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FRP Reinforced Concrete Slabs in Fire: A Parametric Analysis

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Applications of FRP reinforcement as the main reinforcing material for reinforced concrete buildings are facing the challenge of adequate fire performance. Steel reinforced concrete members usually perform better compared to FRP reinforced concrete members. In this study the application of GFRP bars in one-way slabs is investigated. Results for fire endurance of FRP reinforced concrete slabs are presented using a thermo-mechanical model. The numerical model is capable of predicting the temperature field inside concrete member at any stage during fire. Requirements of ACI-440.1R are considered to design and examine the validity of the model. This paper offers charts to determine fire endurance of slabs by employing strength-domain failure criteria. In addition, placement of FRP reinforcement in two layers results in an increase in fire endurance compared to corresponding slab with one layer of FRP even if the same amount of reinforcement is implemented in both cases. The amount of fire endurance gained by placing FRP at two layers increases as the thickness of slab increases. Specifically, fire endurance increased by 15, 35, and 45 minutes for slabs with 180, 250, and 300 mm thicknesses respectively.

1 INTRODUCTION

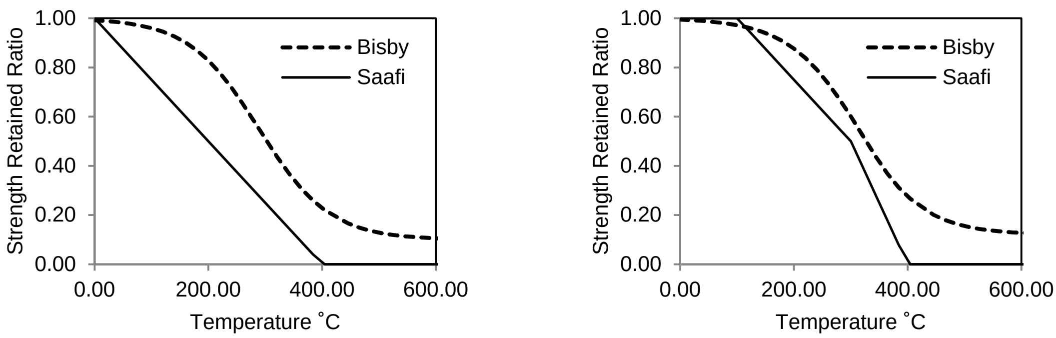
There has been a great deal of interest in the civil engineering community regarding applications of fibre reinforced polymer (FRP) reinforcements in concrete structures as an alternative to steel reinforcement during the past decade. Outstanding characteristics of FRP materials such as high strength-to-weight ratio and resistance to corrosion make FRPs suitable for structures subjected to severe environments. On the other hand, progress in FRP manufacturing technology has reduced the material cost and increased the confidence in FRPs for civil engineering applications.

Among the many areas of application for FRPs, their use for the strengthening of buildings has gained more attention, given their advantages in fast construction. Nevertheless, application of FRPs as internal reinforcement of concrete structural members has been blooming lately. FRP reinforcing bars are now available in different forms for flexural and shear reinforcements. Demand for their use as internal reinforcement in highly-corrosive environments such as bridges, barrier walls, parking lots, buildings in coastal areas and industrial structures has increased. Apart from notable advantages of FRPs, application of FRP in structures has a few drawbacks. One of the drawbacks of FRP materials is their performance in fire. Degradation of strength and stiffness of FRPs induced by high temperatures could cause substantial loss of load carrying capacity in concrete structures, specifically when they are the primary form of reinforcement, while conventional concrete structural members with internal steel reinforcement generally exhibit good performance in fires. The strength loss could be substantial even at mildly increased temperatures, Blontrock (1999). Therefore, a better understanding of the performance of FRP reinforced concrete structures in fire is required. This paper aims to address this research need.

