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Reply to Discussion of "The impact of fire on seismic resistance of fibre reinforced polymer strengthened concrete structural systems"*

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Introduction

First, we would like to thank the authors of the Discussion for their interest in our paper. We think the points that they raised are important and allow us to clarify some of the ideas in the original paper.

Modelling of Loading Cycles

The first point raised in the Discussion was that only one lateral loading cycle was considered. Only one cycle was predicted by the model because the column essentially failed on the first loading cycle. As shown in Fig. 2 of the original paper, the second loading cycle was much smaller than the first. The concrete in the column failed in compression during the first loading cycle, and thus the second loading cycle basically represented the residual capacity of the reinforcing steel and the concrete core. Fig. 3 of the paper showed a photo of the failure mode indicating that the cover concrete had completely crushed and spalled off. The model was developed to predict the performance of the reinforced concrete column up to its ultimate capacity. Once concrete crushing was simulated in the model, no further predictions were able to be made with the model.

Although numerical models to predict post-failure behaviour could be developed, such modelling was beyond the scope of the paper. Also, the axial–shear–flexural interaction (ASFI) method used in the paper did not have the ability to consider cyclic hysteresis loops, but other approaches could be developed. Once again, such modelling was beyond the scope of the paper.

Material Properties at High Temperature

The other main comment in the Discussion related to the material properties at high temperature. For the 400×400 mm column 5 that was wrapped with FRP and protected with insulation, the temperatures were not given in the paper but were previously published (Kodur et al. 2005). At the end of the four hour fire test, the measured temperatures at the surface of the concrete ranged between 270 and 290 °C at the corners of the column and between 380 and 420 °C for thermocouples located on the side faces of the column. As a result of this observation and for simplicity in modelling, the temperature throughout the concrete was assumed to be 300 °C for the purpose of producing the curve shown as “Column in Fire with FRP” in Fig. 9 of the paper. In reality, the temperature in the concrete inside the column would be much lower than the temperatures measured at the surface and thus this assumption was both reasonable and conservative.

Based on this assumed average temperature in the column, the material properties in the concrete and reinforcing steel were adjusted based on models implemented in SAFIR (Franssen 2007) which are based on Eurocode 3 (EC3 1995) for steel and Eurocode 2 (EC2 1993) for concrete as shown in Table 1. The modelling included the effects of temperature on modulus and strength but did not consider the effect of temperature on bond strength of steel to concrete.

Predictions of performance after fire exposure

As mentioned in the discussion and the paper, the numerical simulations showed that the FRP strengthened and insulated column displayed more ductility than did the unstrengthened column that was not exposed to fire. The additional ductility was mainly due to the effect of the temperature on the steel reinforcement. For example, Table 1 shows that the modulus of elasticity of the steel reinforcement is reduced to 81% of its room temperature value at 300 °C. At this same temperature, Table 1 shows that the steel has not lost any strength and the concrete has lost approximately 14% of its original structure. Thus, the predicted increases in ductility are consistent with the changes in the material properties at high temperature.

The Discussion also asks about the effect of a long period of fire exposure. For the columns considered in the original paper, the fire exposure was four hours and thus longer exposure periods are not practical. However, if the insulated columns were exposed to a longer period of fire exposure, then the temperatures would increase and the behaviour would approach that of the uninsulated column (indicated as the "Bare Column in Fire" in Fig. 9 of the original paper).

Conclusions

This Discussion has clarified the importance of high temperature material properties for modelling the performance of structural members during and immediately after fire exposure.

References:

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EC2 (1993), Eurocode 2: Design of Concrete Structures, ENV 1992-1-2: General Rules- Structural Fire Design. European Committee for Standardization, Brussels, Belgium.

EC3 (1995), Eurocode 3: Design of Steel Structures, ENV 1993-1-2: General Rules- Structural Fire Design. European Committee for Standardization, Brussels, Belgium.

Table 1. Temperature-dependent material properties for reinforcing steel and concrete.

Temperature	Steel		Concrete	
°C	Relative Modulus of Elasticity	Relative Yield Strength	Peak Strain	Relative Compressive Strength
20	1.00	1.00	0.0028	1.00
100	1.00	1.00	0.0039	0.96
200	0.91	1.00	0.0047	0.91
300	0.81	1.00	0.0065	0.86
400	0.71	1.00	0.0078	0.76
500	0.60	0.78	0.0097	0.61
600	0.34	0.54	0.0129	0.46
700	0.13	0.22	0.0151	0.30