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Chapter 5

Elevated Temperature Performance of Multiple-Blended Binder Concretes

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Abstract

Concretes that contain binary-blended binders (BBB) and ternary-blended binders (TBB) incorporating thermally activated alum sludge ash (AASA), silica fume (SF), ground-granulated blast-furnace slag (GGBS) and palm oil fuel ash (POFA) are exposed to temperatures as high as 800 °C. The water-binder ratio of the multiple-blended binder (MBB) concretes was 0.30, and the total binder and polypropylene (PP) fibre contents were 493 and 1.8 kg/m^3, respectively. The elevated temperature performance of the MBB concretes is evaluated in terms of the mass loss, compressive strength, ultrasonic pulse velocity (UPV) and surface cracks. The concrete strength deteriorated significantly due to elevated temperature up to 800 °C, but the residual strength of the BBB containing 15 % AASA was higher than that of the control and 20 % AASA concretes. High-temperature exposure decreased measured UPV values. The concrete weight loss was more pronounced for TBB concretes. The elevated temperature performance of all of the TBB concretes was better than that of the BBB concretes with the same AASA replacement levels. It was observed that PP fibres help reduce spalling. BBB concrete containing 15 % AASA combined with either SF or GGBS or POFA exhibits superior performance at elevated temperature than Portland cement concrete at the same mix design proportion.

Keywords: elevated temperature, alum sludge, thermal activation, pozzolanic materials, multiple-blended binders

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1. Introduction

Concrete may be exposed to elevated temperatures upon the outbreak of fire or when they are located near furnaces or reactors. The exposure of concrete to high temperatures (above 200 °C) from accidental fire or elevated temperatures in industrial plants leads to high internal tensile stresses that can cause cracks and damage to concrete structures. The compressive strength of concrete also decreases significantly when subjected to elevated temperatures. Therefore, the residual compressive strength (RCS) is an appropriate factor to consider when assessing the strength of concrete after accidental fire exposure. It is vital for concrete that has been exposed to fire to maintain a high RCS to preserve the safety of the whole structure. The mix proportion, curing period, aggregate type and presence of pozzolanic materials are all features that affect the performance of concrete at elevated temperatures. The use of pozzolanic materials such as metakaolin (MK), silica fume (SF), fly ash (FA), ground-granulated blast-furnace slag (GGBS) and palm oil fuel ash (POFA) in concrete mixtures has been described as an efficient technique for improving the strength and other material properties of concrete. However, the temperature resistance of concrete incorporated with pozzolanic materials such as SF and MK was found to be lower than the resistance of control concrete [1, 2]. Over the last decade, therefore, there has been extensive research into the performance of high-strength concrete (HSC) that incorporate FA, GGBS and SF at elevated temperatures, such as those produced by fire. Phan and Carino [3] compiled experimental reports of the mechanical properties of concrete when exposed to rapid heating, as occurs in a fire. They found that the material properties of HSC vary with temperature differently than do those of normal-strength concrete (NSC) in the range between room temperature and approximately 450 °C and noted that the differences narrow at temperatures above 450 °C. Morsy and Shebl [4] found that a composition including 15 % MK and 5 % SF demonstrated improved fire resistance. It appeared that the MK had a more pronounced effect on the residual compressive strength of the concrete. Behnood and Ziari [5] showed that the addition of SF had no significant effect on the relative residual compressive strength when the concrete was subjected to temperatures of 100 and 200 °C, but that the amount of SF had considerable influence on the residual compressive strength when the concrete was subjected to temperatures of 300 and 600 °C.

Phan et al. [6] reported that high-performance concrete (HPC) with higher original strength (lower w/c) and with SF retained more residual strength after exposure to elevated temperature than did those HPC with lower original strength (higher w/c) and without SF. There was a significant reduction in the weight of the specimen and the relative strength of the concrete at elevated temperatures (200–1200 °C) [7]. Ghandehari et al. [8] evaluated the residual mechanical properties of HPC after exposure to elevated temperatures by using SF. They found that after heating the concrete to 200 °C, the strength of all of the concrete samples showed a slight improvement compared with the strength of concrete at 100 °C. Mohammad et al. [9] reported the residual compressive strength of concrete containing 20 % POFA after exposure to elevated temperatures and subsequent cooling. They found that there was a continuous decrease in the residual compressive strength with increasing temperature. The highest reductions were observed in ordinary Portland cement (OPC) concrete: 22.5, 33 and 78 % at 300, 500 and 800 °C,

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respectively. They also found that residual performance was higher in POFA concrete than in the OPC concrete.

Demirel and Keleştemur [10] demonstrated that adding pozzolanic materials (finely ground pumice (FGP) and SF) to concrete decreased both unit weight and compressive strength. Morsy et al. [11] evaluated the effects of high temperature on cement paste mixes containing 15–30 % MK and 5–15 % SF OPC replacement. They concluded that the best performance was achieved with cement paste containing 10 and 15 % SF, which increased in strength at 400 °C by 39 and 48 % but decreased in strength at 600 °C by 33 and 43 %, respectively. Rahel et al. [12] studied the replacement of cement by high-volume fly ash combined with colloidal nanosilica to produce high-strength mortars after exposure to temperatures of 400 and 700 °C. They found that high-strength mortars that have equivalent residual strength after exposure to 700 °C to that of control (unheated) cement mortar specimens can be produced by replacing cement with high-volume fly ash and by using colloidal nanosilica. Because of the increasing use of HSC in columns, resistance to spalling has become one of the crucial components of effective fire resistance [13]. Surface spalling occurs when a low-permeability paste is subjected to a high rate of heating and the vapour pressure in the pores consequently develops stresses greater than the material's tensile strength [14].

The internal stresses in compression members make them more vulnerable to spalling. In particular, there is a greater risk that HPC, with its lower permeability, will spall at high temperature compared with conventional concrete. To combat the spalling effect in HSC, it is necessary to add polypropylene (PP) fibres to the concrete mixes. Polypropylene fibres melt at approximately 160–170 °C and become capable of producing moisture escape channels to release the vapour pressure. One major consideration in the design of buildings is the safety of the occupants in case of an outbreak of fire. As such, thorough knowledge of the behaviour of all construction materials is required before incorporating them into structural elements. Additionally, the growing prevalence of engineering structures characterized as large span, high rise and ultra-high rise has necessitated a continual increase in performance requirements.

Water treatment plants (WTPs) produce waste residual sludge, alum sludge (AS), when aluminium sulphate is used as a coagulant in the process of making drinking water for human consumption. Thermally activated alum sludge ash (AASA) is a new pozzolanic material that is acquired from the calcination of AS at 800 °C. Owaid et al. [15] studied the feasibility of using AASA as a pozzolanic material for replacing cement in binary-blended binder (BBB) and ternary-blended binder (TBB) concretes that incorporate SF, GGBS and POFA. According to their results, AASA exhibits pozzolanic behaviour and can be classified as a Class natural (N) pozzolan [15]. The BBB containing 15 % AASA increased the compressive and tensile strength of concrete up to 85.3 and 5.38 MPa at 28 days, respectively, but further increases in AASA content gradually reduced these strengths [15]. The mechanical properties of the ternary combinations are better than those of the binary mixes at the same AASA replacement levels [15].

The performance of multiple-blended binder (MBB) concretes incorporating AASA, SF, GGBS and POFA at elevated temperature is presented in this chapter. The fire resistance of MBB concretes with thermally activated alum sludge ash (AASA) and pozzolanic materials as

replacements for cement in both binary and ternary blends can be used to determine the suitable application of the concrete.

2. Experimental programme

2.1. Materials

The cement used in the concrete mixtures was ordinary Portland cement (OPC) type I from Orang Kuat Berhad, which conforms to ASTM C150-1992 [16].The chemical composition and physical properties of OPC are as reported previously [15]. Alum sludge (AS) is the raw material used in the present research that was obtained from the drinking water purification process. The AS was collected at the ABASS Consortium water treatment plant and then oven dried at 105 °C for 24 h. The dried sludge was crushed and sieved through a 10 mm sieve to remove coarse and foreign particles.

