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Properties of fly ash-modified cement mortar-aggregate interfaces

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Abstract

This paper investigates the effect of fly ash on strength and fracture properties of the interfaces between the cement mortar and aggregates. The mortars were prepared at a water-to-binder ratio of 0.3, with fly ash replacements from 15 to 55%. Notched mortar beams were tested to determine the flexural strength, fracture toughness, and fracture energy of the plain cement and fly-ash modified cement mortars. Another set of notched beams with mortar-aggregate interface above the notch was tested to determine the flexural strength, fracture toughness, and fracture energy of the interface. Mortar-aggregate interface cubes were tested to determine the splitting strength of the interface. It was found that a 15% fly ash replacement increased the interfacial bond strength and fracture toughness. Fly ash replacements at the levels of 45 and 55% reduced the interfacial bond strength and fracture toughness at 28 days, but recovered almost all the reduction at 90 days. Fly ash replacement at all levels studied increased the interfacial fracture energy. Fly ash contributed to the interfacial properties mainly through the pozzolanic effect. For higher percentages of replacement, the development of interfacial bond strength initially fell behind the development of compressive strength. But at later ages, the former surpassed the latter. Strengthening of the interfaces leads to higher long-term strength increases and excellent durability for high-volume fly ash concrete.

Keywords: Fly ash; Mortar; Aggregate; Bond strength; Fracture toughness

1. Introduction

In concrete prepared with plain Portland cement (PC), the interfaces between the hydrated cement matrix and the aggregates are the weakest links. The interfacial zones generally have a thickness of about 50 μm. They are porous and rich in preferentially oriented calcium hydroxide (CH), but poor in calcium silicate hydrates [1,2]. The formation of these zones is due to (1) the poor packing of the anhydrous materials along the aggregate surface, leading to a locally higher water-to-cement ratio, and (2) the less efficient filling of the hydration products in the interfaces [3,4]. The former also facilitates the crystallization and growth of CH that exhibits low strength between cleavage planes [5].

It is known that some pozzolanic materials (for example, silica fume) can strengthen the interfaces when used in concrete [1,4,6]. According to the results of some studies [7,8], addition of silica fume increased the interfacial bond strength and interfacial fracture energy by about 100%. The interfacial bond improvement effect of such materials is due to their small particle size and pozzolanic reactivity, leading to denser microstructures and stronger interfacial bond.

Fly ash is another type of pozzolanic material widely being used as a cement replacement to produce high-performance concrete and high-volume fly ash concrete [9,10]. Many researchers indicated that low-calcium fly ash (ASTM Class F) also improves the interfacial zone microstructures, although it is generally coarser and less reactive than silica fume. Mehta and Monteiro [11] indicated that fly ash is effective in reducing the thickness of the interfacial zone and porosity in the interfacial zone after prolonged curing. Saito and Kawamura [12] demonstrated that fly ash significantly reduced the degree of orientation of CH crystals and suppressed the precipitation of CH crystals and formation of ettringite in the interfacial zone. Bijen and Selst [13] indicated that fly ash reduced not only the preferential orientation but also the quantity of CH. Besides, Bentz and Garboczi [3] predicted through computer simulation studies that replacing 20% of cement with fly ash with smaller particle size resulted in higher interfacial strength than that of the control Portland cement paste. However, experimental data justifying the improvement effect of fly ash replacements on the interfacial bond are still unavailable, especially when large volumes of fly ash (around 50%) are used.

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The present study concentrated on the interfacial bond behavior between coarse aggregate and mortar-incorporating fly ash. It is a part of a comprehensive investigation on high-volume fly ash concrete carried out at the Hong Kong Polytechnic University (HKPU). Our previous publications [14,15] showed that fly ash increased the porosity of pastes, but reduced the porosity of cement mortars and concrete, and that fly ash contributed to concrete strength more than paste strength [14,15]. These observations imply that fly ash has an effect on the improvement of interfacial microstructure and mechanical bonding; details are presented in this paper.

2. Experimental study

2.1. Test program

The mechanical and fracture properties of PC and fly ash-modified cement mortars were determined first. The flexural strength (f_f^m), Young's modulus (E^m), fracture toughness (critical stress intensity factor K_{IC}^m), and fracture energy (G_F^m) of the mortars were determined from the three-point bending test of notched mortar beams. The compressive strength of the mortars (f_c^m) was determined from the compression tests of mortar cubes.

The properties of the mortar-aggregate interfaces were determined from the three–point bending test of notched beams with mortar-aggregate interface above the notch. The parameters studied included: (1) interfacial flexural strength (f_f^{if}), (2) interfacial fracture energy (G_F^{if}), and (3) interfacial fracture toughness (critical stress intensity factor of interfaces, K^{if}). In addition to the three-point bending test of the mortar-aggregate interface beams, splitting tensile test on mortar-aggregate interface cubes was performed to determine the interfacial splitting tensile strength (f_{sp}^{if}).

Based on the test results, the effects of fly ash on the interfacial bond behavior of concrete were quantified. Further discussions were concentrated on the interfacial bond strength development and internal structure characteristics of high-volume fly ash concrete.

