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Comparison of the strength and durability performance of normal- and high-strength pozzolanic concretes at elevated temperatures

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Abstract

The strength and durability performance of normal- and high-strength pozzolanic concretes incorporating silica fume, fly ash, and blast furnace slag was compared at elevated temperatures up to 800°C. The strength properties were determined using an unstressed residual compressive strength test, while durability was investigated by rapid chloride diffusion test, mercury intrusion porosimetry (MIP), and crack pattern observations. It was found that pozzolanic concretes containing fly ash and blast furnace slag give the best performance particularly at temperatures below 600°C as compared to the pure cement concretes. Explosive spalling occurred in most high-strength concretes (HSCs) containing silica fume. A distributed network of fine cracks was observed in all fly ash and blast furnace slag concretes, but no spalling or splitting occurred. The high-strength pozzolanic concretes showed a severe loss in permeability-related durability than the compressive strength loss. Thirty percent replacement of cement by fly ash in HSC and 40% replacement of cement by blast furnace slag in normal-strength concrete (NSC) was found to be optimal to retain maximum strength and durability after high temperatures.

Keywords: High temperature; Pozzolan; Compressive strength; Permeability; Microstructure

1. Introduction

Pozzolanic concretes are used extensively throughout the world; the oil, gas, nuclear, and power industries are among the major users. The applications of such concretes are increasing day by day due to their superior structural performance, environmental friendliness, and energy-conserving implications [1]. Apart from the usual risk of fire, these concretes are exposed to high temperatures and pressures for considerable periods of time in the abovementioned industries. Although concrete is generally believed to be an excellent fireproofing material, many recent studies have shown extensive damage or even catastrophic failure at high temperatures, particularly in high-strength concrete (HSC) [2]. During the last decade, there has been extensive research on the fire performance of normal-strength concrete (NSC) and HSC; however, most studies emphasized on aspects like types of aggregate, addition of fibers, heating rate and maximum temperature level, methods of testing, etc. [3]. There is very little research work available on the comparative performance of different pozzolanic concretes at elevated temperatures. Moreover, these studies considered only the residual strength properties and no attention was given on the durability loss, which can reduce the remaining service life of the structure very rapidly. In this research, a comprehensive experimental work has been done to consider the effects of pozzolan type and content on the fire performance of NSCs and HSCs at macro- and microlevels.

The following sections present a brief review of the previous research conducted on the fire performance of pozzolanic concretes.

1.1. Fire performance of silica fume concrete

An earlier research by Hertz [4] indicated that silica fume concrete is highly prone to spalling and cracking at elevated temperatures. He prepared a special 170-MPa concrete containing 14–20% silica fume. Five of the fifteen 100×200-mm cylinders exploded when heated to 650°C.

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Table 1
Chemical composition and physical properties of binders

	OPC	CSF	PFA	GGBS
Chemical composition (%)				
Silicon dioxide (SiO_2)	19.61	90.26	56.79	38.14
Aluminum oxide (Al_2O_3)	7.33	0.63	28.21	6.53
Ferric oxide (Fe_2O_3)	3.32	0.33	5.31	0.40
Calcium oxide (CaO)	63.15	3.18	< 3.00	35.77
Magnesium oxide (MgO)	2.54	0.33	5.21	13.65
Sodium oxide (Na_2O)	–	–	–	0.36
Potassium oxide (K_2O)	–	–	–	0.40
Sulphur trioxide (SO_3)	2.13	0.4	0.68	–
Loss on ignition	2.97	4.84	3.90	0.76
Physical properties				
Specific gravity	3.16	2.22	2.31	2.52
Specific surface (cm^2/g)	3519	–	4120	4700

The test results indicated that on average, the residual compressive strength of silica fume concrete increased with temperature up to 350°C and then decreased sharply. In a later study, Hertz [5] prepared another series of specimens with granite aggregate and silica fume contents of 0%, 5%, 10%, and 15% of cement by weight, and a moisture content in equilibrium with air conditions. None of these specimens exploded when heated at rates of 1°C/min and 5°C/min to 600°C. He concluded that concretes densified by means of silica fume at high moisture contents are more likely to explode and suggested an upper limit of 10% by weight of cement on silica fume to avoid spalling. The same conclusions were also verified by Sanjayan and Stocks [6] after conducting a fire test on a monolithic beam-slab (T-beam) specimen containing 8% silica fume.

Sarshar and Khoury [7] found that replacement of ordinary Portland cement (OPC) by 10% silica fume did not offer any additional benefits at elevated temperatures. The residual strength of the concretes containing silica fume appeared to be worse than the concretes containing 100% OPC. Hammer [8] prepared five-series of specimens containing 0–5% silica fume with normal-weight crushed gravel or lightweight aggregates and subjected them to a maximum temperature of 600°C at a heating rate of 2°C/min. Concrete without silica fume showed slightly better performance in terms of lower strength loss and reduced spalling. Felicetti and Gambarova [9], who prepared concretes with 6.7% and 9.7% silica fume, flint aggregates, and Type V Portland cement, also obtained the similar results.

In short, it can be concluded that the addition of silica fume highly densifies the pore structure of concrete, which results in explosive spalling due to the buildup of pore pressure by water vapors. Since the evaporation of physically absorbed water starts at 80°C, which induces thermal cracks, such concretes show inferior performance as compared to pure OPC concretes at elevated temperatures.

