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Compressive behavior of fiber reinforced high-performance concrete subjected to elevated temperatures

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Abstract

In this paper, the effects of elevated temperatures on the compressive strength stress–strain relationship (stiffness) and energy absorption capacities (toughness) of concretes are presented. High-performance concretes (HPCs) were prepared in three series, with different cementitious material constitutions using plain ordinary Portland cement (PC), with and without metakaolin (MK) and silica fume (SF) separate replacements. Each series comprised a concrete mix, prepared without any fibers, and concrete mixes reinforced with either or both steel fibers and polypropylene (PP) fibers. The results showed that after exposure to 600 and 800 °C, the concrete mixes retained, respectively, 45% and 23% of their compressive strength, on average. The results also show that after the concrete was exposed to the elevated temperatures, the loss of stiffness was much quicker than the loss in compressive strength, but the loss of energy absorption capacity was relatively slower. A 20% replacement of the cement by MK resulted in a higher compressive strength but a lower specific toughness, as compared with the concrete prepared with 10% replacement of cement by SF. The MK concrete also showed quicker losses in the compressive strength, elastic modulus and energy absorption capacity after exposure to the elevated temperatures. Steel fibers approximately doubled the energy absorption capacity of the unheated concrete. They were effective in minimizing the degradation of compressive strength for the concrete after exposure to the elevated temperatures. The steel-fiber-reinforced concretes also showed the highest energy absorption capacity after the high-temperature exposure, although they suffered a quick loss of this capacity. In comparison, using PP fibers reduced the energy absorption capacity of the concrete after exposure to 800 °C, although it had a minor beneficial effect on the energy absorption capacity of the concrete before heating.

Keywords: High-performance concrete; Steel fibers; PP fibers; Elevated temperatures; Compressive behavior

1. Introduction

High-performance and durable concrete structures have been studied in various aspects. More and more attention has been paid to the mechanical properties of concrete at high temperature, or the residual properties of concrete after exposure to elevated temperatures [1,2]. High temperature causes dramatic physical and chemical changes, resulting in the deterioration of the concrete [3,4]. An assessment of the degree of deterioration of the concrete structure after exposure to high temperatures can help engineers to decide whether a structure can be repaired rather than required to be demolished [5].

Applications of silica fume (SF), fly ash (FA) and ground slag in concrete are effective ways to prepare high-performance concrete (HPC; [6]). Recently, a lot of interest is being paid on the possible use of metakaolin (MK) in concrete [7,8]. Although HPC has been shown to have a number of advantages when used in concrete structures, it suffers from one major weakness: higher brittleness. When exposed to high temperatures, HPC exhibits more serious degradation than normal concretes do, such as spalling and cracking [9,10]. A past research study by the authors suggested that the HPC prepared with MK could increase the possibility of concrete spalling [10]. The buildup of high internal vapour pressure in the dense HPC system has been suggested to be the major cause [11].

Fibers have extensively been used to improve the ductility of concrete. Recently, it has been found that a number of fibers can also improve the residual properties of concrete after exposure to elevated temperatures. Polypropylene (PP) fibers and steel fibers have been used to reduce spalling and cracking and to enhance the residual strength [12,13]. But minimal or even negative effects of PP fibers on the residual performance of the heated concrete were also observed [14].

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The initial moisture state of the specimens and the rate of heating may be the main parameters determining the effect of PP fibers [15].

A few recent studies have been conducted on the effects of high temperatures on the residual compressive and fracture properties of the high-strength concrete [16,17]. The objective of this study is to increase the insight of the mechanical behaviour of high-strength concrete (HPC) prepared with fiber reinforcement after exposure to elevated temperatures (up to 800 °C). In particular, the results of the effects of pozzolanic materials (SF and MK) and different types of fibers (steel and PP fibers) on the load bearing and energy absorption capacities of concretes are discussed.

2. Experimental details

2.1. Materials

The cementitious materials used in this study were ordinary Portland cement (PC) equivalent to ASTM Type I, SF sourced commercially and MK obtained from Imerys Minerals. The chemical compositions of the cement, MK and SF are shown in Table 1.

The coarse aggregate used was a crushed granite with nominal sizes of 10 and 20 mm. The specific gravity of the aggregate was 2.62 g/cm^3. In the concrete mixtures, the 10- and 20-mm coarse aggregates were used in a proportion of 1:2. Natural river sand, with a fineness modulus of 2.11, was used as fine aggregate. The steel fibers used were hooked fibers with a length of 25 mm and an aspect ratio of 60. The polymer fibers used were PP fibers with a length of 19 mm and an aspect ratio of 360. A naphthalene-based superplasticizer was used to achieve the required workability of the concrete mixes.

