Structural performance of waste-based reinforced alkali-activated concrete

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15.1 Introduction

Every year, the world economy uses more than 100,000 MT of natural resources of which only 8400 MT are recycled. Also, in developed countries, the demand for construction materials will continue to increase (Deetman et al., 2020). More specifically, the global building sector's demand for steel and cement is estimated to be 769 MT/year and 11.9 GT/year, respectively. As such, the European Union assumed, early on, a leading role in setting policies that may lead to a more sustainable future. In fact, in its Europe 2020 Strategy and flagship initiative "A Resource Efficient Europe" (COM, 2011), the EU created an early blueprint of the path to this transformation. In particular, the mentioned flagship called for a roadmap "to define medium and long-term objectives and the means needed to achieve them." Furthermore, the Europe 2020 Strategy for smart, sustainable, and inclusive growth (COM, 2014; COM, 2015) also inserted the important concept of Circular Economy, which emerged with the aim of increasing resource use efficiency, while minimizing resource inputs, waste, and emissions generation. Moreover, the circular economy process also presupposes: "open production systems - in which resources are extracted, used to make products, and become waste after the product is consumed - should be replaced by systems that reuse and recycle resources and conserve energy" (Huysman, De Schaepmeester, Ragaert, Dewulf, & De Meester, 2017). Nevertheless, other authors warned about the problems associated with recycling waste streams with high waste content. In reality (Lee, Pedersen, & Thomsen, 2014) warned that the full implementation of European waste legislation will increase unwanted micropollutant recycling and the studies of Knapp, Allesch, Müller, and Bockreis (2017) also confirm this problem, as they showed that certain contaminants can be critical. Moreover, since Portland cement concretes are known to have some shortcomings in what concerns the recycling of industrial wastes (Miraldo, Pacheco-Torgal, Lopes, & Lopes, 2021), and also, since alkali-activated materials that are produced through the reaction of an aluminosilicate powder (precursor) with an alkaline activator, are usually composed of hydroxide, silicate,

carbonate, or sulfate, and lead to the formation of an amorphous aluminosilicate gel and secondary nano crystalline zeolite-like structures (Provis, 2014), and which have shown to be more effective for the reuse of industrial wastes (Abdulkareem, Havukainen, Nuortila-Jokinen, & Horttanainen, 2021; Obenaus-Emler, Falah, & Illikainen, 2020; Wan et al., 2018), then, the contribution of alkali-activated concrete would be especially important in that context. Thus this chapter reviews the structural performance of waste-based alkali-activated concretes.

15.2 Structural applications for waste-based alkali-activated binder concretes (AABC)

This section contains a review of the most important current knowledge on the structural performance of waste-based alkali-activated concrete. It is structured according to the general underlying activation model of the binder, and on a second level, based on the materials which compose the mentioned binder. This organization arises from the realization that the mentioned variables may constitute the most decisive factors on the performance of the concretes, as different precursor materials lead to different binder reaction products, and thus, different concrete behavior.

15.2.1 Calcium-rich systems

The first group of concretes addressed in this review is the one addressing waste-based concrete in which an alkali solution activates precursors which contain high levels of calcium. From a chemical standpoint, in the alkali activation of calcium-rich precursors, and although other secondary phases can be detected, the main reaction product is a C-(A)-S-H gel, and is similar to the (C-S-H) gel involved in the hydration of Portland cement (García-Lodeiro, Fernández-Jiménez, & Palomo, 2013). The most important waste for use as a precursor in this family of AABC is ground-granulated blast furnace slag (GGBFS). In fact, as seen below, its importance can be attested to the fact that it is the only waste material used in every structural element covered in the current section.

15.2.1.1 GGBFS as sole primary precursor

GGBFS is the most popular waste for use as a primary (or otherwise secondary) precursor in calcium-rich structural geopolymer concretes. One of the most important reasons to explain this popularity is the fact that, when present in a highly granulated, vitreous form, this material demonstrates high potential for geopolymerization. As to the available research concerning the use of GGBFS as sole precursor in structural concretes, Shojaei, Behfarnia, and Mohebi (2015) studied the structural performance of prestressed monoblock railway sleepers modeled after the German, ordinary Portland cement (OPC)-based, Vossloh B70 sleeper (Fig. 15.1) and based on the activation of GGBFS through the use of a sodium silicate/hydroxide solution. More specifically,

following the results yielded by the application of the Taguchi method, a 6 M NaOH concentration, an activating solution to binder ratio of 0.5 and a sodium silicate to sodium hydroxide ratio of 1.0 in the activating solution to binder and sodium hydroxide to silicate ratios, respectively, and also, a 1800 kg/m³ aggregate content was used to manufacture the concrete. Moreover, the optimal mixture, cured at 50% relative humidity and a 20°C temperature, displayed a 28-day compressive strength of 65.80 MPa, which, as the authors mention, is superior to the minimum established by several standards. However, for static testing purposes, the concrete compositions meant for the sleepers were instead subjected to a 6-h steam curing regimen at 60°C–70°C. Furthermore, the sleeper was reinforced with eight 7-mm 27.5 MPa prestress wires.

With respect to the structural performance of the sleeper, the authors only performed the verification of the maximum bending moment observed at the rail-seat section at the moment of the appearance of the first midspan crack. For the calculation of maximum admissible stress, the researchers followed the determinations stated in the International Union of Railways (UIC713) code, and used the models previously designed by other authors for the study of the structural behavior of OPC concrete B70 sleepers (Mohammadzadeh & Vahabi, 2011). After considering the same premises of the original study, that is, that the maximum train speed of travel would be of 120 km/h and the heaviest axle load would not exceed 25 T, the researchers arrived at the conclusion that the maximum bending moment would be of 17.2 kN.m. To verify the compliance of the AABC sleepers with these values, the authors performed three-point bending tests to two sleepers and arrived at a maximum load of 65 kN (again, up to the appearance of the first crack). For comparison with the maximum permissible load previously referred, a simple calculation was performed [Eq. (15.1)]:

$$F_{C_{on}} = \frac{4 \cdot M d_{c_n}}{(L_c - 0.1)} \quad (15.1)$$

Figure 15.1 Midspan (*left*) and rail-seat (*right*) section of the B70 sleeper (Mohammadzadeh & Vahabi, 2011).

