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# PERFORMANCE IN FIRE OF FIBRE REINFORCED POLYMER STRENGTHENED CONCRETE BEAMS AND COLUMNS: RECENT RESEARCH AND IMPLICATIONS FOR DESIGN

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**Keywords**: Fire, Fibre Reinforced Polymers, Concrete, Strengthening, Repair, Rehabilitation.

**Abstract**. *This paper considers the fire performance of concrete beams and columns that have been strengthened with fibre reinforced polymers (FRPs). Results from four recent full-scale tests are presented. A newly developed type of insulation was employed and the thickness of the insulation (15 to 20 mm) was approximately half that provided in earlier tests. All of the members survived four hours of the fire exposure. A conceptual model for design is also presented. Further research needed to fully develop the conceptual model to a more practical design tool is outlined.*

## 1 INTRODUCTION

Fibre reinforced polymer (FRP) materials are increasingly being applied in many areas of construction, particularly for strengthening of concrete beams and columns. However, concerns associated with fire remain an obstacle to applying FRP materials in buildings and parking garages due to their susceptibility to degradation at elevated temperatures [1,2].

The first research in the area of fire and FRP strengthening was conducted in Switzerland. Deuring [3] tested beams strengthened with Carbon FRP (CFRP) sheets in fire conditions, and found that insulated beams obtained satisfactory fire endurance. Additional work was conducted in Belgium on CFRP plated beams using multiple insulation schemes [4]. More recent work has considered the performance in a real compartment fire [5], and these authors found that the FRP strengthening schemes performed poorly even with insulation systems. On the other hand, Palmieri et al. [6] found that concrete beams strengthened with surface mounted FRPs could achieve a 2 hour fire endurance when appropriate insulation was provided.

Over the past ten years, Queen's University and the National Research Council have collaborated to investigate the performance of FRP strengthened concrete structures in fire [2,7,8,9]. Four recent full-scale tests have been conducted to expand knowledge from earlier work, and the results from these tests are presented in this paper. These new results extend previous knowledge because the loads on the members were higher than the design capacity of the unstrengthened member, and the thickness of the insulation was approximately half that provided in earlier tests.

In addition to the paucity of test data on FRP strengthened concrete members in fire, design guidance for fire endurance is extremely limited [1,10,11]. To address this deficiency, a conceptual model for the design of FRP strengthened concrete structures for fire conditions is presented. The model is based on the experience gained from conducting 17 full-scale tests over the past ten years and from numerical models developed to predict the performance of FRP strengthened concrete structures in fire.

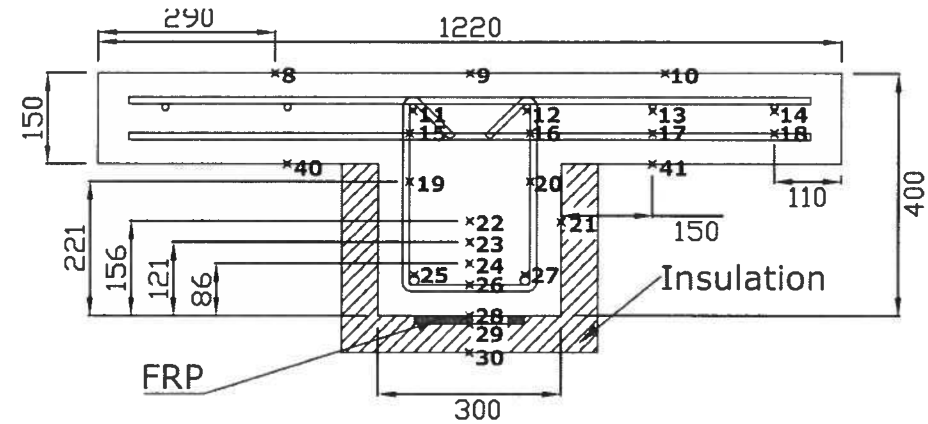



Figure 1. T-beam dimensions and thermocouple locations (shown by **x**).

## 2 RECENT TESTS AND RESULTS

Four recent full-scale tests have been conducted to expand knowledge from earlier work. Two full-scale T-beams (3.9 m span), one circular column, and one square column were all strengthened with external FRP. Fire protection for the FRP was provided by sprayed insulation.