2.2. Mix proportioning

A total of 14 concrete mixes were prepared in three series with different cementitious materials constitutions. While Series I mixes were prepared with plain ordinary PC, Series II mixes were prepared with MK at a cement replacement level of 20% by weight, and Series III mixes were prepared with SF at a cement replacement level of 10% by weight. Each series comprised a control mix prepared without any fibers and three fiber reinforced concrete mixes that were prepared with 1% steel fibers, 0.22% PP fibers and a combination of 1% steel fibers and 0.22% PP fibers (by volume), respectively. Series II and III each comprised one more mix with 0.11% PP fibers. These percentages were based on the total volume of the concrete. All the concrete mixes were prepared at a water-to-cementitious materials ratio of 0.29. The details of the mix proportions are shown in Table 2.

Table 1
Properties of cement, metakolin and silica fume

Materials	PC	MK	SF
Chemical composition (%)			
SiO_2	19.61	53.20	90.26
Al_2O_3	7.33	43.90	0.63
Fe_2O_3	3.32	0.38	0.33
CaO	63.15	0.02	3.18
MgO	2.54	0.05	0.33
Na_2O		0.17	
K_2O		0.10	
SO_3	2.13		0.40
Loss of Ignition	2.97	0.50	4.84
Physical properties			
Specific gravity	3.16	2.62	2.22
Specific surface (cm^2/kg)	3519	12680	

Table 2
Mix proportions

Mix		Water	PC	MK	SF	Sand	Aggregate	Steel fibre	PP fibre
		Mix proportions (kg/m^3)							
Series 1	PC-0	145	500			725	1075		
	PC-1	145	500			725	1075	78	
	PC-2	145	500			725	1075		2
	PC-3	145	500			725	1075	78	2
Series 2	MK-0	145	400	100		695	1075		
	MK-1	145	400	100		695	1075	78	
	MK-2	145	400	100		695	1075		1
	MK-3	145	400	100		695	1075		2
	MK-4	145	400	100		695	1075	78	2
Series 3	SF-0	145	450		50	710	1075		
	SF-1	145	450		50	710	1075	78	
	SF-2	145	450		50	710	1075		1
	SF-3	145	450		50	710	1075		2
	SF-4	145	450		50	710	1075	78	2

2.3. Specimen preparation and test

The concrete mixtures were prepared in a pan mixer. For each mix, a total of 12 specimens, including three 100-mm cubes and nine 100 (diameter) × 200 mm (height) cylinders, were cast in steel molds. The specimens, after removal from the steel molds at 1 day, were cured in water at 27 °C until the age of 28 days, and then conditioned in an environmental chamber at a temperature of 20 °C and a relative humidity of 75% for another 28 days. The three cubes and three of the nine cylinders were tested immediately after the conditioning, and the results will be referred to as those obtained under normal curing (unheated). The remaining six cylinders were subjected to two temperature exposure conditions in an electrical furnace. Three were heated to 600 °C and the other three to 800 °C. In the furnace, the cylinders were heated at a constant rate of 2.5 °C/min to reach the prescribed temperatures. After that, the heat was applied for an additional hour before the samples were allowed to cool down naturally to room temperature.

Table 3
Test results of concrete cube and cylinders of unheated concrete (normal curing)

Mix	Addition of fibers	Cube tests		Cylinder tests			
		Apparent density (kg/m^3)	Compressive strength (MPa)	Compressive strength (MPa)	Strain at cylinder peak stress (%)	Area under stress–strain curve ($MPa \times 10^{-2}$)	Specific toughness (%)
PC-0	No	2409	84.3	69.1	0.374	34.20	0.495
PC-1	1% steel	2442	86.6	71.4	0.410	66.41	0.930
PC-2	0.22% PP	2384	83.4	68.5	0.360	36.8	0.537
PC-3	1% steel + 0.22% PP	2411	83.9	69.6	0.433	62.43	0.897
MK-0	No	2382	105.1	86.1	0.397	35.97	0.418
MK-1	1% steel	2426	109.5	87.5	0.420	75.49	0.863
MK-2	0.11% PP	2370	105.4	86.1	0.397	37.64	0.437
MK-3	0.22% PP	2365	98.4	84.6	0.391	39.76	0.470
MK-4	1% steel + 0.22% PP	2409	100.3	86.0	0.402	73.35	0.853
SF-0	No	2395	97.3	82.8	0.388	35.83	0.433
SF-1	1% steel	2433	99.9	83.7	0.390	64.53	0.771
SF-2	0.11% PP	2384	96.7	81.8	0.371	39.46	0.482
SF-3	0.22% PP	2374	95.8	81.2	0.368	42.28	0.521
SF-4	1% steel + 0.22% PP	2415	97.6	82.9	0.391	71.19	0.859