1.2. Fire performance of fly ash concrete

The research on performance of pulverized fly ash (PFA) concretes at elevated temperatures dates back to 1969 [10]; however, due to limited applications of PFA concrete before the last decade, only few studies were carried out. Nasser and Marzouk [11] conducted a series of research on 25-MPa mass concrete containing 25% lignite fly ash by weight. The specimens were exposed to high temperatures of 21–232°C for periods of over 6 months. They found an increase in strength of concrete in the temperature range of 121–149°C that was as high as 152% of the original strength. However, the strength of PFA concrete was reduced to about 27% of the original value when the exposure temperature was raised to 232°C. They suggested that the increase in strength for the concrete containing fly ash is due to the formation of tobermorite (a product of lime and PFA at high pressure and temperature), which was reported to be two to three times stronger than the CSH gel. A similar increase in strength was reported by Diederichs et al. [12] between 200°C and 250°C in a 90-MPa concrete incorporating Class F fly ash. However, the residual strength was below the original value at all temperatures.

Sarshar and Khoury [7] tested OPC–PFA paste containing 30% PFA by weight under a series of temperatures till 650°C. The relative residual compressive strength was 88%

Table 2
Mix proportions of HSC mixtures

Mix	SF (%)	FA (%)	GGBS (%)	W/B	Batched quantities (kg/m^3) Water	Cement	Fine aggregate	Coarse aggregate	SP[a]	28 days compressive strength (MPa)
HS-CC[b]	–	–	–	0.30	150	500	758	927	0.5	85.9
HS-SF5	5	–	–	0.30	150	475	710	1066	0.6	96.5
HS-SF10	10	–	–	0.30	150	450	620	1151	0.8	108.3
HS-FA20	–	20	–	0.30	150	400	618	1147	0.8	82.7
HS-FA30	–	30	–	0.30	150	350	615	1143	0.7	80.2
HS-FA40	–	40	–	0.30	150	300	613	1139	0.7	76.7
HS-SF + FA	10	20	–	0.30	150	350	615	1142	0.8	105.3
HS-BS30	–	–	30	0.30	150	350	616	1145	0.7	83.9
HS-BS40	–	–	40	0.30	150	300	615	1142	0.7	80.9

[a] SP content in percentage by weight of binder.
[b] Control concrete.

Table 3
Mix proportions of NSC mixtures

Mix	SF (%)	FA (%)	GGBS (%)	W/B	Batched quantities (kg/m^3)					28 days compressive strength (MPa)
					Water	Cement	Fine aggregate	Coarse aggregate	SP[a]	
NS-CC[b]	–	–	–	0.50	195	390	768	917	–	35.8
NS-FA30	–	30	–	0.50	195	273	626	1133	–	39.3
NS-FA40	–	40	–	0.50	195	234	625	1129	–	36.9
NS-BS30	–	–	30	0.50	195	273	626	1135	–	46.4
NS-BS40	–	–	40	0.50	195	234	625	1132	–	39.8

[a] SP content in percentage by weight of binder.
[b] Control concrete.

at 450°C and 73% at 600°C, which was almost double than the residual strength shown by pure OPC pastes.

Ghosh and Nasser [13] conducted research to investigate the effects of high temperature and pressure on the strength and elasticity of HSC incorporating silica fume and high-calcium lignite fly ash together. The fly ash content was 20% or 60% of the weight of binder, while silica fume was added in each mix with a constant dosage of 10% by weight of the total binder. The exposure temperature was varied from 21°C to 232°C, while pressure was varied from 5.2 to 13.8 MPa. A gradual deterioration of strength and static modulus of elasticity was observed with a rise in temperature at all pressures. The 20% PFA replacement showed more loss; however, in any case, the residual compressive strength was more than 60% of the original strength.

In a recent research, Wong et al. [14] studied the effects of PFA replacement level, water/binder ratio (W/B), and curing conditions on the residual properties of concrete at elevated temperatures. An increase in strength was observed at 250°C. All PFA concrete specimens showed better performance till 650°C than pure OPC concrete specimens; however, after that, there was no significant difference in the residual strength of all specimens. It was found that a high dosage of PFA enhanced the residual properties of concrete at elevated temperatures. The results were also verified by porosity analysis done by mercury intrusion porosimetry (MIP) technique.

Conclusively, it was found that the PFA improved the performance of concrete at elevated temperatures as compared to silica fume or pure OPC concretes. However, this improvement was more significant at temperatures below 600°C. Moreover, it was discovered that the PFA also reduces the surface cracking of concrete both at elevated temperatures and after postcooling in air or water [15].

1.3. Fire performance of blast furnace slag concrete

An earlier work on the performance of slag cement at elevated temperatures was done by Grainger [16]. He tested four cement pastes containing 0%, 50%, 70%, and 90% replacement of slag by weight with OPC. The maximum tested temperature was 500°C with an interval of 100°C. All slag–cement paste specimens experienced an increase in strength between 100°C and 250°C. The 70% slag replacement showed the best results with a residual compressive strength of 190% of the original strength at 110°C. Moreover, the residual strength of this paste was higher at all temperatures as compared to the original strength. The other two slag–cement paste specimens also showed better residual strengths as compared to the pure cement paste specimen.