### 2.1 T-beams

The T-beams (Figure 1) were chosen to be 400 mm deep, with a web breadth of 300 mm and a 1220 mm × 150 mm flange. Two 15 mm diameter steel reinforcing bars were provided as tensile reinforcement in the web at an effective depth of 340 mm. The clear cover to the reinforcement was 40 mm. The reinforcement had a tested yield strength of 406 MPa while the concrete was carbonate aggregate with a tested 28 day strength of 36 MPa. Each beam was fitted with a 100 mm wide, single-layered strip of carbon FRP for flexural strengthening (Tyfo SCH-41), which ran longitudinally along the bottom of the web as shown in Figure 2. U-shaped glass FRP sheets were wrapped around the ends of the beams to provide anchorage for the longitudinal FRP. The carbon FRP had a design strength of 834 MPa, a design modulus of 82 GPa, a design failure strain of 0.85%, and a thickness of 1.0 mm. This FRP strengthening scheme provided a 23% strength increase according to ACI 440 [10]. The beams were extensively instrumented with thermocouples as shown in Figure 1.

Figure 2 shows the insulation being applied to the beams. One beam was insulated with a 13 mm thick layer of insulation, which covered the entire web and extended 100 mm along the bottom of the flange on either side of the web. The other beam was insulated with a 19 mm thick layer, but it was to be placed more sparingly. Near the U-wraps, the insulation covered the entire web, but in the central part of the beam, the insulation only covered a portion of the side of the web. The insulation in this location only extended approximately 150 mm up the side of the web from the bottom of the beam. The thickness of the insulation for these beam tests was approximately half the thickness provided in earlier work [2,7,8,9].

For the fire tests, a superimposed load was applied such that the total load on the beam during the fire exposure was 10% greater than the design load of the beam without considering the strength of the FRP. This resulted in a test load ratio of 0.72 calculated as the total load during the test divided by the strengthened design capacity according to ACI 440 [10]. Furthermore, the beams were tested in a simply supported condition without any lateral or moment restraint. The beams were allowed sufficient space to expand at the ends without touching the sides of the supporting frame. Thus, the testing conditions were more severe than those considered in previous work [8] where the beams were restrained laterally and subjected to lower loads. The beams were then exposed to a standard ASTM E119 [12] fire scenario from below.

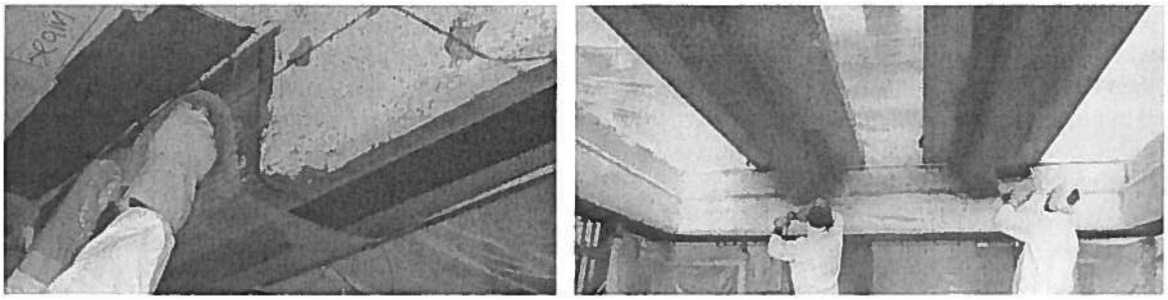


Figure 2. FRP strengthening and insulating procedures.

