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ORIGINAL RESEARCH

Thermal analysis of GFRP-reinforced continuous concrete decks subjected to top fire

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Abstract This paper presents a numerical study that investigates the behavior of continuous concrete decks doubly reinforced with top and bottom glass fiber reinforced polymer (GFRP) bars subjected to top surface fire. A finite element (FE) model is developed and a detailed transient thermal analysis is performed on a continuous concrete bridge deck under the effect of various fire curves. A parametric study is performed to examine the top cover thickness and the critical fire exposure curve needed to fully degrade the top GFRP bars while achieving certain fire ratings for the deck considered. Accordingly, design tables are prepared for each fire curve to guide the engineer to properly size the top concrete cover and maintain the temperature in the GFRP bars below critical design values in order to control the full top GFRP degradation. It is notable to indicate that degradation of top GFRP bars do not pose a collapse hazard but rather a serviceability concern since cracks in the negative moment region widen resulting in simply supported spans.

Keywords Finite elements · Thermal-stress analysis · GFRP bars · Fire simulation curves · Concrete cover · Design tables

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Introduction

GFRP bars have been considered and used worldwide as negative and positive reinforcement in concrete decks, such as bridge decks as shown in Fig. 1, especially in cold regions to resist the corrosion imposed on conventional steel bars by the frequent use of high dosage of deicing salts leading to fast deterioration of bridge decks due to the ingress of chlorides (Koch and Karst 2013).

The problem of studying fire resiliency in reinforced concrete (RC) beams strengthened with CFRP composites and subjected to bottom and top fire has been addressed numerically by the authors and co-workers (Hawileh et al. 2009, 2011; Naser et al. 2014, 2015). These numerical studies were benchmarked against several experimental investigations related to the same subject (Blontrock et al. 1999; Williams et al. 2006; Tan and Zhou 2011). This problem was extended to investigate the high-temperature effects in bridge decks externally strengthened with FRP when subjected to accidental events like top fire or maintenance activities like surfacing with bituminous paving materials (Del Prete et al. 2015).

The behavior of concrete beams reinforced with GFRP bars under fire is experimentally investigated by Abbasi and Hogg (2006). The first author and a co-worker (Hawileh and Naser 2012) developed a three-dimensional finite element (FE) model that predicted with a good level of accuracy the fire resistance of a beam reinforced with glass (GFRP) bars that was tested by Abbasi and Hogg (2006). The validated model was utilized in a design oriented parametric study to examine the effect of the bottom concrete cover thickness and different fire scenarios on the fire resistance of beams reinforced in flexure with GFRP bars. Yu and Kodur (2013) studied the same problem numerically to identify factors affecting the fire response of

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Fig. 1 GFRP *bars* used for negative and positive reinforcement in bridge decks, Courtesy of Hughes Brothers

concrete beams reinforced with FRP bars. The two papers reported that the rebar type, concrete cover thickness, and the fire scenario are the key parameters affecting fire endurance.

The behavior of concrete slabs reinforced with FRP bars or grids, in concrete buildings subjected to fire loading, is addressed experimentally and numerically in a two-part paper (Nigro et al. 2011a, b). Shortly after that, the same research group published a paper proposing guidelines for the flexural resistance of concrete beams and slabs subjected to fire in accordance to Eurocode2 (2004) guidelines.

In this study, a numerical investigation is performed to examine continuous concrete decks doubly reinforced with top and bottom glass fiber reinforced polymer (GFRP) bars subjected to top surface fire. A detailed transient thermal FE analysis is carried out under the effect of various fire curves. A parametric study is performed to examine the top cover thickness and the critical fire exposure curve needed to fully degrade the top GFRP bars while achieving certain fire ratings for the bridge deck considered.

