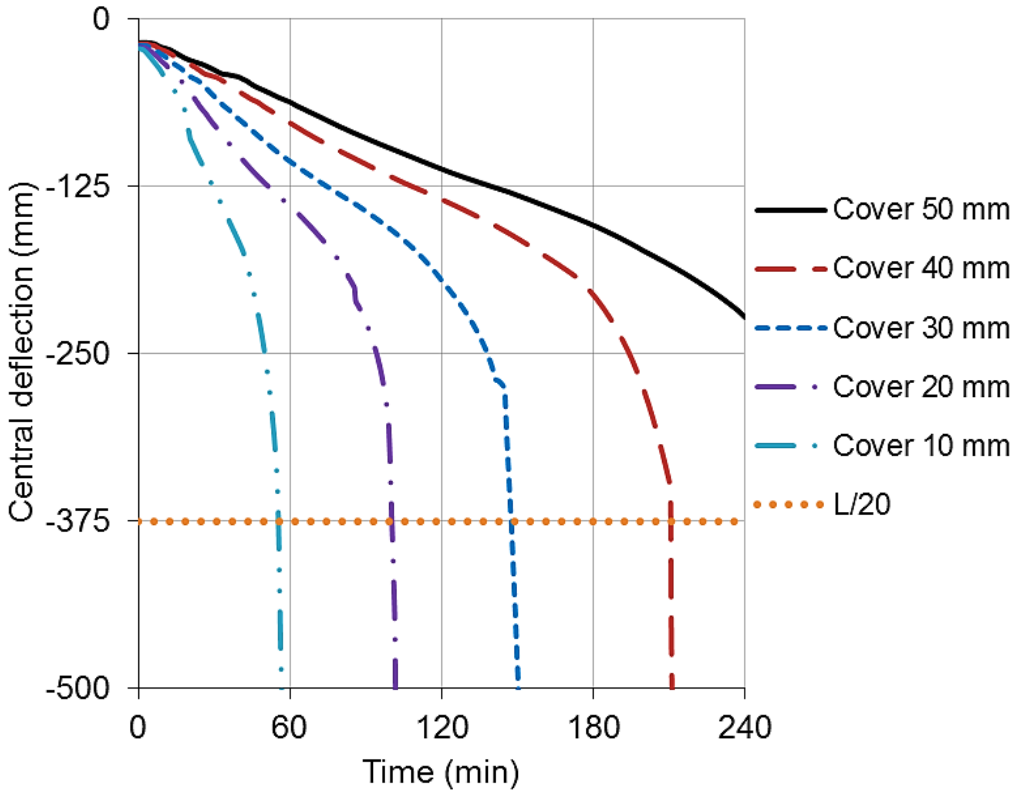

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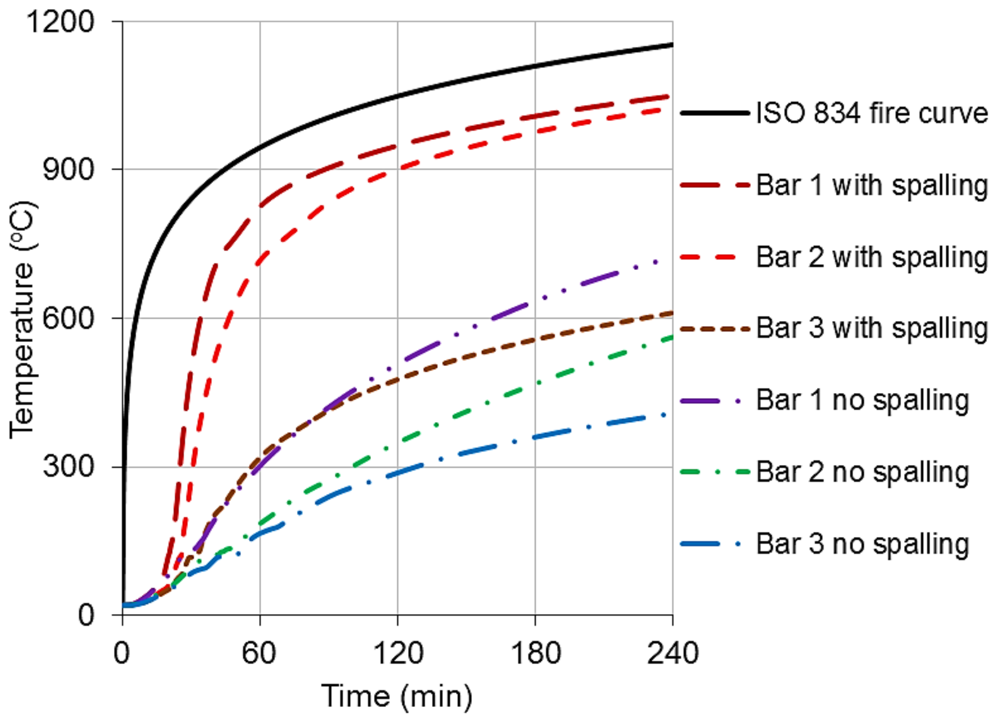

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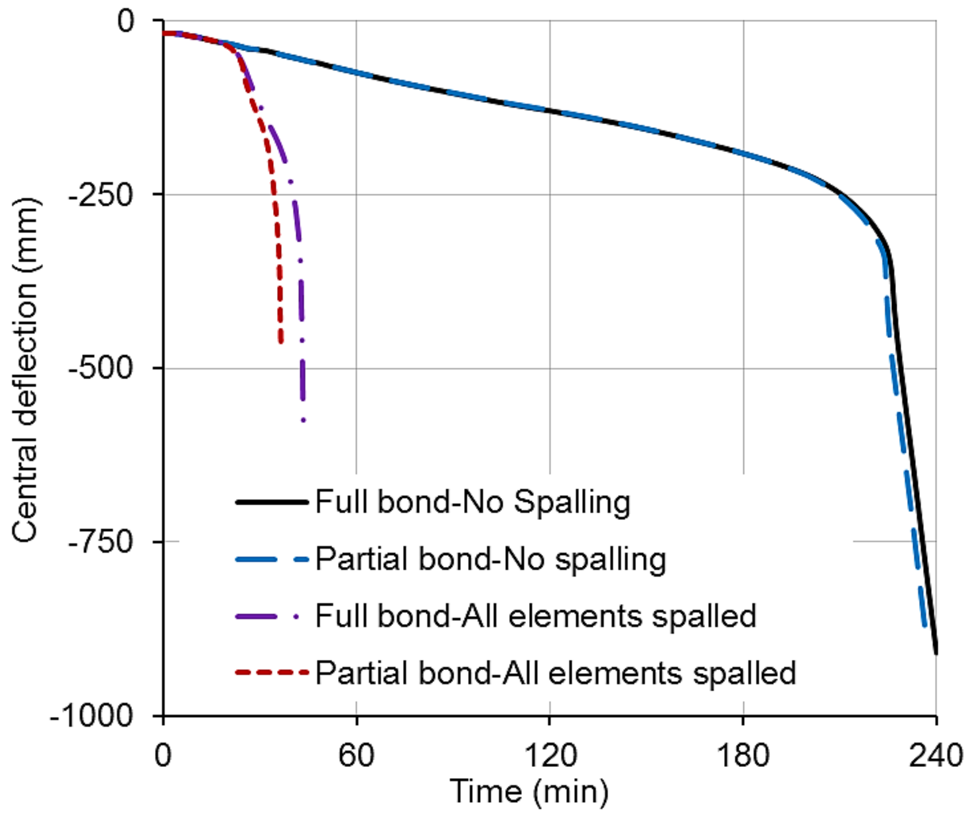

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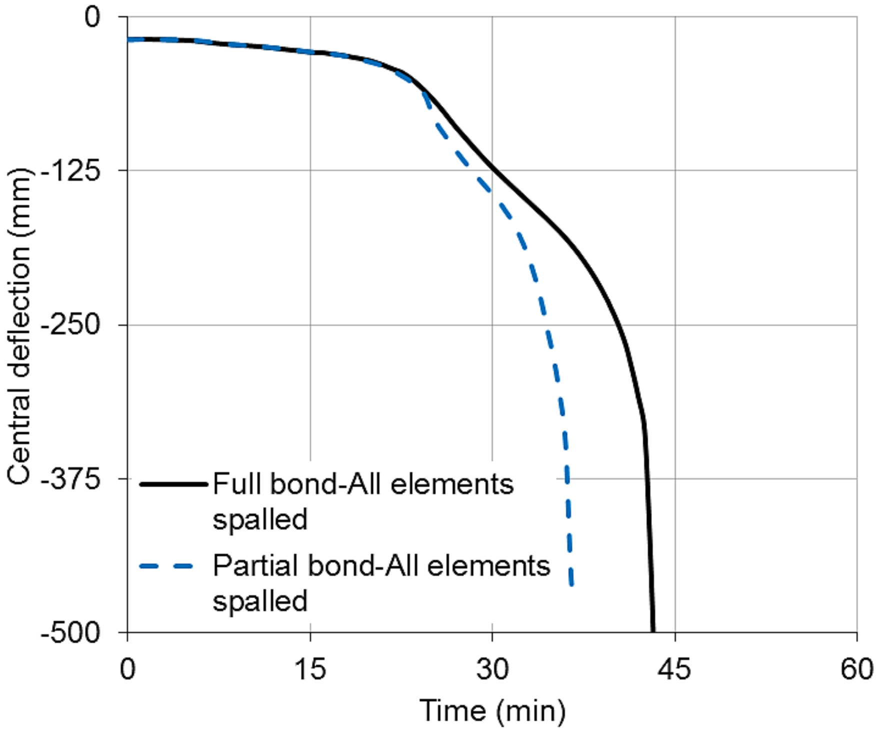