Compression strength tests were carried out on the concrete cubes and cylinders using a Denison compression machine with a 3000-kN capacity. An axial load was applied at a constant axial displacement ratio of 0.1 mm/min. The axial shortening of the cylinder under compression was measured using two linear variable displacement transducers (LVDT) set between the platens. The test was terminated after a shortening of 4 mm was reached, which corresponded to an axial compressive strain of 2%. The load and displacement data were collected using a computer with a "Global Lab" data acquisition and processing system. The results of the compression tests are summarized in Table 3 for the specimens cured normally (unheated), and in Table 4 for the specimens tested after exposure to the elevated temperatures. In these two tables, each value represents the average result of three specimens.

3. Behavior of unheated concrete

3.1. Compressive strength

For the unheated specimens, both the cubes and cylinders were tested. It can be seen from Table 3 that the compressive strength obtained from the cylinder tests is, on average, 0.83 times of that obtained from the cube tests. A 20% replacement of cement by MK resulted in an increase of 20% to 25% in the compressive strength, while a 10% replacement of cement by SF increased the strength by 15 to 20%. Comparing with the use of steel fibers at the level of 1%, which resulted in a small increase in the compressive strength, the use of 0.11% or 0.22% PP fibers had negative effects on the compressive strength for both the PC mixes and the mixes prepared with PC–MK or PC–SF. Similar results had been reported by other researchers [18].

Table 4
Test results of concrete cylinders after exposure to elevated temperatures

Mix	Addition of fibers	After exposure to 600 °C			After exposure to 800 °C		
		Compressive strength (MPa)	Strain at peak stress (%)	Area under stress–strain curve ($MPa \times 10^{-2}$)	Compressive strength (MPa)	Strain at peak stress (%)	Area under stress–strain curve ($MPa \times 10^{-2}$)
PC-0	No fiber	32.79	0.838	28.59	17.64	0.931	17.91
PC-1	1% steel	38.95	1.049	40.73	23.80	1.234	23.66
PC-2	0.22% PP	34.31	0.817	28.27	17.07	0.946	16.14
PC-3	1% steel + 0.22% PP	36.19	1.042	36.69	22.53	1.310	21.88
MK-0	No fiber	33.28	0.832	26.89	15.76	1.049	15.94
MK-1	1% steel	38.72	1.027	38.62	21.41	1.334	21.81
MK-2	0.11% PP	35.10	0.856	26.96	14.98	1.069	15.74
MK-3	0.22% PP	31.69	0.901	25.75	14.49	1.078	14.34
MK-4	1% steel + 0.22% PP	38.65	1.049	37.84	17.81	1.403	18.45
SF-0	No fiber	37.84	0.825	28.54	20.55	1.000	19.82
SF-1	1% steel	39.19	0.838	36.05	23.46	1.170	23.97
SF-2	0.11% PP	36.85	0.814	28.69	15.55	1.093	16.92
SF-3	0.22% PP	33.76	0.812	28.69	14.09	1.039	15.45
SF-4	1% steel + 0.22% PP	37.69	0.974	38.76	19.25	1.225	21.65

The lower compressive strength of the concrete prepared with the PP fibers might be due to the insufficient dispersing of the fibers in the concrete during mixing. In general, microfibers, such as carbon and PP fibers, are difficult to be completely dispersed in concrete prepared with a conventional mixing procedure. Such microfibers tend to form a so-called multifilament structure [19] in concrete during mixing and increases the local porosity. As a result, the compressive strength of concrete is reduced. For sufficient dispersing of microfibers, a special concrete mixing procedure is required [20].