Approach of numerical analysis

In order to perform transient thermal FE analysis on a continuous concrete deck subjected to top surface fire loading, a three-dimensional FE model for a segment of a typical concrete bridge deck without surfacing materials (like bituminous) taken at an interior support is modeled and analyzed as follows:

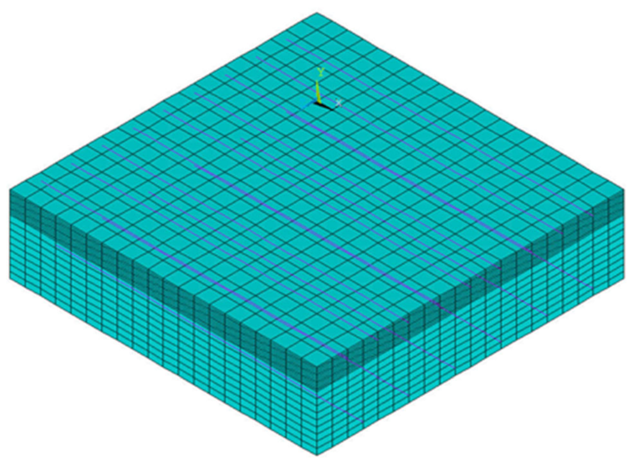
1. Develop a FE model of the concrete deck reinforced with top and bottom GFRP bars using thermal brick and link elements for the concrete and GFRP bars, respectively.
2. Vary the input data for the thermal material properties of the concrete deck as a function of temperature according to Eurocode2 (2004) guidelines.
3. Apply a temperature versus time curve to the top surface of the concrete deck and perform transient thermal analysis to simulate the heat transfer throughout the deck by conduction, convection, and radiation due to the applied fire curve scenario.
4. The needed output of the thermal analysis is the progression of temperature along the top GFRP bars for the entire fire exposure.

It should be noted that the authors analyzed one way slab for FE modeling in this study. However, the progressions of temperature in the top GFRP bars are also applicable to two-way slabs. The top concrete cover is measured from the top concrete surface to the center of the top GFRP bars layer.

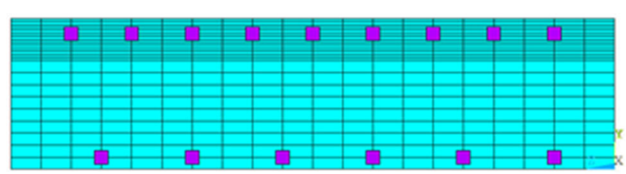
FE model description

Geometry and material properties

Figure 2 shows the developed FE model for a typical segment of a bridge concrete deck at an interior support, having a width and thickness of 1000 and 250 mm, respectively. The model is created and analyzed using the finite element software ANSYS-Release Version (2013).



(a) Isometric view



(b) Cross-section

Fig. 2 Developed FE model of a concrete deck

The deck is reinforced with top and bottom GFRP bars. The negative moment at the interior support is resisted by 9 #25 (Area = 510 mm^2 per bar, total area = 4590 mm^2) diameter GFRP bars located with a typical top cover thickness of 25 mm as shown in Fig. 2. It should be noted that the top cover thickness measured from the top concrete surface to the center of the top GFRP bars is one of the parameters that will be varied in this study to examine its effect on the fire resistance of bridge decks when subjected to top fire loading. The deck is also reinforced with 6 #25 (Area = 510 mm^2 per bar, total area = 3060 mm^2) diameter GFRP bars with a bottom concrete cover of 25 mm. The center-to center spacing between the bars located at the top and bottom of the deck is 100 and 150 mm, respectively.

The brick SOLID70 and spar LINK33 thermal elements (ANSYS-Release Version 2013) are used to model the concrete and GFRP bars, respectively. The thermal brick SOLID70 element has a total of eight nodes with a temperature degree of freedom (dof) at each node (ANSYS-Release Version 2013). The thermal spar LINK33 element (ANSYS-Release Version 2013) is a three-dimensional uniaxial element defined by two nodes and has a temperature dof per node. Both elements have the capability of transferring heat in a transient thermal analysis throughout the deck due to applied fire that will be initiated in this study at the top surface of the concrete deck.