In which F_{con} is the initial reference test load, Md_{cn} is the maximum allowable moment, and L_c is the distance between the supports (gauge). In the present case, the maximum test load allowable was 51.5 kN, and the sleepers successfully meet this requirement as the test yielded the appearance of the first crack only for loads superior to 65 kN. Finally, the authors end the study with the evaluation of the CO₂ footprint associated with the production of the alkali-activated binder designed (based on individual emission values of each constituent) versus an OPC binder of identical compressive strength performance and found a 28.9% improvement in CO₂ emissions (to 249 kg/m³ of concrete produced). In another set of experiments concerning the structural use of AABC, more specifically the ones based on GGBFS as a sole primary precursor (Saranya, Nagarajan, & Shashikala, 2019, 2020) utilized concrete containing binders utilizing dolomite waste powder as secondary precursor/mineral admixture, while being activated with a sodium hydroxide/silicate activating solution, for the production of beams and short columns. The experimental optimization of the concrete mix design was performed and concluded that a 70%–30% GGBFS/dolomite ratio for the content of the precursors would optimize the compressive strength of the mixture and thus, this ratio was adopted as optimal. Furthermore, a binder content of 400 kg/m³, a silicate to hydroxide ratio of 2.5, and a 12 M NaOH concentration, and also, a 6 kg/m³ content of a naphthalene-based superplasticizer were also deemed ideal. Moreover, curing was performed under ambient conditions, which in the present case, and with temperature values in the range of 25°C–28°C, can be considered a higher than usual interval (which may skew the results). Additionally, for the improvement of the modulus of elasticity and tensile strength of the concrete, the researchers added crimped 30 mm × 0.5 mm steel fibers in a proportion of 0.25%–0.75% of the total volume.

The resulting concrete demonstrated a mean 28-day compressive strength of 74.7 MPa, a slight upgrade from the strength observed for the mean value of the three corresponding cubic ordinary Portland cement concrete (OPCC) samples of 70 MPa (although the tested reference OPC samples did not contain steel fibers). Furthermore, the results from slump, split tensile, and flexural strength testing all hint at a concrete of superior performance (when compared to the reference OPCC composition). Moreover, the study of the concrete's stress–strain relationships also revealed higher ultimate and peak strains (0.0038–0.0033 and 0.0025–0.0023, respectively, in AAB and OPC concrete samples with no fiber addition) and slightly lower modulus of elasticity and Poisson's ratio (38.6–39.1 GPa and 0.184–0.185, respectively). It must be noted that the results for the Poisson's ratio of both concretes is lower than would be expected, as Eurocode 2 (ECS, 2004) estimates a 0.2 Poisson's ratio for OPC concrete. The first testing of the abovementioned AABC in structural applications was performed in five pairs of 1.2-m reinforced concrete beams, one pair for each tested proposition (OPCC compositions, and also, AABC containing 0.00%, 0.25%, 0.50%, and 0.75% steel fibers). Moreover, the steel reinforcement consisted of four longitudinal 10-mm HYSD bars, and also, 6-mm two-legged stirrups with a 80-mm spacing as shear reinforcement (Fig. 15.2).

The beams were subjected to a four-point bending test, with the load being applied at a rate of 400 kg/min until failure and the load and crack observation

Figure 15.2 Structural detail of beams (Saranya et al., 2019).

Figure 15.3 Load–deflection curves (Saranya et al., 2019).

results were analyzed and later compared to numerical analysis using finite element models. The load–deflection curves resulting from the bending test of the beams can be seen in Fig. 15.3.

As can be gathered from the displayed data, the best performing concrete is, as expected, the AABC possessing maximum fiber content, also, a reduction of fibers content leads to a proportional performance downgrade. Additionally, the AABC samples possessing no fibers also yielded (slightly) higher first and ultimate load, when compared to OPCC samples (18.0 and 58.9 kN for the first and 16.0 and 56.9 kN for the latter). Furthermore, the authors also recorded the midspan deflection and crack width at ultimate load, and the results also confirm the superior performance of AABC over OPCC (8.0 and 3.1 mm vs 8.5 and 3.8 mm, respectively).

Meanwhile, and as previously mentioned, the same authors also tested the designed concrete for producing five steel reinforced $200 \text{ mm} \times 200 \text{ mm} \times 1100 \text{ mm}$ short columns with 12-mm and 6-mm longitudinal reinforcement bars, and two-legged 6-mm-diameter transverse reinforcement stirrups at varying spacings (Fig. 15.4), which were later subjected to axial load tests.

Once more, after the experimental tests were performed, the elements were modeled through the finite element technique and the final results were compared to

Figure 15.4 Structural details of the short columns (Saranya et al., 2020).

one another. The stress–strain relationships yielded by axial loading the specimens up until failure at a constant rate of $140 \text{ kg}/(\text{cm}^2 \cdot \text{min})$ can be seen in Fig. 15.5. As can be inferred from the figure, and similarly to what could also be observed in the behavior of the same concrete when applied in beams, the addition of steel fibers returns proportional gains in strength and ductility for the columns, and also, AABC structural elements with no fiber additions perform better than the equivalent OPCC elements. Furthermore, a high correlation between the numerical modeling data and experimental results was observed. Finally, the researchers also calculated that the estimated cost of the concrete per ultimate load and determined that, even for the case of the highest steel fibers addition (6%), this factor is lower in AABC compositions. Also, after modeling the same structural element with OPCC possessing ductile detailing, the study found that an AABC without the same detailing will offer the same performance, while reducing the cost.

Figure 15.5 Stress–strain relationships for the columns under axial loading (Saranya et al., 2020).