3.2. Stress–strain curve

Complete stress–strain curves of the concrete of the unheated specimens were obtained from the compression tests of the cylinders with a controlled displacement rate. For each mix, three cylinders were tested. As the test results reproduced well, each stress–strain curves shown in Fig. 1 represents the average results of three tests. It should be noted that the axial strains of concrete in compression were obtained from the full height shortening of the cylinders using LVDTs. Such strains are generally larger than those obtained at the midheight region because of the end effects [21].

For the concrete prepared without incorporating any fibers, the replacements of cement by MK or SF resulted in higher strains at the peak stresses, but steeper descending paths in the stress–strain curves. The addition of 1% steel fibers resulted in a significant change in the shape of the stress–strain curves. The concrete mixes containing 1% steel fibers all showed flattened descending paths in the stress–strain curves. The steel fibers also resulted in an increase in the strain at the peak stress for the PC and PC–MK mix, but not for the PC–SF mix.

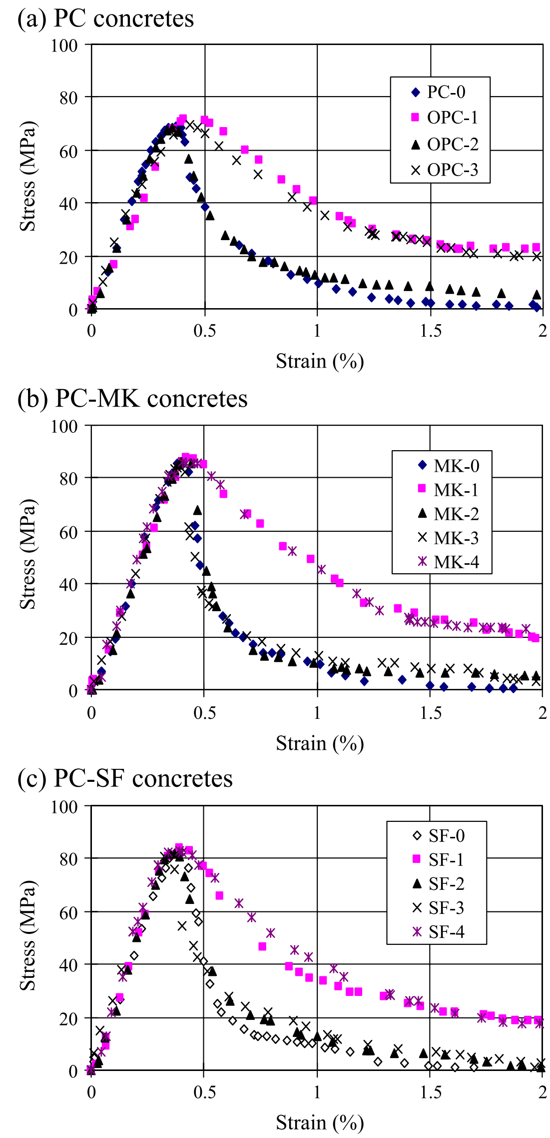

Fig. 1. Stress–strain curves of unheated concrete cylinders.

3.3. Energy absorption capacity (toughness)

The energy absorption capacity or toughness of concrete in compression has been defined as the area under the stress–strain curve calculated up to a specified strain value [22,23]. The specific toughness of concrete in compression has been defined as the ratio of the area under the stress–strain curve to the cylinder compressive strength of the concrete [22]. In this paper, the area under the stress–strain curve was calculated up to a strain value of 1.5%. Such a strain value was also used in the study of Nataraja et al. [23].

It can be seen from Table 3 that among the three mixes without any fibers, the PC concretes have the highest specific toughness (PC-1) and the PC–MK concrete the lowest (MK-1), with the PC–SF concretes (SF-1) in between. The incorporation of 1% steel fibers in the concrete approximately doubled the toughness when compared with the concrete mixes prepared without the fibers. However, the specific toughness of the PC concrete was still higher than that of the PC–MK and PC–SF concretes (MK-2 and SF-2) because the compressive strengths of the PC–MK and PC–SF concretes are higher. When used at the 0.11% and 0.22% levels, the PP fibers only slightly increased the toughness and the specific toughness. Compared with the use of 1% steel fibers alone, the combined use of 1% steel fibers and 0.22% PP fibers did not show additional benefits for the PC and PC–MK concretes. However, it increased the toughness of the PC–SF concrete.

