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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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1 **Introduction**

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

5 **Modelling of Loading Cycles**

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

18

19 Although numerical models to predict post-failure behaviour could be
20 developed, such modelling was beyond the scope of the paper. Also, the
21 axial–shear–flexural interaction (ASFI) method used in the paper did not

22 have the ability to consider cyclic hysteresis loops, but other approaches
23 could be developed. Once again, such modelling was beyond the scope of
24 the paper.

25

26 **Material Properties at High Temperature**

27 The other main comment in the Discussion related to the material properties
28 at high temperature. For the 400×400 mm column 5 that was wrapped with
29 FRP and protected with insulation, the temperatures were not given in the
30 paper but were previously published (Kodur et al. 2005). At the end of the
31 four hour fire test, the measured temperatures at the surface of the concrete
32 ranged between 270 and 290 °C at the corners of the column and between
33 380 and 420 °C for thermocouples located on the side faces of the column.
34 As a result of this observation and for simplicity in modelling, the temperature
35 throughout the concrete was assumed to be 300 °C for the purpose of
36 producing the curve shown as “Column in Fire with FRP” in Fig. 9 of the
37 paper. In reality, the temperature in the concrete inside the column would be
38 much lower than the temperatures measured at the surface and thus this
39 assumption was both reasonable and conservative.

40

41 Based on this assumed average temperature in the column, the material
42 properties in the concrete and reinforcing steel were adjusted based on
43 models implemented in SAFIR (Franssen 2007) which are based on
44 Eurocode 3 (EC3 1995) for steel and Eurocode 2 (EC2 1993) for concrete as

45 shown in Table 1. The modelling included the effects of temperature on
46 modulus and strength but did not consider the effect of temperature on bond
47 strength of steel to concrete.

48

49 **Predictions of performance after fire exposure**

50 As mentioned in the discussion and the paper, the numerical simulations
51 showed that the FRP strengthened and insulated column displayed more
52 ductility than did the unstrengthened column that was not exposed to fire.
53 The additional ductility was mainly due to the effect of the temperature on the
54 steel reinforcement. For example, Table 1 shows that the modulus of
55 elasticity of the steel reinforcement is reduced to 81% of its room
56 temperature value at 300 °C. At this same temperature, Table 1 shows that
57 the steel has not lost any strength and the concrete has lost approximately
58 14% of its original structure. Thus, the predicted increases in ductility are
59 consistent with the changes in the material properties at high temperature.

60

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

68

69 **Conclusions**

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

References:

Franssen, J.M. 2007. User's Manual For SAFIR 2007, a Computer Program For Analysis of Structures Subjected to Fire, University of Liege, Belgium.

Kodur, V.K.R., Bisby, L.A., Green, M.F., and Chowdhury, E.U. 2005. Evaluating the Fire Endurance of FRP-Strengthened Square Reinforced Concrete Columns. ACI Special Publication SP-230, American Concrete Institute, Farmington Hills, Michigan, pp. 1253-1268.

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 °C	Steel		Concrete	
	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