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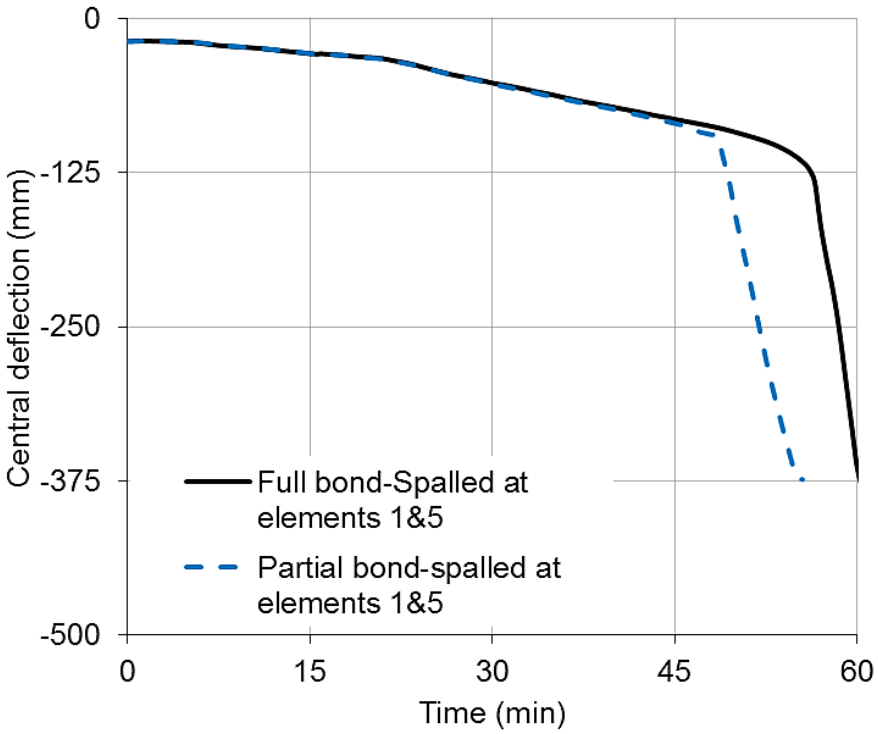

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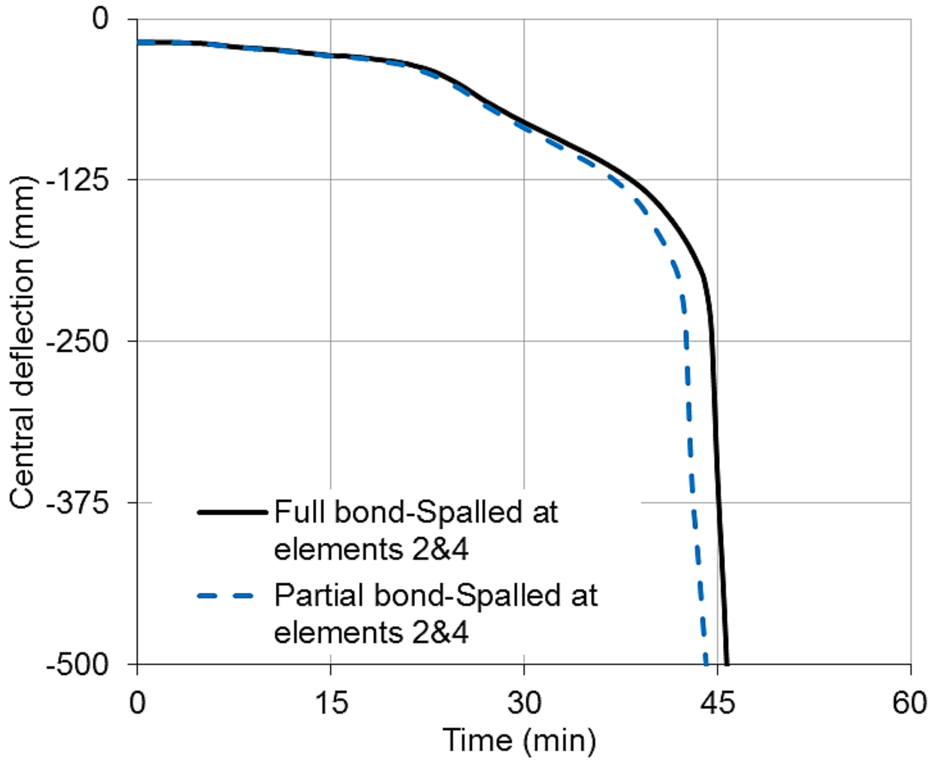

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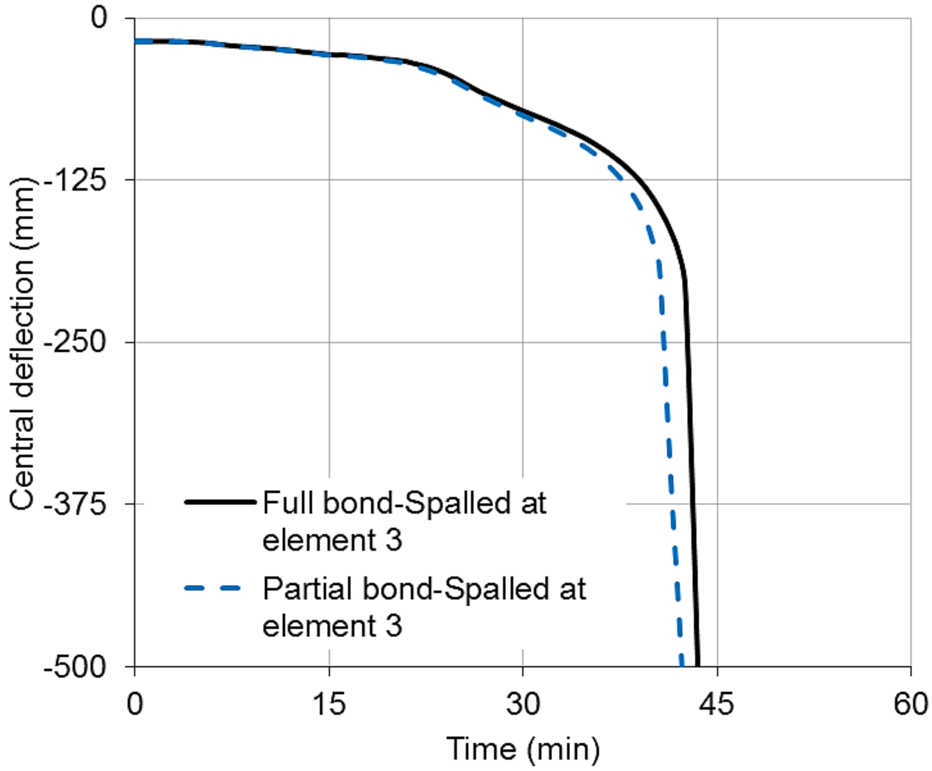

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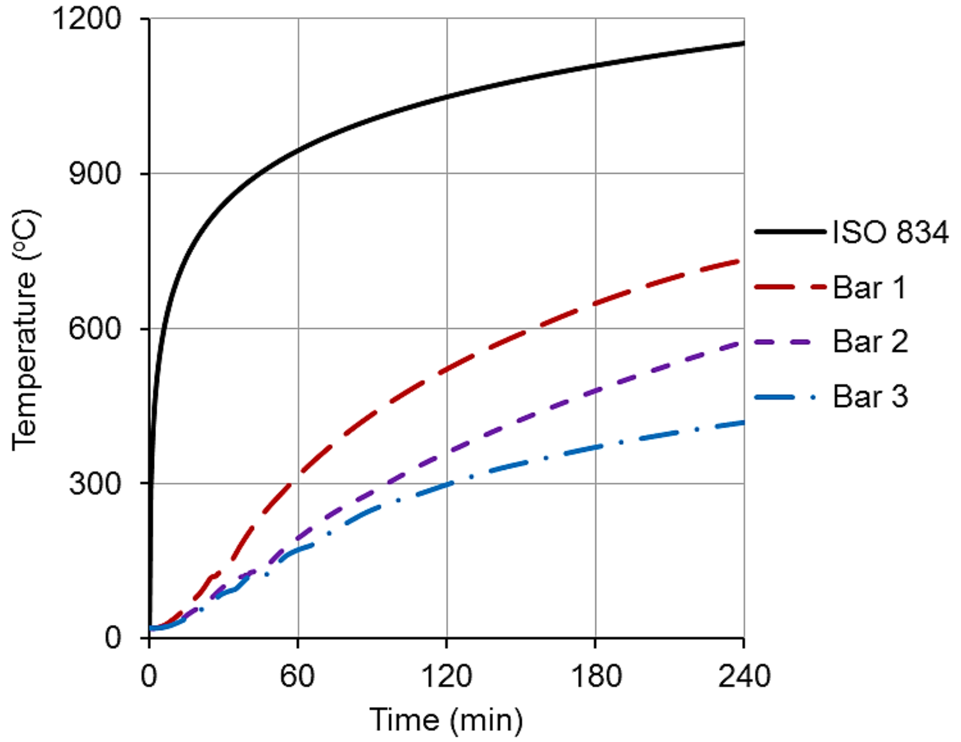