4. Behavior of concrete after exposure to elevated temperatures

4.1. Compressive strength

Only the cylinders were tested after they are exposed to the elevated temperatures. It can be seen from Table 4 that an average of 45% of the compressive strength of the unheated concrete was retained after exposure to 600 °C, and was further reduced to 23% after exposure to 800 °C. For the majority of the mixes, more than 50% and 70% of the compressive strength were lost after the exposure to the temperatures of 600 and 800 °C, respectively. Only the two mixes prepared with the use of steel fibers in the PC series retained more than 50% their compressive strength after exposure to 600 °C and 30% after exposure to 800 °C.

In comparison, the MK series concretes showed the highest losses in strength after exposure to elevated temperatures. Although the unheated PC–MK concretes had the highest compressive strength, the residual strengths of the MK series concretes were similar with that of the PC concretes after exposure to 600 °C, and they became lower than that of the PC concretes after exposure to 800 °C. The quick loss in compressive strength for the PC–MK concrete has been attributed to the dense microstructures in this type of concrete, which led to the buildup of high internal pressure during heating [8,10].

Steel fibers are seen to be useful in minimizing the damage effect of high temperature for all the three series of concrete mixes. However, although the use a small percentage (0.11%) of PP fibers slightly increased the residual strength of the PC and PC–MK concretes after exposure to 600 °C, it had negative effects on the compressive strength of the concrete at 800 °C. When the PP fibers were used together with steel fibers, the residual strength of the concrete was lower than that of the concrete with steel fibers alone, but was slightly higher that of the concrete without any fibers in the PC and MK series.

In a study conducted by Kalifa et al. [13], PP fibers were shown to be beneficial to the residual strength of concrete after exposure to high temperature. This is not consistent with the observations of the present study. The different observations between the two studies might be due to the differences in the experimental conditions. In the referenced study [13], the specimens were heated following a standard curve specified in a Japanese standard [12], which requires that a temperature as high as 800 °C has to be reached within 20 min. However, in the present study, the specimens were heated at a much lower rate of 2.5 °C. This means that it takes 320 min to reach 800 °C. On the other hand, the specimens in the present study underwent a 28-day conditioning, and an air-dry state was achieved before they were subjected to the high-temperature exposures.

It has been shown in the previous studies [8,10,16] that heating to a temperature of 200 °C does not have significant effects on the compressive strength of concretes. Fig. 2 shows the effect of temperature on the residual compressive strength of the HPCs, where the compressive strengths were given as relative values, with reference to the compressive strength of the unheated concrete mixes. Fig. 2a shows that for the PC concrete prepared without any fibers, the relationship between the residual compressive strength and exposure temperature can be approximated by the dash line,

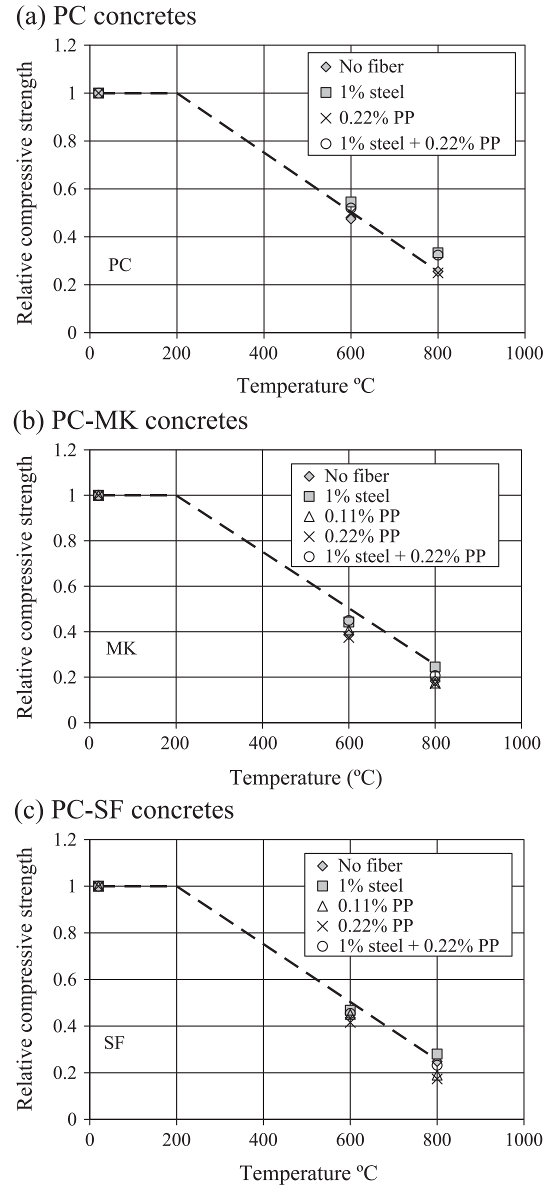

Fig. 2. Effect of temperature on compressive strength.