2 MODEL DESCRIPTION

2.1 Material behaviour at high temperatures

The behaviour of concrete at elevated temperatures is well understood (Lie 1992, Buchanan 2001, Purkiss 2006, and Bazant 1996). There are certain difficulties dealing with FRPs, firstly the properties of commercially available materials can vary widely. Additionally, time-dependent visco-elastic behaviour of matrix or adhesive makes experimental characterization difficult. In general, the mechanical properties of FRP degrade due to high temperatures depending mainly on the properties of the matrix. Blontrock et al. (1999) suggested the tensile strength of CFRP and AFRP remains unaffected up to 100 °C but that the tensile strength of GFRP bars decreases consistently with the increase of the temperature. In this paper, two models proposed by Saafi (2002) and Bisby (2003) for degradation of GFRP bars at elevated temperatures are used. Figure 1 compares the strength and elastic modulus degradation for GFRP composites in Saafi and Bisby's models. Saafi's model produces conservative results compared to Bisby's model.



a b

Figure 1. Comparing Bisby's and Saafi's model; a) Strength reduction; b) Elastic modulus degradation for GFRP at elevated temperatures

2.2 Heat conduction simulation in reinforced concrete members

High temperatures cause damage to concrete and FRP, but the major challenge is the simulation of the heat transfer in concrete due to its complicated chemical and structural composition. Portland cement paste may experience various changes such as dehydration, porosity increase, thermal cracking, spalling during heating. Several models have been proposed for modelling hydrothermal-mechanical simulation of concrete, for example see Gawin (1998) and Mounajed (2004). If only the temperature field is required, drastic simplifications can be made, which lead to uncoupled field equations. The model used in this paper solves the uncoupled field equations. The following assumptions are made for the theoretical model:

- Local thermodynamic equilibrium exists and the response time for the local heat transfer between the fluid and the solid is much smaller than the times of interest.
- The energy transferred by mass diffusion is negligible
- Evaporation of chemically and physically bound water is neglected (Di Capua et al. 2007).

The model developed in this paper is a finite-volume (FV) code that is capable of predicting temperature in any concrete section. The partial differential equation of heat conduction can be expressed as

$$\rho c \frac{\partial T}{\partial t} = \nabla.(k\nabla T) = \frac{\partial}{\partial x}\left(k\frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(k\frac{\partial T}{\partial y}\right)$$

Where k is thermal conductivity, ρ is density, c is heat capacity, T is temperature, t is time, and x and y are spatial coordinates. The effect of moisture is taken into account by increasing the heat capacity for moist concrete. After performing a mesh sensitivity analysis, a mesh size of 1 mm was chosen. To achieve stability in the finite volume solution, a time step of 0.2 sec was used. More information on the model verification can be found in Adelzadeh et al. (2012).

Realistic simulation of fire in structures could be very complicated; for this paper, the ASTM E119 time temperature curve has been used to simulate the temperature rise due to compartment fire in the heat transfer model. ASTM E119 temperatures and temperature predictions at different concrete depths are presented in Figure 2.

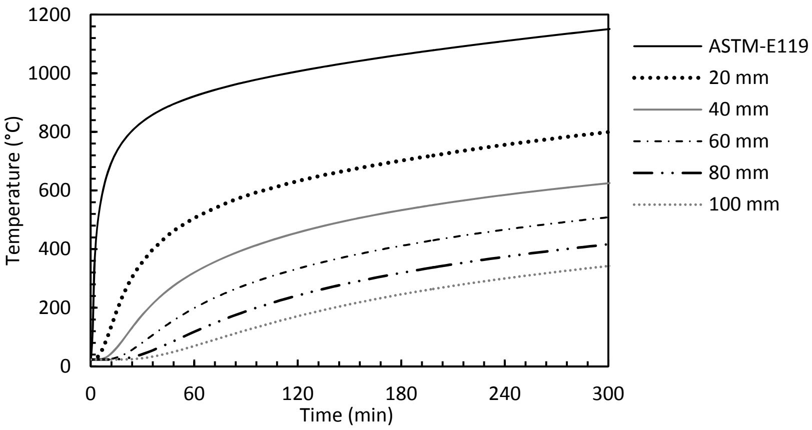


Figure 2 Temperature predictions at different concrete depths vs. exposure time.