2.2. Materials and specimen preparation

The materials used to prepare the mortars were a commercially available Portland cement equivalent to ASTM Type I cement, a commercially available low-calcium fly ash equivalent to ASTM Class F fly ash, and medium-grade natural river sand. The cement and the fly ash had specific surface areas of 3519 and 3860 g/cm^2, respectively. A naphthalene-based high-range water-reducing admixture was used to obtain proper workability of the mortars. The mortars were prepared in the proportion of 0.3:1:1.5 (water:cementitious materials:sand) by mass. Fly ash was used as replacement of cement, at the levels of 0, 15, 25, 45, and 55% by mass. The model aggregates used for the mortar-aggregate interface specimens were local granites of medium and coarse grain, with Young's modulus of 45.9 and 52.3 GPa, and Poisson's ratio of 0.204 and 0.324, respectively, determined according to the ASTM standard method [16].

The dimension of mortar cubes was 70.7 × 70.7 × 70.7 mm. The dimension of the mortar beam and the mortar-aggregate interface beam was 275 mm in length (L), 50 mm in thickness (t), and 75 mm in depth (d), with a notch of 20 mm in depth (a) and 2 mm in width (Fig. 1a). The test span (S) was 225 mm. Each mortar-aggregate interface beam contained mortar and a thin layer of granite, 6 mm thick (h), which was sandwiched at the midspan. This type of specimen was similar to that used in the work of Lee et al. [17]. The mortar-aggregate interface splitting specimen had the dimension 50 × 50 × 50 mm, each containing mortar and a

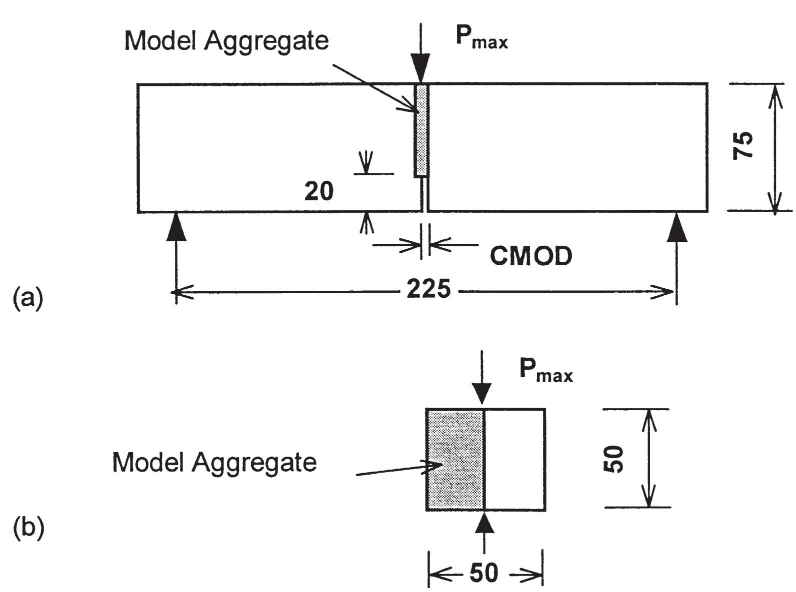


Fig. 1. Mortar-aggregate interface test specimens: (a) notched beam for three-point bending test and (b) for splitting test.

piece of granite 50 mm square by 25 mm thick (Fig. 1b). The model aggregates were cut from core samples of the granites using a diamond saw and were immersed in water before use to become saturated surface dry.

The test setup for the mortar beams was similar to the mortar-aggregate interface beams. All specimens were cast and moist cured in steel moulds at room temperature (21 ± 3°C) during the first 3 days, and then were demoulded and cured in water at 27°C until the time of testing.

2.3. Surface roughness of aggregate layers

The surface roughness of five representative samples of the 6-mm thick aggregate layers, including two samples of medium grain and three samples of coarse grain, were determined by a Talysurf Model 4 surface roughness tester with a sharply pointed stylus tracing the profile of surface irregularities. The measured results were expressed as the values of the Center-Line-Average (CLA). The CLA values reflect the roughness irregularities on the surface and are defined as the average value of the departures, both above and below its centerline, throughout a prescribed sampling length. The tested aggregate layers were found to have average CLA value of 2.21 μm, with a standard deviation of 0.75 μm. The roughness of 25-mm thick square aggregates was similar to the 6-mm thick aggregate layers. A graphic representation of the surface profiles with the vertical height of the irregularities magnified by 1,000 and the horizontal sampling length magnified by 100 is shown in Fig. 2.

2.4. Testing procedures

The compressive tests of the mortar cubes and the splitting test of the mortar-aggregate interface cubes were performed using a Denison compression machine. The loading rates for these two tests were 100 kN/min and 10 kN/min, respectively.