15.2.1.2 GGBFS and metakaolin as precursors

In another example of a recent investigation focusing on the use of high calcium GGBFS-based AAB in structural concretes, Liu et al. (2019) studied the behavior of reinforced concrete walls, made with an AAB in which the precursor was composed of a blend of GGBFS and MK, when subjected to different kinds of TNT blasts. With this study, the authors intended to address the typical limitations of conventional OPC concrete in resisting shear and concrete spalling arising from blast loadings by replacing it with a cost-efficient concrete with, expectedly, higher strength and ductility. For this purpose, the researchers also made use of steel wire mesh (SWM), which the investigation states possesses higher toughness and energy absorption capacity, and also, may more effectively prevent crack propagation through the so-called bridging effect. It was also expected that, due to a membrane phenomenon, this material would provide improved spall and crater resistance to the structural element. Additionally, aluminum foam (AF) was used as buffer to the blast. The first step taken by the researchers to properly design the structural element was to optimize the design of the concrete mixture. In AABC, this procedure usually encompasses the experimentation of several mixture designs in order to maximize the compressive strength of the samples. In this particular study, the researchers also subjected 400 mm × 100 mm × 100 mm beams containing several of the designed mixtures to four-point bending tests and found that the optimal mixture was the one containing alkaline solution to binder and water to binder ratios of 0.38 and 0.40, respectively, and also, an equal content of MK and GGBFS as precursor materials. Furthermore, 20-layer SWM reinforcement with a mesh grid size of 6.35 mm × 6.35 mm was considered (particularly after the observation of the midspan displacement curves of samples submitted to flexural tests revealed that

the upgrade from 20- to 30-layer SM could compromise the ductility of the specimens) as being the solution which maximizes mechanical resistance while minimizing the cost of the concrete element. Moreover the samples were covered and cured for 48 h in a humid environment at a room temperature of $(20 \pm 5)^{\circ}\text{C}$, after which period the covers were removed and the samples were left curing until testing. As to the production of structural elements, five $2260 \text{ mm} \times 2260 \text{ mm} \times 150 \text{ mm}$ (Fig. 15.6) and two $2260 \text{ mm} \times 1000 \text{ mm} \times 150 \text{ mm}$ walls were prepared for blast testing. The first group contained two OPCC walls, two SWM-reinforced geopolymer walls, and finally, one SWM/AF wall. The second possessed one SWM and

Figure 15.6 Structural details of the wall (Liu et al., 2019). Representations of the dimensions (a), and cross section, (b) of the tested walls.

another SWM/AF geopolymer concrete wall. Furthermore, two sets of blast tests were performed, one in which 50 kg of TNT were used to test four walls, and another, in which 100 kg were used to test the remaining three walls produced.

The results of the blast tests show that the AABC walls yielded a better response post-blast, in terms of structural integrity, when compared to their OPCC counterparts (Fig. 15.7). In fact, even for a 100 kg blast, which caused the complete failure of OPCC walls, and although several brittle damages and cover peeling on the rear surface of the wall could be detected after the event, the structural integrity of the AABC wall was preserved. Moreover, an increase in residual deflection was observed in AABC walls reinforced with AF, which the authors attribute, among other reasons, to the AF's low stiffness. Finally, the study suggests that covering

Figure 15.7 Rear view of the structural damage on: (A) the OPCC wall and (B) the SW/AF-reinforced AABC wall.

Source: Adapted from Liu, J., Wu, C., Li, C., Dong, W., Su, Y., Li, J., . . . Pang, J. (2019). Blast testing of high performance geopolymer composite walls reinforced with steel wire mesh and aluminium foam. *Construction and Building Materials*, 197, 533–547.

the rear wall surface with fiber-reinforced polymer would prevent projected fragments and, as such, further upgrade the material's response to blasts.

15.2.1.3 GGBFS and FA as precursors

Binders comprising GGBFS (as primary precursor) and Fly-ash (FA) (as a secondary precursor) are, nowadays, the most widely adopted material in the production of concrete compositions based on the geopolymerization of waste-based aluminosilicates. Such is the case of the AABC designed in [Wu, Hwang, Shi, Li, and Du \(2020\)](#), in which 18 reinforced AABC and 3 OPCC beams were produced for the purpose of studying these element's structural performance when subjected to shear testing. As to the optimization of the concrete mixtures, the researchers defined target compressive strengths to be achieved by the specimens and designed the materials employed, their respective proportions and curing methods, accordingly. The main characteristics of the mixtures and the results from the compressive strength tests performed can be observed in [Table 15.1](#).

For the structural analysis of the beams, and other than the contrast in the nature of the binder material (OPC or AAB) at various target compressive strengths (40, 60, and 80 MPa), different variations of section sizes (200 mm $\times$ 400 mm vs 200 mm $\times$ 300 mm), beam length (4000, 2950, 2450, and 2250 mm), longitudinal reinforcement ratios (1.8%, 2.5%, and 2.8%), type of rebar strengths (normal and high strength), and transverse reinforcement spacing (125, 175, and 350 mm) were employed ([Fig. 15.8](#)). Minimum spacing for stirrups was implemented according to ACI 318-19. The setup for the shear tests consisted of a simply supported beam, subjected to a two-point load, which was applied, initially, at a 10 kN increase rate until the appearance of the first diagonal crack.

Afterwards, a 20 kN load increase stage was implemented, up to 90% of the estimated nominal shear strength, followed by a final stage, in which the load increments returned to 10 kN, until failure was reached. The resulting load–deflection curves of the tested beams can be observed in [Fig. 15.9](#).

As can be gathered by the observation of the load–deflection relationships, the shorter beam (2250 mm) displayed the greatest load capacity, also, the increase in the concrete's resistance class led, as expected, to proportional improvements in the load carrying capacity and midspan deflections. Moreover, except in two cases (in which early shear failure occurred), in the load–deflection curves of the beams that possessed the highest strength concrete, a large yield plateau and significant midspan deflections could be observed. However, for other high-strength concrete beams, flexural (or flexural-shear) failure modes were reported. In relation to shear strength, and although the crack distribution and failure modes were similar to the ones displayed by their OPC counterparts, AABC beams performed, in general, slightly worse than their OPCC equivalents. Moreover, an upgrade in the concrete resistance class of the AABC beams led to higher shear strengths. Nevertheless, a clear pattern did not emerge when studying the relationship between shear strength and midspan displacements and the increase in the longitudinal reinforcement ratio. In contrast, the increase in shear span-to-depth ratio led to a reduction in shear

Table 15.1 Measured versus target compressive strength of concrete samples.

Binder	GGBFS/ FA ratio	Liquid to binder ratio	Superplasticizer (%)	Curing	Samples (<i>n</i>)	Target compressive strength (MPa)	Average compressive strength (MPa)
OPC	—	0.36	—	Ambient	1	40	29.2
	—	0.28	0.185	24 h 80°C		60	39.5
	—	0.234	0.623	24 h 80°C		80	64.2
AAB	2.5	0.588	—	Ambient	6	40	30.2
	2.5	0.605	—	24 h 80°C		60	59.3
	4	0.573	—	24 h 80°C		80	69.4

Source: Adapted from Wu, C., Hwang, H. J., Shi, C., Li, N., & Du, Y. (2020). Shear tests on reinforced slag-based geopolymer concrete beams with transverse reinforcement. *Engineering Structures*, 219, 110966.