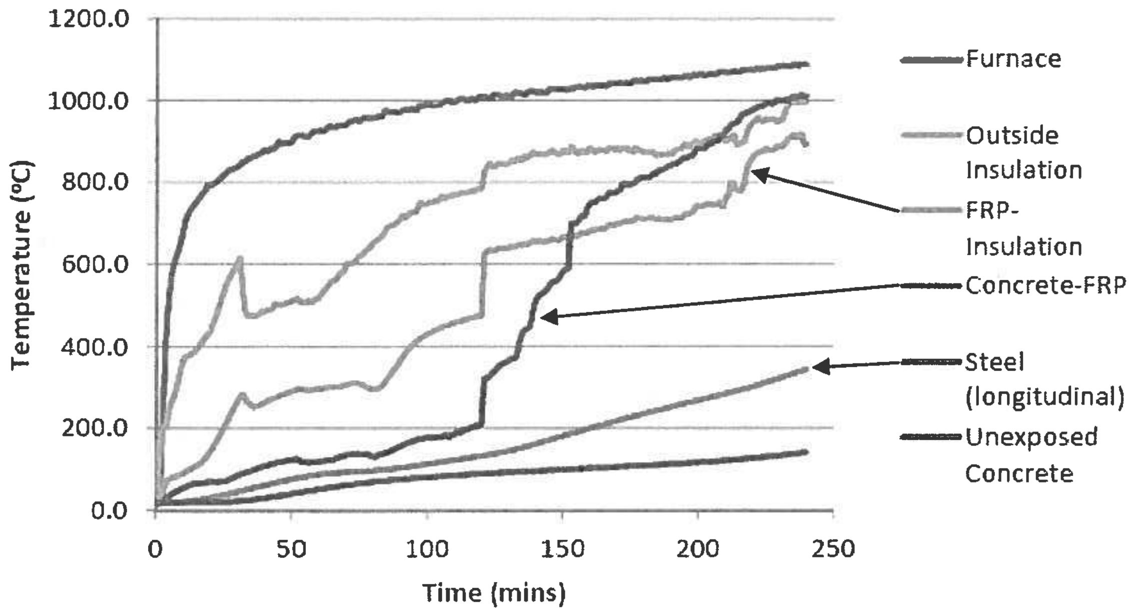



Figure 3. Temperatures measured in the T-beam with 19 mm of insulation.

During the test, the most significant area of interest was the slow deterioration of the insulating materials. Over the course of the four hours, insulation protecting certain key areas began cracking, delaminating from the concrete, and eventually falling off completely. After approximately 15 minutes of the test, cracking of the insulation and flames coming from the cracks were noticed. As the test progressed, pieces of insulation, bordered by the aforementioned cracks, began delaminating and eventually falling from the beams. This occurred more quickly with the beam with 19 mm of insulation because only part of the web was protected and the insulation could start to delaminate on the side of the web. At 120 minutes into the test, a large portion of insulating materials fell from one side of this beam. In Figure 3, the temperatures in the FRP increased considerably after this point. At the end of the test, it was clear that both beams had completely lost their CFRP strengthening because most of the strips had burned away.

The temperatures at various locations on the beam with 19 mm of insulation are shown in Figure 3. This beam experienced much more significant debonding of the insulation than the other beam. The temperatures in the FRP increased fairly steadily over the first 120 minutes. At this point, the temperature at the interface between the concrete and the FRP reached approximately 200 °C. Based on previous material testing [13], the FRP is expected to start to lose its bond to the concrete at approximately 50 °C while it can retain approximately half of its strength up to 200 °C. After 120 minutes, the temperatures in the FRP increased rapidly because of delamination of insulation. The FRP is undoubtedly ineffective after this point. Nevertheless, the insulation is still effective in reducing temperatures in the concrete and reinforcing steel because the temperature in the reinforcing steel does not exceed 400 °C at any point in the four hour test.

At the end of the four hour fire exposure, the load on the beams was increased but the beams did not fail. As a result, the beams will be tested at ambient temperature at a later date for residual strength.

### 2.2 Columns

Figure 4 shows the dimensions and details of the two column specimens. One column was circular with spiral reinforcement while the other was square with tied lateral reinforcement. The height of the columns was 3810 mm. Two 38 mm thick square steel plates were attached above and below the columns. These plates were attached to the loading machine to provide fixed-fixed loading conditions.

The circular column was constructed with 35 MPa concrete while the square column had a concrete strength of 28 MPa. Carbonate aggregate was used in both concrete mixes. Clear cover to the reinforcement was 40 mm. Eight 20 mm diameter steel bars with a tested strength of 456 MPa were provided as longitudinal reinforcement for the circular column while four 25 mm bars with a tested strength of 477 MPa were provided for the square column. The circular column was strengthened with two layers of the same carbon FRP used for the T-beams while three layers were applied to the square column. The calculated strength increase, according to ACI [10], was 27% for the circular column and 20% for the square column. The square column was protected with a 19 mm thick layer of insulation while the circular column had an insulation thickness of 15 mm.