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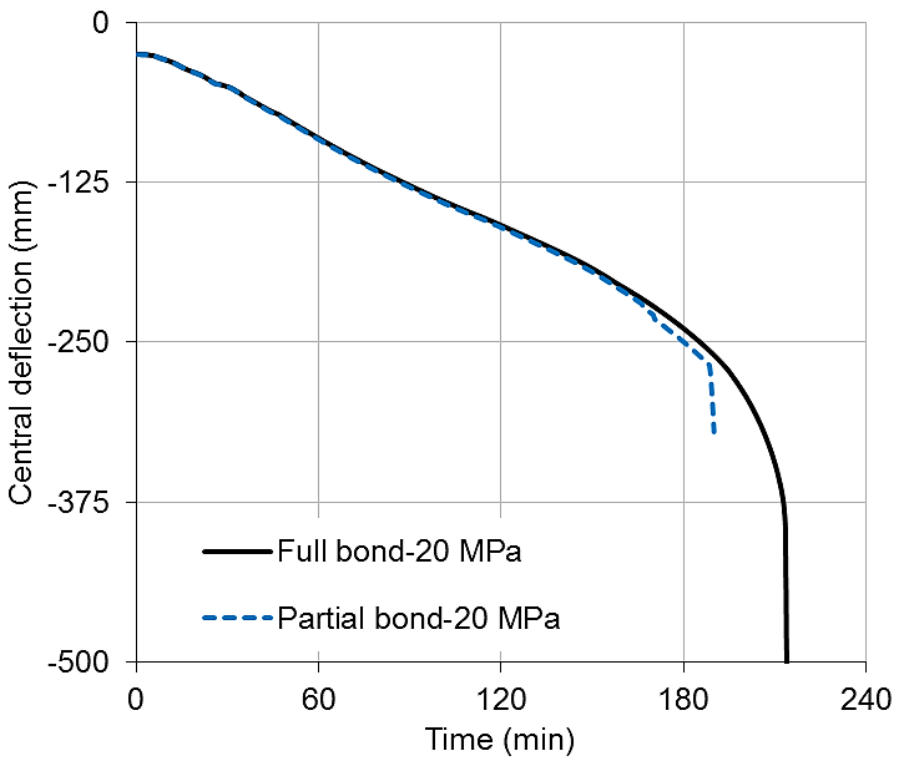

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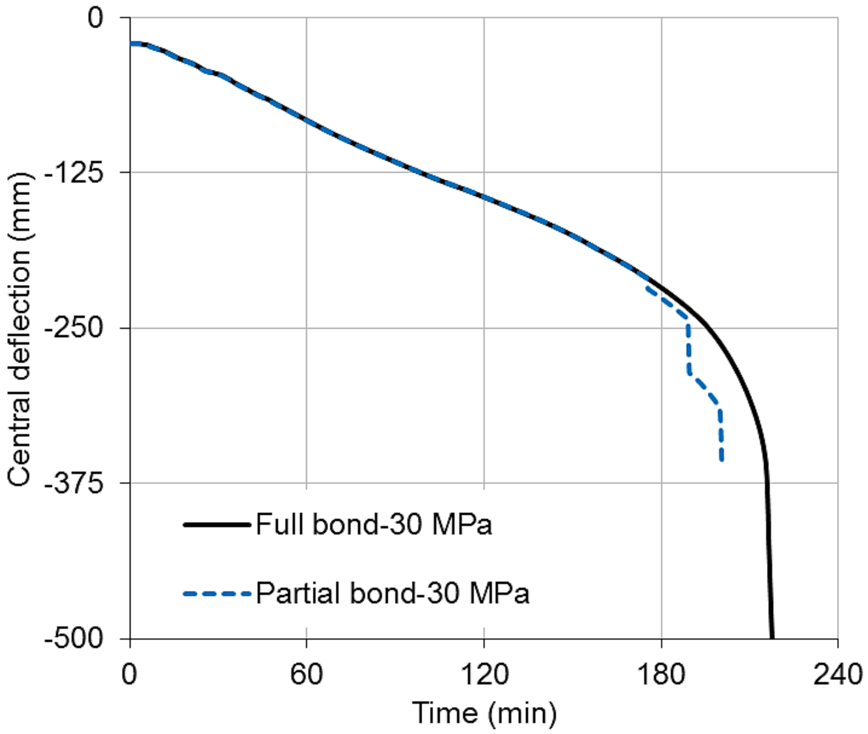

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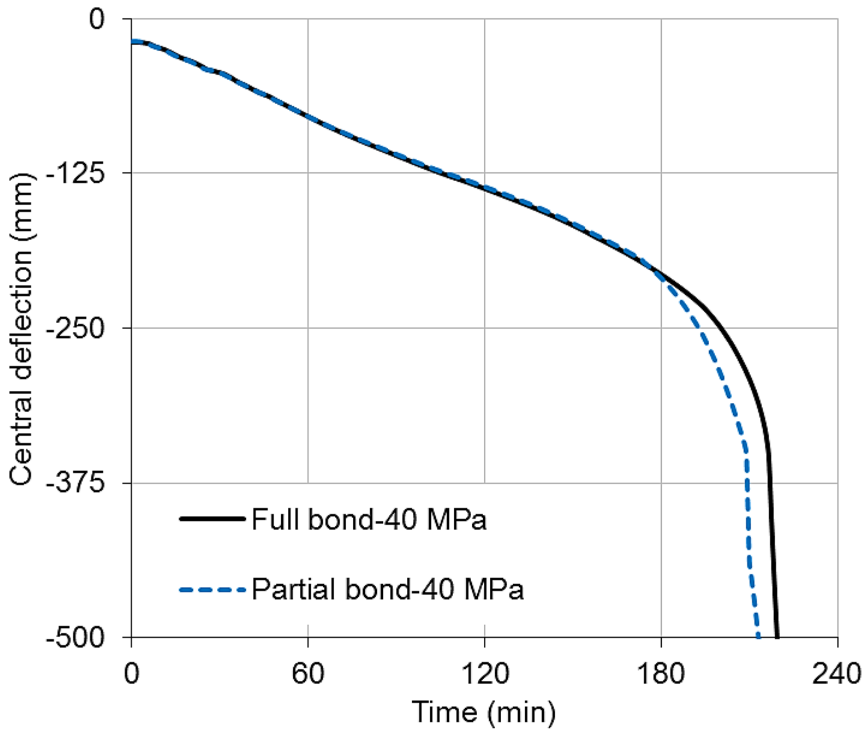

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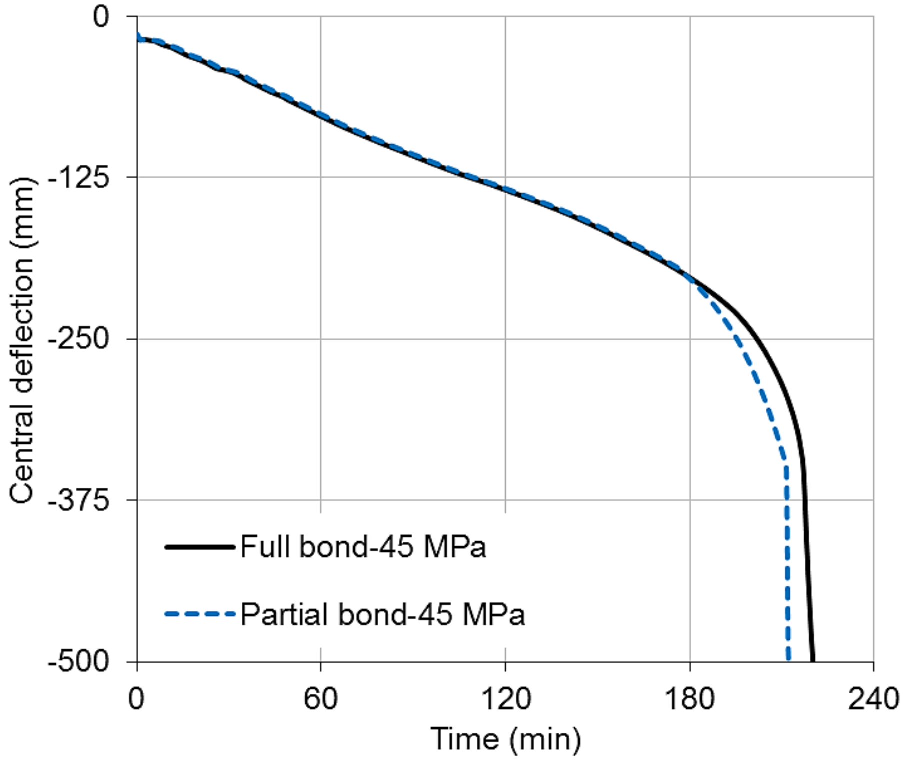

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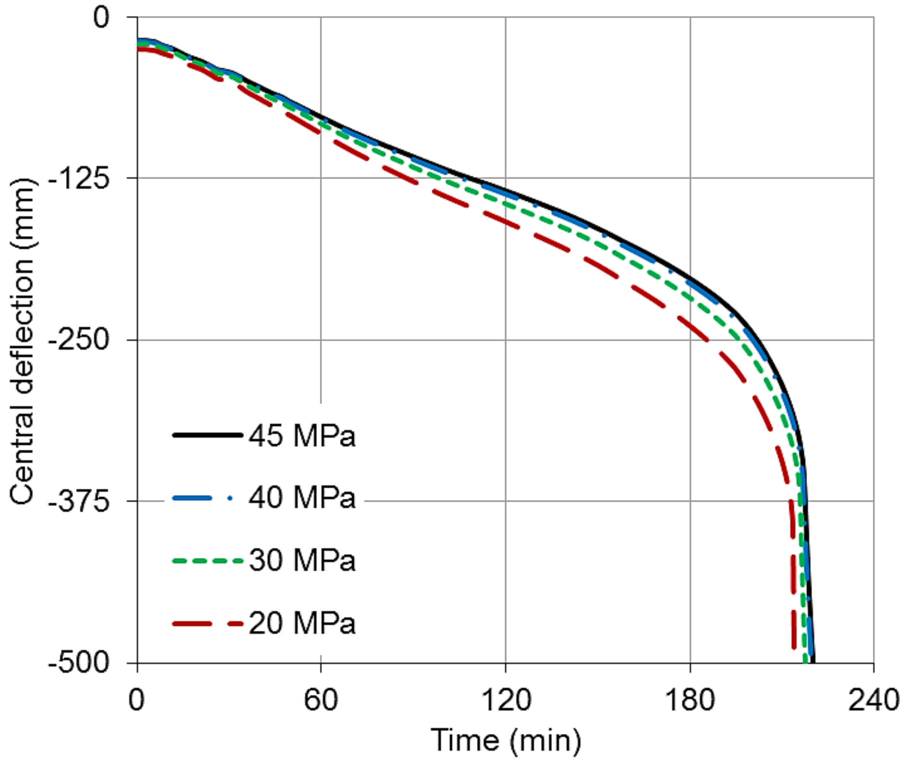