The three-point bending tests were performed using a closed loop servo-controlled MTS testing system. The loading frame had 2,000-kN capacity and the maximum load applied was less than 3 kN. The central load was measured using a 25 kN load cell calibrated for a working range of 5 kN. The crack mouth opening displacement (CMOD) was measured using a crack opening displacement (COD) gauge held by two knife edges, which were glued on the notch mouth of the beam. The midspan deflection was measured using two linear variable displacement transducers. The load was applied at constant rates of CMOD of 0.0002 mm/s for notched mortar beams, and 0.0001 mm/s for notched beams with mortar-aggregate interface above the notch, until the beams were broken. Data were collected at a load interval of 0.01 kN using a data acquisition system.

2.5. Calculations

The flexural strength of mortar (f_f^m) and the flexural strength of interface (f_f^{if}) were calculated from the maximum load ($P_{\max}$) in the three-point bending test, using the net depth (d-a) of the notched beam. The Young's modulus and fracture toughness of the mortars (E^m and K_{IC}^m) were determined from the load-CMOD curves of the three-point bending test of the mortar beams, based on the equations of linear elastic fracture mechanics [18,19]. The mortar fracture energy (G_F^m) and the interfacial fracture energy (G_F^{if}) were determined from the areas under the load-deflection curves of the three-point bending test of the mortar or mortar-aggregate interface beams, according to the RILEM recommendation [20].

The interfacial fracture toughness (K^{if}) was determined based on the theory of interface fracture mechanics for bimaterials [17,21–24]. According to this theory, the interfacial stress intensity factor has the complex form of $K^{if} = K_1^{if} + K_2^{if}$. The two components K_1^{if} and K_2^{if} denote the normal stress intensity and the shear stress intensity, respectively. Details of the determination of K_1^{if} and K_2^{if} are given in the Appendix of this paper.

3. Results

3.1. Properties of PC and fly ash mortars

The test results of the properties of PC and fly ash mortars are shown in Table 1. Three specimens were used for the compression test and four specimens were used for the three-point bending test. It should be emphasized that the flexural strength obtained from the notched beam test was only an approximation, which was affected by the stress singularity at the notch.

The results showed that fly ash replacement at the levels of 15 to 25% did not have a significant effect on the com-

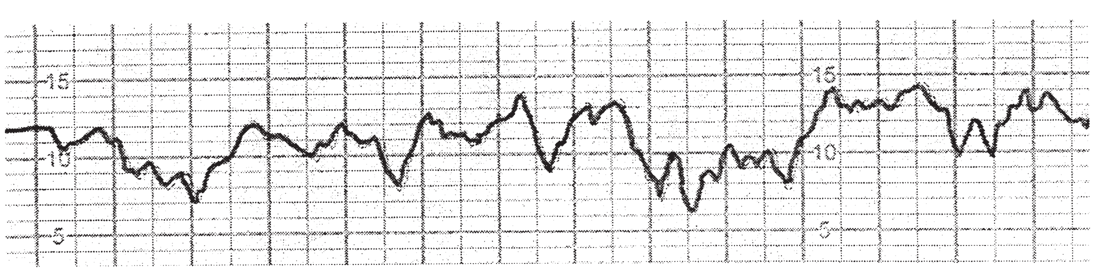


Fig. 2. Typical surface texture profile of the granite aggregate layers (coarse grain granite with CLA = 2.76, the vertical height of the irregularity was magnified by 1,000, and the horizontal sampling length was magnified by 100).

Table 1
Results of the strength and fracture tests of the mortars

Mix	f_c^m (N/mm^2)	P_{max} (KN)	f_f^m (N/mm^2)	E^m (10^{10} N/m^2)	G_F^m (N/m)	K_{IC}^m (10^6Nm$^{-3/2}$)
28 days						
PC	86.8 ± 2.9	2.72 ± 0.18	7.33 ± 0.21	3.46 ± 0.20	94.3 ± 13.0	0.83 ± 0.06
15% FA	86.4 ± 4.4	2.67 ± 0.18	7.21 ± 0.49	3.13 ± 0.20	78.5 ± 13.1	0.83 ± 0.05
25% FA	77.5 ± 6.9	2.62 ± 0.06	7.08 ± 0.17	3.22 ± 0.27	87.0 ± 12.0	0.84 ± 0.02
45% FA	71.3 ± 1.4	2.01 ± 0.20	5.42 ± 0.53	2.79 ± 0.31	57.5 ± 14.2	0.62 ± 0.06
55% FA	66.3 ± 3.1	2.30 ± 0.19	6.20 ± 0.52	3.10 ± 0.19	68.0 ± 10.2	0.70 ± 0.06
90 days						
PC	90.5 ± 9.7	2.87 ± 0.06	7.74 ± 0.17	3.17 ± 0.15	80.0 ± 1.7	0.88 ± 0.02
15% FA	90.0 ± 8.2	2.84 ± 0.22	7.68 ± 0.60	3.36 ± 0.34	74.2 ± 19.2	0.86 ± 0.08
25% FA	99.0 ± 2.6	2.75 ± 0.07	7.43 ± 0.20	3.35 ± 0.04	75.9 ± 7.2	0.84 ± 0.02
45% FA	85.4 ± 8.2	2.41 ± 0.35	6.50 ± 0.95	2.97 ± 0.26	66.0 ± 9.0	0.72 ± 0.12
55% FA	74.9 ± 2.3	2.07 ± 0.09	5.59 ± 0.23	2.79 ± 0.19	75.2 ± 1.8	0.63 ± 0.02

pressive strength (f_c^m), flexural strength (f_f^m), and fracture toughness (K_{IC}^m) of the mortars. Higher fly ash replacements (45 and 55%) resulted in lower values of these properties at both ages. The observations on the strength development of fly ash-modified cement mortars are similar to those for the fly ash concrete at the same water/binder ratio [25].

