

Cracking Potential of Alkali-Activated Concrete Induced by Autogenous Shrinkage

Li, Zhenming; Zhang, Shizhe; Liang, Xuhui; Kostiuchenko, Albina; Ye, Guang

DOI 10.1007/978-3-030-76551-4_22

Publication date 2021 **Document Version** Accepted author manuscript

Published in Proceedings of the 3rd RILEM Spring Convention and Conference (RSCC 2020)

Citation (APA)

Li, Z., Zhang, S., Liang, X., Kostiuchenko, A., & Ye, G. (2021). Cracking Potential of Alkali-Activated Concrete Induced by Autogenous Shrinkage. In I. B. Valente (Ed.), *Proceedings of the 3rd RILEM Spring Convention and Conference (RSCC 2020)* (pp. 239-245). (RILEM Bookseries; Vol. 33). Springer. https://doi.org/10.1007/978-3-030-76551-4_22

Important note

To cite this publication, please use the final published version (if applicable). Please check the document version above.

Copyright

Other than for strictly personal use, it is not permitted to download, forward or distribute the text or part of it, without the consent of the author(s) and/or copyright holder(s), unless the work is under an open content license such as Creative Commons.

Takedown policy Please contact us and provide details if you believe this document breaches copyrights. We will remove access to the work immediately and investigate your claim.

1 2	Cracking potential of alkali-activated slag and fly ash concrete subjected to restrained autogenous shrinkage
3	Zhenming Li ^{a,*} , Shizhe Zhang ^a , Xuhui Liang ^a , Guang Ye ^{a,b}
4 5	^a Department of Materials and Environment (Microlab), Faculty of Civil Engineering and Geoscience, Delft University of Technology, Delft, the Netherlands
6	^b Magnel Laboratory for Concrete Research, Department of Structural Engineering, Ghent University, Ghent, Belgium
7	

8 Abstract

9 This study aims to investigate the cracking potential of alkali-activated slag (AAS) and alkali-10 activated slag-fly ash (AASF) concrete subjected to restrained autogenous shrinkage. 11 Temperature Stress Testing Machine (TSTM) is utilized, for the first time, to monitor the stress 12 evolution and to measure the cracking time of alkali-activated concrete (AAC) under restraint 13 condition. The stresses in AAS and AASF concrete are calculated based on the experimental 14 results while taking into consideration the influence brought by creep and relaxation. It is 15 found that AAS and AASF concrete showed lower autogenous shrinkage-induced stress and 16 later cracking compared to ordinary Portland cement (OPC) based concrete with similar 17 compressive strength, despite the higher autogenous shrinkage of AAS and AASF concrete. 18 The low autogenous shrinkage-induced stress in the AAC is mainly attributed to the 19 pronounced stress relaxation. A good prediction of the stress evolution in AAC is obtained by 20 taking into account the elastic part of the autogenous shrinkage and the stress relaxation. In 21 contrast, calculations ignoring the creep and relaxation would lead to a significant 22 overestimation of the stress in AAC.

- 23 Keywords: cracking, shrinkage, stress, alkali-activated materials, concrete, modelling
- 24

25 **1. Introduction**

Alkali-activated materials (AAMs) have emerged as eco-friendly alternatives to OPC as binder materials in engineering practice [1]. In comparison to OPC, AAMs can significantly reduce the environmental impacts of concrete products by lowering greenhouse gas emissions and the embodied energy [2]. Furthermore, using AAMs as binders could also contribute to the repurposing of industrial by-products [3,4].

31 Currently, granulated blast-furnace slag (hereinafter termed slag) and powered coal fly ash 32 (hereinafter termed fly ash) are the two most widely utilized precursors for AAMs. Among two 33 types of fly ash, namely Class C and Class F fly ash according to ASTM C618 [5], Class F fly ash 34 with reactive CaO \leq 10% has been intensively studied worldwide due to its wide availability 35 and high content of amorphous aluminosilicate phases [6]. Alkali activator is usually an alkali 36 metal hydroxide and/or silicate [7]. Although Na₂CO₃ and Na₂SO₄ can also be used as 37 activators [8,9], the majority of studies have shown that activation with Na₂SiO₃ and/or NaOH 38 provides high mechanical properties for slag and fly ash-based AAMs [10]. NaOH activators are found to accelerate early-age activation but tend to present a barrier to advanced
 reactions, therefore limiting the later-age strength [11]. In contrast, the reaction of Na₂SiO₃ activated slag is comparatively slow and results in the formation of very dense products with

42 improved mechanical strength [12].