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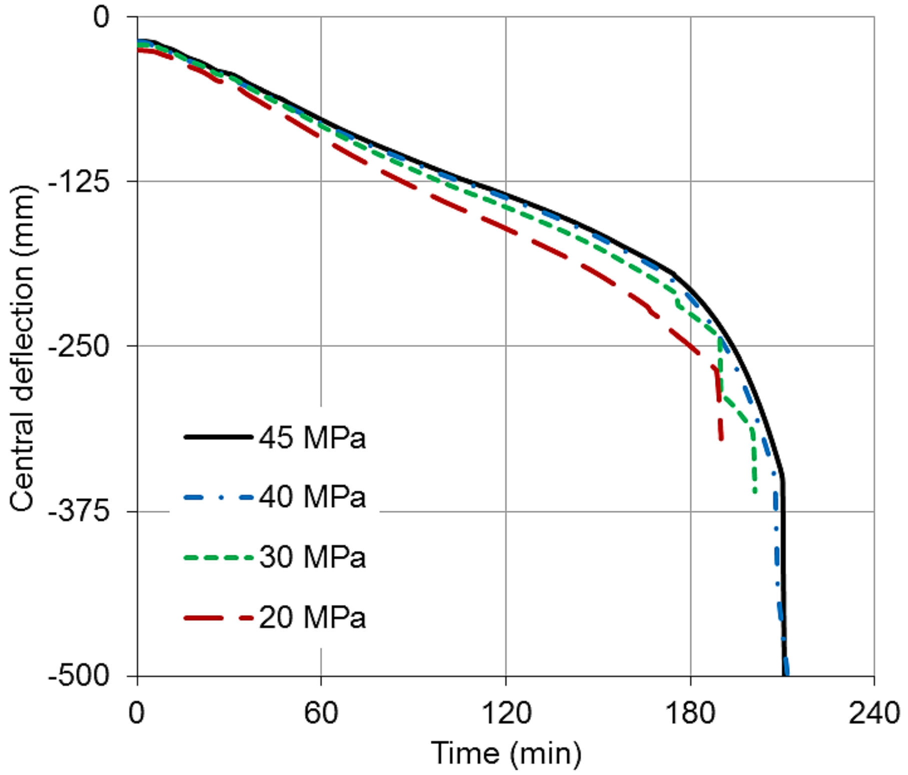

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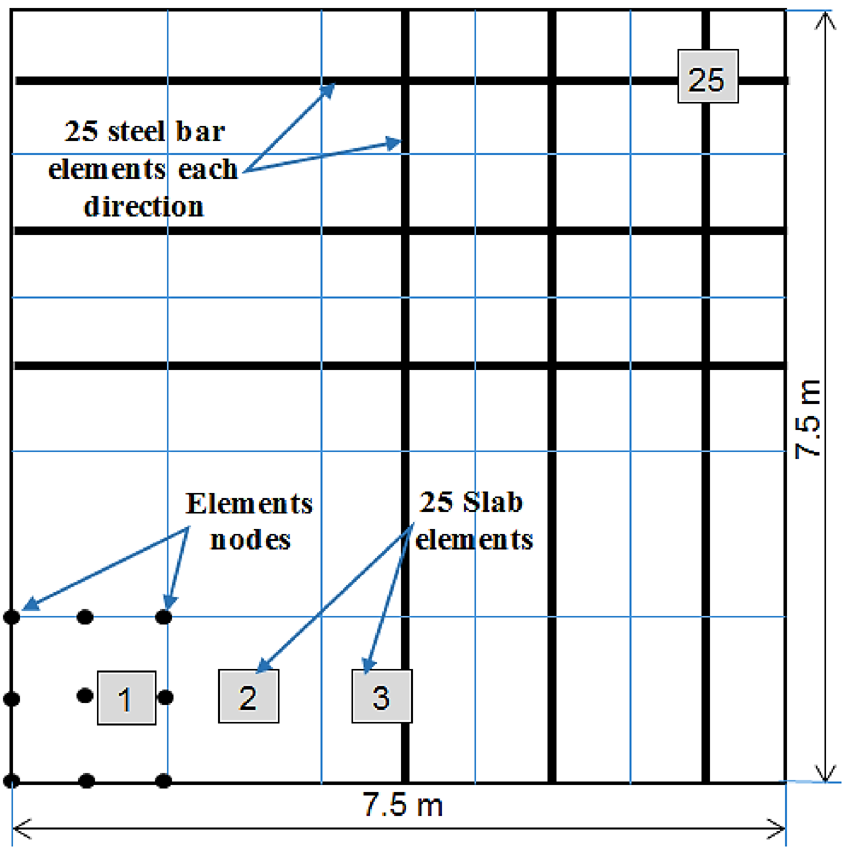

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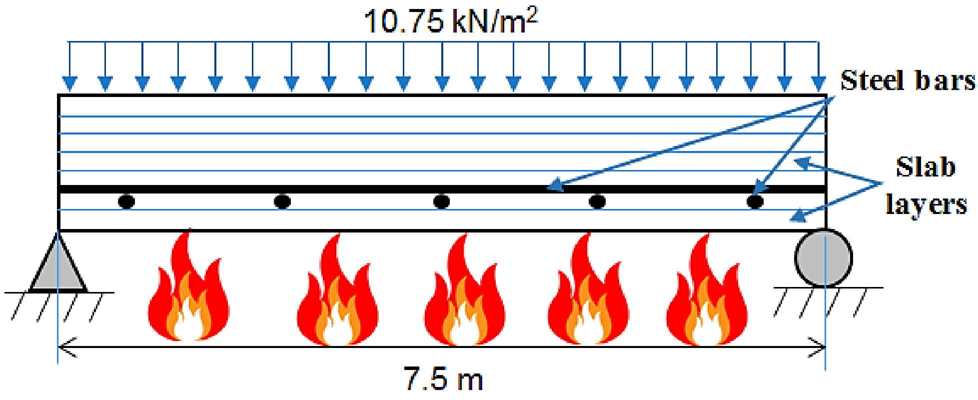

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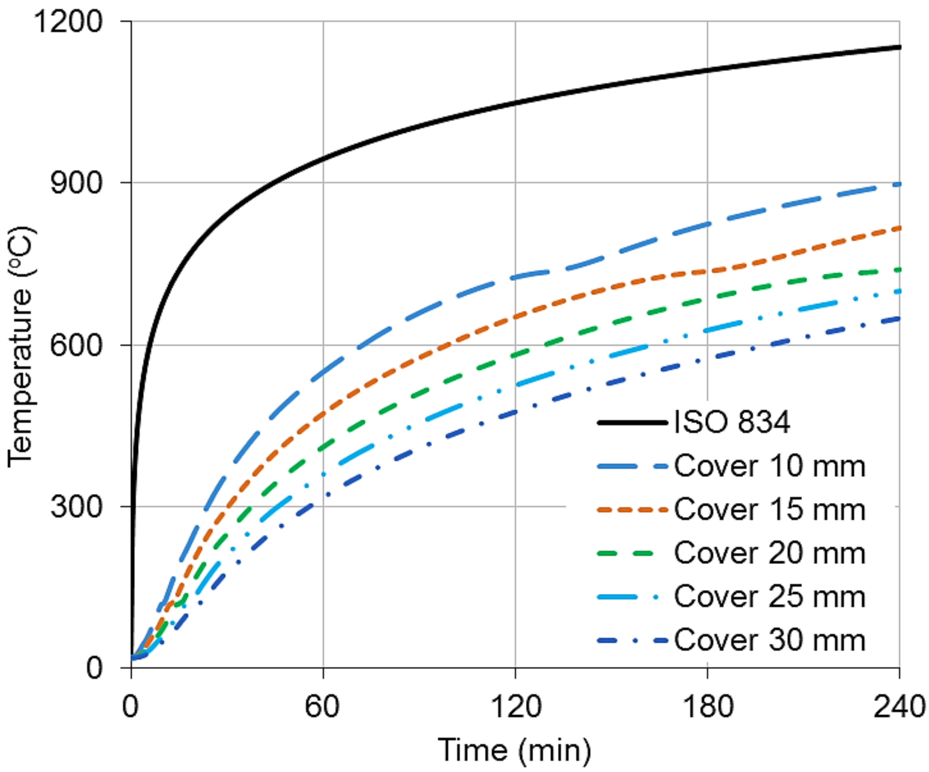