The fire test scenario for these columns was the ASTM E119 standard fire. The load applied during the fire test was 5% greater than the unstrengthened design strength for the circular column (test load ratio of 0.85) and 10% greater than the unstrengthened design strength for the square column (test load ratio of 0.97). All of these calculations were based on ACI 440 [10].

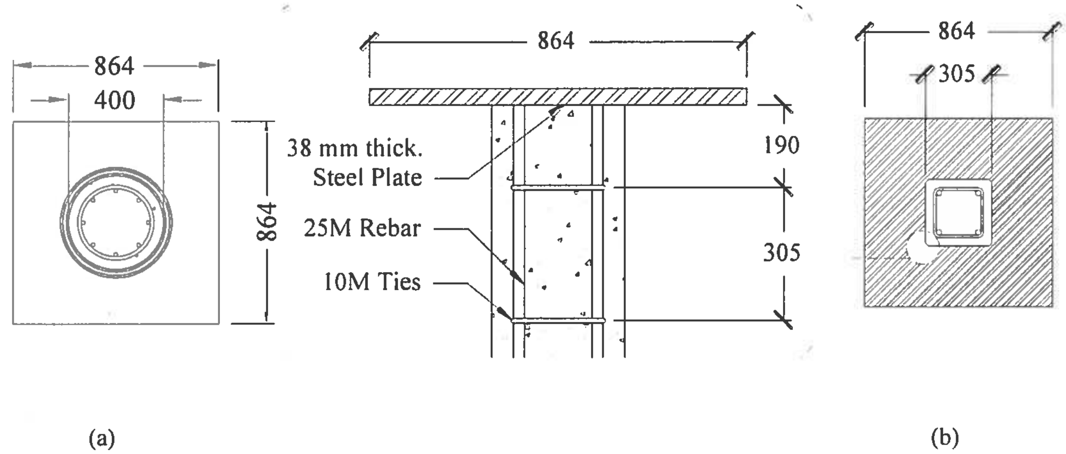



Figure 4. Dimensions of columns (a) Circular (35 MPa concrete, 8-20M longitudinal bars, 10M spiral with 50 mm pitch, 40 mm clear cover to spiral) (b) Square (28 MPa concrete, 4-25M longitudinal bars, 40 mm clear cover to ties).