In a previous study by Owaid et al. [15], an effective way of preparing thermally activated alum sludge ash (AASA) by thermal activation of AS was to incinerate dry alum sludge in a laboratory electric furnace at 800 °C for a period of 2 h with the heating rate of 5 °C/min. The chemical composition and physical properties of the AS and AASA are as given in Ref. [15]. Three types of pozzolanic materials, namely, SF, GGBS and POFA, were employed as a partial replacement of OPC by weight in different combinations of binary and ternary cementitious blends. The type of condensed silica fume (SF) was Force 10,000 D microsilica. The chemical and physical properties of these materials have previously been reported [15].

Fine and coarse aggregates obtained from local sources were in accordance with the ASTM standard. The local natural sand used as a fine aggregate had a maximum aggregate size of 4.75 mm and a fineness modulus of 2.89. The maximum size of the local coarse aggregate (crushed granite) was 10 mm; its specific gravity was 2.64, and its water absorption value was 0.48 %. The fine aggregate had a specific gravity of 2.61 and a water absorption value of 0.72 %. The superplasticizer (SP) used in this study was an aqueous solution of modified polycarboxylate-based superplasticizer (ViscoCrete-2044). The specific gravity of 1.08 was utilized to achieve the desired workability in all HPC mixtures. The polypropylene (PP) fibres used in this study were obtained from Timuran Engineering and bore the brand name of fibrillated polypropylene fibre. They were white and 12.19 mm long, and they had a specific gravity of 0.9. The PP fibres were used to eliminate the spall effect for all specimens that were subjected to high temperatures and were added at an amount of 1.8 kg/m^3 for concrete mixes.

2.2. Mix proportions and preparation of specimens

The mix proportions were based on recommendations by Owaid et al. [15]. **Table 1** shows the mix proportions for both categories of concrete. Nine types of concrete mixtures were prepared to explore the effects of elevated temperatures on the properties of MBBC specimens containing AASA in both binary- and ternary-blended cement with the same binder content of 493 kg/m^3, and the ratio of water-binder (w/b) was kept at 0.30. The aggregates used were

in accordance with ASTM standards and comprised crushed granite gravel with a nominal maximum size of 10 mm and local natural sand with a maximum size of 4.75 mm. The superplasticizer used was an aqueous solution of modified polycarboxylates with two OPC mass fractions, 1.5 and 1.8 %.

Mix description (%)	w/b	Water (kg)	Cement (kg)	AASA (kg)	SF (kg)	GGBS (kg/m^3)	POFA (kg)	FA* (kg)	CA** (kg)	SP*** dosage (%)	PP**** (kg)
Control-OPC	0.30	147.9	493	0	0	0	0	684	1027	1.5	1.8
AASA15	0.30	147.9	419.0	74.0	0	0	0	684	1027	1.5	1.8
AASA20	0.30	147.9	394.4	98.6	0	0	0	684	1027	1.5	1.8
SF6	0.30	147.9	463.4	0	29.6	0	0	684	1027	1.5	1.8
GGBS20	0.30	147.9	394.4	0	0	98.6	0	684	1027	1.5	1.8
POFA15	0.30	147.9	419.0	0	0	0	74.0	684	1027	1.5	1.8
AASA20 SF6	0.30	147.9	364.8	98.6	29.6	0	0	684	1027	1.8	1.8
AASA20GGBS20	0.30	147.9	295.8	98.6	0	98.6	0	684	1027	1.8	1.8
AASA20POFA15	0.30	147.9	320.4	98.6	0	0	74.0	684	1027	1.8	1.8

FA* stands for fine aggregates; CA* for coarse aggregates; SP*** for superplasticizer; PP**** for polypropylene fibres.

Table 1. Concrete mix proportions [15].

The content of polypropylene fibres was 1.8 kg/m^3 for all of the concrete mixtures that contained AASA to eliminate the spall effect. All concrete materials were mixed in a rotating pan mixer for approximately 5 min to conform with the mixing process described in ASTM C192-2002 [17]. The mixtures were cast into specimens by using 100 mm standard cube moulds and were compacted with a vibrating table to reduce the air voids content in mixes. The specimens were subsequently covered to prevent evaporative water loss. After casting, the moulded specimens were left in the casting room at a temperature of 26 °C for 24 h. After demoulding, the specimens were cured in a water tank for 28 days. Later, they were removed from the tank and left in the laboratory to air and cure naturally for up to 56 days, under similar conditions of temperature and relative humidity.

2.3. Heating and cooling regimens

After a curing period of 56 days, the concrete specimens were conveyed to an electrical furnace. The specimens were kept in the furnace for 3 h at maximum temperature; the rate of temperature increase in the automatic electric furnace was 5 °C/min [18], as observed in **Figure 1**. Subsequently, the power was shut off, and the specimens remained in the furnace until the temperature dropped to room temperature to prevent the specimens from experiencing thermal shock. For this research study, the concrete specimens were heated in an electric furnace to 400, 600 and 800 °C.



Figure 1. Automatic electric furnace used to heat concrete specimens.

2.4. Testing procedures

The residual properties of the unheated control mix were compared with the properties of the specimens for multiple-blended binder (MBB) concretes containing OPC, AASA and pozzolanic materials. Also, the crack patterns on the surface of the multiple-blended binder concretes were inspected after the heated specimens had cooled down. An electronic digital balance with an accuracy of ±0.1 g was used to determine the concrete mass loss ($M_{\mathbf{loss}}$) before and after each heating temperature. The calculation of the concrete mass loss of specimens was based on Eq. (1); subsequently, specimens were taken out from furnace and weighed (W_d). After the concrete specimens had cooled to room temperature, they were removed from the furnace to determine their mass loss, residual compressive strength and ultrasonic pulse velocity of MBB concrete specimens. For each type of concrete, the residual properties were subsequently compared with the properties of the unheated control specimens. Additionally, crack patterns on the surface of the MBB concrete specimens were inspected after the heated specimens had cooled down. An electronic digital balance with an accuracy of ±0.1 g was used to determine the concrete mass loss ($M_{\mathbf{loss}}$) before and after each heating step and the final weight (W_d). Calculation of the concrete mass loss of specimens was based on Eq. (1):

$$M_{loss} = \frac{M_{initial} - M_{heated}}{M_{initial}} \tag{1}$$

where $M_{\mathbf{initial}}$ and $M_{\mathbf{heated}}$ are the initial mass (before heating) and heated mass (after heating) weighed in air, respectively. The compressive strength of the concrete was determined by crushing three 100 mm^3 cubes for each mix. The test was carried out according to BS EN 12390-3 [19]. The compression load was applied at a rate of 3 kN/s using a compression machine with

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a capacity of 5,000 kN. The residual compressive strength (RCS) was calculated using the following Eq. (2):

$$RCS = \frac{\sigma_{elev}}{\sigma_{26}} \times 100 \tag{2}$$

where σ_{elev} is the compressive strength (MPa) of the cubes subjected to elevated temperature and σ_{26} is the compressive strength (MPa) of the cubes kept at room temperature (26 °C).

The portable equipment for ultrasonic non-destructive indicating test (known as PUNDIT) was used to measure ultrasonic velocity (V), in accordance with BS 1881: Part 203 (1986) [20]. Ultrasonic velocity (V) was determined by measuring the ultrasonic pulse transmission time by means of the direct transmission method. The ultrasonic pulse transmission time is the time required for longitudinal vibrations of ultrasonic frequency to travel a known distance through the material. Wave speed was calculated as:

$$V = \frac{L}{T} \tag{3}$$

where L is the transmission distance (m), T is the transmission time in the concrete (s), and V is the velocity of pulse transmission in the concrete (m/s) [20].