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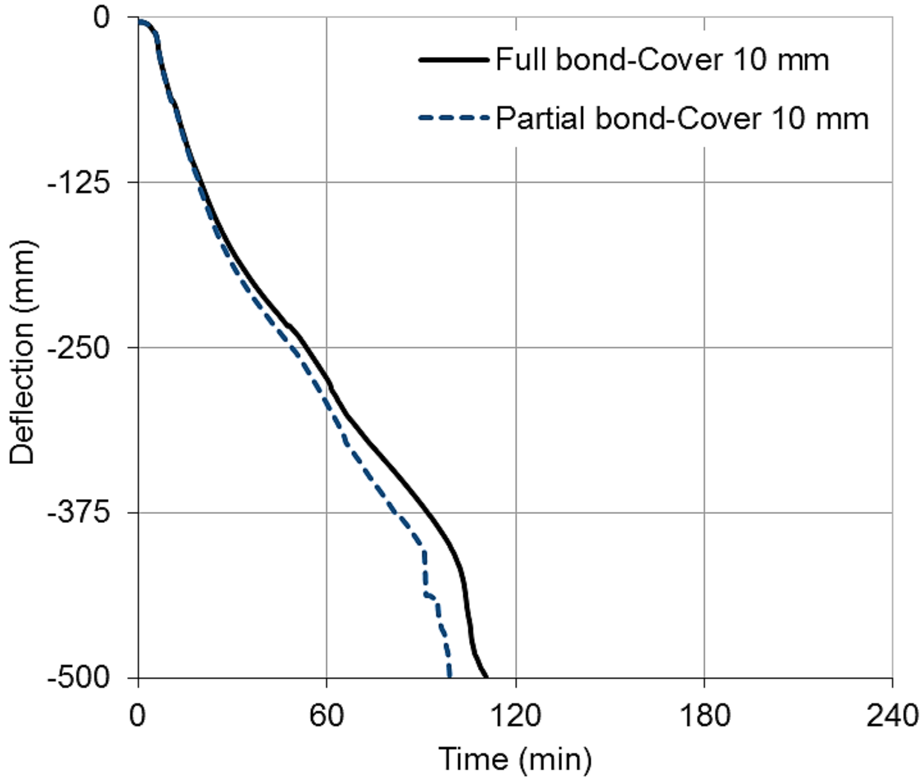

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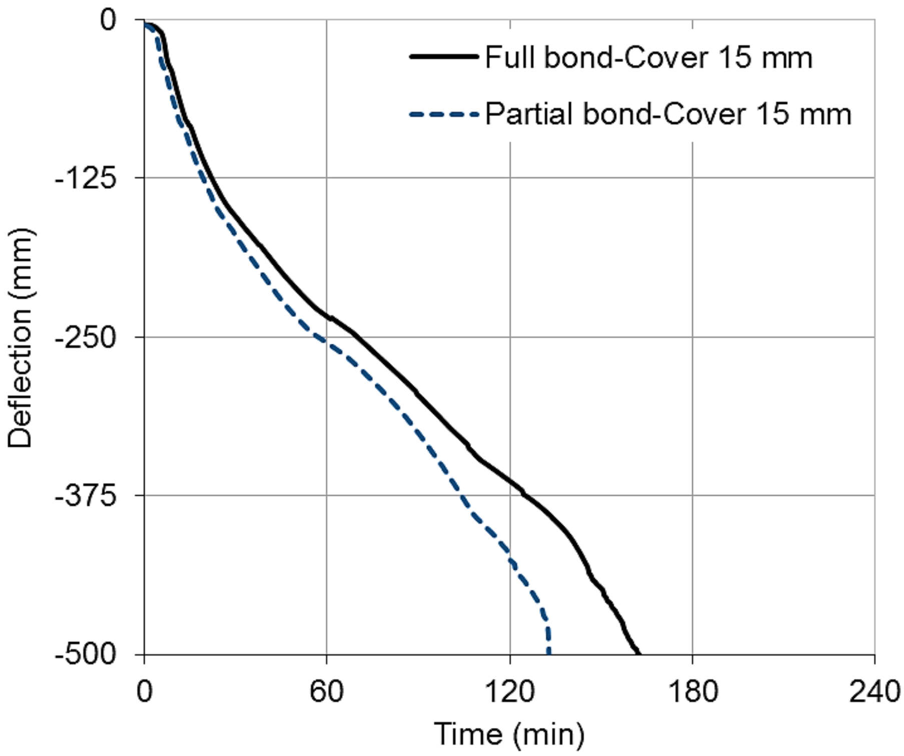

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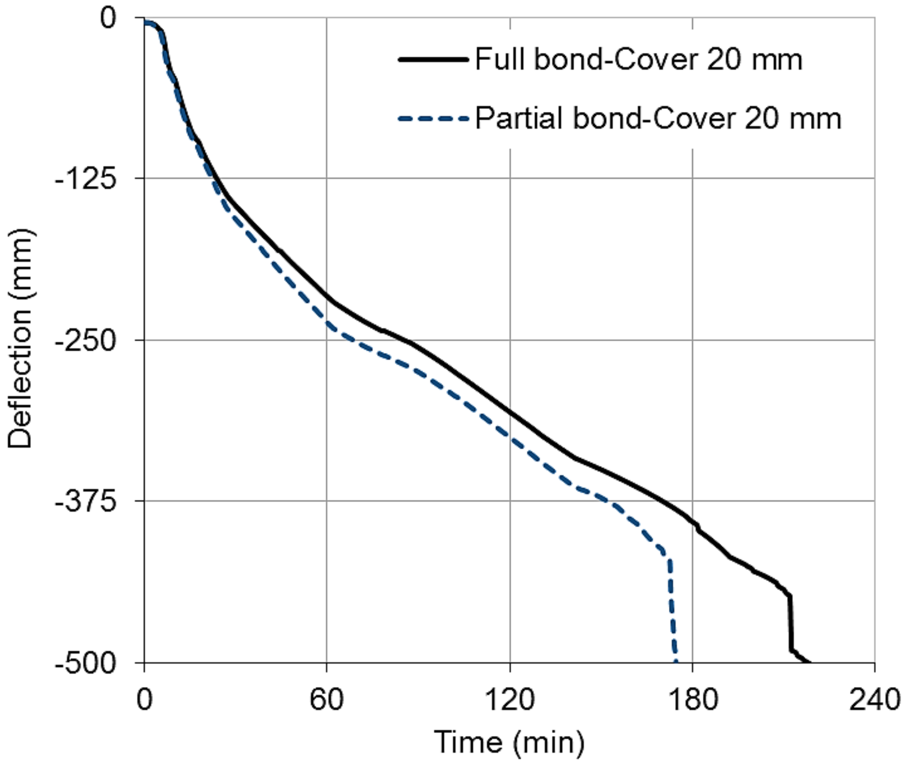

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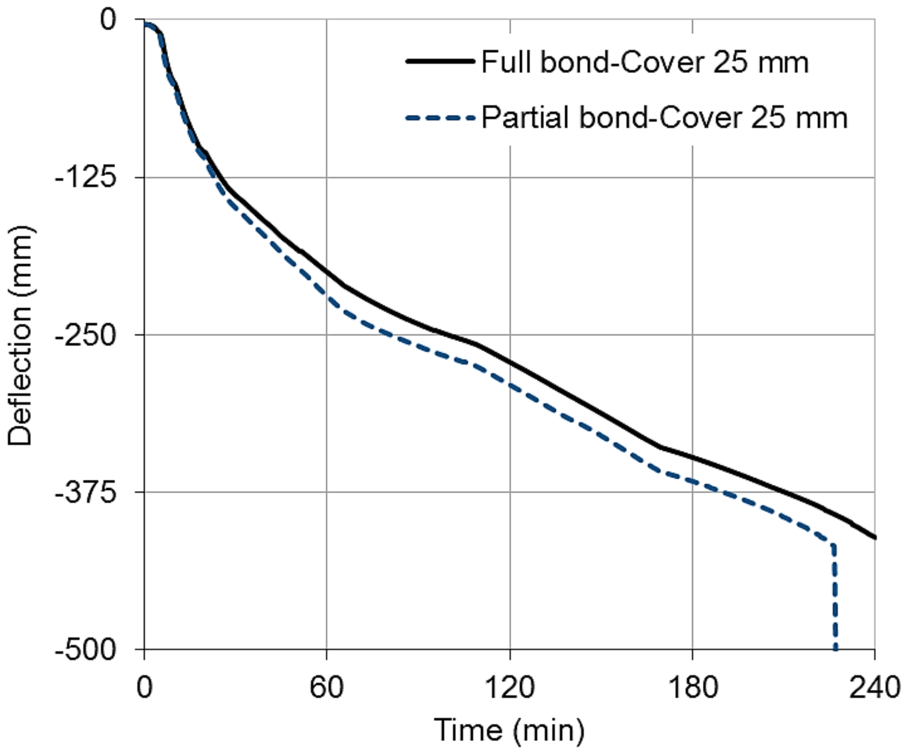

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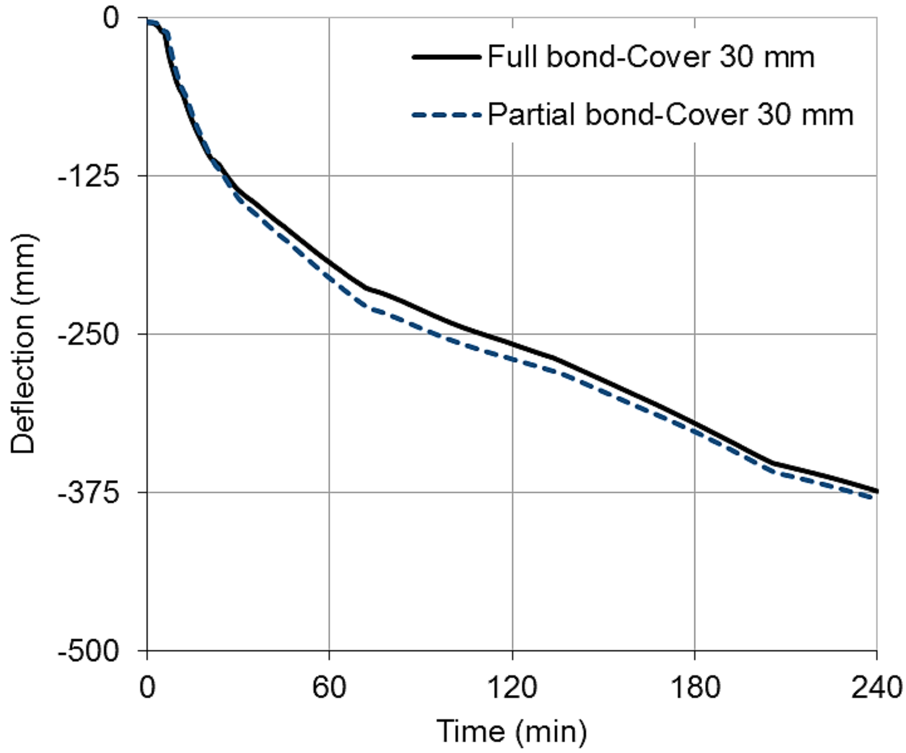