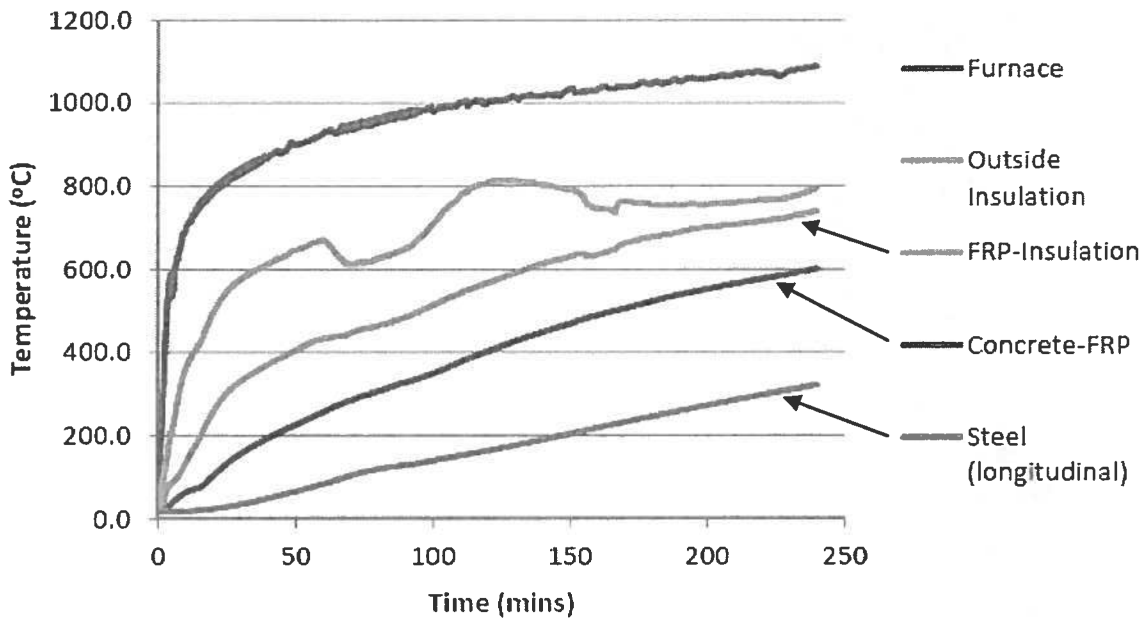
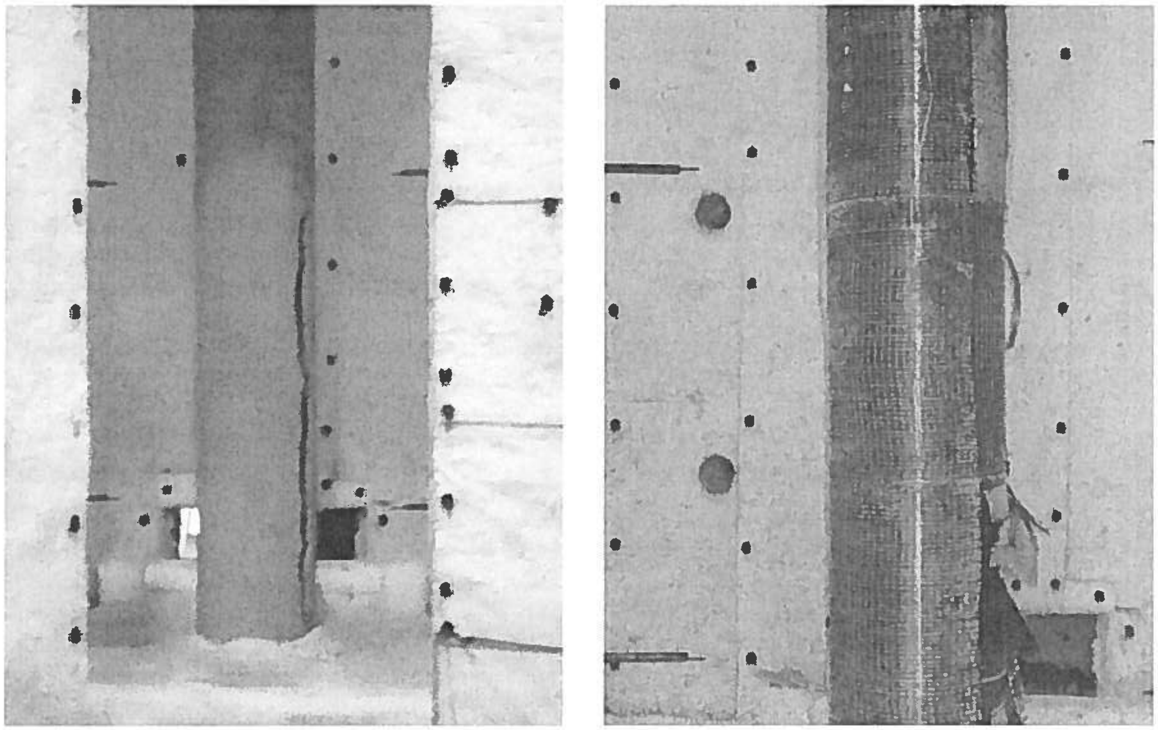



Figure 5. Temperatures measured in the circular column with 15 mm of insulation.

Figure 5 shows the temperatures at various locations in the circular column. With only 15 mm of insulation, the temperature in the FRP increases rapidly but steadily to 150 °C after 30 minutes and to almost 400 °C after 120 minutes. During the testing of this column, a single small vertical crack was first noticed at 57 minutes at the mid-height of the front-right side of the column (with respect to the furnace doors). Flames were also observed coming out from the crack, which initially rose in intensity, but eventually ceased after 140 minutes. As the test progressed, this sole crack elongated and widened. After 184 minutes, the crack was over one metre long, and 3-4 mm wide. After the full 240 minutes, upon opening the furnace doors, this crack opened even more as the insulating materials cooled. Despite this, no insulation fell from the column, unlike in the beam tests.