3. Results and discussion

The saturated surface-dry specimens were heated to 400, 600 and 800 °C for 3 h and subsequently cooled to room temperature (26 °C). **Figures 2–11** show the results obtained from the tests (losses in weight, compressive strength and ultrasonic pulse velocity) of MBB concrete specimens subjected to elevated temperatures. The finding is analyzed and discussed below.

3.1. Concrete mass loss for different elevated temperatures

Figure 2 and **Table 2** present the percentage of the mass loss relative to the original weight (weight before heating) of the mixtures of OPC, AASA, SF, GGBS and POFA in the binary- and ternary-blended concrete with increasing temperature. It can be observed that the mass loss in all specimens showed a gradual increase from 3.72 to 7.41 % with the increase in temperature from 400 to 800 °C. The results also show that the mass loss increased with increasing amounts of AASA and pozzolanic materials. It is observed that the mass loss was less than 7.5 % for the specimens that did not exceed 800 °C. After being subjected to temperatures of 400, 600 and 800 °C, the mass losses of OPC concrete were 3.78, 5.24 and 6.46 %, respectively. Additionally, the mass losses of AASA 15 % and AASA 20 % concrete were measured to be 3.98, 5.41 and 6.84 % and 4.1, 5.82 and 7.06 %, respectively. This small difference

in the mass loss has not caused significant effects when the AASA replacement ratios are at 15 and 20 %.

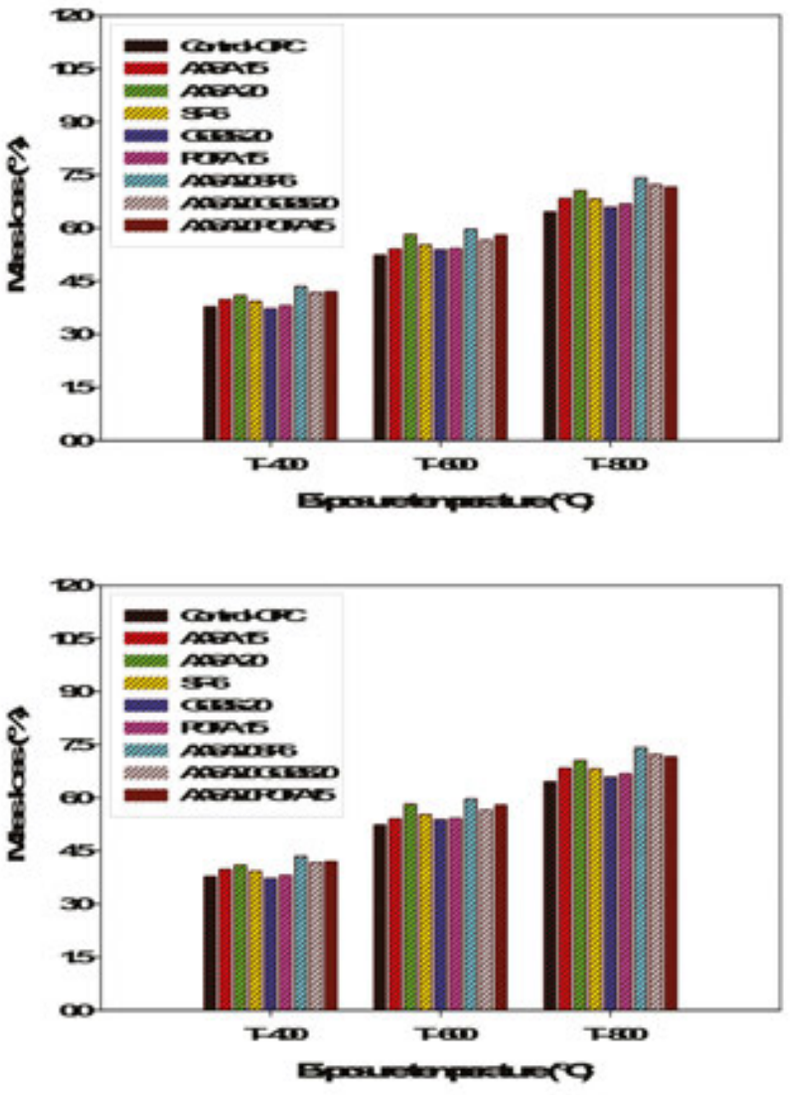


Figure 2. Mass loss of BBB and TBB concrete mixtures subjected to different elevated temperatures.

Mass loss occurs in the specimens due to the loss of water. Because of the loss of bound water from the cement paste, air voids are formed in the concrete. The structural integrity of the specimens deteriorates corresponding to the increase in mass loss with increasing temperature. Similar observations were reported by Janotka and Nurnbergerova [21]. The rate of concrete mass loss was slower when the heating temperature increased from 400 to 800 °C. These findings of the current study agree with observations by Hanaa et al. [22], who reported that approximately 70 % of the water contained in the concrete had evaporated at 300 °C. **Figure 2** shows that the incorporation of SF, GGBS and POFA at 6, 20 and 15 % by total binder

weight, respectively, resulted in a decrease in mass loss of the pozzolanic material concrete with increasing temperature. The observations show that there were higher mass losses in SF concrete compared with GGBS and POFA concretes. The mass loss associated with the SF, GGBS and POFA concretes was 3.93, 3.72 and 3.81 % at 400 °C; 5.53, 5.39 and 5.42 % at 600 °C; and 6.82, 6.59 and 6.68 % at 800 °C, respectively. Similar observations were reported in various studies [9, 10, 23].As observed, all of the ternary binder mixtures shown in **Figure 2** lost more mass compared with the OPC concrete. In this study, the weight losses in the concrete containing both AASA and SF were higher than the losses in AASA + GGBS and AASA + POFA concrete with an increasing temperature from 400 to 800 °C. This result can be attributed to the specific gravity of the selected pozzolanic materials, with the specific gravity of SF being the lower than that of GGBS and POFA. The average mass loss of the ternary-blend binder concrete was 4.2 % at 400 °C, 5.8 % at 600 °C and 7.3 % at 800 °C.

Mix description (%)	Percentage of the mass loss relative to the original weight at different heating regimens (°C) (%)		
	26–400	400–600	600–800
Control-OPC	3.78	5.24	6.46
AASA15	3.98	5.41	6.84
AASA20	4.10	5.82	7.06
SF6	3.93	5.53	6.82
GGBS20	3.72	5.39	6.59
POFA15	3.81	5.42	6.68
AASA20 SF6	4.35	5.97	7.41
AASA20 GGBS20	4.17	5.66	7.23
AASA20 POFA15	4.20	5.80	7.16

Table 2. Percentage of the mass loss relative to the original weight of the BBB and TBB mixtures.

3.2. Residual compressive strength of concrete subjected to elevated temperatures

Figures 3–5 show the results of residual compressive strength (RCS) measurements of the specimens for multiple-blended binder concretes containing OPC, AASA and pozzolanic materials that were subjected to elevated temperatures followed by cooling in ambient air until the 56th day after the heating process. **Figures 6–8** indicate the relative residual compressive strength (ratio of residual compressive strength after elevated temperature to initial compressive strength at room temperature) of the concrete specimens. The residual strength of the MBB concrete specimens decreased as the temperature was increased. **Figure 3** shows that regardless of the presence of PP fibres, the strength of AASA concrete deceased when the temperature was increased from 400 to 800 °C. Hence, there was a significant increase in the compressive strength of the binary concrete blends with 15 % AASA at room temperature (26 °C) compared with the compressive strength of the control mix and concrete mixture with 20 % AASA. The

increase in compressive strength of the concrete mixture with 15 % AASA was approximately 12.4 % compared with the compressive strength of the control mix. However, the concrete with 20 % AASA cement exhibited reduced compressive strength.