assuming that the compressive strength maintains constant up to 200 °C and then drops linearly with increasing temperatures. This dash line is also shown in Fig. 2b and c as a reference to compare the performance of the MK and SF series mixes. Clearly, the use of MK and SF resulted in a quicker loss of compressive strength, although they increased the compressive strength of the unheated concrete. The use of steel fibers slightly reduced the rate of degradation of compressive strength.

4.2. Elastic modulus (stiffness) and strain at peak stress

The stress–strain curves of the concretes after exposure to the temperatures of 600 and 800 °C are shown in Figs. 3 and 4, respectively. These two figures show that after exposure to the elevated temperatures, the initial ascending

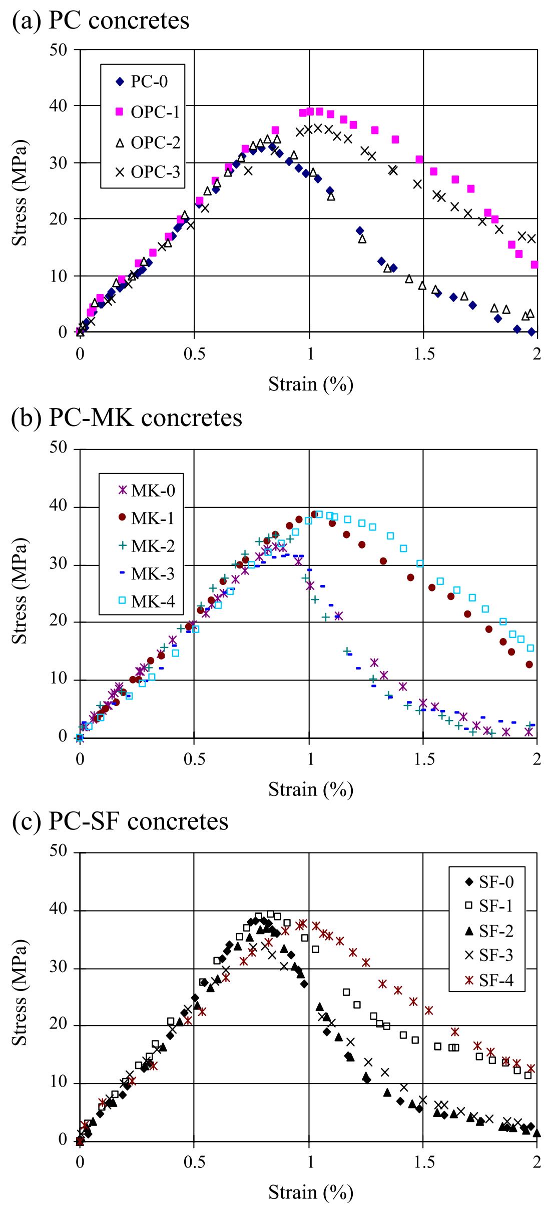

Fig. 3. Stress–strain curves of concrete cylinders after exposure to 600 °C.

parts of the curve for all the concretes were approximately linear, but the stiffness magnitudes were much lower. The decrease in elastic modulus with increasing temperature is shown in Table 5, where the elastic moduli of concrete were obtained within one third the peak stress, and were given as relative values with reference to the elastic modulus of the unheated concrete mixes. It should be noted that in this paper, only the relative elastic modulus values are presented because the test method adopted was slightly different from the standard procedures. The results show that the degradation of elastic modulus is much quicker than that of compressive strength. Only 18% of the elastic modulus original sample was retained, on average, after the concretes were exposed to 600 °C. This was further decreased to 11% after exposure to 800 °C. Wu et al. [16] observed that for high-strength concrete prepared without any fibers, the elastic modulus dropped quickly from 200 to 600 °C, but the decrease was more gradual from 100 to 200 °C and from 600 to 900 °C. The slower drop of elastic modulus after heating to 600 °C was also observed in this study for both the concretes prepared with and without the use of fibers (as shown in Table 5). The use of MK or SF did not appear to have a significant effect on the stiffness of the concrete before and after exposure to the elevated temperatures. In comparison, the mixes with 1% steel fibers seemed to preserve the elastic modulus to a higher percentage than those concretes prepared with the PP fibers or a combination of the PP and steel fibers.