The temperature-dependent thermal conductivity, specific heat, and density are required input material properties to perform transient thermal analysis. The temperature-dependent input thermal properties of the concrete material in the developed FE model are based on Eurocode2 (2004) guidelines. The concrete slab is conservatively assumed to be made of siliceous aggregates and cast with a moisture content of 3%, by weight. Figures 3, 4, and 5 show the variation of the thermal conductivity, specific heat, and density with temperature for the concrete deck. The thermal conductivity, specific heat, and density of the

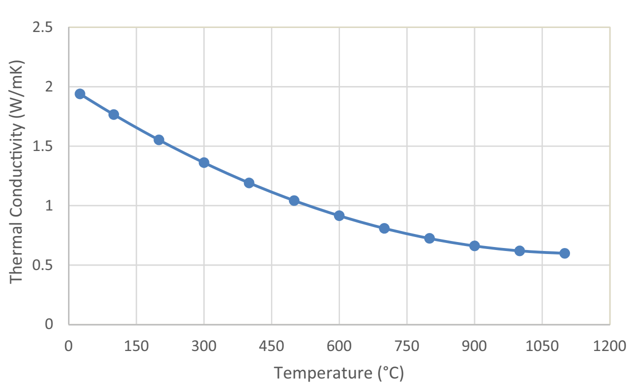


Fig. 3 Thermal conductivity of concrete as a function of temperature (Eurocode2 2004)

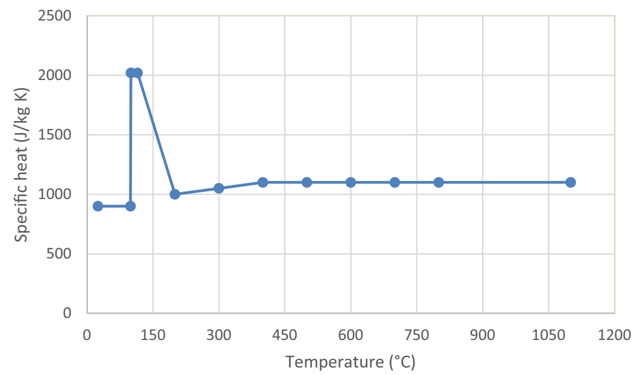


Fig. 4 Specific heat of concrete as a function of temperature (Eurocode2 2004)

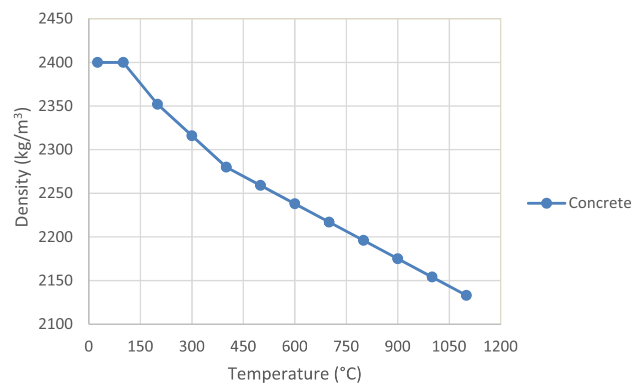


Fig. 5 Density of concrete as a function of temperature (Eurocode2 2004)

GFRP bars were taken at room temperature as 4.0×10^{-5} W/mm K, 1310 J/kg K, and 1600 kg/m^3, respectively (Hawileh and Naser 2012).

Top fire scenarios

The top surface of the concrete deck will be subjected to different fire exposure scenarios in the form of temperature versus time curves with a convective heat transfer coefficient of 20 W/m^{2}K. The model also accounts for heat transfer by radiation using a Stefan–Boltzman radiation coefficient and concrete emissivity constants of 5.669×10^{-8} W/m^{2}K^4 and 0.7, respectively (Del Prete et al. 2015). A transient thermal analysis is performed for every fire exposure with several time load steps and substeps. The temperature distribution throughout the deck and progression of temperature throughout the GFRP bars are the main output of the thermal analysis. The output of the transient thermal analyses includes temperature distribution throughout the slab specimen for the entire fire exposure.