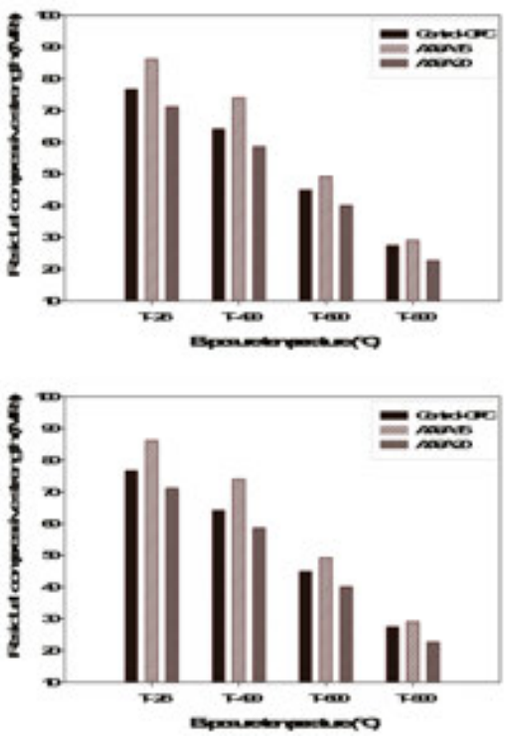


Figure 3. Influence of AASA on the compressive strength of concrete subjected to different elevated temperatures.

The change in the strength of concrete specimens appeared to follow a common trend. Initially, as the temperature was increased to 400 °C, the strength decreased relative to that at room temperature; the relative decrease was approximately 14.2–17.7 %, as shown in **Table 3**. This effect could be due to variations in pore structure, including porosity and pore size distribution, or to an increase in pore diameter [24]. According to several researchers [25–27], this reduction is primarily due to the release of free and physically bound water from the pores of the hydrates and to the first stage of dehydration in the hydrated products (calcium hydroxide, calcium silicate hydrates and calcium aluminosilicate hydrates) and the breakdown of tobermorite gel. The concrete containing 15 % AASA performed better and showed higher residual strength compared with the control-OPC and 20 % AASA concretes. A severe loss in strength was observed in the concrete as the temperature increased from 400 to 600 °C. The average strength loss was of 42.6 % (**Table 3**). The quick loss in compressive strength for concrete mixtures has been attributed to the dense microstructure of this type of concrete, which is the direct cause of the excessive build-up of vapour pressure. This pressure produces large cracks in the specimens during heating. The decomposition of calcium hydroxide and calcium carbonate in cement paste, which occurs from 430 to 600 °C, is an additional reason for the loss of strength [26, 28].

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Mix description (%)	Percentage of the strength loss relative to the original weight at different heating regimens (°C) (%)		
	26–400	**400–600**	**600–800**
Control-OPC	12.3	41.3	64.1
AASA15	14.2	42.9	66.2
AASA20	17.7	43.6	68.1
Average	**14.7**	**42.6**	**66.1**
SF6	15.7	43.3	65.2
GGBS20	14.8	42.1	63.4
POFA15	16.1	42.6	64.7
Average	**15.5**	**42.7**	**64.1**
AASA20 SF6	17.6	45.8	69.7
AASA20 GGBS20	16.2	44.2	66.1
AASA20 POFA15	17.7	45.3	67.3
Average	**17.2**	**45.1**	**67.7**

Table 3. Percentage of the strength loss relative to the original weight of the BBB and TBB mixtures.

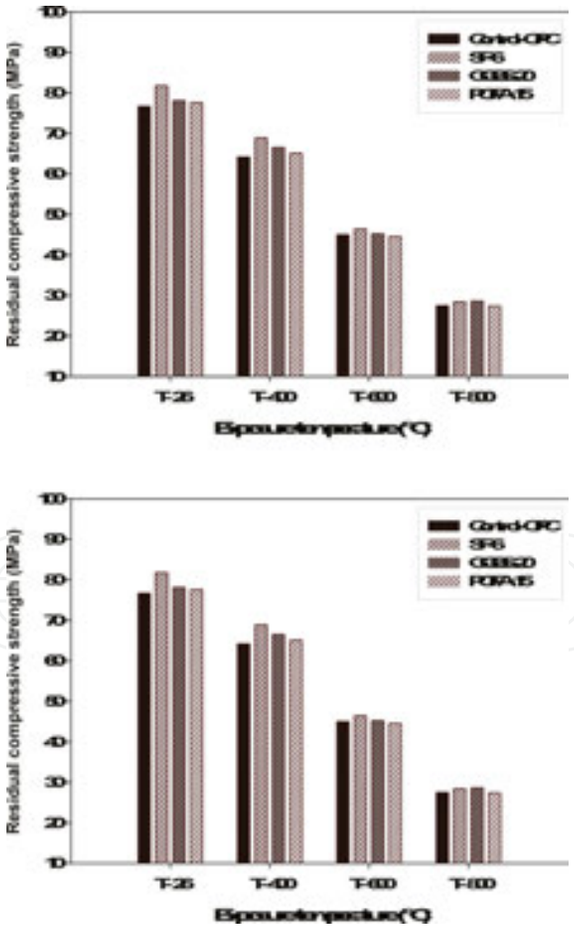


Figure 4. Influence of pozzolanic materials on the compressive strength of concrete subjected to different elevated temperatures.

All types of the MBB concretes showed severe deterioration at high temperatures between 600 and 800 °C temperature ranges, with further reduction in the strength. This reveals that the decomposition of C—S—H greatly affects the loss in the strength of concrete with severe deterioration for all mixtures of concrete [1]. The average loss of strength for the heated specimens was 66.1 %, compared with the strength of the unheated specimens. Thus, the sharp reduction in strength may be due to the formation of microcracks in the specimen, which weakens the interfacial transition zone and bonding between the aggregate and the cement paste. Therefore, the contractions of the paste lead to appear the cracks following the loss of water and expansion of the aggregate [29, 30].

Concrete mixtures that contained AASA exhibited extensive cracking and spalling, and their residual compressive strength was less than that of the control mixture. These effects are attributed to the presence and amount of filler additives in concrete mixtures that produce very dense transition zones between aggregates and paste due to their ultra-fine particles as filler materials and their pozzolanic reactions. During the expansion of the aggregates and contraction of the paste, higher stress concentrations are produced in the transition zone. These stresses worsen the bonding between aggregate and paste that contains filler additives compared with that of the control mixture [27]. The residual strength at 800 °C ranges from 23 to 29 % relative to unheated controls, as shown in **Figure 3**.

Figure 4 presents the changes in the compressive strength of concrete that contained 6, 20 and 15 % of SF, GGBS and POFA, respectively, with increasing temperature from 400 to 800 °C. The observations show a decrease in the strength of the pozzolanic concretes at temperatures from 26 to 400 °C. This drop in strength is quantified as a 14.8–16.1 % reduction of the original strength, as given in **Table 3**. Once again, the concrete containing pozzolanic materials performed better and exhibited higher residual strength [1, 27].

Generally, it can be concluded that the loss in strength of pozzolanic concrete is caused by the dense microstructure in this type of concrete, which causes the build-up of high internal pressures due to the water-vapour transition in the water interlayer. The observations also reveal a severe loss in strength for all four types of concrete at temperatures ranging from 400 to 600 °C. This loss recorded is 41.3, 43.3, 42.1 and 42.6 % of the initial values for OPC, SF6, GGBS and POFA concretes, respectively (see **Table 3**). When the concrete specimens are subjected to high temperatures, the cement paste contracts and the aggregates expand. This response causes the transition zone to weaken and results in bonding between aggregates and cement paste. As a result, this process as well as the chemical decomposition of hydrated products causes severe deterioration and loss of strength in concrete after exposure to high temperatures. All the concrete containing pozzolanic materials exhibited severe deterioration up to 800 °C, and the average loss in strength was 64.1 % because of the decomposition of C—S—H gel. A similar observation has been reported by Demirel and Kelestemur [10]. From their results, it is shown that the pozzolanic material exerts a considerable influence on the residual strength. The concrete mixture containing SF performed poorly compared with the concrete mixtures containing GGBS and POFA. Although the addition of SF increased the initial strength of the concrete, there was a considerable compressive strength loss when the concrete was subjected to high temperatures. This decrease in strength likely arose from the

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very dense structure of SF concrete, which resulted in a build-up of vapour pressure due to the evaporation of physically and chemically bound water.