Figure 15.8 Structural detailing of the tested beams (Wu et al., 2020).

Figure 15.9 Load-deflection relationships for the groups of beams tested (Wu et al., 2020). Load-deflection relationships for the seven groups of beams tested: OPC concrete-based, Intermediate-size beams (A); geopolymer concrete-based, intermediate-size beams (B), (C) and (D); geopolymer concrete-based, deep beams (E); geopolymer concrete-based, slender beams (F); geopolymer concrete-based, intermediate-size, small section beams (G).

strength while the reduction in the section's depth caused a decrease in the normalized shear stress. Finally, five design models (among them the ones recommended in ACI 318-19, Eurocode 2 or CEB-FIP Model Code 2010) were used to predict the

theoretical shear strength of the beams and the results show that the models tend to underestimate this property in beams containing AABC.

Another investigation on the theme of structural concrete which makes use of AABC based on GGBFS/FA precursors is [Cong, Zhou, and Elchalakani \(2020\)](#). Besides this waste material, the researchers also utilized water-quenched slag (WQS), another industrial by-product, as a replacement for fine natural aggregate in AABC mixtures. Furthermore, the typical NaOH/Na₂SiO₃ solution was used as activating solution, and also, a mix of zinc nitrate and sodium gluconate actuated as set retarders. As to the results from the compressive strength tests, the investigation found that the WQS addition slightly downgraded the performance of the concretes. In fact, when compared to the AABC containing only river sand, and for a 90-day curing period, the full replacement of sand for waste slag yielded an 8.4% strength decrease to this property, and also, returned identical compressive strength to the one demonstrated by the reference OPC concrete. In relation to splitting tensile strength, the full sand replacement resulted in a 13% lower strength result when compared to both the geopolymer concrete containing only river sand and OPC concrete. It is necessary to stress that the mentioned results were obtained for the 90-day curing period, as the strength development of these compositions proved to be substantially slower than the one displayed by the OPC concretes (which was confirmed by the analysis of the reaction products through SEM imaging). The data resulting from the pullout tests performed to the compositions designed by the authors (in which a first batch of concrete mixtures was reinforced with round 12-mm steel bars) revealed low maximum bond strengths (<2 MPa) and slip values (in peak bond strength).

Additionally, the authors observed that all samples exhibited a failure by the pullout mode. Of the group, the best performing design was the one incorporating a 50% WQS replacement which, the authors argue, is due to being the mixture that optimizes the two main factors influencing the bond stress: the radial confining stress and the roughness of the concrete matrix. In the second set of tests, the researchers made use of ribbed 10, 20, and 25-mm-diameter bars. As to the test comprising the 12-mm ribbed bars, and according to the authors, the failure also happened in a pullout fashion, while for the 20-mm bars' concrete, the authors reported a cracking failure, and lastly, for the 25 mm bars, a fracturing failure (in which the concrete in front of the bar ribs tends to get crushed) was observed. In fact, for the larger diameters (20 and 25 mm), the descending branch of the bond stress–slip curves could not be recorded as the failure occurred in a brittle fashion ([Fig. 15.10](#)).

As to the values of the ultimate bond stress, the data show that the increase in rebar diameter leads to lower stress results. Also, the OPC samples yielded the lowest ultimate bond stress in each bar category, which the authors attribute to the AABC's superior chemical adhesion and friction effect (which was upgraded by the presence of WQS). Furthermore, the authors also performed beam-end tests to the concrete samples and concluded that, when a thin concrete cover (20 mm) was used, the type of concrete affected the bond stress and slip results (which means that narrowing the adjacent stirrup spacings improved bond strength). In the case of

Figure 15.10 Failure modes of pullout specimens. (A) Pullout failure mode; (B) cracking failure mode; (C) fracturing failure mode (Cong et al., 2020).

a thick concrete cover (40 mm), however, the dispersion in bond strength and ultimate slip was greatly reduced and the bond performance was similar for all concretes. Moreover, the bond performance of concrete containing 12-mm bars was highly influenced by the bond length (contrary to the behavior displayed by the concretes containing 20-mm bars) and, for a 10Φ bond length, was similar to that possessed by the 20-mm bars' concrete. The study of the important creep phenomenon in structural elements produced with AABC, more specifically, AABC based on GGBFA/FA, was addressed by Un et al. (2015). In this investigation, the binder was mostly composed of GGBFS (95% of the precursor weight, in mass), whilst fly ash was used to complete the remaining 5% of the total precursor weight. Furthermore, only sodium silicate was used as activator solution and a total of 0.4 water to binder ratio was used in the mixture. The designed concretes were used to produce two reinforced AABC $600\text{ mm} \times 240\text{ mm} \times 6500\text{ mm}$ beams, and also, several cylinders (for compressive strength and creep tests) and prism (to study shrinkage and flexural strength) of various sizes. The nonstructural elements were cured at an ambient temperature of 20°C – 23°C in a lime-saturated bath, and after 24 h of sealed curing, the samples were coated with two layers of a wax emulsion curing compound. Furthermore, the cylinder samples destined for creep analysis were further cured in the lime bath for 6 days at 23°C and then placed in a climate-controlled chamber (at 23°C and 50% relative humidity). Alternatively, with respect to the curing regimen implemented for the beams, the side and end faces of the beam's formworks were removed 3 days after casting, and then the same two layers of the emulsion agent were applied on the respective surfaces (thus simulating the conditions in which wide slabs are left for curing in real-world applications). Furthermore, temporary supports were removed at the end of 14 days and the beams were supported and subjected to a uniformly distributed 1 kPa load located at the top of the beams. As to the long-term behavior of the AABC beams, three methods were used to predict their performance: the rate of creep method (which assumes the independence of the rate of creep in relation to the age of the loading); the effective modulus method (in which the main assumption is that the effective elastic modulus depends on the modulus at the moment of loading and of instantaneous and creep effects); and finally, the age-adjusted effective modulus method (which derives of the previous method and accounts with the ageing of the concrete).

Additionally, the prediction of the long-term deflection of the beams was also performed, through a double integration of the curvature values, with a 0.5 m spacing along the beam's length. The 28-day compressive strength and flexural strength of concrete samples subjected to the standard bath curing was reported to be of 46.7 MPa and 6.8 MPa, respectively (this last value being substantially superior to the 4.1 MPa reported for the same property measured for an OPCC sample of identical strength class). Furthermore, it is also stated that the samples subjected to drying in an environmental chamber demonstrated lower performance in both properties.