Figure 6 shows the heat loading fire scenarios which include the standard building fire curve ASTM Test

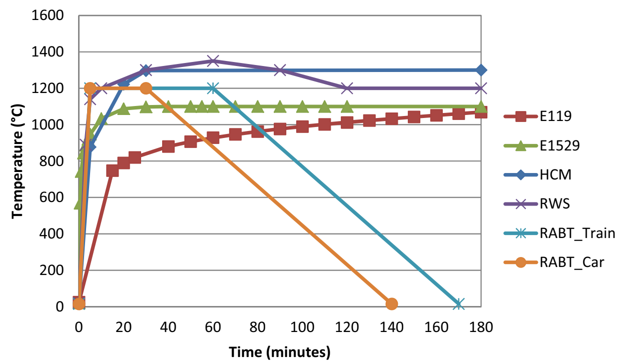


Fig. 6 Applied top surface fire scenarios

Method E119 (2002) which is quite similar to ISO834 (1975) fire curve, ASTM E1529 (1993) hydrocarbon fire curve, hydrocarbon modified curve (HMC), RWS fire curve that is usually used to simulate possible fire in tunnels, RABT_Train and RABT_Car fire curves (2008).

The hydrocarbon ASTM E1529 (1993) and HCM fires (2008) represent possible fire scenarios from petrochemicals such as fire from car fuel tanks, gasoline and oil tankers. It should be noted from Fig. 6 that HCM fire curve is more severe than that of the ASTM E1529 where the temperature can reach 1300 °C instead of 1100 °C for the ASTM E1529 fire curve. The road tunnel RWS fire curve (Fehérvári 2008) was developed by the Ministry of Transport in the Netherlands and represents a fire lasting up to 120 min of a 50-m^3 fuel, oil or petrol tanker with a fire load of 300 MW. The RABT fire curves (train and car) shown in Fig. 6 were developed in Germany (2008) with shorter fire exposure compared to other scenarios and a very rapid temperature rise up to 1200 °C within 5 min. As shown in Fig. 6, the temperature drop for RABT_Train and RABT_Car started to occur after 60 and 30 min of fire exposure, respectively.

Fire resistance

In this study, the fire resistance of the concrete deck due to the applied top fire exposure is assumed to occur when the temperature of the GFRP bars reaches a specified critical temperature (fire rating). The ACI 440.1R-15 (2015) guidelines did not specify a critical temperature for GFRP bars since there is a lot of debate about it that still warrants further research investigations. The reported critical temperature ranges from 65 °C to about 350 °C (ACI 440 1R 2015). In this study, the time to failure (fire resistance) for the entire range (65–350 °C) will be reported for the investigated fire case scenarios shown in Fig. 6.

Results and discussions

Validation model

As mentioned earlier, the first author and a co-worker (Hawileh and Naser 2012) developed a FE model that was capable of predicting the fire resistance of simply supported concrete beams reinforced in flexure with GFRP bars and subjected to ISO834 bottom fire exposure. The numerical results were in close agreement with that of the experimental results conducted by Abbasi and Hogg (2006). A comparison between the predicted and measured progression of temperature in the GFRP bars is shown in Fig. 7.

It clearly indicated from Fig. 7 that there is a close agreement between the measured and predicted temperature at all stages of fire loading. In addition, the authors (Hawileh and Naser 2012) predicted with a high level of accuracy the mid-span deflection response results for the entire fire exposure. It should be also noted that the authors developed in a previous study (Hawileh et al. 2009) a FE model that simulated the thermal and mechanical response of reinforced concrete beams externally strengthened with CFRP laminates subjected to fire loading. The predicted and recorded temperatures at different depths of the beam's cross-section were in close agreement (Hawileh et al. 2009). Thus, the developed FE model in this study can predict with a good level of accuracy the temperature distribution in the RC slab during the entire fire exposure.

It should be noted that the literature is lacking data on the fire resistance of continuous concrete bridge decks when subjected to top fire loading. Thus, it could be confidently extrapolated that the developed FE model can predict with a reasonable level of accuracy the fire