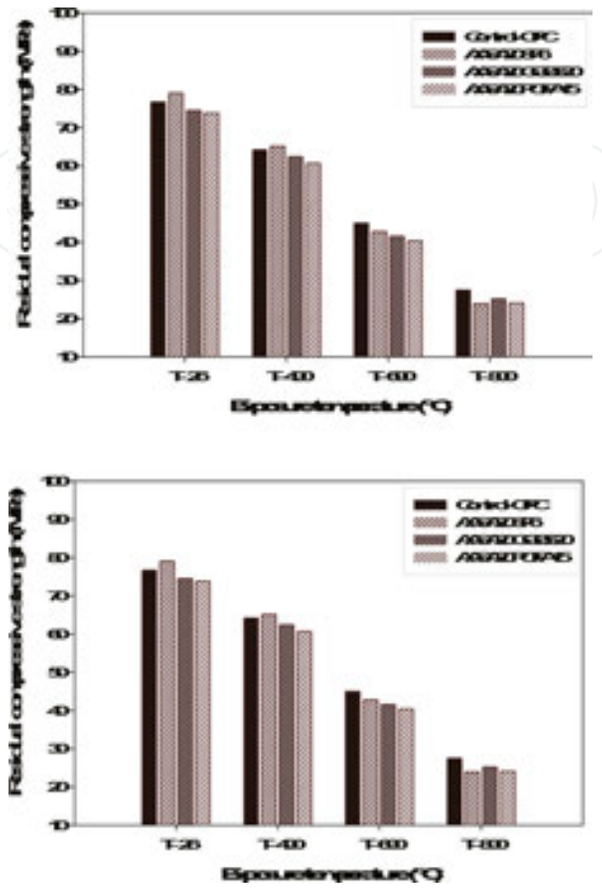


Figure 5. Influence of ternary-blended binders on the compressive strength of concrete subjected to different elevated temperatures.

Figure 5 illustrates the comparison of the residual compressive strengths of ternary-blend binder concrete specimens and the control specimens that were subjected to elevated temperature. It is clear that the residual compressive strength decreased as the treatment temperature increased to 800 °C. The observations show that the performance of the ternary blends with AASA and pozzolanic materials in terms of compressive strength is better than the performance of binary blends with AASA for the same replacement levels of the unheated specimen (26 °C). **Figure 5** shows that the OPC + AASA + SF mix exhibited the highest strength, followed by OPC + AASA + POFA and OPC + AASA + GGBS. This result is due to the transformation of calcium hydroxide, which leads to the formation of calcium silicate hydrate on the surface of the aggregate particles. This transformation occurs because the average particle size of SF is very small compared with the particle sizes of other pozzolanic materials. This has led to the refinement of grains in ternary mixtures that contain AASA. From 26 to 400 °C, a decrease was observed in the performance of ternary blends in terms of the loss of compressive strength in mixtures containing AASA and pozzolanic materials. The loss in strength is from 16.2 to 17.7 % of the original strength, as shown in **Table 3**.

The OPC + AASA + SF mix exhibited the highest loss in strength, followed by OPC + AASA + POFA and OPC + AASA + GGBS. This loss is due to the ultra-fine particles of SF that are used as fillers, along with the pozzolanic reactions of the particles. All concretes that contain ternary blends of AASA and pozzolanic materials lose their strength at a faster rate when they are subjected to temperatures ranging from 400 to 600 °C. The loss is from 44.2 to 45.8 % of the original strength. At these temperatures, the dehydration of the cement paste results in its gradual disintegration. Because the paste tends to shrink and aggregates tend to expand at high temperatures, the bond between the aggregate and the paste is weakened, thereby reducing the strength of the concrete. At 800 °C, the average strength loss is 67.7 % for the ternary-blend binder concrete (see **Table 3**). The test results revealed that all of the tested concretes deteriorated at a temperature over 600 °C, as indicated in previous studies [10, 31].

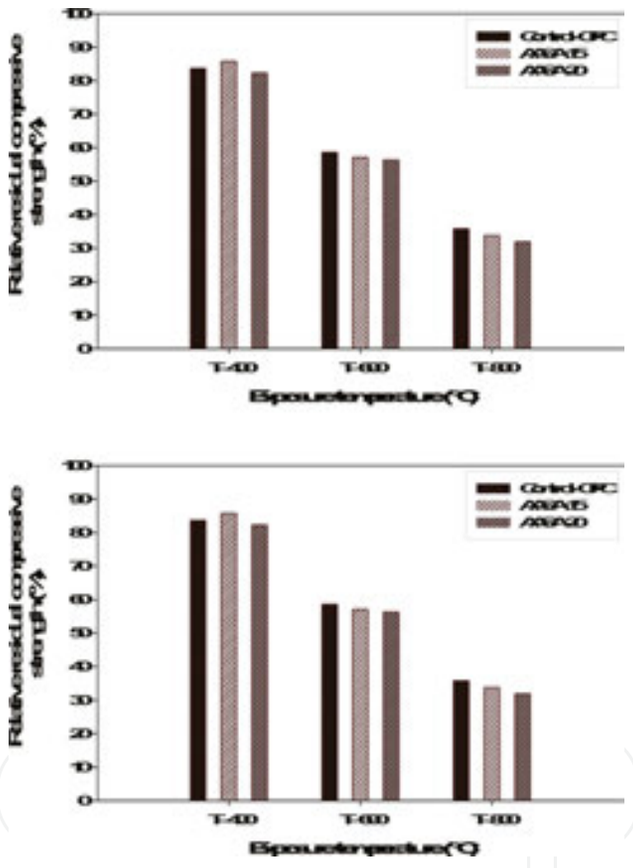


Figure 6. Relative compressive strength of control and AASA specimens subjected to different elevated temperatures.

Figures 6–8 show the values of the relative compressive strengths of the concrete mixtures containing OPC, AASA and pozzolanic materials in the form of binary and ternary blends after being subjected to high temperatures of 400, 600 and 800 °C. The relative strength was calculated as the percentage of strength retained by the concrete relative to the strength of the unheated specimen (26 °C). **Figure 6** shows that the relative compressive strength of the AASA concrete increases slightly when it is heated to 400 °C and then decreases slightly at 600 and 800 °C compared with the control concrete. At 400 °C, the relative compressive strength of

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concrete that contains 15 % AASA drops by 85.8 %, whereas that of the control-OPC and 20 % AASA concretes drop by 83.7 and 82.3 %, respectively. Finally, it is observed that a sharp reduction in relative strength occurs when the temperature increases beyond 600 and 800 °C due to the loss of crystal water, which leads to the reduction of the $Ca(OH)_2$ content, morphological changes and the formation of microcracks. The average relative compressive strengths of concrete mixtures are approximately 57.3 and 33.8 % when the concrete is subjected to temperatures of 600 and 800 °C, respectively. Thus, the compressive strength of concrete decreases significantly when the temperature rises beyond 400 °C. Similar results were obtained by Arioz [7].

Figure 7 shows that the average relative compressive strengths of concrete specimens subjected to temperatures at 400, 600 and 800 °C with 6, 20 and 15 % of SF, GGBS and POFA were 84.2, 57.3 and 35.5 % of the strengths at room temperature, respectively. From the results, it can be concluded that below 400 °C, the relative residual compressive strength does not change significantly but that it does drop significantly at temperatures above 400 °C [24].