With respect to the long-term variation of the creep coefficient of samples loaded after 14 and 28 days of curing (Fig. 15.11), the authors highlight the lasting (1 year) downgrade in creep strains in the case of the early load application (14 days). Also, and in a behavior that is still to be explained, the researchers stress that this phenomenon is somewhat contradictory to the fact that the ABBC's strength and elastic modulus are already, at 14 days, close to the values displayed at 28 days.

Finally, the comparison of long-term deflection values with the predictive models (originally developed for OPCC elements) with the experimental data demonstrates that both the effective elastic and age-adjusted effective modulus methods can be utilized in the structural design of AABC beams. In opposition, RCM is reported to overestimate the deflection by not accounting for delayed creep recovery and by wrongfully assuming parallel creep curves at different ages. Furthermore, the researchers also state that the use of $\beta = 1$ (for accounting the effect of tension stiffening) for beam analysis up to 56 days, and also, the cracking condition must be used for the beam after loading in order to properly determine long-term deflections.

Figure 15.11 Long-term evolution of the AABC's creep coefficient when loaded at 14 and at 28 days (Un et al., 2015).

When considering the material design of AABC structural elements containing a calcium-rich binder, there is also the possibility, such as is the case of the one used in Xie, Zhou, Lu, and Chen (2017), that the main constituent of the binder is a material with a low calcium content. In fact, the authors of the mentioned study made use of a binder constituted of a mixture of low calcium FA (65% of binder content in weight) and GGBFS (35%). Furthermore, an activating solution with an unusually high NaOH concentration (18 M) and a NaOH:Na₂SiO₃ mass ratio of 3. In this investigation, twelve (5100 mm × 100 mm × 100 mm single-edge notched) beams (Fig. 15.12) with a notch-depth ratio (a_0/d) varying from 0.2 to 0.5, at an increment of 0.1 (with three beam samples per ratio) were subjected to three-point bending tests (to complete failure, at a loading rate of 0.05 mm/min) and digital image correction (DIC) techniques were implemented for analyzing the fracture behavior of the beams. More specifically, DIC procedures were first used to determine the evolution of the displacement field of the beam during the test. Afterwards, these results were used to obtain the crack mouth and crack tip opening displacements (CMOD and CTOS, respectively), and also, the crack extension length. Based on these results, the crack formation and propagation were modeled, and finally, the fracture energy and toughness of each beam were evaluated.

Moreover, as the researchers stress, the load–displacement curves produced with the aid of full-field displacement results, namely CMOD values, indicate that the notch depth does not meaningfully influence the deformation behavior of the beams. Also, the analysis of the load–displacement curves of the AABC beams reveals that the general shapes of these curves are similar to the ones resulting from the same tests, when performed to OPCC beams. Nevertheless, and although a (mostly) prepeak linear P – δ relationship can be observed in both types of beams, and also, in a phenomenon that may almost be unnoticeable by the observation of the load–deflection curve, as peak load approaches the curves yielded by AABC beams start to deviate from a linear elastic deformation and cracking begins to propagate (thus demonstrating the quasi-brittle nature of the material). One interesting observation made by the study is that the nonlinearity of the AABC beam's behavior can be more easily noticed by the observation of the P –CTOD graph. Fig. 15.13 details the several P –CTOD relationships obtained for the AABC beams tested, with emphasis on the one possessing a notch-depth ratio of 0.4. In this

Figure 15.12 Structural details of the single-edge notched beams (Xie et al., 2017).

Figure 15.13 P –CTOD graphs for the beams (Xie et al., 2017).

representation, the authors discriminated four stages of the beam's behavior and, in the second one (designated ab), the gradual reduction in the curve inclination is clearly visible (sign that cracking has already initiated). Afterwards, the data from the load–deflection relationships was used, through the implementation of RILEM guidelines, to determine the fracture energy involved in the cracking process and the critical intensity factor (also known as fracture toughness). However, as the authors mention, the unusually high variability in results renders obtaining reliable relationships between the properties of the tested beams and the resulting observations, next to impossible.

In another example of a structural application of an AABC based on a FA/GGBFS calcium-rich binder, Meng et al. (2020) analyzed the response of slabs composed of this material to a natural gas explosion. In relation to the components of the mixture, and besides the inclusion of the FA/GGBFS matrix, the researchers also added silica fumes to the binder. Furthermore, concrete possessing 2% $12\text{ mm} \times 0.12\text{ mm}$ steel fibers (with a 2850 MPa yield) with C 30 OPC samples were produced. Initial strength testing performed to the AABC samples revealed an average compressive and flexural strength of 61.11 and 4.24 MPa, respectively (and 73.90 and 15.06 MPa for the fiber-reinforced specimens). Furthermore, the elastic modulus of the specimens was found to be of 27.7 and 34.8 GPa for the standard and fiber-reinforced AABC, respectively, while for the OPCC samples the result of the same test yielded 34.8 GPa (considerably higher than for standard AABC specimens). For the purpose of studying the structural response of a concrete slab containing the previously mentioned AABC to a gas explosion, the authors designed a full-sized closed gas tunnel, of which three of the top panels were the specimens to

be tested (one fiber-reinforced AABC, one standard AABC, and one reference OPCC slab). These 1800 mm × 400 mm × 90 mm reinforced concrete elements, of which the structural detailing can be seen in Fig. 15.14, possessed a 10-mm cover on both surfaces and 12-mm HRB335 steel reinforcement bars in both directions.

Additionally, thick steel plates filled the rest of the tunnel's top face, and the slabs were connected to the tunnel's concrete walls through 20-mm-thick angle steel with bolts. Finally, the ignition of the 9.5% methane gas (blended with air) was performed with the help of an electrical spark. As to the structural response of both, the OPCC and the fiber-reinforced AABC slabs to the explosion, no permanent deflection or structural crack was recorded. In opposition, minor parallel cracks were observed on the bottom side of the standard AABC slab, in an indication of a more brittle behavior of standard AABC elements. In the final part of the study, the researchers report the application of the finite element analysis performed to simulate the impact of the explosion on the slabs (using the single degree of freedom method to generate the analytical load–time curve. Lastly, the data from the simulation indicates that the fiber-reinforced AABC slab may demonstrate good static and dynamic responses to such an explosion (it is to be noted, as stated by the authors, that the gas leaks that occurred during the experimental study prevent the conclusions of this study to be generalized to gas detonations, but rather, deflagrations).