Diederichs et al. [12] prepared three HSCs mixes incorporating condensed silica fume (CSF), PFA, and granulated blast furnace slag (GGBS) independently. The mixes were subjected to a maximum temperature of 900°C. The GGBS

Table 4
Unstressed residual compressive strength of HSCs

Mix	Compressive strength (MPa)				
	20°C	200°C	400°C	600°C	800°C
HS-CC	91.3	88.0 (96%)	81.5 (89%)	53.0 (58%)	21.9 (24%)
HS-SF5	106.1	105.5 (99%)	98.7 (93%)	55.2 (52%)	22.3 (21%)
HS-SF10	119.9	117.7 (98%)	104.3 (87%)	52.8 (44%)	19.2 (16%)
HS-FA20	96.6	110.2 (114%)	92.9 (96%)	59.8 (62%)	27.0 (28%)
HS-FA30	102.8	124.6 (121%)	100.7 (98%)	68.9 (67%)	32.9 (32%)
HS-FA40	107.7	131.5 (122%)	112.2 (104%)	61.4 (57%)	32.3 (30%)
HS-SF + FA	123.9	135.3 (109%)	116.5 (94%)	63.2 (51%)	23.5 (19%)
HS-BS30	111.9	126.7 (113%)	108.5 (97%)	59.3 (53%)	30.2 (27%)
HS-BS40	115.5	133.3 (115%)	114.9 (100%)	70.5 (61%)	33.6 (29%)

The values in brackets indicate the relative increase or decrease in residual compressive strength as compared to the original strength before heating.

Table 5
Unstressed residual compressive strength of NSCs

Mix	Compressive strength (MPa)				
	20°C	200°C	400°C	600°C	800°C
NS-CC	38.2	35.5 (93%)	28.3 (74%)	11.5 (30%)	3.8 (10%)
NS-FA30	49.1	50.1 (102%)	42.2 (86%)	18.2 (37%)	7.9 (16%)
NS-FA40	55.6	58.9 (106%)	46.7 (84%)	25.0 (45%)	10.0 (18%)
NS-BS30	61.7	60.5 (98%)	52.5 (85%)	31.5 (51%)	12.9 (21%)
NS-BS40	66.8	61.5 (92%)	54.1 (81%)	36.1 (54%)	13.4 (20%)

The values in brackets indicate the relative increase or decrease in residual compressive strength as compared to the original strength before heating.

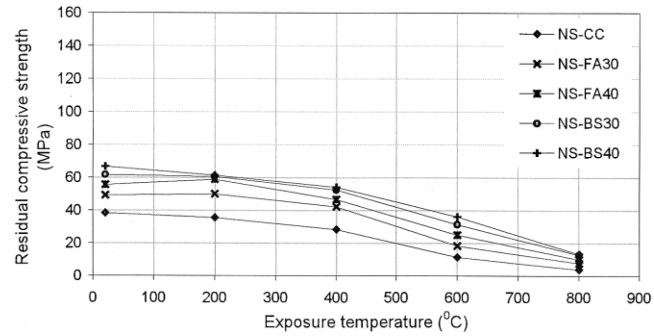


Fig. 2. Residual compressive strength of NSCs.

concrete showed the best performance followed by PFA and CSF concretes.

Sarshar and Khoury [7] prepared cement paste and concrete specimens incorporating 65% slag by weight of cement and firebrick aggregates. The results were compared with pure OPC cement paste/concrete and 30% PFA cement paste. The maximum temperature was 700°C, while the residual properties were measured at every 100°C interval. They found that the slag–cement paste and concrete gave the best results among all the specimens tested. The residual compressive strengths of slag concrete were 102% and 80% of the initial cold strength at 450°C and 600°C.

Additionally, Sullivan and Sharshar [17] studied the properties of silica fume and blast furnace slag concretes prepared with two "thermally stable" aggregates (viz., Lytag and crushed firebrick). The results indicated that concretes with the lightweight aggregate (i.e., Lytag) had lower residual strength at temperature above 150°C than the corresponding concrete prepared with firebrick aggregate. The use of firebrick with blast furnace slag in concrete also resulted in superior elevated temperature performance.

2. Research objectives

The preceding review reveals that most of the studies used only one or maximum two pozzolans at different dosage levels in concrete to evaluate its fire performance. The types of aggregate, curing and testing conditions, heating rate, and strength levels were different in different studies, which makes it difficult to generalize the results for a given condition. Moreover, these analyses were limited to residual mechanical properties only, and no durability studies were conducted. To compare the strength and durability performance of different pozzolanic concretes at elevated temperatures, it is necessary to prepare and test them under the same set of material and environmental conditions. In this study, an attempt is made to achieve this objective by preparing nine HSC and five NSC mixes incorporating CSF, PFA, and ground GGBS, which were then subjected to heating levels of 200°C, 400°C, 600°C, and 800°C. In addition to measuring residual compressive strength, durability analysis was also carried out using rapid chloride diffusion test, MIP, and crack pattern observations.

3. Experimental details

3.1. Materials

3.1.1. Binders

A locally manufactured OPC complying with ASTM Type I, a low-calcium fly ash equivalent to ASTM Class F, CSF, and GGBS were utilized as binders. All the binders are commercially available in Hong Kong. The chemical composition and physical properties of these materials are shown in Table 1.