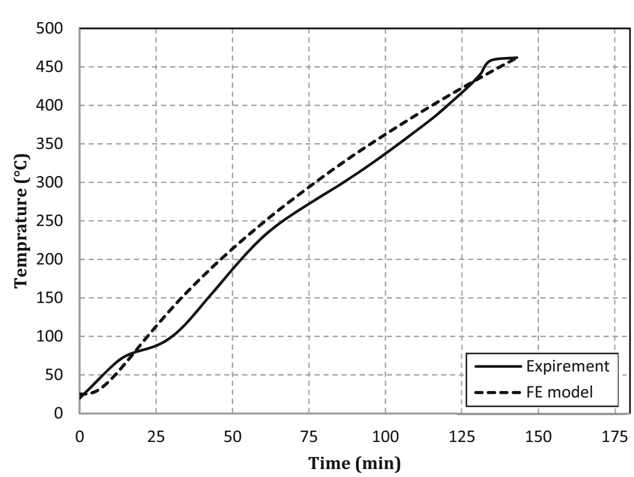


Fig. 7 Measured and predicted temperature in the GFRP *bars* (Hawileh and Naser 2012)

resistance of concrete bridge decks when subjected to top fire loading.

Fire resistance of concrete deck under top fire loading

A total of 60 cases are analyzed to examine the effect of top concrete cover on the fire resistance of concrete decks reinforced with GFRP bars and subjected to the six different fire curve scenarios discussed in the preceding section. The fire loading in the form of temperature versus time curves is applied to the top surface of the concrete deck. The top concrete cover is varied from 25 to 70 mm with an increment increase of 5 mm. This will examine the effect of top concrete cover thickness and different possible fire scenarios on the fire resistance of concrete bridge decks when subjected to top fire loading.

Figures 8, 9, 10, 11, 12, and 13 show the progression of temperature in the top GFRP bars due to the applied ASTM E119, ASTM E1529, HMC, RWS, RABT_Train, and RABT_Car fire scenarios respectively.

Figures 8, 9, 10, 11, 12, and 13 display the results for top concrete cover of 25, 30, 35, 40, 45, 50, 55, 60, 65, and 70 mm, respectively. As expected, it is clearly indicated in Figs. 8, 9, 10, 11, 12, and 13 that as the top concrete cover thickness increases, the temperature in the GFRP bars decreases which would thus lead to an increase in the time for the GFRP bars to reach their critical specified temperature limit. In addition, the plotted results in Figs. 8, 9, 10, 11, 12, and 13 indicate that the ultimate temperature attained in the top GFRP bars is higher for the modified hydrocarbon HMC fire curve than that for the other five studied fire exposures. It should be noted that the increase of temperature in the GFRP bars would lead to a reduction in the elastic modulus and tensile strength of the top GFRP bars. However, the degradation in the mechanical properties of the top GFRP bars do not cause a collapse of the concrete deck but rather a serviceability concern since cracks in the negative moment region of continuous spans widen resulting effectively in adjacent simply supported spans. It should be also noted from Figs. 12 and 13 that there is a recovery (reduction) in the progression of

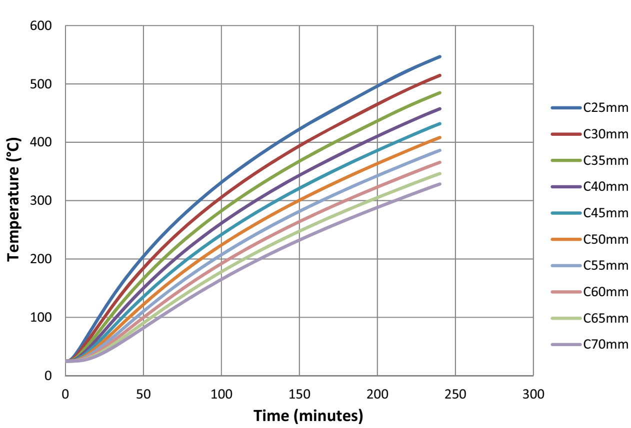


Fig. 8 Progression of temperature in the top GFRP *bars* due to ASTM E119 fire exposure

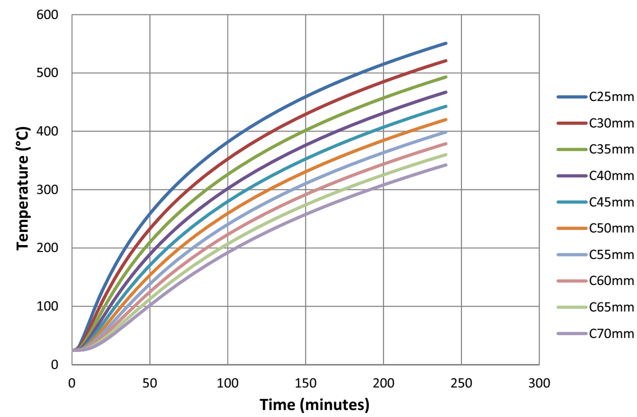


Fig. 9 Progression of temperature in the top GFRP *bars* due to ASTM E1529 fire exposure