Figure 6(a) shows a photo of the column at the end of the fire test. The insulation on this column was later knocked off with a long stick in order to inspect the state of the underlying FRP as shown in Figure 6(b). The FRP appeared intact; however the epoxy had presumably burned off by this point, effectively negating its strengthening benefits. The free-ends of the confining wraps were also completely debonded from the layer below, which supported the theory that the epoxy had been consumed in the fire.

Despite the lack of contribution from the FRP at the end of the fire test, both columns were able to carry the applied load for the full four hours of fire testing. Although the insulation did not protect the FRP completely, it reduced temperatures in the reinforcing steel and concrete. For example, the temperature in the reinforcing steel was approximately 350 °C after four hours of the fire exposure. At this temperature, neither the steel nor the concrete would have lost much of their original room temperature strength. As a result, the strength of the column after four hours of fire exposure is expected to be close to the room temperature strength of the unstrengthened column.



(a) (b)

Figure 6. Circular column after fire testing.

## 3 DESIGN IMPLICATIONS

Based on the research conducted over the past ten years, a reasonable understanding of the performance in fire of FRP strengthened concrete structures has been obtained through full-scale testing [2,7,8,9]. To extend this knowledge from the constraints of the specific fire tests, numerical models have been developed for slabs, beams, and columns [14,15,16,17]. These models can predict both the thermal and structural performance of the members, and have been validated against the full-scale tests conducted as part of the research program [14,15,16,17]. The details of these numerical models are presented in other publications [14,15,16,17] and thus are not reproduced in this paper.

The models, however, have demonstrated that the fire behaviour of FRP strengthened concrete members can be predicted with reasonable accuracy with standard structural fire endurance procedures as long as the thermal properties and material performance at high temperature are known. For concrete and steel, both the thermal properties and material properties are characterized with sufficient accuracy [18], and thermal properties of insulation can generally be obtained from manufacturers or from testing [14,15,16,17]. A major impediment to the accuracy of models for strength prediction is the characterization of the strength and bond degradation of FRP at high temperatures.

FRP materials are susceptible to high temperatures mainly because of their polymer resins. The glass or carbon fibres that provide the strength and stiffness of the composite material are relatively unaffected by high temperature up to 500°C [2]. However, the polymer resin that helps transfer load between individual fibres and bonds the FRP to the concrete starts to lose strength and stiffness when the resin reaches its glass transition temperature. For FRP materials applied in concrete strengthening, the glass transition temperature is generally between 60°C to 80 °C [2]. Material testing as part of this overall research program has shown that FRP materials lose approximately half their tensile strength near the glass transition temperature of the resin [13]. If bond is not an issue, FRP materials do not lose any additional strength up to temperatures of 200°C [13]. For bond-critical applications, almost all of the bond strength may be lost at temperatures near the glass transition temperature.

With the knowledge gained from the full-scale tests, material tests, and numerical modelling, the general understanding of FRP strengthened concrete structures is starting to reach the point at which rational fire design procedures can be proposed. An approach suggested by ACI 440 [10] is to estimate the loss in strength in fire of an FRP strengthened concrete member by using procedures similar to those recommended by ACI 216 [18] for reinforced concrete members.

In this paper, a conceptual model for fire design is proposed that follows the philosophy advocated by ACI 440 [10]. Figure 7 provides an illustration of this conceptual model with the vertical axis representing the amount of strengthening and the horizontal axis the required fire resistance. The essential idea behind the conceptual model is that a trade off exists between the amount of FRP strengthening that is provided and the required fire resistance. When little or no fire resistance is required then only flame spread resistance is required, regardless of the amount of FRP strengthening. For example, one column tested as part of the overall research program obtained a fire endurance rating of over three hours without any insulation [19]. At this low level of strengthening and fire endurance requirements, most of the information is available in existing design procedures [10,18] to conduct these calculations for design purposes. The only unknown is that burning of the FRP on the surface of the concrete will increase internal temperatures in the concrete and reinforcing steel. This effect is likely to be minor as long as the surface burning does not cause spalling of the concrete. For high performance concrete structures where spalling may be a concern regardless of the presence of FRP, insulation should likely be provided.