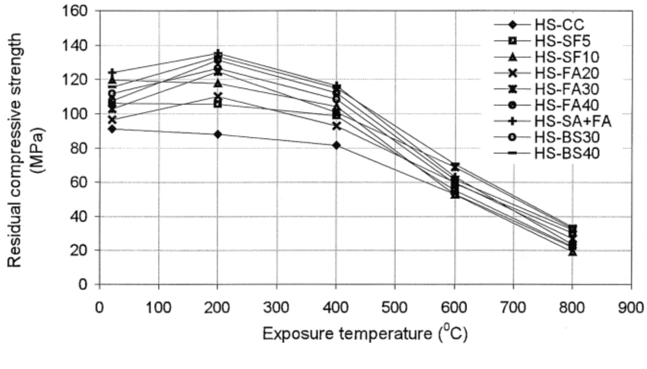


Fig. 1. Residual compressive strength of HSCs.

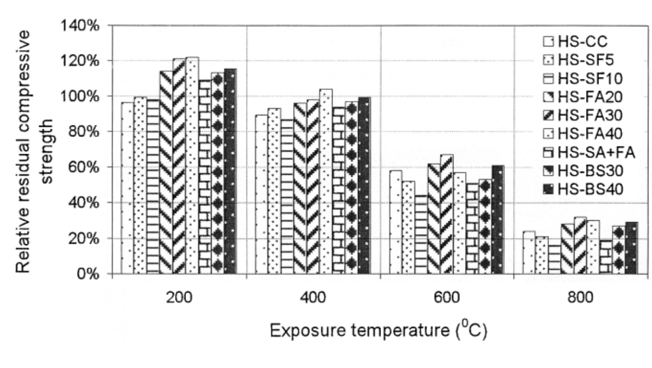


Fig. 3. Relative residual compressive strength of HSCs.

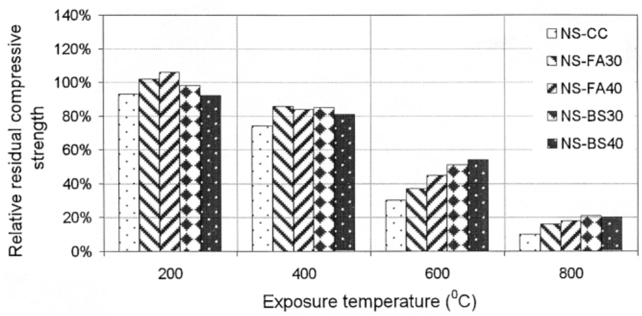


Fig. 4. Relative residual compressive strength of NSCs.

3.1.2. Aggregates

The fine and coarse aggregates were local natural river sand and crushed granite with maximum nominal sizes of 10 and 20 mm, which were mixed in a ratio of 1:2.

3.1.3. Superplasticizer

A sulphonated, naphthalene–formaldehyde condensate was used as a superplasticizer (SP) in HSC mixes. This SP was a dark brown liquid containing 38.6% solids.

3.2. Mix proportions

The mix proportions of nine HSC and five NSC mixes are shown in Tables 2 and 3. All pozzolans were introduced as cement replacement materials and their proportions were decided on the basis of previous research [18] to achieve the optimum strength and durability. One control mix incorporating pure OPC was also prepared for each type of concrete for comparison purposes. All the mixtures were produced at a slump of over 200 mm and no air entraining admixture was used.

3.3. Curing conditions

The specimens were demoulded 24 h after the casting and placed in a water tank at 20°C. After 28 days of water curing, they were transferred to an environmental chamber maintained at 20°C and 75% relative humidity, which are the average climatic conditions in Hong Kong. The specimens were kept there for 1 month until heating.

Table 6
Chloride resistance of high-strength pozzolanic concretes

Mix	Total charge passed (C) 20°C	600°C	800°C
HS-CC	941	12,534	23,396
HS-SF5	610	8619	18,390
HS-SF10	285	10,080	25,170
HS-FA20	533	4947	9405
HS-FA30	449	5373	9049
HS-FA40	369	5739	11,625
HS-SF + FA	122	8928	23,951
HS-BS30	334	6772	10,673
HS-BS40	245	7575	11,044

Table 7
Chloride resistance of normal-strength pozzolanic concretes

Mix	Total charge passed (C) 20°C	600°C	800°C
NS-CC	2941	21,792	35,724
NS-FA30	1826	11,724	24,240
NS-FA40	1454	8550	20,666
NS-BS30	1389	7917	15,836
NS-BS40	1181	6306	15,368

3.4. Heating and cooling regimes

At an age of 60 days, the specimens were heated in an electric furnace up to 200°C, 400°C, 600°C, and 800°C. Each temperature was maintained for 1 h to achieve the thermal steady state [19]. The heating rate was set at 2.5°C/min based on the experience of our previous research [20]. The specimens were allowed to cool naturally to room temperature.

3.5. Specimen dimensions and testing details

1. Unstressed residual compressive strength test was performed on 100-mm concrete cubes according to BS 1881: Part 120:1983. Three specimens were tested at each stage and average values are reported.

2. Rapid chloride diffusion test was conducted on 100×50-mm concrete slices to determine permeability and resistance to chloride ion penetration.

3. The porosity and pore size distribution was measured using MIP, which has a measuring pressure ranging from 0.01 to 207 MPa. The contact angle selected was 140°, so the measurable pore size range was from 0.007 to 144 μm. The samples in the form of pellets of about 5 mm in size, consisted of hardened cement paste, were collected from the crushed concrete cubes and immediately soaked in acetone to stop the further hydration. The samples were dried in an oven at 60°C for 48 h before testing.