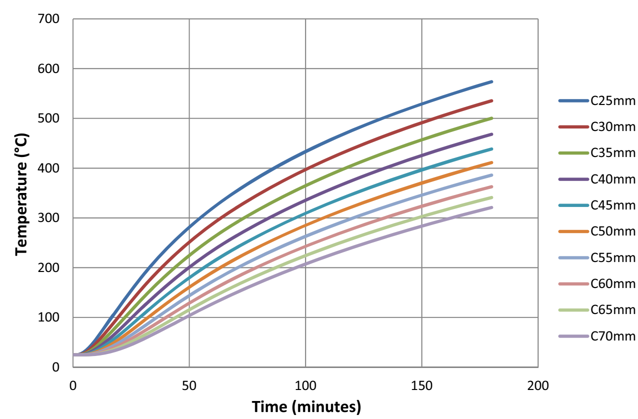


Fig. 10 Progression of temperature in the top GFRP *bars* due to HCM fire exposure

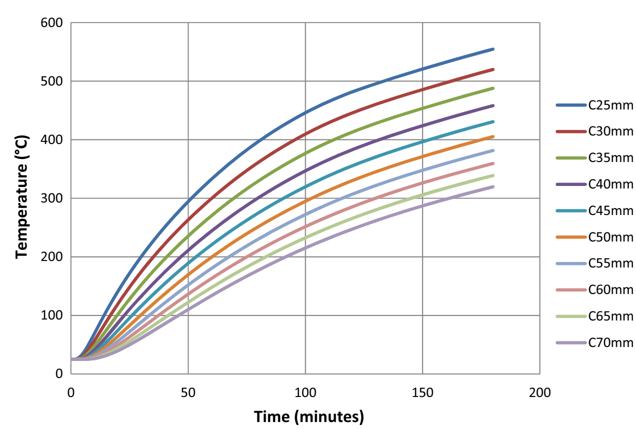


Fig. 11 Progression of temperature in the top GFRP *bars* due to RWS fire exposure

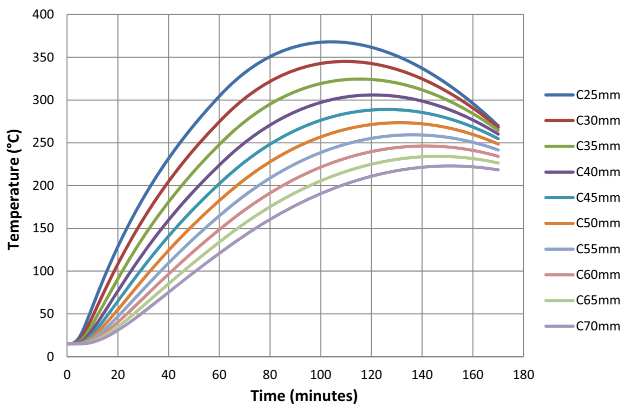


Fig. 12 Progression of temperature in the top GFRP *bars* due to RABT_Train fire exposure

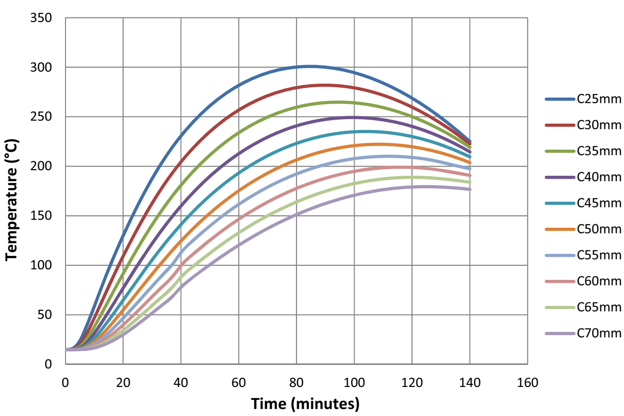


Fig. 13 Progression of temperature in the top GFRP *bars* due to RABT_Car fire exposure

temperature in the top GFRP bars for the two slabs subjected to RABT_Train and RABT_Car fire exposure, respectively. This recovery is caused by the presence of the cooling phase in the applied RABT_Train and RABT_Car fire curves as shown in Fig. 6.