2.3 Load capacity model

Once the distribution of temperatures throughout the slab is known at each time step during fire exposure, the flexural capacity of slab can be calculated using Euler-Bernoulli beam theory. The following assumptions were made in the model,

- Slabs are exposed to fire from the bottom of the slab
- Slabs are bending in one direction (one-way slab),
- Slabs are simply-supported and there is no axial restraint and restrictions against expansion
- The bond of FRP bars to concrete is unaffected by heat.
- Plane-sections remain plane throughout the analysis

In order to calculate the flexural capacity of the section during fire, concrete and FRP characteristics have been altered at each step to account for the loss of strength due to fire exposure according to the models presented in Figure 1.

3 SLABS WITH ONE LAYER OF FRP

3.1 Design of FRP reinforced slabs considering serviceability limits of ACI

In order to study the effect of fire on slabs, several different slab configurations have been considered as shown in Table 1. Since serviceability limitations are the governing criteria in FRP reinforced slabs, the slabs are designed for crack width and deflection serviceability criteria. The crack width of the slabs has been limited to 0.7 mm for interior exposure as suggested by ACI-440.1R. The permissible deflection is $L/360$. For the two above defined limits, the required FRP reinforcement ratio is found for the all slabs. Slab thicknesses are 180, 250, and 300 mm and concrete cover ranges from 30 to 70 mm.

For the purposes of illustration, consider a 250 mm thick concrete slab with a 28 day concrete compressive strength of 30 MPa. The aggregate of the concrete is carbonate which affects the thermal behaviour of the slab. If the concrete cover to the centre of the FRP bars is 50 mm, the required reinforcement area assuming M_a/M_{cr} =1.50 is 1719 mm^2. Placing the required amount of reinforcement to satisfy serviceability criteria gives nominal moment resistance 150 kN.m. The cracking moment (M_{cr}) of the slab is 34 kN.m. Exposed to fire from below, the slab loses its moment capacity as a consequence of thermal degradation of the mechanical properties of the FRP. The initial flexural capacity of the slab drops to the applied moment (M_a =51 kN.m) at 140 minutes. It should be mentioned that the resistance model given by the model does not include member reduction factors. Crack width is calculated using the following equation:

$$w_{cr} = 2\frac{f_f}{E_f}\beta k_b\sqrt{{d_c}^2 + (\frac{s}{2})^2}$$

In which w = maximum crack width; f_f = reinforcement stress; β= ratio of distance between neutral axis and tension face to distance between neutral axis and centroid of reinforcement; d_c = thickness of cover from tension face to centre of closest bar; and s = bar spacing,

The crack width is a function of stress in the FRP bars. Therefore, for different levels of service loads or M_a/M_{cr}, different amount of reinforcements should be used. Three common ratios of 1, 1.25, and 1.5 were selected for the M_a/M_{cr}.

Table 1. Characteristics of FRP reinforced slabs investigated in this study

Slab number	Thickness (mm)	Rebar type	f'_c (MPa)	cover (mm)	L(mm)	Spacing (mm)	$A_{f,req}$ *	M_n ** (kN.m)	M_{cr}	Deflection (mm) *Ma/Mcr*=1.5
1	180	GFRP	30	30	3600	150	1006	77.6	17.8	4.1
2	180	GFRP	30	40	3600	150	1243	74.1	17.8	4.1
3	180	GFRP	30	50	3600	150	1576	70.2	17.8	4.1
4	180	GFRP	30	60	3600	150	2051	65.6	17.8	4.0
5	180	GFRP	30	70	3600	150	2754	59.9	17.8	4.0
6	180	GFRP	30	80	3600	150	3845	53.1	17.8	3.9
7	250	GFRP	30	30	5000	150	1235	156.4	34.2	5.7
8	250	GFRP	30	40	5000	150	1446	153.5	34.2	5.7
9	250	GFRP	30	50	5000	150	1719	149.7	34.2	5.6
10	250	GFRP	30	60	5000	150	2072	145.4	34.2	5.6
11	250	GFRP	30	70	5000	150	2530	140.2	34.2	5.5
12	250	GFRP	30	80	5000	150	3136	134.0	34.2	5.4
13	300	GFRP	30	30	6000	150	1409	222.6	49.3	6.9
14	300	GFRP	30	40	6000	150	1617	227.2	49.3	6.8
15	300	GFRP	30	50	6000	150	1879	223.9	49.3	6.7
16	300	GFRP	30	60	6000	150	2206	220.2	49.3	6.7
17	300	GFRP	30	70	6000	150	2614	215.7	49.3	6.6
18	300	GFRP	30	80	6000	150	3122	210.0	49.3	6.5