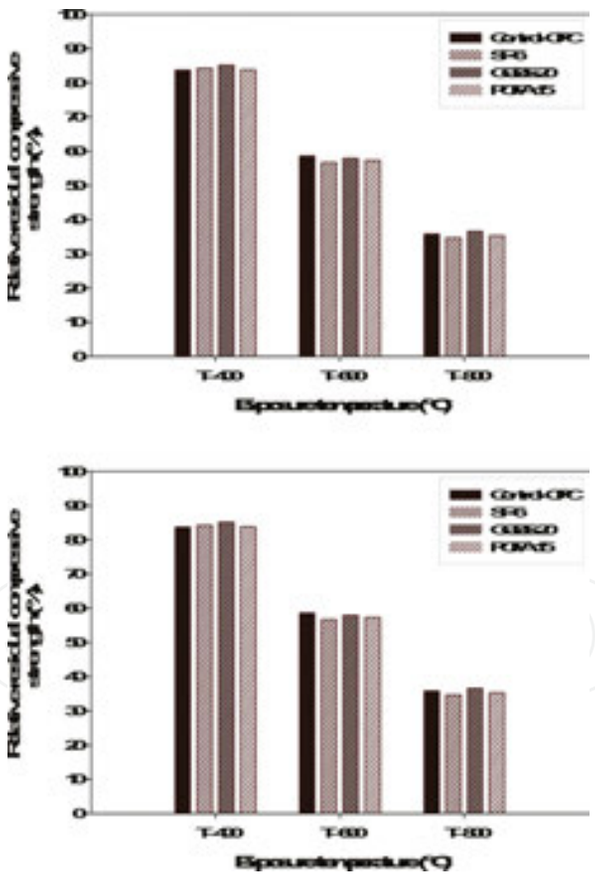


Figure 7. Relative compressive strength of control and pozzolanic material specimens subjected to different elevated temperatures.

Figure 8 shows that the relative compressive strength of TBB concrete exhibits a significant decrease after being subjected to elevated temperatures. The relative compressive strength of

mixtures OPC + AASA + SF, OPC + AASA + GGBS and OPC + AASA + POFA is approximately 82.3, 83.8 and 82.2 %, respectively, at 400 °C compared with 26 °C. A significant reduction of the relative compressive strength for mixtures OPC + AASA + SF, OPC + AASA + GGBS and OPC + AASA + POFA occurred at 600 °C. The loss recorded for each of the mixtures was 54.1, 55.7 and 54.7 %, respectively. The reduction of compressive strength of concrete is primarily attributed to the evaporative loss of free and physically bound water [11, 28]. After the temperature was increased up to 800 °C, the relative compressive strengths of OPC + AASA + SF, OPC + AASA + GGBS and OPC + AASA + POFA concretes were approximately 30.2, 33.8 and 32.7 % of the control concrete. It is clear that the compressive strength of concrete decreased significantly when the temperature was raised above 400 °C, as reported in several studies [32, 33]. The loss in strength was due to the excessive build-up of vapour pressure, which produced large cracks in the specimens [12]. Moreover, the binder products in cement paste dehydrate at this temperature, which causes a reduction in strength. However, specimens that contained AASA and SF had lower relative compressive strength compared with those of the ternary-blend binder specimens that contained GGBS and POFA materials.

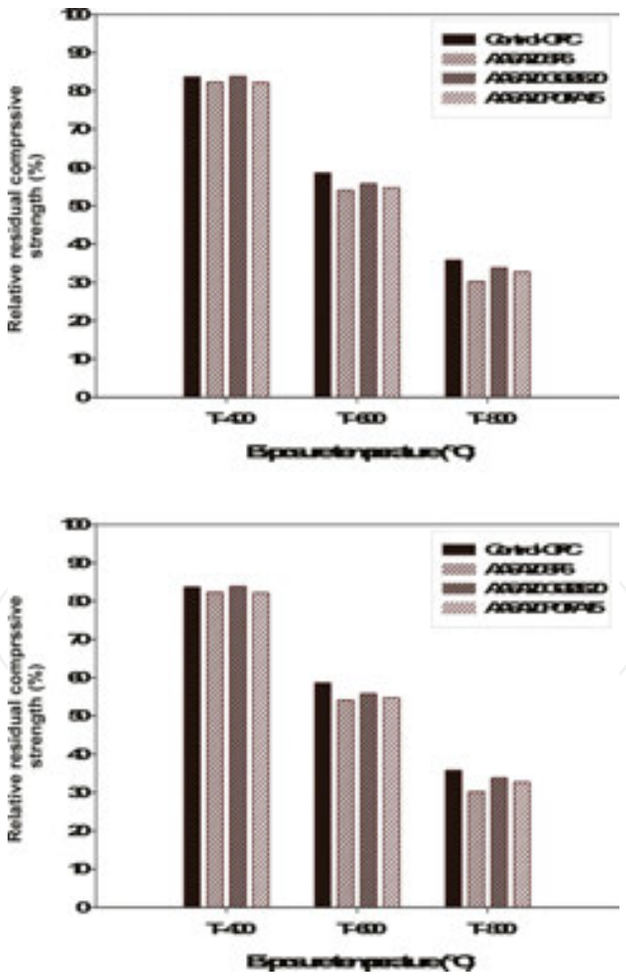


Figure 8. Relative compressive strength of control and ternary-blended binder specimens subjected to different elevated temperatures.

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3.3. Ultrasonic pulse velocity of concrete subjected to elevated temperatures

The ultrasonic pulse velocities (UPVs) of the specimens of binary and ternary AASA and pozzolanic material MBB concretes that were subjected to different elevated temperatures are given in **Figures 9–11**. Each data point represents the average of three measurements. With regard to strength, the UPV values of both the binary and ternary AASA and the pozzolanic material MBB concrete specimens decreased with increasing temperatures. However, the rate of reduction in UPV was slightly different from that of strength. **Figure 9** reveals that the UPV values of the binary concrete blend with 15 % AASA were greater than those of the specimens with 20 % AASA and the control concrete at room temperature (26 °C). However, the values of the ultrasonic pulse velocities for AASA concrete were lower than those for the control concrete with increasing temperature, and there was a notable reduction in UPV after the specimens were subjected to elevated temperatures (i.e. higher than 400 °C).

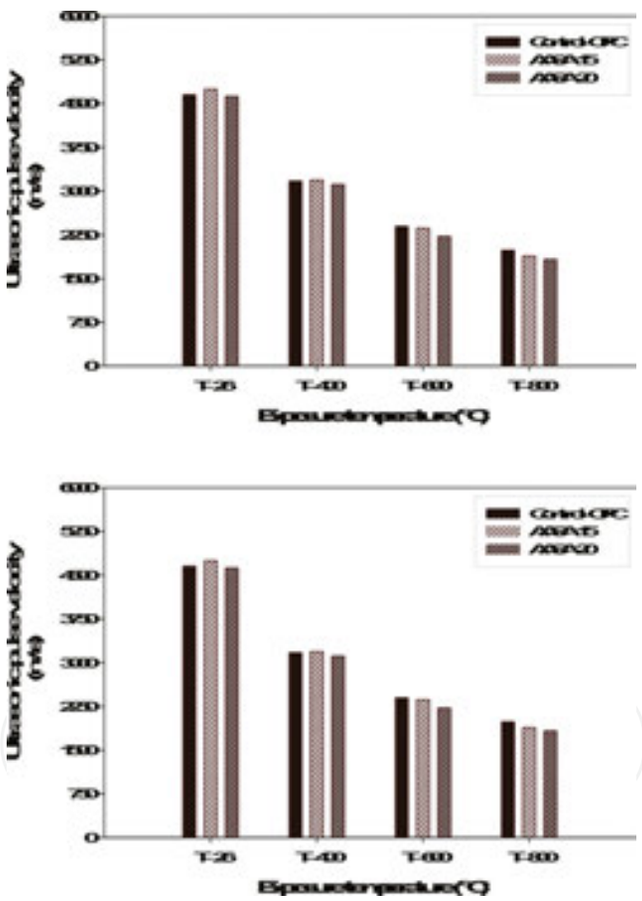


Figure 9. UPV of control and AASA specimens subjected to different elevated temperatures.