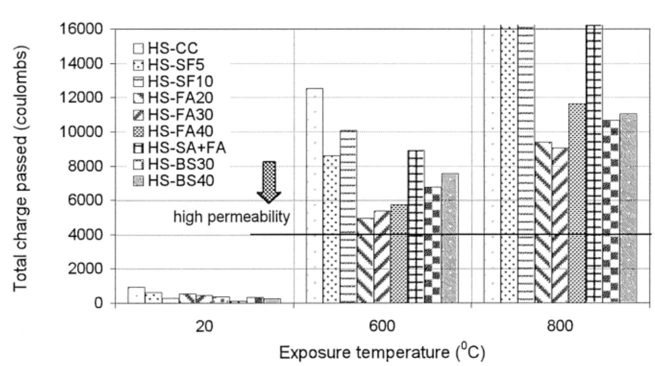


Fig. 5. Resistance of HSC against chloride ion penetration.

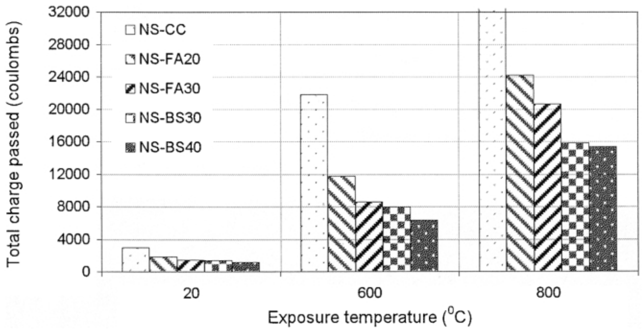


Fig. 6. Resistance of NSC against chloride ion penetration.

4. Crack widths in heated concrete cubes were measured using a digital microscope that can measure the surface crack width up to an accuracy of 1 μm.

4. Test results and discussion

4.1. Residual compressive strength

There are three test methods available for finding the residual compressive strength of concrete at elevated temperatures: *stressed test*, *unstressed test*, and *unstressed residual strength test*. The first two types of test are suitable for accessing the strength of concrete during high temperatures, while the later is excellent for finding the residual properties after the high temperature. It was found that the last method gives the lowest strength and is therefore more suitable for getting the limiting values and hence selected for this research [2]. The residual compressive strength test results are shown in Tables 4 and 5 and Figs. 1–4.

The test results indicate that each temperature range showed a distinct pattern of strength loss or gain. From 20°C to 200°C, the PFA and GGBS concretes showed increases in strength with a greater increase in PFA concretes. The maximum strength gain was shown by the HSC containing 40% PFA, which was 122% of the original strength. A slight strength loss was observed in OPC and CSF concretes. The strength gain was probably due to the formation of tobermorite, which was formed by the reaction between unhydrated PFA or GGBS particles and lime at high temperatures [11]. However, this increase in strength was more pronounced in HSC as compared to NSC. The possible reason is the greater percentage of unhydrated PFA or GGBS particles in HSC due to its dense structure. No visible cracking or spalling was observed in this temperature range.

From 200°C to 400°C, most HSCs maintained their original strength, while a significant decrease was observed in NSCs, which was 19–26% of the original strength. This reduction is due to the pore structure coarsening, which was found to be more in NSC [20]. Again, the pozzolanic concretes performed better and showed higher residual strength. Hairline cracks were observed in CSF concretes, while no spalling occurred in any specimen.

A severe loss in strength was observed in the 400–600°C temperature range. The average loss was 44% in HSC and 60% in NSC. The CSF concretes experienced extensive cracking and spalling and their residual compressive strength was less than the pure OPC concrete. This was probably due to the very dense structure of CSF concrete, which results in a buildup of vapor pressure formed by the evaporation of physical and chemically bound water. The PFA and GGBS concretes performed better and showed no spalling or cracking except hairline cracks. The maximum strength was retained by the HSC containing 30% PFA replacement, which was 67% of the original value. The better performance of PFA and GGBS concretes in this temperature range is due to the reduced amount of $Ca(OH)_2$, which otherwise results in strength loss and disintegration [20].

At 800°C, all the concretes showed severe deterioration due to the decomposition of CSH gel [21]. The average residual strength was 26% for HSC and 17% for NSC.

In each temperature range, HSC maintained a higher percentage value of residual compressive strength than NSC. The reason is the coarsening of the pore structure and increase in pore diameter, which was found to be more

Table 8
Porosity and average pore diameter of high-strength pozzolanic concretes

	Exposure temperatures					
	20°C		600°C		800°C	
Mix	Porosity (%, v/v)	Average pore diameter (μm)	Porosity (%, v/v)	Average pore diameter (μm)	Porosity (%, v/v)	Average pore diameter (μm)
HS-CC	9.52	0.0329	18.32	0.0632	29.13	0.1102
HS-SF5	7.68	0.0294	13.71	0.0526	22.21	0.0938
HS-SF10	5.73	0.0273	9.87	0.0470	32.71	0.1559
HS-FA20	7.73	0.0316	12.77	0.0521	23.31	0.0945
HS-FA30	6.69	0.0309	11.28	0.0522	21.96	0.0921
HS-FA40	6.23	0.0302	10.63	0.0516	22.46	0.0870
HS-SF + FA	5.49	0.0247	8.91	0.0400	30.01	0.1351
HS-BS30	6.29	0.0306	12.71	0.0630	23.01	0.0947
HS-BS40	6.02	0.0299	11.21	0.0562	24.42	0.0899