Note: * Required FRP reinforcement area to meet serviceability criteria

** Nominal moment capacity (No reduction factor)

3.2 Parametric analysis for slabs with one layer of GFRP

3.2.1 Effect of concrete cover

As expected, by increasing the concrete cover the initial moment capacity decreases. Because the defining design criteria relate to serviceability rather than strength, all slabs fulfil the strength requirements at room temperature. Fire performance of slabs considerably increases by increasing their concrete cover as shown in Figure 3, but this performance improvement comes at the expense of higher reinforcement ratio due to crack width limitation. For example, a 180 mm slab with 30 mm cover has approximately 1 hour of fire endurance while the slab with 60 mm concrete cover has in excess of 4 hours of fire endurance. However, the required reinforcement ratio is 2 times higher in the slab with 60 mm concrete cover to meet serviceability design criteria.

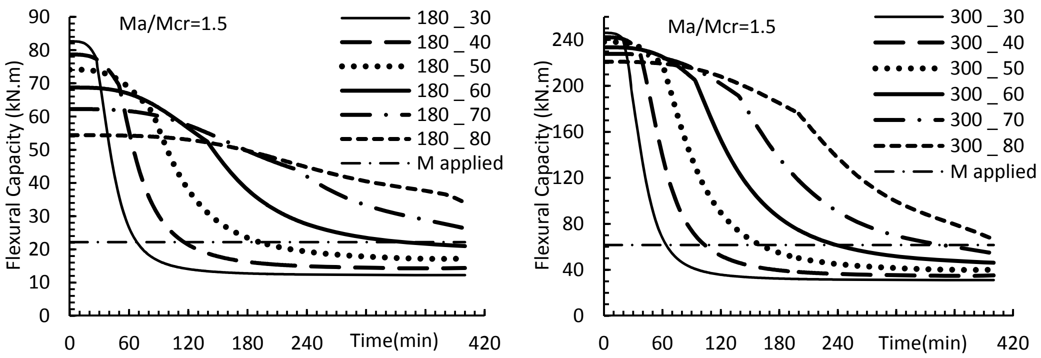


Figure 3. Simulation results for reductions in moment capacities of 180 and 300 mm thick slabs with various cover depths during standard fire exposure , M_a/M_{cr} =1.5

An interesting observation is that the fire endurance of slabs is independent of their level of applied service load. In other words, changing the M_a/M_{cr} ratio during the design process does not significantly affect the failure time of the slab within the range of M_a/M_{cr} between 1.0 and 1.5. This effect is illustrated in Figure 4 where moment resistance curves are normalized versus applied load or service load (M_a). While slabs with different M_a/M_{cr} ratios behave differently in the beginning, they approach each other when the moment capacity reaches the service load level. So for example a slab with initial M_a/M_{cr} = 1 has approximately the same fire endurance as a slab with M_a/M_{cr} =1.5. Obviously in a slab with M_a/M_{cr} = 1.5 the amount of reinforcement is higher due to higher crack width but fire endurance is independent of service load level.