Clearly, the transmission of pulse waves through a concrete mass is greatly influenced by the microcracking of the concrete. Thus, the decrease in pulse velocity with increasing temperature is a sensitive measure of the progress of cracking in the material [23, 27, 34]. **Figure 9** also shows that the UPV displayed a continuous drop when the temperature was raised. It was noted that the reduction of pulse velocity for all concrete mixtures was due to the content of PP fibres in

the concrete mixtures. When the temperature was above 162 °C, the melting point of the PP fibres, these fibres created more randomly distributed pathways or voids in the concrete specimens. Moreover, thermal expansion and dehydration of the concrete due to high temperatures tend to cause the formation of fissures in the concrete. With more fissures, the cracks or micro-pathways delayed the pulse velocity in the concrete [35]. Therefore, micro-cracks reduce the UPV, resulting in low UPV values. The UPV values of OPC, AASA15 and AASA20 concrete mixtures were approximately 3173, 3189 and 3113 m/s at 400 °C; 2397, 2364 and 2224 m/s at 600 °C; and 1986, 1886 and 1832 m/s at 800 °C, respectively.

As illustrated in **Figure 10**, replacing part of the OPC with pozzolanic materials with 6, 20 and 15 % of SF, GGBS and POFA at 400, 600 and 800 °C caused a reduction in the UPV values. The specimens deteriorated due to increasing temperature, particularly at 800 °C, as reported by 23, 27, and the UPV values decreased substantially with increasing temperature. The measured UPVs of SF6, GGBS20 and POFA15 concretes after exposure to 400, 600 and 800 °C were approximately 3092, 3188 and 3151 m/s at 400 °C; 2287, 2332 and 2352 m/s at 600 °C; and 1829, 1963 and 1942 m/s at 800 °C, respectively.

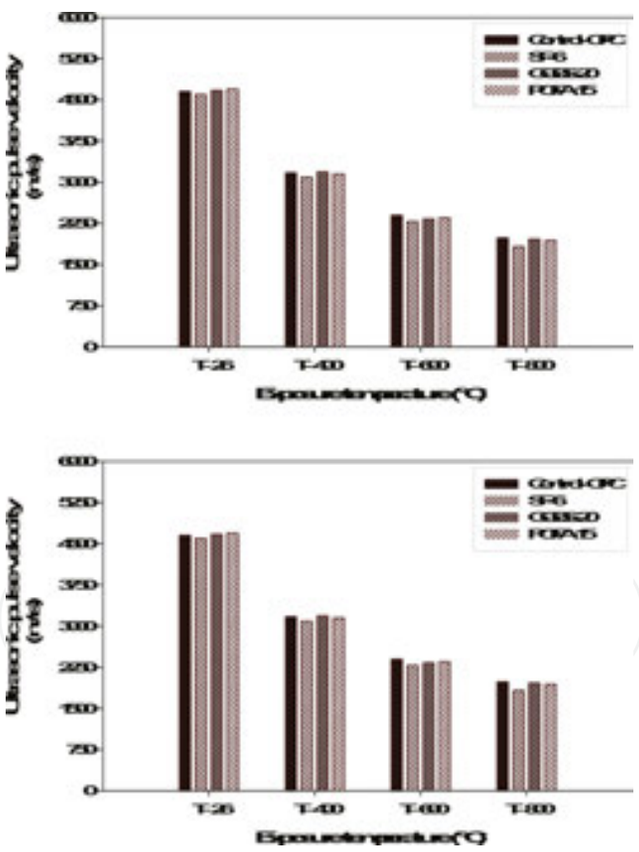


Figure 10. UPV of control and pozzolanic material specimens subjected to different elevated temperatures.

Figure 11 presents the measured UPV of TBB concretes in comparison to control concrete. **Figure 11** shows that there was a noticeable decrease in the UPV of the ternary blends of concrete with AASA and pozzolanic material after being subjected to temperatures higher than

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400 °C. This decrease in the UPV values of the concrete specimens that have been subjected to high temperatures is due to the degeneration of the C—S—H gel at temperatures above 600 °C, which increases the amount of air voids and decreases the transmission speed of sound waves through the specimens. The decrease in UPV values was observed to be higher for SF-entrained concrete specimens, especially at 600 and 800 °C. A similar conclusion was reported by Demirel and Kelestemur [10]. This decrease results from the formation of a more porous structure due to the decomposition of the C—S—H gel, which is more abundant in samples containing SF.

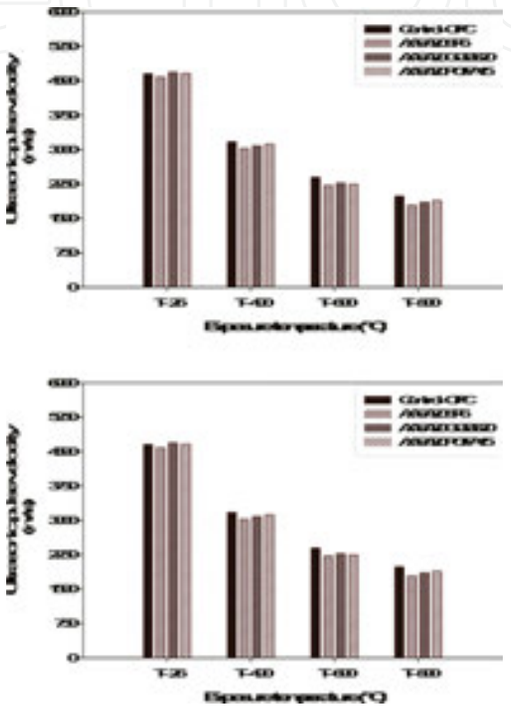


Figure 11. UPV of control and ternary-blended binder specimens subjected to different elevated temperatures.

3.4. Effect of elevated temperatures in multiple-blended-binder concretes on UPV and quality of concrete

The heating regimens of UPV measured after exposure to elevated temperatures could be divided into three stages, ranging from 26 to 400 °C, 400 to 600 °C and 600 to 800 °C. The effects of elevated temperatures on the quality of MBB concretes containing AASA, SF, GGBS and POFA are illustrated in **Table 4**. Whitehurst [36] provided varying ranges of UPV ratings to describe the quality of concrete. For concrete of excellent quality, the UPV must be greater than 4500 m/s; for good-quality concrete, the UPV must be in the range of 3500–4500 m/s; for medium-quality concrete, the UPV should be in the range of 3000–3500 m/s, while for poor-quality concrete, the UPV is in the range of 2000–3000 m/s; and finally, for very poor-quality concrete, the UPV is less than 2000 m/s.

Table 4 illustrates the effects of elevated temperatures in MBB concretes that contain AASA, SF, GGBS and POFA on the quality of concrete within different heating regimens (°C). This

table evaluates the quality of concrete processed at different temperatures for all of the types of mixtures that were subjected to the heating regimens of 26–400, 400–600 and 600–800 °C. The findings indicate that the specimens of the multiple-blended binder concretes degraded from excellent- to good- or medium-quality concrete, from medium- to poor-quality concrete and from poor- to almost very poor-quality concrete, respectively (see **Table 4**). The reason for this degradation is that the quality of concrete depends on the compressive strength. It is observed that the UPV values show a falling trend. The UPV values decrease with decreasing compressive strength for all of the mixtures after being subjected to temperatures of 400, 600 and 800 °C. **Table 5** tabulates the reduction of the UPV values for all concrete types. The UPV values for all mixtures at room temperature (26 °C) range from 4582 to 4743 m/s. Thus, all types of concrete produced at this temperature are classified as excellent-quality concrete, as shown in **Figures 9–11**.