As the required fire resistance increases, even structures with a minor amount of strengthening may require some insulation to protect the structural performance of the original reinforced concrete. This is the intermediate range of fire performance, and many strengthening applications in buildings will fall into this range. For designs in this range, the FRP would not be considered structurally active during the fire, but the effect of the insulation would need to be taken into account to estimate the structural performance of the concrete and steel. Numerical simulation models would need to be used to include the effect of the insulation. Alternatively, such numerical models could be employed to develop design charts to predict internal temperatures in reinforced concrete members with different amounts of insulation. A significant amount of research and development is still required to develop such design charts.

The final range of fire performance occurs when high levels of fire endurance are needed for structures that also require a significant amount of strengthening. In this case, the strength of the FRP will need to be protected during the fire event and thus the insulation requirements are even higher than in the intermediate range. The numerical simulation programs developed as part of this overall research program could be used to estimate fire performance in this range [14,15,16,17]. Nevertheless, many unknowns still exist regarding the material behaviour of FRP and how the strength of the FRP strengthened concrete member will deteriorate in fire conditions. More research is needed in this area.

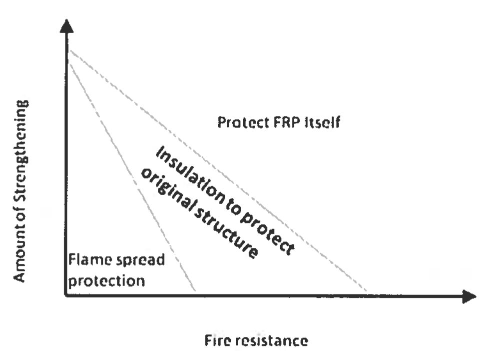
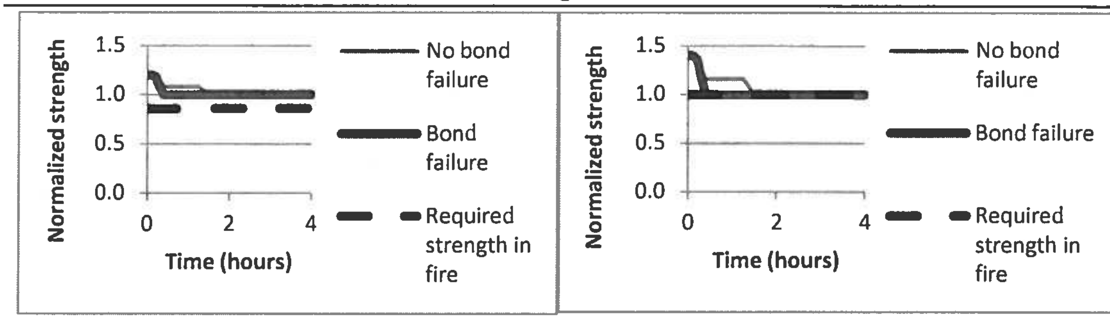
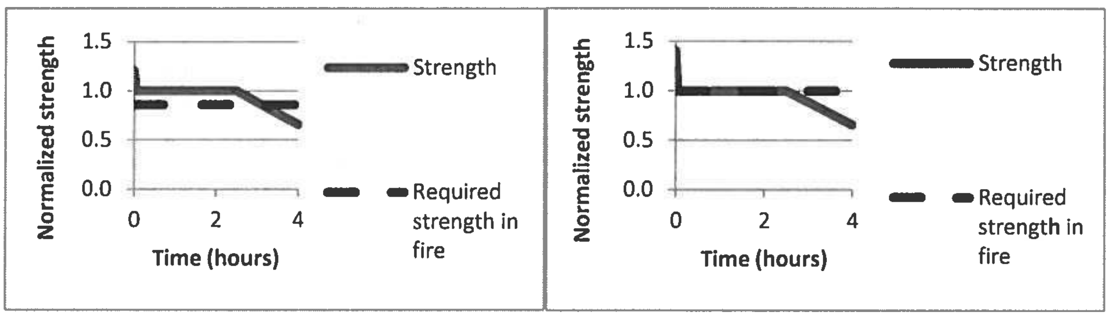



Figure 7. Conceptual fire design model.




(a) (b)

Figure 8. Fire endurance estimates for FRP strengthened slabs with 40 mm clear cover to internal reinforcement **with insulation** (a) 20% strengthening (b) 40% strengthening.