Tables 1, 2, 3, 4, 5, and 6 provide the results for the fire resistance of concrete decks when subjected to the six different top fire loading scenarios. The fire resistance is defined in this study as the time for the top GFRP bars to reach a critical specified temperature, which in turn referred to fire rating of the concrete deck. Since the critical temperature limit in the GFRP bars is still debatable and ranges between 65 and 350 °C, the time to failure is reported in Tables 1, 2, 3, 4, 5, and 6 for the critical temperature values of GFRP bars of 65, 80, 100, 125, 150, 180, 200, 250, 300, and 350 °C, respectively. For example, the fire resistance for a concrete deck with a 25-mm top cover and subjected to ASTM E119, ASTM E1529, HMC, RWS, RABT_Train, and RABT_Car top fire exposures is 49, 35, 33, 30, 33, and 33 min, respectively, if the critical specified temperature in the top GFRP bars is 200 °C. Similarly, if the designer is aiming to achieve a fire rating of 90 min for the concrete deck with the same type of GFRP reinforcement, a minimum top cover thickness of 55, 65, 70, 70, 65, and 60 mm is required for the ASTM E119, ASTM E1529, HMC, RWS, RABT_Train, and RABT_Car top fire exposures, respectively.

Tables 1, 2, 3, 4, 5, and 6 could be used as design tables to guide engineers to properly size the top concrete cover and maintain the temperature in the GFRP bars below critical design specified values when subjected to six different possible top fire loading scenarios. Thus, properly sizing the top concrete cover would control the full degradation of the top GFRP bars during fire exposure.

Summary and conclusions

A 3D FE model was developed to conduct a detailed transient thermal analysis of continuous concrete bridge decks doubly reinforced with GFRP bars subjected to top

Table 1 Fire resistance (in minutes) of top GFRP bars when subjected to ASTM E119 fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	13[a]	17	22	28	34	43	49	66	86	110
30	16	20	25	32	39	49	55	75	99	124
35	18	23	29	36	44	55	63	84	110	139
40	21	26	33	41	50	62	70	94	122	155
45	24	30	37	46	56	69	78	105	136	171
50	27	33	41	51	62	77	87	116	150	189
55	30	37	45	57	69	85	96	127	165	206
60	33	41	50	63	76	93	105	140	180	225
65	37	45	55	68	83	102	115	152	195	>240
70	40	49	60	75	90	110	125	164	211	>240

[a] Fire endurance

Table 2 Fire resistance (in minutes) of top GFRP bars when subjected to ASTM E1529 fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	10[a]	13[a]	15[a]	19	24	30	35	47	65	84
30	12	15	18	23	28	35	40	56	75	99
35	14	18	21	26	32	41	46	65	87	114
40	16	20	24	31	38	48	54	74	99	131
45	18	23	28	35	43	54	61	84	114	149
50	21	26	32	40	49	61	69	95	126	167
55	23	29	36	44	55	68	78	106	141	186
60	26	32	40	49	61	76	87	119	158	206
65	29	36	44	55	68	84	96	130	174	228
70	32	40	49	61	75	92	105	144	192	>240

[a] Fire endurance

Table 3 Fire resistance (in minutes) of top GFRP bars when subjected to HCM fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	11[a]	14[a]	17	20	24	30	33	43	55	69
30	13[a]	16	19	24	28	34	38	49	65	81
35	15[a]	18	22	27	32	39	44	57	75	94
40	17	21	25	31	36	44	50	65	84	108
45	20	24	28	35	42	50	56	75	96	123
50	22	27	32	39	47	56	63	84	108	138
55	25	30	36	44	52	63	72	94	120	153
60	28	33	40	49	58	70	79	105	135	171
65	31	36	44	54	65	77	87	115	149	>180
70	34	40	48	59	70	86	96	126	162	>180