Table 9
Porosity and average pore diameter of normal-strength pozzolanic concretes

	Exposure temperatures					
	20°C		600°C		800°C	
Mix	Porosity (%, v/v)	Average pore diameter (μm)	Porosity (%, v/v)	Average pore diameter (μm)	Porosity (%, v/v)	Average pore diameter (μm)
NS-CC	14.82	0.0412	26.56	0.0935	42.12	0.1747
NS-FA30	12.17	0.0379	20.29	0.0760	31.09	0.1458
NS-FA40	11.71	0.0358	18.78	0.0688	29.98	0.1281
NS-BS30	11.46	0.0369	19.39	0.0734	28.89	0.1355
NS-BS40	11.08	0.0347	17.87	0.0671	27.79	0.1260

in NSC at elevated temperatures than in the HSC [19]. Due to this effect, NSC showed a gradual decrease in strength, while sharp decrease was observed in HSC between 400°C and 800°C. This topic will be further discussed in the following sections.

The beneficial effect of pozzolans was more pronounced at temperatures below 600°C for both types of concrete. In HSC, PFA concretes gave the best performance followed by GGBS, OPC, and CSF concretes, while in NSC, this sequence was GGBS, PFA, and OPC. The optimum replacement levels were 30% PFA for HSC and 40% GGBS for NSC. Spalling was observed in 28% of CSF concrete specimens and 5% of OPC concrete specimens, while PFA and GGBS concretes only showed visible network of fine surface cracks.

4.2. Resistance against chloride ion penetration

This test determines the electrical conductance of concrete to provide a rapid indication of its resistance to the penetration of chloride ions. The chloride ion resistance of concrete gives an indirect measure of its permeability and internal pore structure, as more current passes through a more permeable concrete. The details of this test can be seen in ASTM C1202-94 and its results can be used to assess the durability of concrete.

The results of rapid chloride diffusion test are summarized in Tables 6 and 7 and Figs. 5 and 6. It is important to note that this test was performed only on specimens subjected to 600°C and 800°C as more damage occurred at those temperatures. Long testing time was another reason to limit the extent of this analysis. Only those specimens that did not show very extensive cracking were selected, as this test is insensitive to cracked specimens.

A close examination of these results indicate a clear relationship between the residual compressive strength and concrete permeability, as more permeable specimens showed more pronounced loss of compressive strength.

In HSC, all specimens showed a very low permeability before subjecting to elevated temperatures. However, as the temperature was increased, a severe loss in impermeability was observed. In percentage of original value, this loss ranges from 500% to 2000% and far more than the loss in compressive strength. The ASTM specifies the concrete as highly permeable if the charge that passes through it is more than 4000°C. Since all the specimens showed higher values than 4000°C after fire, such concretes can be considered as nondurable depending upon the situation, even if they retain a higher proportion of their compressive strength. The NSCs also showed the impermeability loss; however, this loss was smaller than the loss in HSCs.

A comparison among different concretes showed better performance of pozzolanic concretes at elevated temperatures even in the case of CSF. In HSC, the lowest impermeability loss was shown by PFA concretes, while

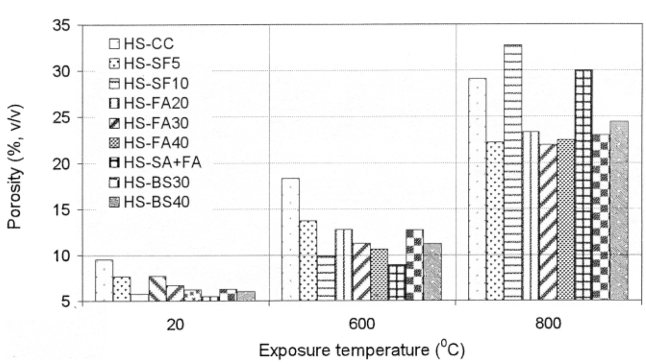


Fig. 7. Residual porosity of HSCs.

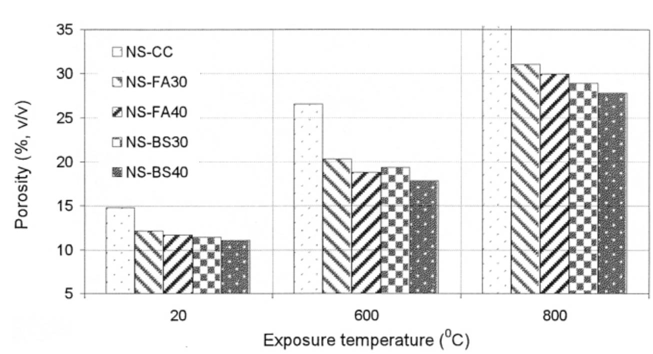


Fig. 8. Porosity of NSCs.

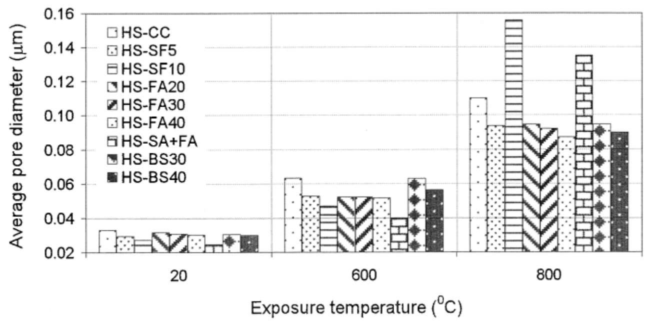


Fig. 9. Average pore diameter in HSCs.

for NSC, GGBS concretes gave the minimum values. This is consistent with the residual compressive strength test results.