15.2.2 Low calcium models

The only waste material that has been used as a primary precursor in the alkali activation of structural concretes containing low-calcium precursors is fly ash and the main reaction product resulting from the alkali activation of these precursors is a N-A-S-H gel (García-Lodeiro et al., 2013). In this section, all the reviewed investigations make use of fly ash as primary, and often sole, precursor.

Figure 15.14 Structural detail of the tunnel section: (A) front view; (B) cross-sectional view (Meng et al., 2020).

15.2.2.1 Fly ash as primary precursor

In the first investigation addressed in the current section (Nguyen, Ahn, Le, & Lee, 2016) submitted low calcium, fly ash-based AABC steel reinforced beams to four-point bending tests and compared the results to the ones returned by the implementation of theoretical models and later solving them with the help of FEM software. In relation to the beam's concrete composition, a 8 M NaOH concentration, in addition to a 2.5 silicate to hydroxide and a 0.4–0.6 alkaline solution to precursor ratios, and also a 60°C, 4-h curing temperature regimen was used. General design details, along with the test setup, can both be observed in Fig. 15.15.

In relation to the concrete compositions which were later used for producing beams, previous testing performed on cylinder samples yielded a (fairly low) Young's modulus (ϵc) in the range of 21.90–25.10 GPa, while the Poisson ratio (ν) results were situated in the 0.18–0.19 scope. As the authors stress, AABC's poor elastic modulus performance has been reported before and may constitute on of these material's foremost handicaps. In opposition, the compressive strength values obtained (46.3–52.0 MPa) are more in line with the results that would be expected from an equivalent OPCC. Moreover, and in order to compare the experimental stress–strain relationships of the AABC compositions to theoretical models, the researchers produced the same previously mentioned mixtures only, this turn, they were cured at temperatures of 60°C, 90°C, and 120°C for a period of 10 h. After assessing the results, the authors report most strain at peak stress values were situated in the range of 0.0024–0.0026 which, they argue, is a similar range to the ones typically yielded in OPCC. The relationships resulting from these procedures, in this case for the concrete cured at 60°C, can be observed in Fig. 15.16. In this representation, the high quality of the theoretical approximation to the experimental result can be clearly seen.

It is also important to mention that the concrete's theoretical stress–strain curve was obtained by making use of the formula proposed by Collins, Mitchell, and Macgregor (1993) for OPCC [Eq. (15.2)]:

Figure 15.15 Test setup, containing structural detailing of the beams (Nguyen et al., 2016).

Figure 15.16 Predicted versus test results for the stress–strain relationships of the beam cured at 60°C (Nguyen et al., 2016).

$$\sigma_c = f_{cm} \frac{\varepsilon_c}{\varepsilon_{cm}} \frac{n}{n - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^{nk}} \quad (15.2)$$

where f_{cm} is peak stress, ε_{cm} is the strain at the mentioned peak stress, and n and k depend on the former variables and are given by the model. In another benchmark, splitting tensile strength, the results from the experiments show a 10%–15% superior performance from the part of AABC, when compared to OPCC. Finally, the load–deflection curves obtained by the investigation, when compared to the ones given by the chosen theoretical models based on the theory of elasticity suggest a better performance than the one implied in the models. Nevertheless, the same results were slightly inferior (although very similar) to the ones obtained by applying the test conditions to the beam in a FEM software. A further example of the use of a fly ash-based low-calcium AABC in a structural concrete is the one present in (Visintin, Mohamed Ali, Albitar, & Lucas, 2017). In this investigation, the authors focused on determining the shear capacity of four AABC beams possessing no transverse reinforcement, and also, four unreinforced AABC prisms. In relation to the composition of the concrete used in this study, the researchers used a sodium silicate to hydroxide ratio of 1.5, with a 14 M NaOH concentration for the activation of a class F fly ash binder. Furthermore, extra water was added to the mixture (in a total liquids-to-binder ratio of roughly 0.51). It is stated that this concrete yielded 29 MPa and 3.1 MPa compressive and flexural strength, respectively. Moreover, a typical four-point beam bending test setup was installed (Fig. 15.17), and four beams (with varying geometries and load application points) were loaded until presenting critical shear failure. Each beam produced a total of two results as, after the first failure was reached, the failed section of the beams was strengthened

Figure 15.17 Test configuration and structural details of the beams (Visintin et al., 2017).

with compression and tension plates and the beam was taken once again to shear failure.

For further analysis of the shear capacity of the concrete, and using a technique designed by Zhang (2015) and in which pure shear failure along the center line of the specimen is induced, four AABC shear blocks were fabricated. In this method, the concrete sample is confined with steel plates and no reinforcement is used, and also, the stress (normal to the sliding plane) is provided by a series of springs, which are being tensioned by external rods. After implementing the testing process, the shear-friction capacity (for each confining stress) was determined by applying the formulation (segmental approach) derived by Chen, Zhang, Visintin, and Oehlers (2015) for OPCC. The results yielded by this investigation for AABC are in line with the ones obtained by Chen et al. (2015), and can be seen in Fig. 15.18. Furthermore, the authors also applied the values for the shear capacity of the concrete obtained by the segmental approach to the conditions of the previously tested beams and confronted both outcomes with the ones predicted by the Fib Level II method and, out of eight beams, only in one situation the failure load of the beams was lower than the one predicted by the method. As the researchers state, the data collected from these methods suggests that the shear-friction failure criteria can, with a moderate degree of confidence, be applied to AABC to predict the failure load (although more research is needed).

In the first of the covered studies dedicated to the analysis of the performance AABC structural elements to extreme heat, Mathew and Joseph (2018) investigated the flexural performance of reinforced fly ash-based AABC beams, when subjected to elevated temperatures. In this investigation, the fly ash binder was activated with a sodium silicate/hydroxide solution in which the hydroxide was present at a 10 m concentration and a silicate to hydroxide ratio of 2.5. Also, extra water and a naphthalene-based superplasticizer were also added to the mixture and a 24 h 100°C curing period was put in place. As to the structural application of the concrete, 15 beams, all possessing a 150 mm × 200 mm × 1100 mm geometry were produced (Fig. 15.19). The mentioned beams were divided in three groups of five, in which each group possesses a different cover size (20, 30, and 40 mm), and in which, for each of the elements of the group, five heat exposure regimens were implemented (0°C, 200°C, 400°C, 600°C, and 800°C with a 200°C for an hour heating period, in which the temperature was increased at 5°C/min increments). After the heat treatment, the beams were subjected to standard four-point bending tests.