As a result of the highly degraded stiffness, peak stresses were reached at higher strains. The strains at the peak

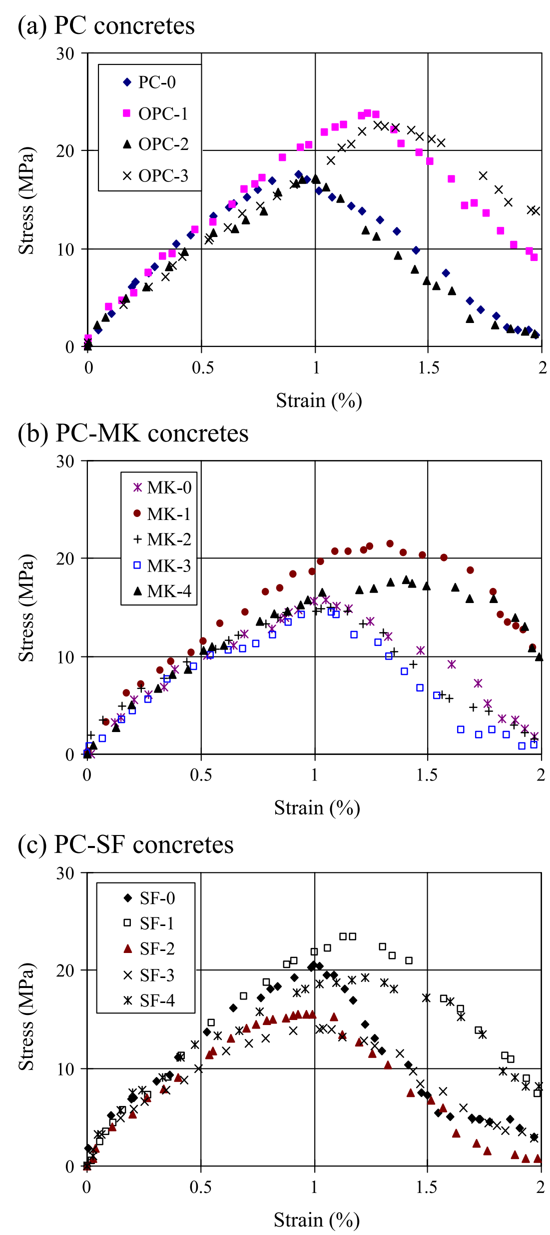

Fig. 4. Stress–strain curves of concrete cylinders after exposure to 800 °C.

Table 5
Effect of temperature on relative strain at peak stress, stiffness and toughness (energy absorption capacity)

Mix	Addition of fibers	Relative strain at peak stress			Relative stiffness			Relative toughness		
		27 °C	600 °C	800 °C	27 °C	600 °C	800 °C	27 °C	600 °C	800 °C
OPC-0	No fiber	1.00	2.24	2.49	1.00	0.47	0.26	1.00	0.84	0.52
OPC-1	1% steel	1.00	2.56	3.01	1.00	0.55	0.33	1.00	0.61	0.36
OPC-2	0.22% PP	1.00	2.27	2.63	1.00	0.50	0.25	1.00	0.77	0.44
OPC-3	1% steel + 0.22% PP	1.00	2.41	3.03	1.00	0.52	0.32	1.00	0.59	0.35
MK-0	No fiber	1.00	2.09	2.64	1.00	0.39	0.18	1.00	0.75	0.44
MK-1	1% steel	1.00	2.44	3.17	1.00	0.44	0.24	1.00	0.51	0.29
MK-2	0.11% PP	1.00	2.16	2.70	1.00	0.41	0.17	1.00	0.72	0.42
MK-3	0.22% PP	1.00	2.31	2.76	1.00	0.37	0.17	1.00	0.65	0.36
MK-4	1% steel + 0.22% PP	1.00	2.61	3.49	1.00	0.45	0.21	1.00	0.52	0.25
SF-0	No fiber	1.00	2.13	2.58	1.00	0.46	0.25	1.00	0.80	0.55
SF-1	1% steel	1.00	2.15	3.00	1.00	0.47	0.28	1.00	0.56	0.37
SF-2	0.11% PP	1.00	2.20	2.95	1.00	0.45	0.19	1.00	0.73	0.43
SF-3	0.22% PP	1.00	2.21	2.83	1.00	0.42	0.17	1.00	0.68	0.37
SF-4	1% steel + 0.22% PP	1.00	2.49	3.14	1.00	0.45	0.23	1.00	0.54	0.30