4.3. Porosity and pore size distribution measurements

The porosity and pore size distribution were measured using MIP. The test was conducted on specimens subjected to temperatures of 600°C and 800°C due to the reasons mentioned earlier. The results are reported in Tables 8 and 9, and plotted in Figs. 7–10.

The MIP test results clearly indicate an increase in porosity and average pore diameter with the increase in temperature. This effect can be pronounced as coarsening of the pore structure [20,22] and was responsible for the strength and impermeability loss. In both HSC and NSC, a significant decrease in porosity and average pore diameter was observed by the addition of pozzolans as compared to the pure OPC concretes even at elevated temperatures. An exception is the mixes with 10% CSF, which suffered severe internal cracking due to the very dense internal structure.

Among HSCs, PFA concretes had the lowest porosity and average pore diameter at all test temperatures. These values were higher in GGBS concretes having the same mix proportions. This explains why the HS-GGSB concretes showed less relative residual compressive strength at 600°C and 800°C, although their overall residual strengths were higher than the PFA concretes. The hydration reaction of GGBS in concrete is fast due to its high fineness as compared to PFA, and results in a dense structure with higher compressive strength at early ages. With the increase in temperature, this dense structure suffered more loss particularly in terms of durability due to the internal cracking produced by evaporable water and expanded siliceous aggregates.

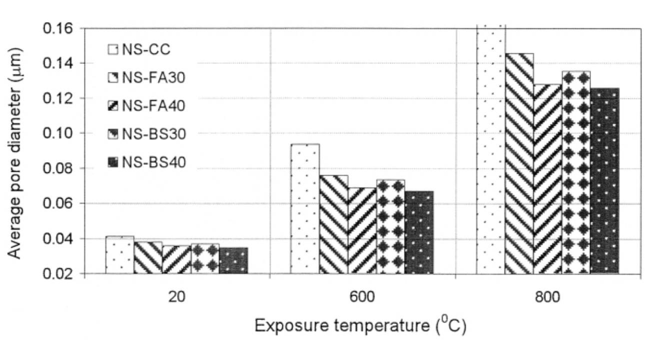


Fig. 10. Average pore diameter in NSCs.

Table 10
Calculated values of worsening indices d_1 and d_2

	Exposure temperature			
	600°C		800°C	
Mix	d_1	d_2	d_1	d_2
HSCs				
HS-CC	0.419	0.620	0.760	0.829
HS-SF5	0.480	0.660	0.790	0.887
HS-SF10	0.560	0.681	0.840	0.913
HS-FA20	0.381	0.605	0.720	0.748
HS-FA30	0.330	0.543	0.680	0.753
HS-FA40	0.430	0.586	0.700	0.764
HS-SF + FA	0.490	0.716	0.810	0.947
HS-BS30	0.470	0.595	0.730	0.792
HS-BS40	0.390	0.587	0.709	0.787
NSCs				
NS-CC	0.699	0.558	0.901	0.731
NS-FA30	0.629	0.500	0.839	0.666
NS-FA40	0.550	0.524	0.820	0.693
NS-BS30	0.489	0.541	0.791	0.651
NS-BS40	0.460	0.520	0.799	0.633

4.4. Integrative analysis of compressive strength and durability

To compare the performance of different pozzolanic concretes integratively, two worsening indices, d_1 and d_2, as defined by Chan et al. [20], are used. The index d_1 is the worsening index of mechanical strength and d_2 is the

Table 11
Surface crack widths in selected concrete specimens

	Crack widths (mm)			
	600°C		800°C	
Mix	Min	Max	Min	Max
HSCs				
HS-CC	0.32	1.26	0.75	1.84
HS-SF10	0.71	1.74	1.30	2.89
HS-FA20	0.15	0.98	0.48	1.32
HS-FA30	0.19	1.09	0.52	1.44
HS-FA40	0.21	1.16	0.52	1.49
HS-BS30	0.21	1.19	0.56	1.46
HS-BS40	0.23	1.24	0.59	1.52
NSCs				
NS-CC	0.28	0.94	0.59	1.44
NS-FA30	0.16	0.95	0.48	0.98
NS-FA40	0.17	1.06	0.52	1.10
NS-BS30	0.16	0.75	0.38	0.82
NS-BS40	0.15	0.90	0.44	0.88

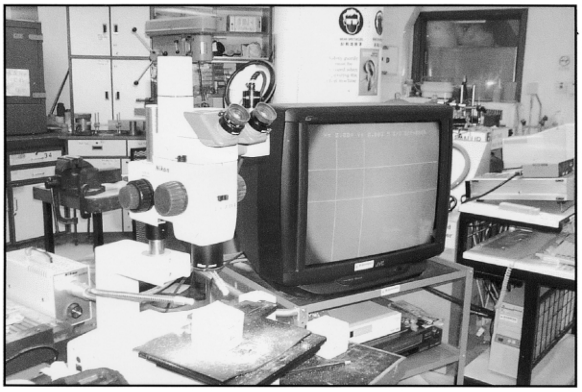


Fig. 11. Cracks observation and measurement apparatus.

worsening index of the permeability-related durability. These indices are defined below in Eqs. (1) and (2):

$$d_1 = 1 - \frac{\text{mean residual compressive strength}}{\text{mean original compressive strength}} \tag{1}$$

$$d_2 = 1 - \frac{\text{cumulative volume of pores larger than } 1.3\ \mu\text{m before fire}}{\text{cumulative volume of pores larger than } 1.3\ \mu\text{m after fire}}. \tag{2}$$

The indices can have values ranging from 0 (*no worsening*) to 1 (*full worsening*). The calculated values of d_1 and d_2 are given in Table 10.