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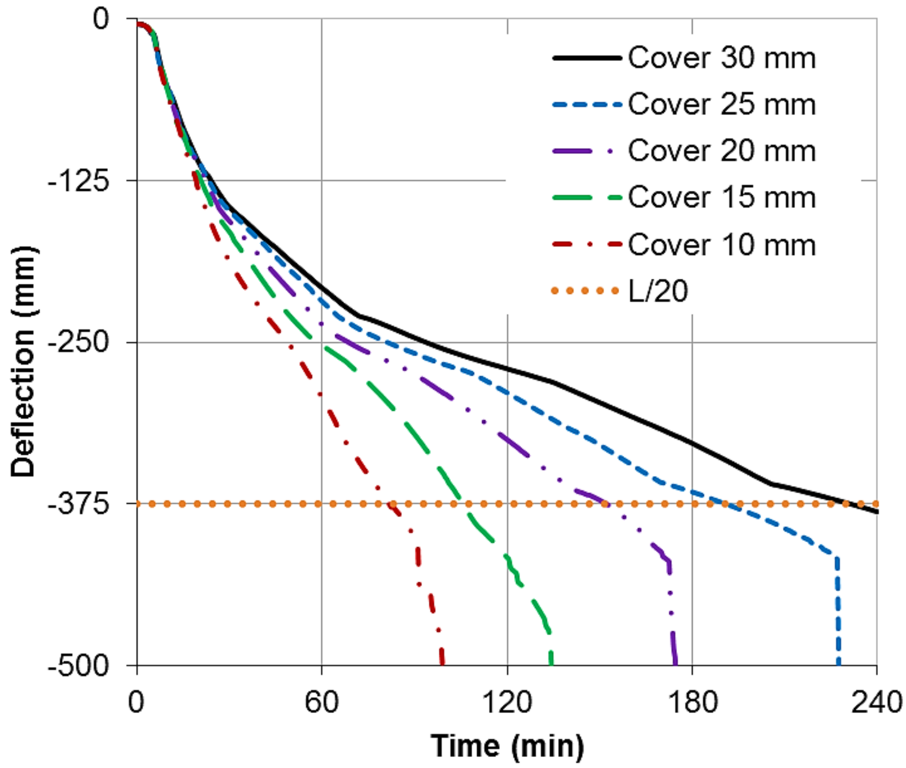

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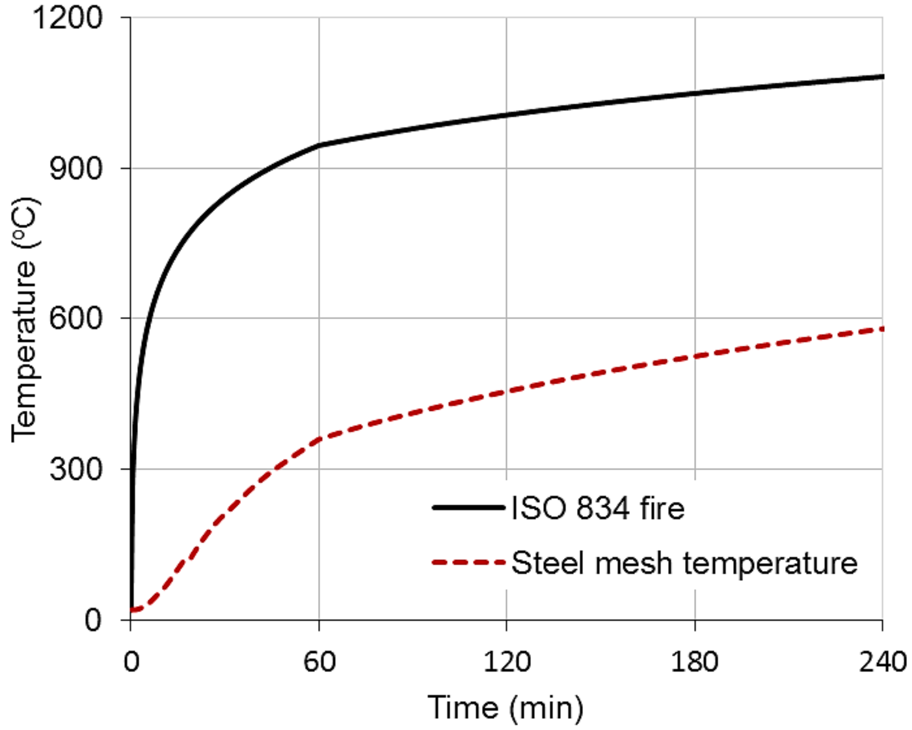

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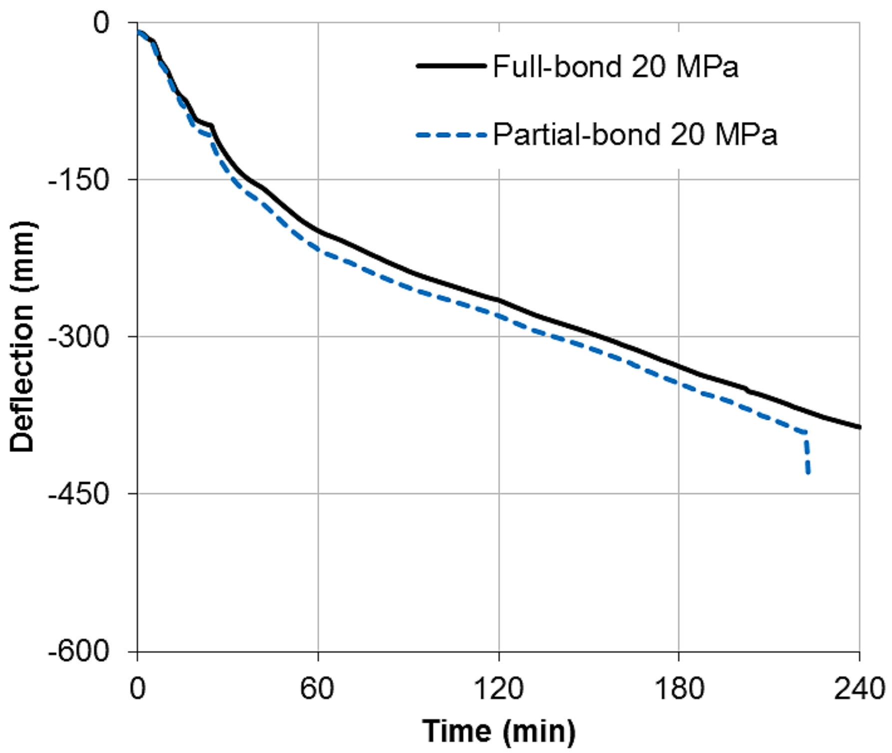

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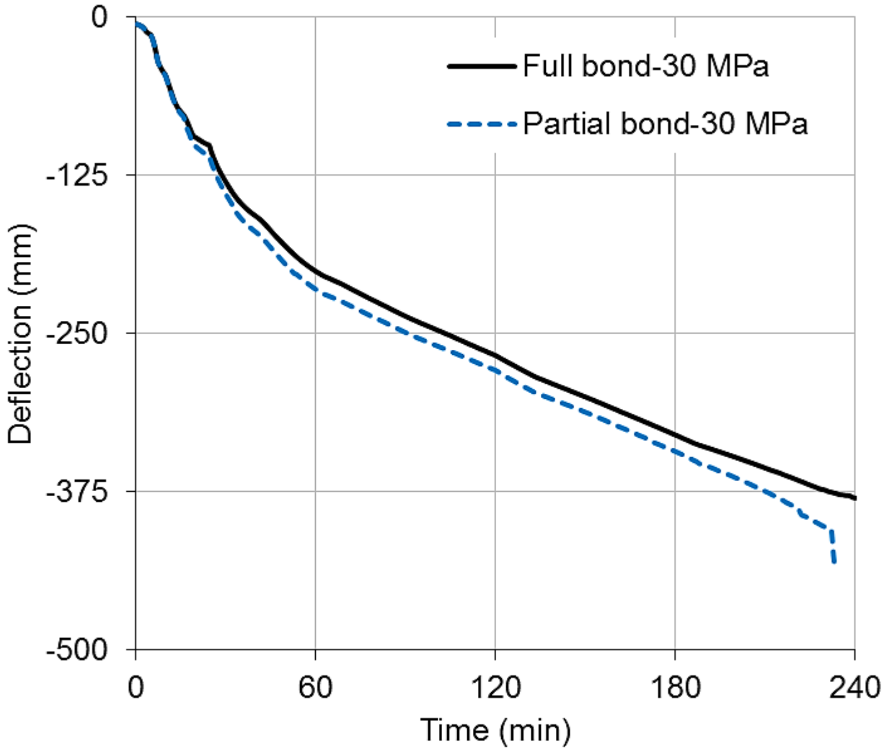