The fly ash-modified cement mortars at all the replacement levels appeared to have lower fracture energy (G_F^m) than the PC mortars. Prolonged curing from 28 to 90 days resulted in decreased fracture energy for the PC mortar and the mortars with 15 and 25% fly ash, but slightly increased values for the mortars with 45 and 55% fly ash.

3.2. Interfacial bond strength

The results of the three-point bending tests on the mortar-aggregate interface beams are shown in Table 2, and the results of the interfacial splitting test are shown in Table 3. The number of specimens for the three-point bending test is given in Table 2, while three specimens were used for the splitting test. In the preparation of test specimens, coarse-grain granite was used in the majority of interface beams and all interface cubes. The medium-grain granite was used only in about 15% of the interface beams. Different moduli of elasticity and Poisson's ratios of the granites were taken into account in the calculation of interfacial toughness (K_1^{if} and K_2^{if}, see Appendix I). Since these parameters did not appear to affect the results significantly, the specimens with different types of granite were not identified in Table 2. Typical load-deflection curves and load-CMOD curves of the mortar-aggregate interface beams are shown in Fig. 3a and b. In this study, interfacial bond strength was determined from both the three-point bending test and the splitting test, denoted as f_f^{if} and f_{sp}^{if}, respectively. It is noted that the magnitude of f_f^{if} was only about one third those of f_f^m, indicating the weakness of the interfacial bonding between the mortars and the coarse aggregates. Moreover, lower values of f_f^{if} were also due to the brittleness of the interfaces.

It can be seen that compared to the PC specimens, the specimens with 15% fly ash had higher interfacial flexural strength (f_f^{if}) at the ages of 28 and 90 days. However, a 25% fly ash replacement appeared to have no significant effect on the interfacial bond strength at both ages. Higher fly ash replacements had negative effects on the interfacial bond strength at the age of 28 days. The specimens with 45 and 55% fly ash showed the f_f^{if} values to be about 25% lower than those of the PC specimens. At 90 days, the interfacial flexural strength of the specimens with high fly ash contents increased and approached the value of the PC specimens.

The results of splitting tensile strength of the interface f_{sp}^{if} were consistent with those of f_f^{if}. However, the specimens with 25% fly ash showed higher f_{sp}^{if} than the PC specimens

Table 2
Results of flexural tests of mortar-aggregate interface beams

Mortar mix	No. of specimens	P_{max} (KN)	f_f^{if} (N/mm^2)	G_F^{if} (N/m)	K_1^{if} (10^6Nm$^{-3/2}$)	K_2^{if} (Nm$^{-3/2}$)
28 days						
PC	10	0.74 ± 0.32	1.99 ± 0.86	5.44 ± 3.16	0.252 ± 0.109	0.009 ± 0.006
15% FA	9	0.94 ± 0.29	2.54 ± 0.75	9.58 ± 7.77	0.329 ± 0.104	0.014 ± 0.006
25% FA	7	0.74 ± 0.21	1.99 ± 0.58	6.72 ± 4.68	0.256 ± 0.076	0.011 ± 0.004
45% FA	7	0.56 ± 0.15	1.51 ± 0.41	4.34 ± 2.41	0.201 ± 0.068	0.010 ± 0.004
55% FA	3	0.56 ± 0.02	1.50 ± 0.06	5.15 ± 6.57	0.198 ± 0.009	0.010 ± 0.000
90 days						
PC	8	0.91 ± 0.32	2.45 ± 0.86	6.18 ± 1.55	0.317 ± 0.110	0.016 ± 0.005
15% FA	9	1.01 ± 0.16	2.72 ± 0.43	7.69 ± 4.20	0.349 ± 0.054	0.016 ± 0.002
25% FA	8	0.90 ± 0.23	2.43 ± 0.63	10.02 ± 5.02	0.309 ± 0.078	0.014 ± 0.004
45% FA	4	0.87 ± 0.15	2.34 ± 0.40	11.32 ± 0.46	0.307 ± 0.055	0.016 ± 0.003
55% FA	3	0.91 ± 0.10	2.46 ± 0.28	13.68 ± 0.26	0.329 ± 0.042	0.019 ± 0.002