Figure 15.18 OPCC versus AABC shear capacity comparison (Visintin et al., 2017).

The results of tests show that beyond 600°C, as the authors stress, the load carrying capacity of the beams drastically declines. This phenomenon is attributed, not to AABC, but to the strength reduction of the reinforcing steel. Moreover, it was observed that, as expected, a reduction in concrete cover also leads to a decrease in value of the load that causes the first crack. Furthermore, due to the similarity in moment–curvature graphs, the strain compatibility approach usually utilized in OPCC may also be applied to AABC elements (although not in the case of elevated temperatures). Finally, the effects of high temperatures on the ductility ratio of the beams are also noteworthy, as it was also observed that an exposure to an 800°C temperature leads to a 64% ductility loss. Lead smelter slag (LSS) is a by-product of the extraction of heavy metals during the lead smelting process. Albitar, Mohamed Ali, and Visintin (2017) used large amounts of this material, in its grounded form, along with class F fly ash, to fill the role of precursors and fine aggregate of an AABC. Furthermore, coarse natural aggregate and a water reducing agent were also used in the mixture. Also, the concrete was cured for 56 days at ambient temperature and nine reinforced geopolymer concrete columns (four short and five slender columns) and one beam, all possessing a $150 \times 150 \text{ mm}^2$ cross sections were produced. The structural detailing of the concrete elements can be seen in Fig. 15.20.

The mentioned columns were subjected to an axial load, applied at varying eccentricities (from 0 to 145 mm). The results from the load tests performed to the control structural elements yielded 35 MPa to the strength of the short columns and the beam, and also, 30 MPa to the slender columns (the difference in the results is

Figure 15.19 Detail of the beam's steel reinforcement (Mathew & Joseph, 2018).

(A) Cross-section of the columns and the beam

(B) Dimensions and reinforcement arrangement of short and slender columns

(C) Dimensions and reinforcement arrangement of the beam

Figure 15.20 Structural detail of the columns and beams (Albitar et al., 2017). Structural detail of the columns and beams: cross-section (A) and side view of the column (B), side-view of the beam (C).

attributed by the authors to the water reducing agent used for each element, in the case Sika ViscoCrete 10 for the first situation and ViscoCrete-5-500 in the second batch). Despite the fact that the failure of the short, and of the slender columns, occurred for a displacement superior to the one corresponding to the peak load, but on the latter case, the excessive buckling led to excessive deformation of the structural element. The study found that the existing analytical models for the ultimate load-bending moment interacting curves overestimate the axial load of short columns by 25% and slender columns by 30% and thus may not be appropriate for the design of geopolymer concrete structural elements.

Wall panels are an example of yet another possible use for structural concrete and the existing research on the topic supports the suitability of AABC for this application. Mohd Ali, Sanjayan, and Guerrieri (2017) investigated the performance of fly ash-based high-strength AABC wall panels when subjected to a hydrocarbon fire. For this purpose, the authors used a binder composed of fly ash, and an activating solution in which NaOH was present at an 8 M concentration and a somewhat unusual silicate to hydroxide ratio of 0.5. Furthermore, the panels were steam cured at $(60 \pm 1)^{\circ}\text{C}$ for 24 h and were otherwise cured at ambient temperature and humidity, which the authors state were situated at $(23 \pm 1)^{\circ}\text{C}$ and $(50 \pm 3)\%$, until testing was performed. As to the steel reinforcement of the four $1075 \text{ mm} \times 1075 \text{ mm} \times 200 \text{ mm}$ panels fabricated, a square, 100 mm side, mesh possessing mild strength 6-mm-diameter steel bars was used. For the hydrocarbon fire testing, the samples were placed within a steel plate support structure that covered one of the sides of a furnace, in such a way that only a $1000 \text{ mm} \times 1000 \text{ mm}$ section of their surface was exposed to a temperature of roughly 1000°C for 120 min. During the test, it is reported that no explosive spalling was heard and following the test, visual inspection of the panels' surface showed that only minor surface spalling (in a depth of up to 2 mm) had occurred. Furthermore, weight measurements demonstrate that the panels' loss of weight due to the fire was only about 2% of the total mass (which the authors mostly attribute to loss of water content). Moreover, dilatometer testing performed to AAB and OPC mortar samples also showed that the incompatibility (the evolution of thermal strains with the increase in the sample's temperature) between the basalt aggregate and the paste is far greater for OPC compositions (Fig. 15.21).

Additionally, using the temperature data relative to the surface exposed to the fire, at a depth of 25 mm, the thermal diffusivity of the concrete at various temperatures was determined, by using the Finite Difference Back Calculation method, and the values compared to the thermal diffusivity of OPC concrete (with a maximum aggregate diameter of 14-mm panels). The results can be seen in Fig. 15.22 and are demonstrative of the superior fire performance of AABC, when compared to OPCC.

In another investigation covering the topic of wall panels produced with a fly ash-based AABC, Ganesan, Indira, and Santhakumar (2013) analyzed the structural performance of these elements in response to the modification of certain parameters of the structural design, most importantly, the slenderness and the aspect ratios (SR and AR, respectively). In terms of the mixture composition, the authors designed an alkaline solution containing a 14 M NaOH solution and a 2.5 silicate to hydroxide

Figure 15.21 Thermal strains versus temperature evolution relationships (Mohd Ali et al., 2017).

Figure 15.22 Thermal diffusivity of the AABC samples, compared to OPCC values (Mohd Ali et al., 2017).

ratio to activate a low calcium class F fly ash binder. Furthermore, the aggregates were used in a saturated surface dry state, and also, extra water and a naphthalene-based superplasticizer were added to the mixture. Additionally, a rectangular grid

of 6 mm high yield strength deformed bars was used in both directions, with a clear side cover of 10 mm, as steel reinforcement. Also, the researchers used vertical and horizontal reinforcement rates of 0.88% and 0.75%, respectively. The general structural details of the panels can be seen in Fig. 15.23.