stresses of the concretes after exposure to 600 and 800 °C were, respectively, on average, about 2.3 and 2.9 times of those of the unheated concretes. The magnitude of the strains at the peak stresses seems to be independent of the cementitious material constitutions (Table 4). While the concretes prepared with the steel fibers (with or without PP fibers) had higher strains at the peak stresses than the other mixes had, the PP fibers did not affect the strain at the peak stress significantly. The effects of temperature on the strain at peak stress are shown in Table 5, where the strains are given as the relative values with reference to the strain at peak stress of the unheated concretes.

4.3. Energy absorption capacity (toughness)

While many studies have been conducted on the effect of incorporating fibers on the compressive toughness of concrete under normal curing (unheated; [22–25]), few studies have documented the compressive toughness of fiber-reinforced concrete after exposure to elevated temperatures.

The areas under the stress–strain curves, which have been referred to as the energy absorption capacity or toughness, of the concretes after exposure to elevated temperatures are given in Table 4. The effects of temperature on the residual energy absorption capacity of concrete are shown in Table 5, where the values curve were averages of three specimens and were given as relative values with reference to the area under the stress–strain curve of the unheated concretes. It can be found by comparing the data given in Tables 3 and 4 that the concrete mixes retained 66% and 39% their energy absorption capacities on average after exposure to temperatures of 600 and 800 °C, respectively. These percentages are higher than those for the residual compressive strength. As a result, the specific toughness values of the concretes, defined as the ratio of the area under the stress–strain curve to the compressive strength of concrete, were all higher after the concrete were exposed to the elevated temperatures.

When comparisons are made between the concrete mixes prepared with different cementitious materials, the PC–SF concrete mixes showed higher energy absorption capacities, particularly after exposure to 800 °C. This is in contrast to the PC–MK concretes, showing the lowest energy absorption capacity after exposure to 800 °C.

In addition, it can be seen from Table 4 that the energy absorption capacities of the concretes prepared with steel fibers (with or without PP fibers) were still the highest after exposure to the elevated temperatures, although the energy absorption capacity of the samples decreased more rapidly after heating (Table 5). The concrete prepared with only the PP fibers had toughness values similar with the concretes without using any fibers after exposure to 600 °C, but lower toughness values than the concretes without any fibers after exposure to 800 °C. These results indicated that the PP fibers contributed little to the residual energy absorption capacity of concrete after exposure to the elevated temperatures, although they slightly improved the toughness of the unheated concretes (see Table 3).

5. Conclusions

In this paper, the results of elevated temperatures on the compressive strength stress–strain relationship (stiffness) and energy absorption capacity (toughness), defined as the area under the stress–strain curve up to an axial strain of 1.5%, were presented. HPCs were prepared in three series with the use of plain ordinary PC, MK and SF, with and without the incorporation of steel and PP fibers. The following conclusions can be drawn from the results:

1. The HPC mixes retained about 45% their compressive strength, on average, after exposure to 600 °C, and this was further reduced to only 23% after exposure to 800 °C. While the loss of the stiffness of the concrete was

much quicker after exposure to the elevated temperatures, the loss toughness was slower as compared with the loss of compressive strength. The strain at the peak stress was about 2.3 and 2.9 times of the original values after exposure to 600 and 800 °C, respectively, as a result of the significant degradation of the stiffness.
2. For the unheated concretes, the use of MK at a replacement level of 20% of cement resulted in higher compressive strength than the use of SF at the level of 10%, but more brittle postpeak stress–strain responses. In addition, the use of MK resulted in a quicker degradation of the compressive strength of the concrete after exposure to the elevated temperatures.
3. Steel fibers approximately doubled the toughness of the unheated concrete. The fibers were effective in reducing the degradation of compressive strength of the concrete after exposure to the elevated temperatures. The steel fiber reinforced concretes also had the highest toughness values after the high-temperature exposures.
4. For the unheated concretes, PP fibers slightly increased the specific toughness, defined as the ratio of the area under the stress–strain curve, and the compressive strength for the concrete. However, they resulted in a quicker loss of the compressive strength and toughness of the concrete after exposure to the elevated temperatures. The combined use of PP fibers and steel fibers showed little benefits compared with the use of steel fibers only.

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