Table 3
Results of interfacial splitting test

Mortar mix	Age (days)	f_{sp}^{if} (N/mm^2)
PC	28	2.42 ± 0.11
15% FA	28	2.63 ± 0.45
25% FA	28	2.92 ± 0.37
45% FA	28	1.77 ± 0.37
15% FA	90	3.23 ± 0.68
45% FA	90	2.45 ± 0.31
45% FA	215	3.77 ± 0.29

and the specimens with 15% fly ash. This was somewhat different from the results of f_f^{if}. Since there were only a limited number of specimens used in the splitting test, the trend in the aforesaid observation should be confirmed when more splitting results are available. The mortar with 45% fly ash showed a weaker bond with the granite at 28 days, but developed a stronger bond at later ages. The test results at 215 days were also available for the specimens with 45% fly ash. The mortar compressive strength (f_c^m) was 94.8 ± 3.4 MPa and the interfacial splitting strength (f_{sp}^{if}) was 3.77 ± 0.29 MPa. It is remarkable that from 28 to 215 days, the interfacial splitting strength between the mortar with 45% fly ash and the granite increased by more than 100%.

The 28-day results of interfacial bond strength are consistent with the observation on the interfacial porosity of mortars, which we reported in another paper [14]. It was found that at the age of 28 days, the interfacial porosity of the mortars was reduced when 15 to 25% fly ash were used, but it was slightly increased when higher percentages of fly ash were used.

3.3. Interfacial fracture toughness and fracture energy

It is noted from Table 2 that the magnitude of the shear stress intensity K_2^{if} is much smaller than that of the normal stress intensity K_1^{if}. This indicates that the fracture of the interface between the cement mortars and the aggregates can be approximately considered as the Mode I fracture (tension mode of fracture), and thus the values of K_1^{if} are comparable with the toughness of mortar (K_{IC}^m).

Similar to the profile of f_f^{if}, the specimen with 15% fly ash showed higher K_1^{if} than the PC specimen and the specimens with higher fly ash contents, at the ages of both 28 and 90 days. At 90 days, the difference in K_1^{if} between the PC specimen and the specimens with high fly ash content was insignificant. The values of K_1^{if} were about one third of the values of K_{IC}^m.

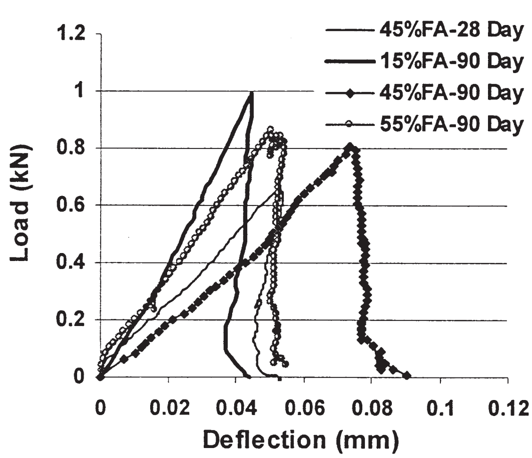
The interfacial fracture energy (G_F^{if}) appeared to be positively affected by fly ash replacement. At the age of 28 days, the specimens with 15 and 25% fly ash had higher G_F^{if} than the PC specimens. At the age of 90 days, fly ash replacements at all the levels studied result in higher G_F^{if} than the PC specimens. It was noted that 55% fly ash replacement resulted in a 90-day G_F^{if} that was double the G_F^{if} of the PC specimen. Relatively higher G_F^{if} of the fly ash-modified mortar-aggregate interfaces may be related to the rougher interfacial fracture surface, with unreacted fly ash particles bonded on the granite surface. This was observed during the tests.

The results showed that the interfacial fracture energy (G_F^{if}) was only about 10% of the corresponding mortar fracture energy (G_F^m). This is consistent with the results obtained by Mistui et al. [8] from the aggregate push out test and by Tschegg et al. [26] from the wedge splitting test. The ratio of G_F^{if} to G_F^m was much smaller than the ratio of K_1^{if} to K_{IC}^m, indicating the fact that fracture energy and fracture toughness reflect different aspects of the interfacial failure.

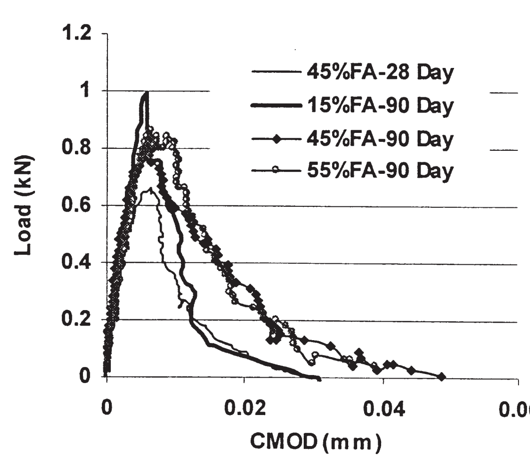
4. Discussion

4.1. Contribution by fly ash to interfacial properties

It is evident that a 15% fly ash replacement enhances the interfacial bond strength (f_f^{if} and f_{sp}^{if}) and fracture properties



(a)



(b)

Fig. 3. Load-displacement curves of mortar-aggregate interface beams: (a) load-mid span deflection curves and (b) load-CMOD curves.