[a] Fire endurance

Table 4 Fire resistance (in minutes) of top GFRP bars when subjected to RWS fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	9[a]	11[a]	14[a]	18	21	26	30	40	52	62
30	11[a]	14[a]	17	21	25	31	35	47	60	76
35	13[a]	16	20	25	30	36	41	54	70	80
40	15[a]	19	23	28	34	42	47	62	80	102
45	17	21	26	32	39	47	53	70	91	117
50	20	24	30	37	44	54	60	79	102	135
55	23	28	34	41	49	60	67	89	116	153
60	26	31	37	46	55	67	75	99	130	171
65	29	34	42	51	61	74	83	110	146	>180
70	31	38	46	57	67	82	92	122	162	>180

[a] Fire endurance

fire loading. The model was benchmarked first against experimental results for concrete beams reinforced in flexure with GFRP bars subjected to bottom fire loading to examine the reliability of its results. After that, a parametric study was conducted to develop design tables that provide fire rating for the concrete bridge decks based on various fire curves considered and different concrete cover thickness values explored. This way, the

Table 5 Fire resistance (in minutes) of top GFRP bars when subjected to RABT_Train fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	11[a]	13[a]	16	19	24	29	33	45	58	80
30	13[a]	15[a]	19	23	28	34	39	52	68	>120
35	15[a]	18	22	27	32	40	45	61	83	>120
40	17	21	25	31	36	45	52	70	105	>120
45	20	24	28	35	43	52	60	80	>120	>120
50	23	27	33	40	49	59	67	94	>120	>120
55	26	30	37	45	55	66	76	111	>120	>120
60	28	34	41	50	61	75	85	>120	>120	>120
65	31	38	46	56	67	83	95	>120	>120	>120
70	35	42	51	62	74	93	109	>120	>120	>120

[a] Fire endurance

Table 6 Fire resistance (in minutes) of top GFRP bars when subjected to fire

Top cover	Critical design temperature of top GFRP bars (°C)									
(mm)	65	80	100	125	150	180	200	250	300	350
25	11[a]	13[a]	15[a]	19	23	28	33	46	79	>120
30	13[a]	15[a]	18[a]	23	27	34	40	57	>120	>120
35	15[a]	18	21	26	32	40	46	72	>120	>120
40	17	20	25	31	38	46	55	>120	>120	>120
45	20	24	29	35	43	54	63	>120	>120	>120
50	23	27	32	40	49	63	75	>120	>120	>120
55	26	30	37	45	55	72	88	>120	>120	>120
60	29	33	40	51	62	82	>120	>120	>120	>120
65	32	37	45	56	70	96	>120	>120	>120	>120
70	35	41	50	63	79	>120	>120	>120	>120	>120

[a] Fire endurance

designer can select the concrete cover thickness that gives a certain deck fire rating (certain time to failure of GFRP bars) when subjected to a specific standard fire scenario. It is important to note that the degradation in the mechanical properties of the top GFRP bars does not pose a collapse threat of the concrete deck but rather a serviceability concern since cracks in the negative moment region of continuous spans widen resulting effectively in adjacent simply supported spans that result in top surface cracks and significantly increased deflections. Based on the results of this study, the following observations and conclusions were drawn:

- The top concrete cover thickness is the most important parameter that influences the fire resistance of concrete slabs when exposed to top fire loading.
- As the concrete cover thickness increases, the temperature in the GFRP bars decreases. Thus, the fire ratings of the slab will increase.
- A minimum top concrete cover thickness of 55, 65, 70, 70, 65, and 60 mm is required to achieve a fire resistance of 90 min for the ASTM E119, ASTM E1529, HMC, RWS, RABT_Train, and RABT_Car top fire exposures, respectively.
- The most severe fire exposure scenario is the modified hydrocarbon (HMC) fire curve. Thus, the top concrete cover thickness of concrete bridge decks should be designed to achieve a specified fire resistance (for example 90 min) when exposed to the HMC fire curve.
- A nominal concrete cover thickness of 70 mm is sufficient to preserve the GFRP bars when subjected to severe fire scenarios.

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