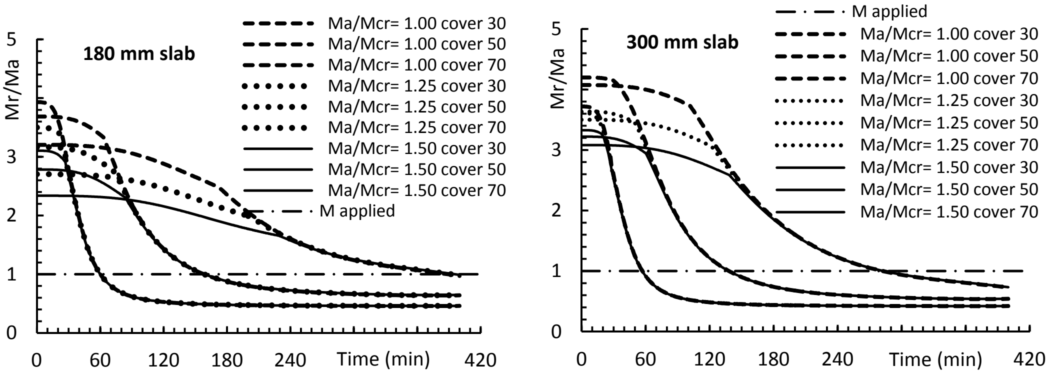


Figure 4. Reductions in flexural capacities of a 180 mm thick slab exposed to standard ASTM E119 fire. Moment capacities are normalised versus service load M_a. M_a/M_{cr} =1.0, 1.25, 1.5.

4 SLABS WITH TWO LAYERS OF FRP

Simulation results for slabs with one layer of FRP shows that in order to fulfill the requirements of serviceability in FRP reinforced slabs, the amount of FRP is considerably larger than the amount needed considering the strength requirements. Since the serviceability requirements during an extreme fire load do not need to be fulfilled, specifically crack width criterion, the abundant FRP could be employed effectively to increase the fire endurance. This could be done by placing the FRP reinforcement in two or more layers. During fire, a slab with two layers of FRP will perform better because the inner layer has more protective cover.

In order to further investigate the effectiveness of the approach placing FRP in two layers, simulations has been performed on slabs with two FRP layers and their behaviour has been compared with slabs with one layer of FRP with the same amount FRP reinforcement.

Slab thicknesses used in the simulations were 180, 250, and 300 mm. The half of reinforcement was placed in one layer and the remaining half in the other layer. The covers chosen for the bottom FRP layers were 30, 40, 50, and 60 mm and the distances between FRP layers were 30, 40, and 50 mm.

The area of FRP reinforcement was determined according to ACI-440.1R serviceability criteria, similar to the design procedure used for slabs with one layer of FRP. The relations for calculating crack width and deflection have been modified for slabs with two layers of FRP. For design, FRP in each layer was considered separately rather than as a single bundled FRP layer. Crack width limit was set to be equal to 0.7 mm similar to slab with one layer of FRP. M_a/M_{cr} in all simulations was equal to 1.5 since fire endurance was found to be independent of the service load level.

4.1 Results for slabs with two layers of FRP

Sample moment capacity curves during fire are shown in Figure 5. As expected, a slab with two layers of GFRP reinforcement outperforms a slab with same amount of reinforcement placed in one layer in terms of fire endurance. For example, a slab with GFRP placed at two layers with covers of 30 and 60 mm achieves approximately 2 hours of fire endurance while in the case of a slab with one layer of GFRP and the same reinforcement ratio the fire endurance is significantly shorter. Additionally, the decline in the moment capacity of a slab with two layers of reinforcement is gentler compared to the corresponding one-layer FRP slab. While the initial strength of the slab is higher for the one-layer slab, the decline in strength is faster during the fire exposure. Thus, the slab with two layers of reinforcement is expected to fail more gradually during fire exposure.