(K_1^{if} and G_F^{if}) at 28 days. The contribution by fly ash to interfacial properties may be mainly due to the pozzolanic reaction. In this study, the fly ash had a specific surface area that was only about 10% higher than that of the Portland cement used. The filler effect of fly ash in this case might not be of any significance. The results of hydration studies at HKPU [27] indicated that the same type of fly ash underwent a degree of reaction of 14% at 28 days and 25% at 90 days when used in a paste with 25% fly ash and a water/binder of 0.3. According to Xu et al. [5], pozzolanic reaction of fly ash has a twofold benefit. It produces calcium silicate hydrates and calcium aluminate hydrates and it consumes $Ca(OH)_2$ that weakens the matrix-aggregate interfaces.

However, with higher volumes of fly ash replacement, the specimens showed lower interfacial bond strength than the PC specimens at the age of 28 days. This was probably due to the lower rate of fly ash reaction in specimens with high volumes of replacement. It has been shown that in a paste with a 55% replacement, the degree of reaction of the fly ash was only about 77% of that in a paste with a 25% replacement at the age of 28 days, and 70% at the age of 90 days [27]. Thus, high volumes of fly ash have a negative effect on the strength of both the matrix and interfaces. Such an effect will be gradually offset by pozzolanic reaction that can take place during prolonged curing.

4.2. Long-term interfacial bond strength development of high-volume fly ash systems

It is interesting to note that the interfacial bond strength of the high-volume fly ash specimens increased proportionally with increasing curing age. Fig. 4 shows a plot illustrating the development of compressive strength and interfacial bond strength of high-volume fly ash mortar and concrete. In this plot, compressive strength of concrete and mortar (f_c and f_c^m) and interfacial splitting strength (f_{sp}^{if}) values are respectively expressed as the percentages relative to the 28-day reference values of corresponding PC specimens. The data of the compressive strength of the PC concrete at the 0.3 water/cement ratio are extracted from our previous paper [25].

Initially, the interfacial bond strength development of the specimens with 45% fly ash fell behind the compressive strength development. At the age of 28 days, the specimens with 45% fly ash developed a compressive strength that was about 80% of the PC mortar and PC concrete, but the interfacial splitting strength was only 70% of the PC specimens. After a long period of curing, the compressive strength development slowed down, while the interfacial bond strength still maintained the rate of increase. From 28 to 215 days, the interfacial bond strength (f_{sp}^{if}) of the specimens with 45% fly ash increased by more than 100%, but the mortar compressive strength increased by only 33%.

Strengthening of interfaces can account for the higher long-term strength increase and excellent durability properties for the high-volume fly ash concrete, as has been demonstrated in many publications [12,28–30].

4.3. Internal structure characteristics of high-volume fly ash concrete

In general, concrete can be considered as a two-phase composite material, with the coarse aggregates embedded in the mortar matrix. The strength of concrete depends on the strength of the matrix, aggregates, and interfaces. In this study, the PC mortars and the fly ash-modified cement mortars are considered as the matrix. Based on the test results, the characteristics of the internal structure of high-volume fly ash concrete are discussed.

It has been noted that with high fly ash contents, the mortars had lower 28-day compressive strength than the PC mortars, and the interfacial bond strength development fell behind the compressive strength development. It can be noted from Fig. 5 that the specimens with 45 and 55% fly ash had lower interfacial bond strength-to-mortar strength ratios (f_f^{if}/f_c^m and f_f^{if}/f_f^m) than those of the PC specimens. Thus, high-volume fly ash concrete at early ages is weak both in the matrix and in the interfaces.

At later ages, the interfacial bond strength development of the specimens with high fly ash contents surpassed the development of the mortar compressive strength (Fig. 4). As the mortar compressive strength development slowed

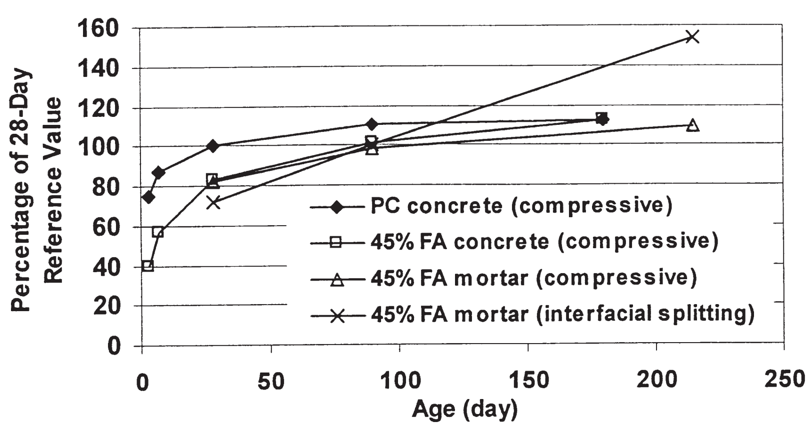


Fig. 4. Compressive and interfacial splitting strength development of high-volume fly ash systems.