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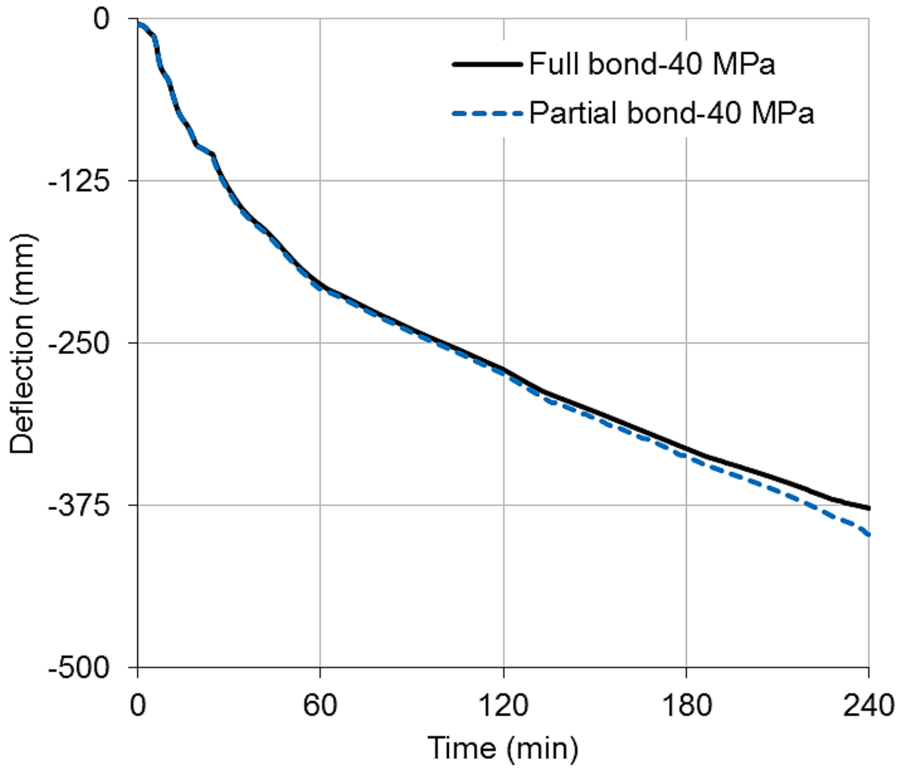

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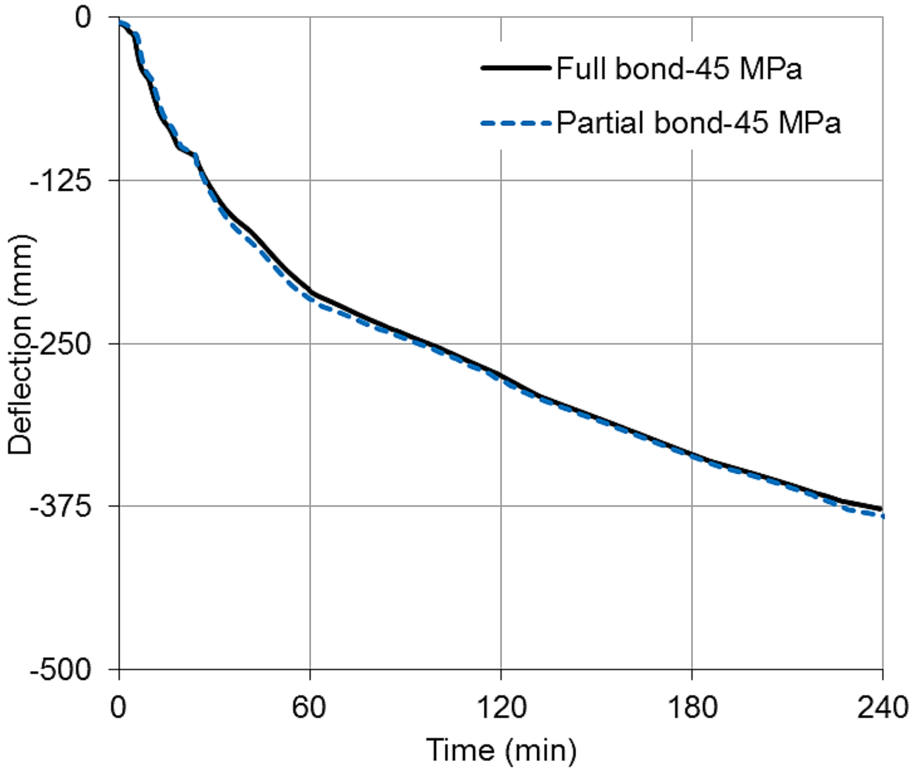

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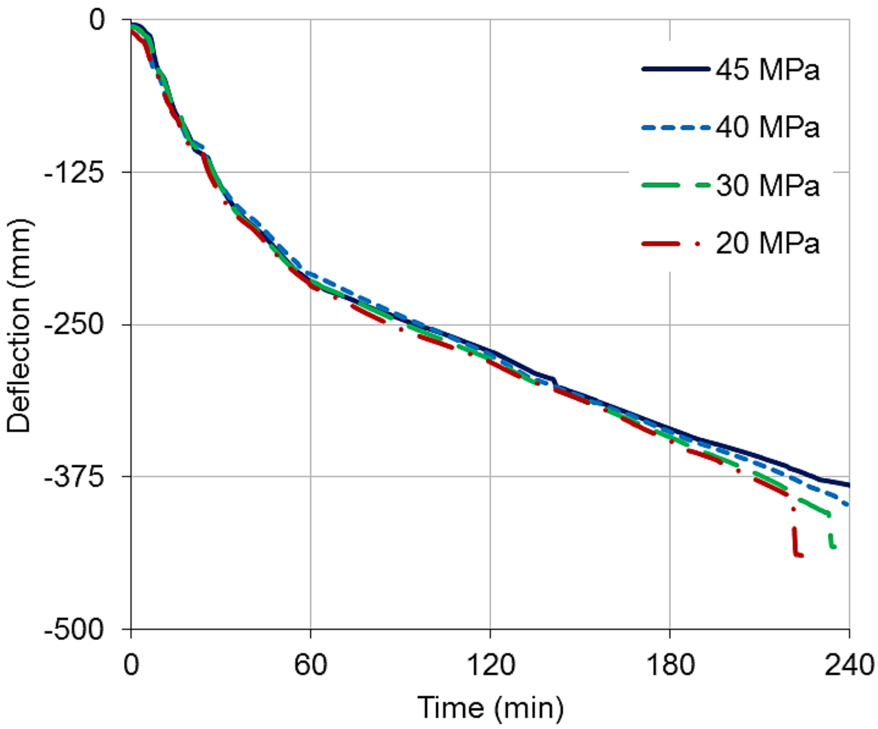

Fig. 44 Influence of different concrete strengths on the central deflections of the slab modelled as partial bond

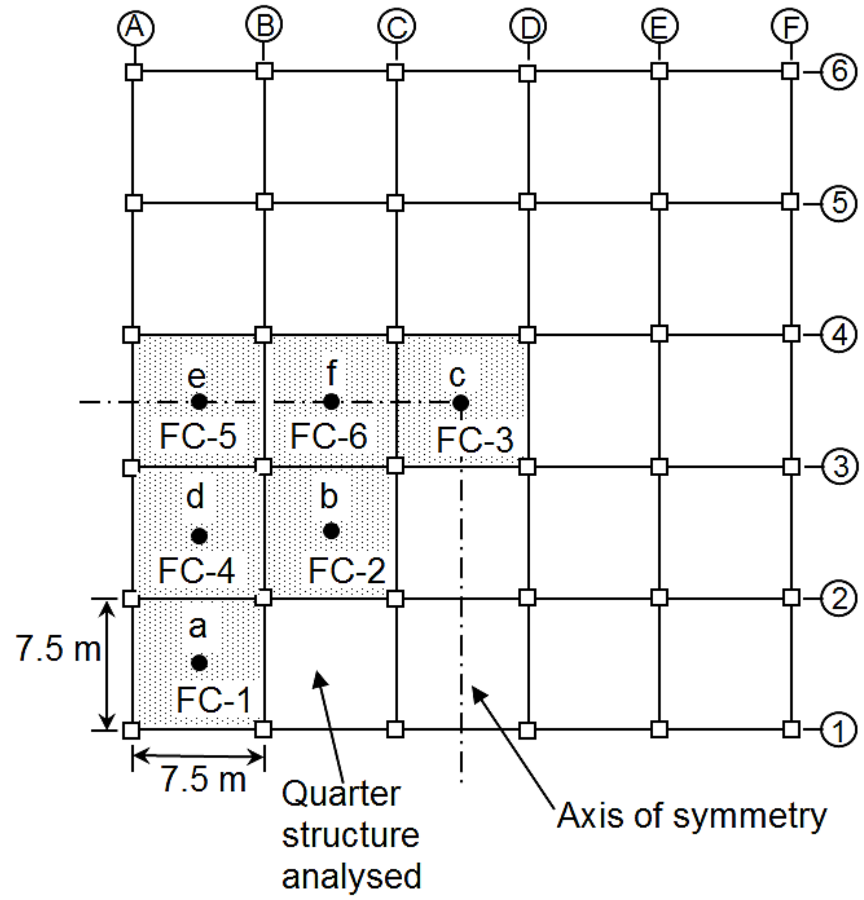

Fig. 45 Floor layout of the building with different locations of the fire compartments

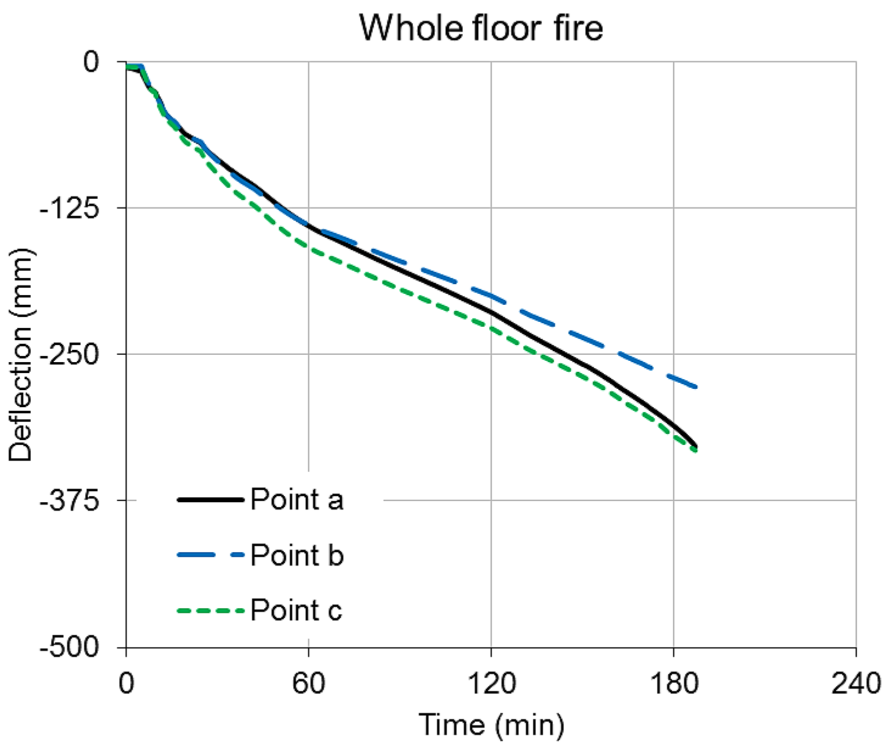

Fig. 46 The deflections of the floor slabs at the position a, b and c under whole floor fire