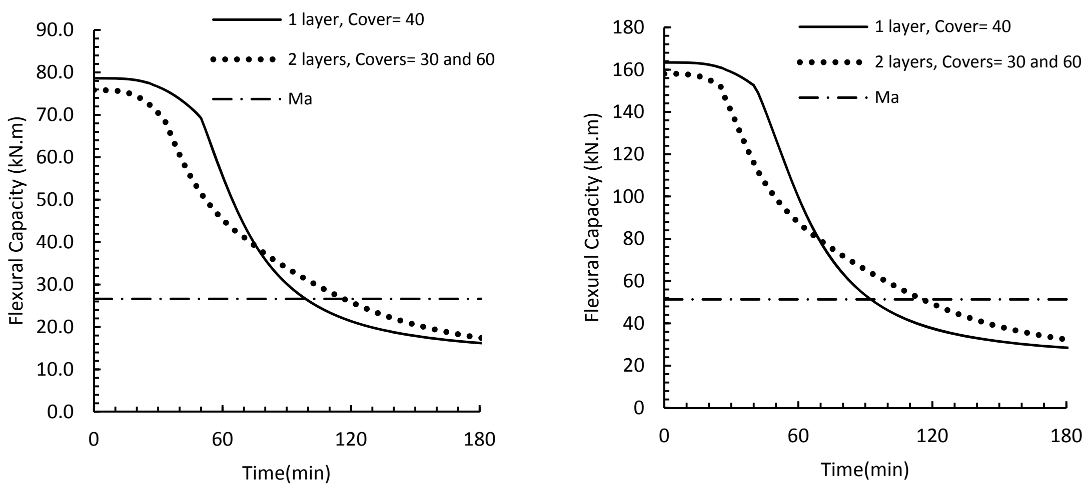


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Figure 5. Prediction of the flexural capacity in fire of a slab with two layers; a) Slab thickness of 180 mm; b) Slab thickness of 180 mm;

The obtained increase in fire endurance by placing reinforcements in two layers varies by slab thickness. For example, in a slab with 180 mm thickness the fire endurance gain was approximately 15 minutes on average, which is not significant considering the amount of effort needed in placing the FRP in two layers. On the other hand, for a slab with 250 mm thickness the average gain is 35 min and for a 300 mm slab it is 45 min. Based on these observations, placing FRP in two layers is more effective in terms of fire endurance for thicker slabs.

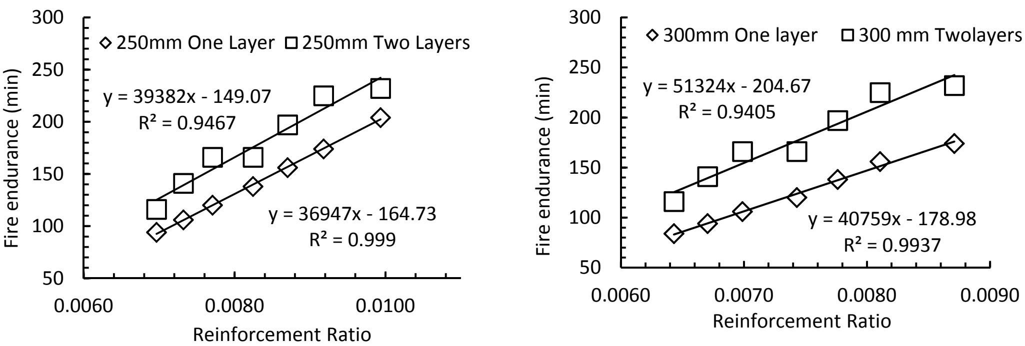

Figure 6. Fire endurance of 2 layer FRP reinforced slabs compared with slabs with 1 layer (Slab thickness= 250 and 300 mm)

Figure 6 shows fire endurance results for two layer slabs and corresponding results for slabs with one layer for slabs with 250 and 300 mm thicknesses plotted against the reinforcement ratio. Therefore two points on a vertical line have the same amount of GFRP reinforcement and their vertical separation is the amount of increased fire endurance in minutes. There is a strong linear relation between reinforcement ratio and fire endurance in one layer slabs. The same linear dependency is generally present for two layer slabs. The fluctuations in two layer data are because of a sudden change in the distance between two layers. For example the distance between two layers in the first three points from left in both figures is 30mm and for the next three points it is 40 mm.

5 CONCLUSIONS

A model to estimate the fire endurance of FRP reinforced concrete slabs has been developed and applied to consider different configurations of reinforcing that may enhance fire resistance. In particular, the fire endurance of slabs with reinforcement placed in two layers was compared to that of slabs with reinforcement in one layer. The two designs had fairly similar behaviour and flexural capacities at room temperature, but slabs with reinforcement placed in two layers were found to have improved fire resistance compared to similar slabs with reinforcement in a single layer.

6 ACKNOWLEDGEMENTS

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