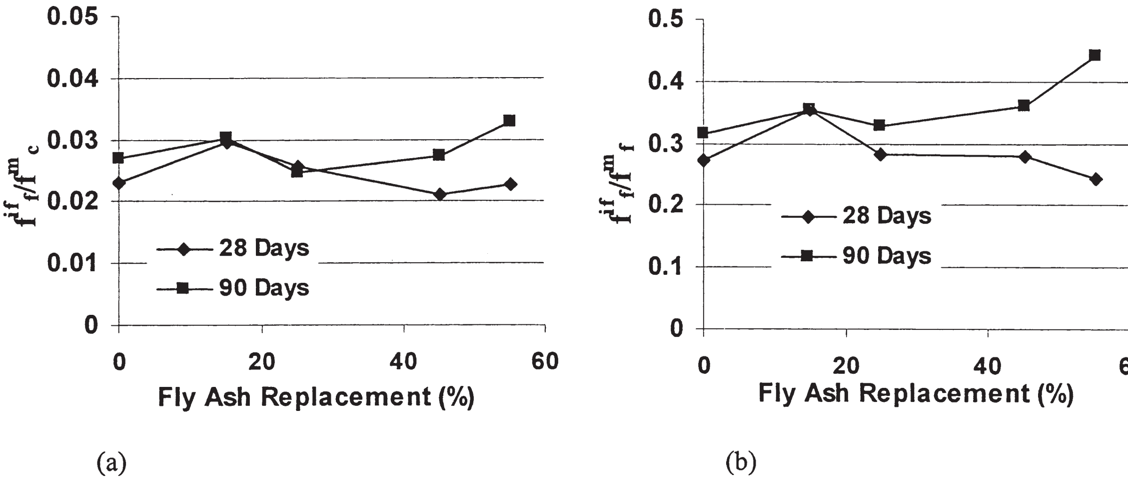


Fig. 5. Effects of fly ash on interfacial bond strength relative to mortar strength: (a) interfacial flexural strength to mortar compressive strength and (b) interfacial flexural strength to mortar flexural strength.

down, the interfacial bond strength continued to increase, leading to higher ratios of f_f^{if}/f_c^m and f_f^{if}/f_f^m at the age of 90 days (Fig. 5).

Moreover, it is also noted that the specimens with 45 and 55% fly ash had lower mortar fracture energy (G_F^m) but higher interfacial fracture energy than the PC specimens (Fig. 6) at 90 days. Thus, after a long period of curing high-volume fly ash concrete is relatively stronger and tougher in interfaces compared to the PC concrete with equivalent strength of matrix.

4.4. Deviation of the present results

Some of the results of the notched mortar-aggregate interface beams showed quite large standard deviations. This was due to the variation in the roughness of the aggregate layers and the difficulties in preparing and handling the specimens. Particularly, the initial conditions of the mortar-aggregate beams had a significant effect on the final results since the link between mortar and aggregate layer was very weak at the initial period of curing. Some precautions have been made, including the use of the saturated-surface dried model aggregates, which showed a better bond with the mortar matrix than the dry aggregates at the initial ages. A large number of specimens were tested, which also made the overall result reasonable.

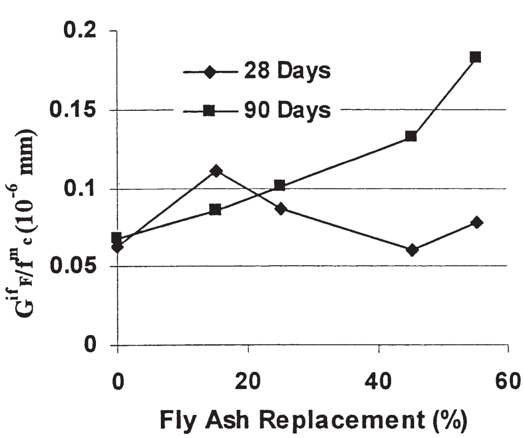


Fig. 6. Effects of fly ash on interfacial fracture energy relative to mortar compressive strength.

5. Summary and conclusions

The effects of fly ash on the mechanical and fracture properties of the cement mortar matrix and the matrix-aggregate interfaces were investigated. A data set of the interfacial flexural strength, the interfacial stress intensity factors in complex form, and interfacial fracture energy were obtained from the notched mortar-aggregate interface beam test. The data of interfacial splitting strength are also presented. The results are generally comparable with available data obtained from other approaches. Based on the results of this investigation, the following conclusions can be drawn:

1. Replacing cement with 15 to 25% fly ash has no significant effects on the compressive strength, flexural strength, and fracture toughness of the mortars at the ages of 28 and 90 days. Higher percentages of the replacement (45 and 55%) have negative effects on these properties.
2. Fly ash replacements seem to result in lower fracture energy (G_F^m) for the mortars. Prolonged curing has a negative effect on G_F^m for the PC mortar and the mortars with 15 and 25% fly ash, but a positive effect on G_F^m the mortars with 45 and 55% fly ash.
3. A 15% fly ash replacement increases the interfacial bond strength (f_f^{if} and f_{sp}^{if}) and interfacial fracture toughness (K_1^{if}) at the ages of 28 and 90 days. Fly ash replacements at the levels of 45 and 55% reduce the f_f^{if}, f_{sp}^{if}, and K_1^{if} at 28 days, but recover almost all the loss at 90 days.