43 Despite the promising mechanical and eco-friendly performance, slag and fly ash-based AAMs 44 have not received broad industry acceptance, primarily due to the uncertain long-term 45 durability and volume stability [13–15]. As reported in [16–25], slag and fly ash-based AAMs, 46 especially those activated by NaOH/Na₂SiO₃, can show higher autogenous shrinkage than OPC 47 based systems. The high autogenous shrinkage of AAMs is concerned by both academia and 48 industrial communities because it can potentially induce cracking of concrete [26,27]. In 49 practice, concrete as building material is normally applied under restrained conditions. The 50 restraint can be either external (e.g. caused by adjoining structure) or internal (e.g., caused by 51 the reinforcement or the non-shrinking aggregates) to the concrete member [28–30]. The 52 volume change of the concrete would be therefore limited to a certain extent and 53 consequently internal tensile stress would develop. Cracking would occur when the tensile 54 stress within concrete exceeds the tensile strength, which can lead to a series of problems 55 regarding mechanical properties, durability and aesthetics of concrete structures [31]. 56 Therefore, the cracking potential of activated slag and fly ash concrete has to be evaluated to 57 provide a solid theoretical basis for safe and reliable application of these materials.

58 From the high autogenous shrinkage of alkali-activated slag and fly ash systems, one may 59 deduce that AAC will show higher cracking potential compared to OPC based concrete. 60 However, the cracking potential of concrete is determined not only by autogenous shrinkage 61 but also by elastic modulus, creep/relaxation and tensile strength of the material. However, 62 very few studies on this topic can be found from the literature [32].

The aim of this study, therefore, is to evaluate the autogenous shrinkage-induced cracking potential of AAS and AASF concrete. Temperature Stress Testing Machine (TSTM) is utilized to track the internal stress evolution and cracking initiation of the concrete under restraint condition. With the measured autogenous shrinkage and elastic modulus as inputs, the internal stresses in AAS and AASF concrete are calculated and compared with the experimental results. The roles of creep and relaxation in influencing the stress evolution and cracking potential of AAC are highlighted.

70 2. Materials and methods

71 2.1 Raw materials and mixtures

The precursors used were slag supplied by Ecocem Benelux B.V and fly ash from Vliegasunie B.V. The chemical compositions of slag and fly ash were determined by X-ray fluorescence (XRF) and shown in Table 1. The fly ash complies with Class F (EN 450, ASTM C618) since it has low CaO content (< 10% reactive CaO) and content of "SiO₂ + Al₂O₃ + Fe₂O₃" higher than 70%. The particle size of slag, as measured by laser diffraction, ranges from 0.1 to 50 µm, with a d₅₀ of

- 18.3 μ m. The particle size of fly ash is between 0.14 and 138 μ m, with a d₅₀ of 48.1 μ m.
- 78
- 79

80 Table 1. Chemical propositions of slag and fly ash.

Oxide (wt. %)	SiO ₂	Al ₂ O ₃	CaO	MgO	Fe ₂ O ₃	SO₃	K ₂ O	TiO₂	Other	LOI
Slag	31.77	13.25	40.50	9.27	0.52	1.49	0.34	0.97	0.21	1.31
Fly ash	56.8	23.8	4.8	1.5	7.2	0.3	1.6	1.2	1.6	1.2

81 LOI= Loss on ignition

82

The alkaline activator was prepared by mixing anhydrous pellets of sodium hydroxide with deionized water and commercial sodium silicate solution. For 1000 g of precursor, an activator containing 384 g of water, 1.146 mol of SiO₂ and 0.76 mol of Na₂O was applied. The water-tosolid ratio is therefore 0.344 if the alkali in the activator is considered as solid. The activator was prepared at least one day before the casting of the concrete so that the solution was cooled down to ambient temperature.

89 The concrete mixture designs are shown in Table 2.

90

91 Table 2. Mixture design of AAS and AASF concrete (kg/m³).

Mixtures	AAS	AASF	
Slag	400	200	
Fly ash	0	200	
Activator	200	200	
Aggregate [0-4 mm]	789	789	
Aggregate [4-8 mm]	440	440	
Aggregate [8-16 mm]	525	525	
Admixtures	-	-	

92

93

94 2.2 Experimental methods

95 2.2.1 Mechanical properties

96 Concrete cubes (150 × 150 × 150 mm³) for compressive and splitting strength tests and prisms
 97 (100 × 100 × 400 mm³) for elastic modulus test were cast and cured in sealed and temperature 98 controlled steel moulds. The moulds were connected with cryostats by parallel circulation
 99 tubes and the upper surface was sealed by plastic film. The temperature of the concrete cubes
 100 was controlled at 20 °C.

101 Compressive strength and splitting strength of the concrete were measured according to NEN-

102 EN 12390 [33]. The measurements were conducted at the age of 1, 3, 7, 28 days and the day

103 when the beam in TSTM cracked. One cube was tested for compressive strength and two for

104 splitting strength.