Moreover, following a 3-day curing period in which the samples were covered (to avoid moisture loss), the walls were subjected to a 60°C, 24-h curing regimen. A total of five different geometries were designed for the panels, and 4 were produced for each geometry (two OPCC and two ABCC panels), which totals 20 panels fabricated. The geometry variations did not include the thickness of the wall, which remained constant at 40 mm, while height varied from 480 to 840 mm, and length from 320 to 560 mm. These modifications served the purpose of testing several slenderness and aspect ratios (and sometimes experiment with different varieties of the same ratio). Afterwards, the walls were placed in a vertical position and subjected to a slightly eccentric uniformly distributed axial load at a rate of 24.5 kN/min. It also important to state that compressive strength performed to both AABC and OPCC cubic concrete samples yielded a compressive strength of little over 40 MPa. Moreover, the authors observed the failure modes of the panels under axial loading and stated that, even though all OPC panels failed due to cracking formation on the top or bottom edges, the maximum increase in the slenderness ratio tested caused the AABC panel to fail through bending (which the researchers attribute to the AABC's larger volume of finer particles). Also, it is stated that an aspect ratio superior to 1.875 induced large lateral deflections to every panel. Moreover, it became clear to the authors, by the observation of the load–deflection curves of both OPC and AAB panels, that the AABC element behaves in a more ductile behavior than the OPC counterpart (much higher deflections

Figure 15.23 Detail of the wall's steel reinforcement (Ganesan et al., 2013).

for approximately the same load carrying capacity). Finally, after comparing the data with the values given by the application of the methods available in the literature for determining the axial strength ratio from the slenderness ratio (which were found to be somewhat conservative), the authors proposed new equations for the calculation of the ultimate load of AABC panels as the existing ones were found to be excessively conservative.

15.3 Conclusions

A thorough review of the existing literature on the topic of the structural performance of waste-based alkali-activated binder concretes is presented in this work. In it, a wide variety of binding material, structural elements, and possible external actions are addressed. As for the adoption by the construction industry of alkali-activated binder concretes, it is still in its early stages and the amount of data concerning the structural performance of these materials is relatively scarce. So far, the bulk of the research efforts have been centered around other extremely important topics, such as mix design, environmental impact analysis, durability performance or economic viability, among others. Nevertheless, the present knowledge on this technology, particularly on the theme of the structural performance of these concretes, already allows for some conclusions to be drawn:

- The alkali activation of different precursors leads to different reaction products, and consequently, to concretes with distinct structural performances. The most relevant chemical distinction between activation models is the calcium content of the starting materials.
- The most important waste material in high calcium activation models is GGBFS. The main reaction product of the activation of these materials (C-A-S-H) is the most similar to the one involved in the hydration of Portland cement (C-S-H), and thus, more parallels can be found between the properties displayed by this family of concretes and the ones presented by OPCCs.
- Following the same principle, fly ash is the most important waste material for use as a precursor in concretes possessing low calcium activation systems. Contrary to their calcium-rich counterparts, the principal reaction products of these concretes (N-A-S-H) are highly distinct from C-S-H gel. As a consequence, special care needs to be taken when considering the curing regimen of low-calcium waste-based AABC specimens, as the temperature, humidity and curing time strongly influence the activation mechanism.
- Other waste materials that have been used in structural AABC, either as secondary precursor or as fine aggregate, are dolomite waste, WQS, or lead smelter slag. Moreover, the addition of set retarders (such as zinc nitrate and sodium gluconate) or steel fibers, or even applying wax emulsions to the exposed concrete surfaces, or implementing a lime-saturated bath curing regimen may also be effective in enhancing the performance of the concretes.
- Regardless of the mixture design, steam, heat, or even, climate-controlled curing tends to benefit the activation reaction of alkali-activated concrete, and thus, improve the structural performance of these materials. The optimal conditions of the curing regimen are highly variable and still need to be better understood, so uniform standardized processes may be implemented.

- Although more research is needed, it is possible to produce blast furnace slag-based alkali-activated railway sleepers with lower CO₂ emissions and superior mechanical properties. Also, this material may be used when considering the design of columns in which ductile detailing is necessary, as the same performance may be obtained in a more cost-efficient fashion.
- AABC walls based on a blend of blast furnace slag and metakaolin (in equal parts) has been shown to be significantly superior to the ones based on Portland cement in resisting TNT blasts. In fact, while, following the explosion, the complete failure of an OPCC wall was observed, the structural integrity of the AABC wall was maintained.
- WQS may be a useful fine aggregate replacement, when considering the pullout strength of AABC as it optimizes the radial confining stress and the roughness of the concrete matrix. On the other hand, it may also significantly slow the geopolymerization reaction.
- In blast furnace slag-based AABC, the evolution of creep strain must be better understood, as the results appear to contradict the early development of strength that these concretes display. Furthermore, the effective modulus and age-adjusted effective modulus methods can be used to understand the creep behavior of these concretes, while the rate of creep method overestimates deflections.
- Meanwhile, digital image correction techniques may be successfully employed to understand the deflections of single-edge notched FA/GGBFS-based beams although the behavior of these elements displays high variability and further research is necessary. Also, the study of the impacts of a methane gas explosion (in reality, deflagration, as leaks were detected to have existed) on tunnel panels composed of these materials demonstrated their brittle nature (contrary to the control OPCC specimens, minor cracks were detected on these elements).
- Heat-cured, fly ash-based AABC beams demonstrated similar results to OPC-based beams. Also, splitting tensile strength is also typically higher in these materials and the load–deflection values of these elements tend to be better than the ones predicted by the elasticity theory.
- The results show that, with great probability, the shear-friction criteria may successfully be applied to designing heat-cured fly ash-based AABC beams. Also, the main limitation to the performance of these elements, when subjected to extreme heat, is the failure of the steel reinforcement and the ductility of the beams is seriously downgraded after 800°C. For lower temperatures, however, the strain compatibility approach usually utilized in the design of OPCC elements may also be applied to the ones based on AABC.
- Structural elements containing a binder composed of fly ash and lead smelter slag show excessive deformation when axial load testing is performed. These binders are, therefore, not recommended.
- Steam-cured fly ash-based AABC wall panels have been shown to be highly resistant to high temperatures as the experienced weight loss resulting from the heat was shown to mainly due to the reduction in water content and explosive spalling was not detected.
- Wall panels composed of heat-cured fly ash-based AABC subjected to one-way action demonstrated higher ductility, when compared to the OPCC equivalents. As old methods for determining ultimate load were found to be too conservative, new equations were proposed.

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