4. Fly ash replacements at all the levels studied result in higher interfacial fracture energy (G_F^{if}) at the age of 90 days.
5. Fly ash contributes to the interfacial properties mainly by the pozzolanic effect. For concrete with high volumes of fly ash, a better interfacial bond requires 90 days of curing. Initially, the interfacial bond strength development falls behind the compressive strength development, but at later ages the former surpasses the latter. Strengthening of the interfaces can account for higher long-term strength and excellent durability properties for the high-volume fly ash concrete.
6. At early ages, high-volume fly ash concrete is weak both in matrix and interfaces, but at the age of 90 days, it is relatively stronger and tougher in interfaces compared to the PC concrete with equivalent matrix strength.

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Appendix I. Calculation of interfacial stress intensity factors

If the thickness of the sandwich layer h is small compared to the crack length and to all other relevant in-plane length quantities, the interface stress intensity factor K^{if} has a universal asymptotic relationship with the stress intensity factor K^{∞} for homogenous problem [24], as shown in Eq. (1):

$$K^{if}h^{i\varepsilon} = \sqrt{\frac{1-\alpha}{1-\beta^2}}K^{\infty}e^{i\omega(\alpha,\beta)} \tag{1}$$

where $\omega = \omega(\alpha,\beta)$, provided by Suo and Hutchinson [24], is given in Appendix II of this paper. The oscillation index ε depends on β according to Eq. (2).

$$\varepsilon = \frac{1}{2\pi}\ln\left(\frac{1-\beta}{1+\beta}\right) \tag{2}$$

where α and β are the moduli mismatch parameters of Dundurs [21], calculated by Eq. (3):

$$\alpha = \frac{\bar{E}^m - \bar{E}^{agg}}{\bar{E}^m + \bar{E}^{agg}},\ \beta = \frac{1}{2}\ \frac{\mu^m(1-2v^{agg}) - \mu^{agg}(1-2v^m)}{\mu^m(1-v^{agg}) - \mu^{agg}(1-v^m)} \tag{3}$$

where $\bar{E} = \dfrac{E}{1-v^2}$,

E, μ, and ν are the Young's modulus, shear modulus, and Poisson's ratio, respectively. The superscript m and agg denote mortar and aggregate, respectively. In the present study, the values of E^m were obtained from the three-point bend test and ν^m was assumed to be 0.2. The parameter α measures the relative stiffness of the two materials and the parameter β causes the linear crack-tip stress and displacement fields to oscillate.

Based on the linear elastic mechanics [18,19] for three-point bending homogenous specimens, mode I fracture is considered, and [see Eq. (4)]

$$K^{\infty} = f_1\sigma_r\sqrt{\pi a} \tag{4}$$

where a is the notch length;

$$\sigma_r = \frac{3PS}{2d^2t}$$

for three-point bend test, with P = maximum load; and f_1 is a geometrical correction factor for the three-point bending specimen, given by Eq. (5)

$$f_1 = \frac{1.99 - \frac{a}{d}\left(1-\frac{a}{d}\right)\left[2.15 - 3.93\frac{a}{d} + 2.7\left(\frac{a}{d}\right)^2\right]}{\sqrt{\pi\left(1+2\frac{a}{d}\right)\left(1-\frac{a}{d}\right)^3}} \tag{5}$$

The interface stress intensity factor is then given by Eq. (6)

$$K^{if} = K_1^{if} + iK_2^{if} = \sqrt{\frac{1-\alpha}{1-\beta^2}}h^{-i\varepsilon}(f_1\sigma_r)\sqrt{\pi a e^{i\omega}} \tag{6}$$

Hence [see Eqs. (7) and (8)]

$$K_1^{if} = \sqrt{\frac{1-\alpha}{1-\beta^2}}(f_1\sigma_r)\sqrt{\pi a}[\cos(\omega - \varepsilon\ln h)] \tag{7}$$

$$K_2^{if} = \sqrt{\frac{1-\alpha}{1-\beta^2}(f_1\sigma_r)\sqrt{\pi a}[\sin(\omega - \varepsilon\ln h)]} \tag{8}$$

Appendix II.

$\omega(\alpha, \beta)$ values in degree [24]

	α								
β	−0.8	−0.6	−0.4	−0.2	0.0	0.2	0.4	0.6	0.8
−0.4	2.2	3.5							
−0.3	3.0	4.0	3.3	1.4					
−0.2	3.6	4.1	3.4	2.0	−0.3	−3.3			
−0.1	4.0	4.1	3.3	2.0	0.1	−2.3	−5.5	−10.8	
0.0	4.4	3.8	2.9	1.6	0.0	−2.1	−4.7	−8.4	−14.3
0.1			2.3	1.1	−0.5	−2.3	−4.5	−7.4	−11.6
0.2					−1.3	−3.0	−4.9	−7.3	−10.5
0.3							−5.8	−7.8	−10.4
0.4									−11.3

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