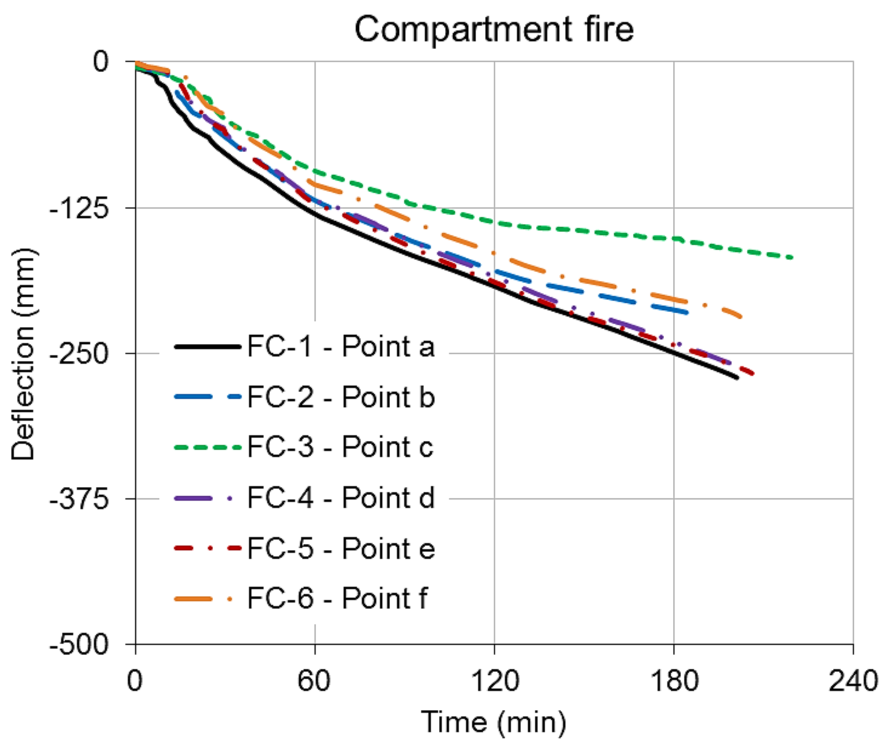

Fig. 47 Comparison of the central deflections of the floor slabs for different fire compartments

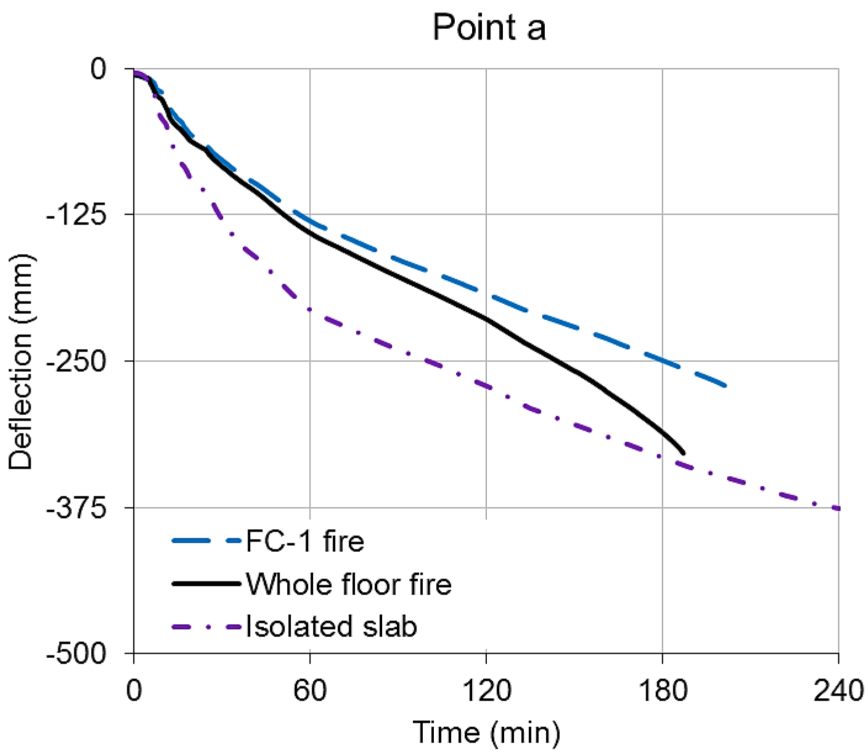

Fig. 48 Comparison of the deflections at Position a for compartment fire FC-1 and whole floor fire, together with the central deflection of isolated simply supported slab.

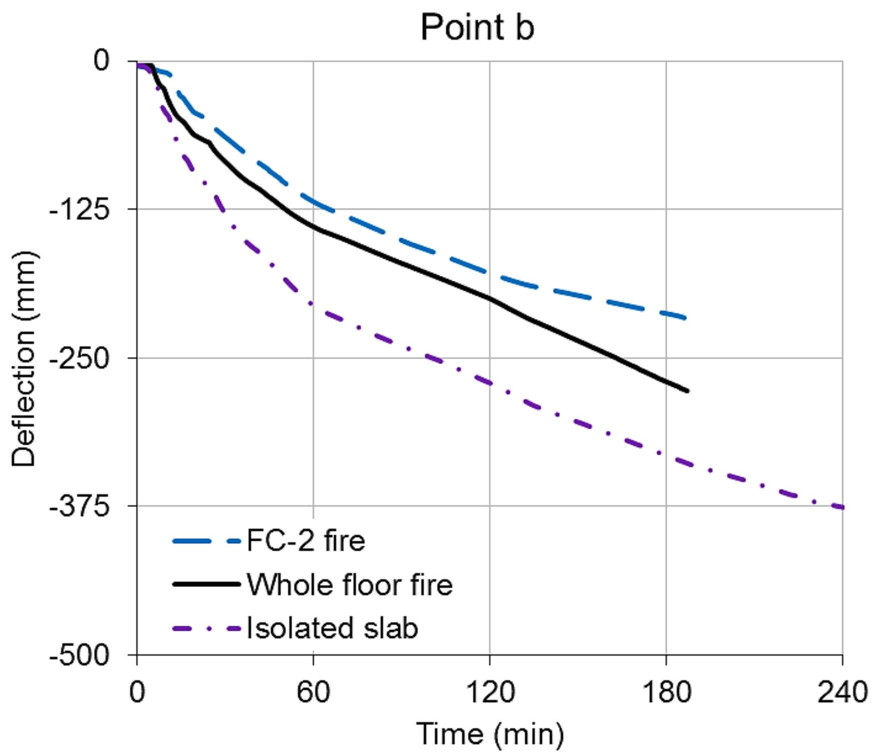

Fig. 49 Comparison of the deflections at Position b for compartment fire FC-2 and whole floor fire, together with the central deflection of isolated simply supported slab.

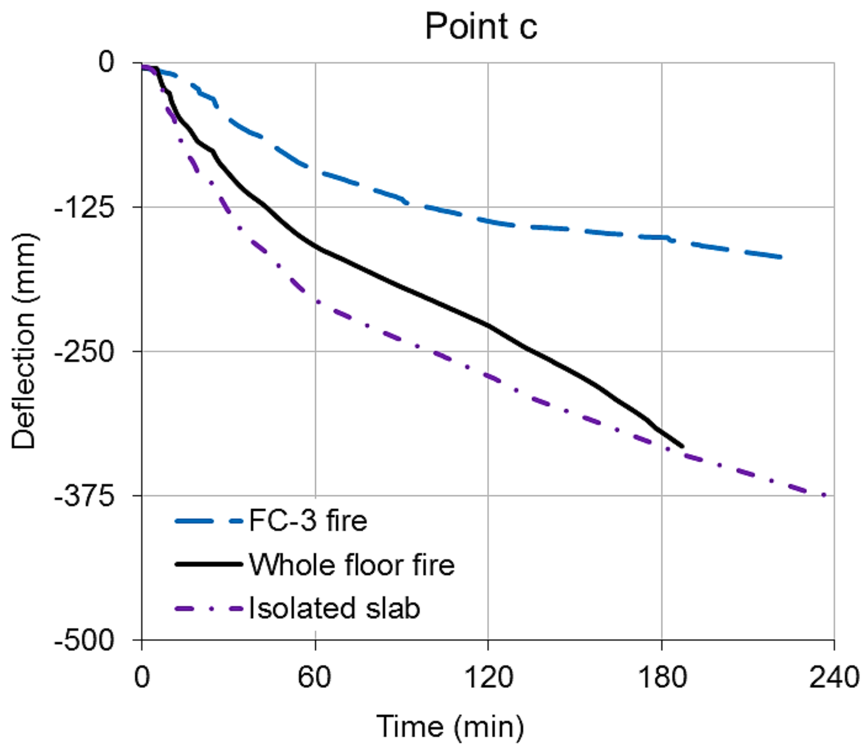

Fig. 50 Comparison of the deflections at Position a for compartment fire FC-3 and whole floor fire, together with the central deflection of isolated simply supported slab.