(a) (b)

Figure 9. Fire endurance estimates for FRP strengthened slabs with 40 mm clear cover to internal reinforcement **without insulation** (a) 20% strengthening (b) 40% strengthening.

To provide more guidance for how the conceptual model could be developed, some estimates for fire performance for a 150 mm thick reinforced concrete slab with 40 mm clear cover to the reinforcement and carbonate aggregate are provided in Figures 8 and 9. Such a slab would be expected to have a four hour fire rating before strengthening. For estimating fire endurance, reinforcing steel temperatures were estimated either from ACI 216 [18] or from fire tests on FRP strengthened slabs. Figure 8 shows that providing 20 mm of insulation to the slabs can allow the slabs to have a fire endurance of over four hours. However, once strengthening is increased beyond 40%, the fire endurance will be reduced because the expected loads during the fire will increase to beyond the capacity of the original reinforced concrete slab. Figure 9 demonstrates the fire endurance of the same FRP strengthened slab when insulation is not provided. In this case, three hours of fire endurance is expected for 20% strengthening and only two hours for 40% strengthening. To provide fire endurance for strengthening above 40%, more insulation would be required to enable the FRP to maintain strength for a longer period in the fire.

Based on this example slab, some numbers can be applied to the conceptual model of Figure 7 as shown in Figure 10. In this case, for strengthening percentages of up to 40%, insulation is not required unless the required fire endurance is greater than two hours. To achieve fire endurance greater than two hours, insulation is required to protect the original member (i.e., these slabs fall into the intermediate range of required fire performance). Additionally, if the required fire endurance is very minimal (less than 30 minutes), insulation to protect the original member is sufficient for any practical level of strengthening. For strengthening above 40%, the FRP itself would need to be protected in the case of the fire because the FRP would need to carry a portion of the load for most of the fire exposure.

For design calculations, the results shown in Figure 10 should not be applied at the present time because they are only meant to be illustrative of the type of approach that could be taken with the proposed conceptual model. Many assumptions have been made about FRP material properties and temperature profiles for insulated reinforced concrete beams. More research and development is required to produce more refined charts that could be applied in design practice.

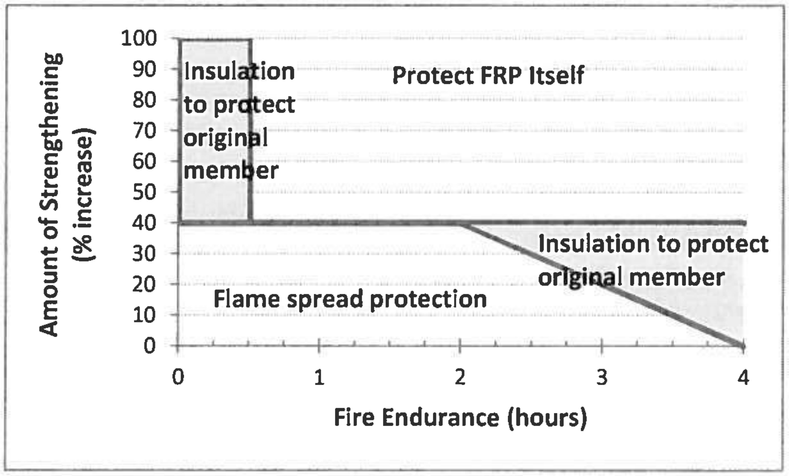



Figure 10. Conceptual design model for FRP strengthened slabs with 40 mm clear cover to internal reinforcement.

## 4 CONCLUSIONS

The results of four full-scale fire tests on reinforced concrete beams and columns strengthened with FRP were presented. All the specimens were insulated with between 15 to 20 mm of insulation. The loads applied on the members in the fire tests were greater than the unstrengthened design capacity of the member. The research demonstrated that insulated FRP strengthened reinforced concrete beams can obtain fire endurance ratings of four hours with approximately half the amount of insulation as previously tested. A conceptual design approach was presented along with numerical computations to demonstrate how it could be applied in fire design. The conceptual design approach was shown to have promise for future design practice. More research is required to develop the approach further. In particular, better information on high temperature material properties, a more complete understanding of the strength deterioration of FRP strengthened members in fires, and design curves for estimating internal temperatures in insulated concrete members are all required to refine the conceptual approach into a practical design tool.

## ACKNOWLEDGEMENTS

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