Mix description (%)	**Concrete quality at different heating regimens (°C)**		
Concrete quality	**26–400**	**400–600**	**600–800**
Control-OPC	Medium	Poor	Very poor
AASA15	Medium	Poor	Very poor
AASA20	Medium	Poor	Very poor
SF6	Medium	Poor	Very poor
GGBS20	Medium	Poor	Very poor
POFA15	Medium	Poor	Very poor
AASA20 SF6	Medium	Poor	Very poor
AASA20 GGBS20	Medium	Poor	Very poor
AASA20 POFA15	Medium	Poor	Very poor

Table 4. The effect of elevated temperatures on types of MBB concrete specimens.

Mix description (%)	**Reduction of the UPV values at different heating regimens (°C)**		
	26–400	**400–600**	**600–800**
Control-OPC	3173	2397	1986
AASA15	3189	2364	1886
AASA20	3113	2224	1832
SF6	3092	2287	1829
GGBS20	3188	2332	1963
POFA15	3151	2352	1942
AASA20 SF6	3024	2218	1783
AASA20 GGBS20	3082	2270	1854
AASA20 POFA15	3119	2243	1887

Table 5. Reduction of the UPV relative to the original weight of the BBB and TBB mixtures.

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3.5. Surface observations of concrete specimens

A thorough visual inspection was performed to evaluate the visible signs of cracking and spalling on the surface of the specimens after being subjected to elevated temperatures. The surface cracks began to appear after the specimens were subjected to elevated temperatures higher than 400 °C and continued to grow until the final rise in temperature up to 800 °C. There was no visible cracking or spalling for concrete specimens in the 26–400 °C temperature range, as shown in **Figure 12** (**a** and **b**). When the temperature was increased to approximately 600 °C, **Figure 12 (c)**, a network of visible fine surface cracks began to appear extensively and become even more pronounced at 800 °C (see **Figure 12 (d)**). In addition, the presence of PP fibres, which were used in all of the concrete mixtures, reduced or eliminated the risk of explosive spalling in all of the MBB concrete that contained AASA, SF, GGBS and POFA. During the process of rapid temperature increase at approximately 162 °C, polypropylene fibres melt and produce escape channels for vapour. The vapour produced in the specimens due to high temperature can be released without any build-up pressure. Hence, this might be the reason for the absence of explosive spalling in the multiple-blended binder concretes with PP fibres [37]. Generally, crack formations, propagations and pattern were similar in all of the multiple-blended binder concrete control-OPC specimens and specimens containing AASA, SF, GGBS and POFA. However, there were noticeable differences in terms of sizes of cracks, including in the length, width and depth of the specimens.

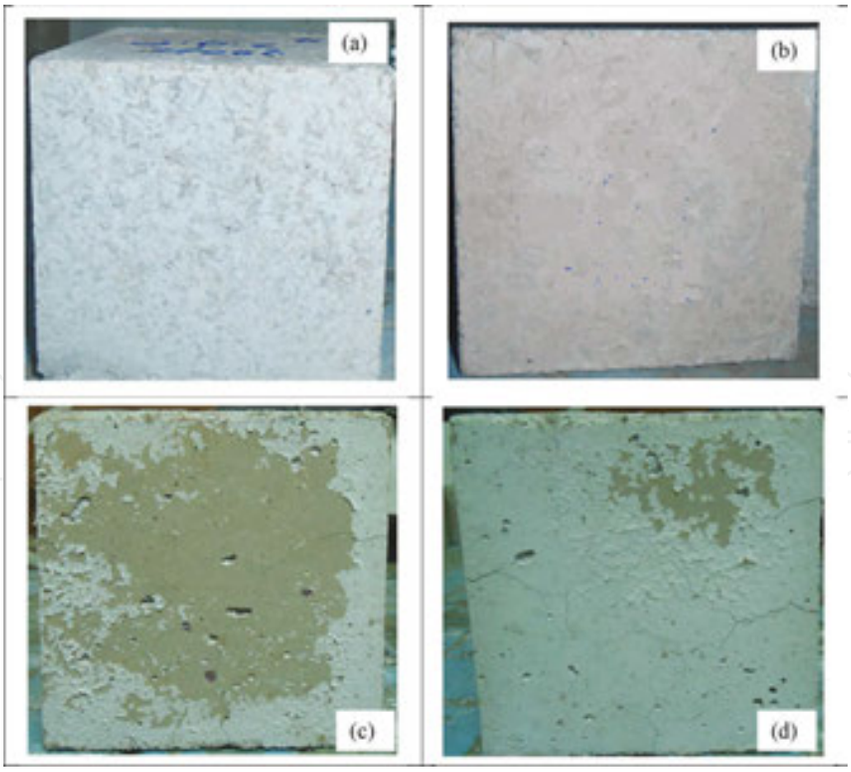


Figure 12. Typical crack patterns observed in multiple-blended binder concretes at different temperatures: (a) 26 °C, (b) 400 °C, (c) 600 °C and (d) 800 °C.

4. Conclusion

This study examines the behaviour of multiple-blended-binder concretes containing AASA and pozzolanic materials to form binary and ternary blends of cement at high temperatures, including the loss in weight, compressive strength and the reduction in ultrasonic pulse velocity as well as observations of surface characteristics of samples. The following conclusions can be drawn from the experimental results presented in this paper.

1. The mass loss decreases when the concrete is subjected to elevated temperature. That is, the mass losses of OPC concrete were recorded at 3.78, 5.24 and 6.46 %, and the mass losses of concrete mixtures of AASA 15 % and AASA 20 % were recorded at 3.98, 5.41 and 6.84 % and 4.1, 5.82 and 7.06% at 400, 600 and 800 °C, respectively. The mass loss of OPC concrete was slightly less than that of the concrete mixtures of AASA 15 % and AASA 20 % replacement levels.

2. As for the replacement levels of SF, GGBS and POFA concrete, the observations revealed a decrease in mass loss with increasing temperature. The loss in mass for the mixtures was in the range of 3.72–6.82 %. All of the ternary binder mixtures tended to show more mass loss compared with OPC concrete. The average mass loss of the concrete of ternary binder mixtures was 4.2 % at 400 °C, 5.8 % at 600 °C and 7.3 % at 800 °C.

3. The residual compressive strengths of specimens were lower than those of the control concrete for all mixtures. The performance of the residual compressive strength of binary-blended concrete mixture with 15 % AASA was observed to be better compared with the performance of the residual compressive strength of control concrete and the binary-blended concrete mixture with 20 % AASA.

4. The pozzolanic concretes containing SF6, GGBS20 and POFA15 exhibited better performance at elevated temperatures than did the control-OPC concrete. The concrete mixture containing SF performed poorly compared with the concrete mixtures containing GGBS and POFA.

5. Based on the experimental results obtained from the ternary mixtures with AASA and pozzolanic materials, the relative compressive strength of concrete exhibited a significant decrease after the specimens were subjected to elevated temperature between 400 and 800 °C.

6. The heated binary and ternary-blended-binder concrete specimens showed decreased UPV values with increasing temperature. UPV values were reduced considerably for concrete specimens that were subjected to temperatures between 400 and 800 °C, which indicates that the physical state of the concrete specimens deteriorated rapidly when the temperature was raised above 400 °C.

7. At high temperatures, the quality of the MBB concretes that contained AASA, SF, GGBS and POFA degraded from excellent to good or medium quality, from medium- to poor-quality concrete and from poor quality to inferior (very poor) quality, respectively, because the quality of concrete depends on the compressive strength.

8. A visual inspection of the surface of the specimens that were subjected to high temperatures in the range of 26–400 °C revealed no visible cracking or spalling on these specimens. However, at 600 °C, visible networks of fine surface cracks were observed on some of the specimens. When the temperature was raised to 800 °C, all of the specimens showed visible spalling and cracking. However, the use of PP fibres reduced or eliminated the risk of explosive spalling and cracking for all specimens of multiple-blended-binder concrete.

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