The results clearly support the findings pointed in the earlier sections as:

1. As compared to NSC, HSC suffered a smaller worsening of its mechanical strength but a greater worsening of the permeability-related durability.
2. The CSF concrete showed greater deterioration of strength and impermeability than all other concretes due to its very dense microstructure.
3. In HSC, 30% PFA replacement, while in NSC, 40% GGBS replacement gives the best strength and durability performance at elevated temperatures.

These results also show a severe loss of durability as compared to the compressive strength, and indicate that even after the fire, if concrete retains a high proportion of its original compressive strength, an examination of durability should be made. This is because the loss in durability can result in deterioration of concrete and rusting of rebars, which may reduce the overall life of the structure.

4.5. Crack width and pattern analysis

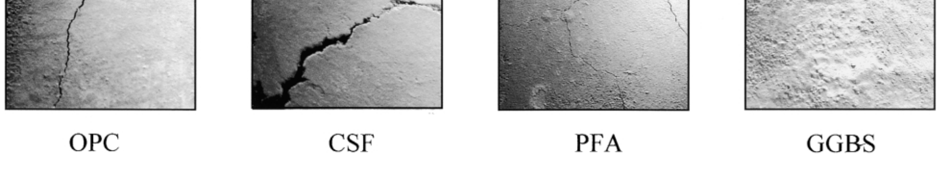
Each pozzolanic concrete showed a distinct pattern of surface cracks regardless of the strength of concrete. The surface cracks started to appear around 300°C and continued to grow till the final rise in temperature. Immediately after cooling, the crack widths were measured using a Nikon digital microscope that can measure the surface crack widths up to 1 μm. The crack widths are reported in Table 11. The test arrangement is shown in Fig. 11, while Fig. 12 shows the typical crack patterns observed in different HSCs. Since almost the same patterns were observed in NSCs, they are not presented here.

The pure OPC concrete specimens showed one or two major cracks usually in the middle of the specimen, but no splitting occurred even at 800°C. In most CSF concrete specimens, usually a single major crack was observed, which often resulted in splitting at 800°C. The PFA and GGBS concretes showed a network of minor cracks, but no major crack was observed that would cause disintegration.

As far as the crack widths after fire are concerned, they were well above the maximum crack widths specified by the ACI (0.10 mm for wet conditions and 0.41 mm for dry conditions). It was also observed that although the surface cracking started around 300°C, the internal cracks commenced after 600°C. As the temperature of the internal concrete during a short heating period is usually lower than the surface temperature, the internal concrete can be considered as durable. This is particularly advantageous for PFA or GGBS concretes, which showed a network of fine cracks that can be refilled by postfire curing [23].

5. Conclusions

1. The pozzolanic concretes showed better performance at elevated temperatures than the pure OPC concretes



OPC CSF PFA GGBS

Fig. 12. Typical crack patterns observed in HSC at 800°C.

except the mixes containing 10% CSF. This better performance was due to the reaction of these pozzolans with free lime, which enhances the strength and durability both at normal and high temperatures by reducing the free lime content.

2. High temperatures can be divided into distinct ranges in terms of effect on concrete strength. In the range of 20–200°C, an increase in strength was observed in PFA and GGBS concretes. At 400°C, most HSCs maintained their original strength, while an average loss of 20% strength was observed in NSCs. After 400°C, both types of concrete lost their strength rapidly and the rate of strength loss was more in HSC.

3. In HSC, the PFA concretes showed the best performance at elevated temperatures followed by GGBS, OPC, and CSF concretes. The mix containing 30% PFA replacement gave the maximum relative residual strength. In NSC, the GGBS concretes gave the best performance followed by PFA and OPC concretes. The 40% replacement level was found to be the optimum.

4. The mechanical strength of HSC decreased in a similar manner to that of NSC when subjected to high temperatures up to 800°C. However, HSC maintained a greater proportion of its relative residual compressive strength than the NSC.

5. The HSC suffered a marginally smaller loss of mechanical strength but a greater worsening of the permeability-related durability than the NSC. Among HSCs, the PFA concretes suffered the least damage in impermeability followed by GGBS, OPC, and CSF concretes. In NSC, the sequence was GGBS, PFA, and OPC concretes.

6. The surface crack pattern gave a good indication about the internal pore structure of the concrete as shown by major cracks in CSF concretes and fine distributed cracks in PFA or GGBS concretes. More surface cracking was observed in HSC than NSC.

7. Severe deterioration and spalling was observed in most CSF concretes and some HS-OPC concretes. Most of the spalling occurred between 400°C and 600°C. No spalling was observed in PFA or GGBS concretes.

Conclusively, the PFA and GGBS concretes were found to be able to retain their properties better at elevated temperatures and can be used in those places where there is a high risk of fire. The CFS concrete with more than 5% replacement should be avoided at such places due to the high risk of explosive spalling.

Acknowledgments

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