- 105 The elastic modulus of the concrete was tested by a Tonibank hydraulic Instron (Figure 1). The
- 106 strain during loading was measured by linear variable differential transducers (LVDTs) aligned
- 107 vertically on the four sides of the concrete prism. The loading and unloading of each sample
- 108 went through four cycles with the stress ranging from 5% to 30% of the compressive strength
- 109 of the concrete. The results obtained from the latter three cycles were used in the calculation
- 110 of the elastic modulus. The loading and unloading rates were 1 kN/s [34]. Two samples were
- 111 tested for each mixture at each age.



- 112
- 113

Figure 1. Testing set-up for elastic modulus measurement on AAC.

114

115 2.2.2 Autogenous shrinkage

The autogenous shrinkage of the concrete was measured with an Autogenous Deformation Testing Machine (ADTM) [35]. The prismatic mould for the concrete is made of thin steel plates and external insulating materials. The size of the mould is 1000 x 150 x 100 mm³, which is illustrated in Figure 2. The mould was connected with cryostats by a series of circulation tubes located between the plates and the insulating material (see Figure 2). The temperature

121 of the beam was controlled at 20 °C.



Figure 2. Front view (left) and cutaway view (right) of the mould of Autogenous Deformation Testing Machine
 (ADTM).

125

The length change of the concrete was measured with two external quartz rods located next to the side mould. LVDTs were installed at both ends of the rods. The LVDTs measured the movement of the steel bars which were cast in the concrete. The distance between the two cast-in steel bars was 750 mm. The instalment of LVDTs was conducted only when the concrete had reached sufficient strength to support them (see Figure 3). The measurement of the deformation of AAS and AASF concrete starts at 8h and 11h, respectively. Attention was

paid to the sealing of the moulds in order to avoid moisture loss to the environment.



133

134Figure 3. Installation of measuring bars and placeholders before casting (left) and installation of LVDTs when135the concrete is stiff enough to support the bars (right), after [35].

136

137 2.2.3 Autogenous shrinkage-induced stress

138 The internal tensile stress in the concrete induced by restrained autogenous shrinkage was 139 monitored by TSTM. A sudden drop of the stress indicated the occurrence of cracking. The 140 TSTM was equipped with a horizontal steel frame (Figure 4) in which the concrete specimen 141 can be loaded in compression or in tension under various temperatures. The whole specimen 142 is of a dog-bone shape and the testing area of interest is of prismatic shape $(1000 \times 150 \times 100)$ 143 mm³). The middle part of the specimen is in a temperature-controlled mould, similar to the 144 ADTM mould as described in Section 2.2.2. Two rigid steel claws were used to grip the 145 concrete specimen. One of the claws was fixed to the steel frame while the other one lied on 146 roller bearings and thereby can move with the hydraulic actuator to provide a compressive or 147 tensile force onto the testing specimen. A short formwork was attached to the claws to 148 provide a smooth and curved transition between the straight insulated mould and the slanting 149 inner sides of the claws. The load was recorded with the load-cell with a loading capacity of 100 kN and a resolution of 0.049 kN. 150

During the first 8 hours after casting, LVDTs were used to control the deformation between the two claws because it was not yet possible to measure the deformations of the fresh concrete with the embedded steel bars. After that, the deformation control was switched to

the LVDTs that measure the displacement between the embedded bars. The deformation of

the concrete was kept at zero (nominally, in reality within 1 μm range) so that a full restraint

156 condition was provided.





Figure 4. Top view of the Temperature Stress Testing Machine (TSTM) [36].

159 **2.3 Calculation of the autogenous shrinkage-induced stress**

160 While the autogenous shrinkage-induced stress in restrained concrete can be monitored by 161 TSTM, the test is very time-consuming and labour-intensive. Besides, TSTM is currently not 162 equipped widely enough to serve as a standard apparatus. Therefore, it would be meaningful 163 if the stress evolution can be predicted based on the experimental results obtained in concrete 164 under free conditions.

165 **2.3.1** Calculation of the stress based on autogenous shrinkage

166 If the autogenous shrinkage is assumed to be purely elastic deformation [29], the stress (σ_{AS}) 167 generated in the concrete due to restrained shrinkage can be calculated using Equation 1. A 168 schematic representation of the calculation process is shown in Figure 5.

169
$$\sigma_{AS} = \varepsilon_{AS} \times E \tag{1}$$

170 where ε_{AS} is the measured autogenous shrinkage of the concrete and *E* is the measured 171 elastic modulus of the concrete.





Figure 5. Schematic representation of stress calculated based on autogenous shrinkage.

174 **2.3.2** Calculation of the stress based on the elastic part of the autogenous shrinkage

175 In fact, the concrete, either OPC based concrete or AAC, is not a purely elastic material [26,37]. 176 Under external or internal load, the concrete tends to generate time-dependent deformation, 177 or so-called creep, due to the viscoelasticity of the material. Therefore, a part of the 178 autogenous shrinkage of the concrete measured under free condition actually belongs to 179 creep deformation [38]. When the concrete is under externally restrained condition, the creep 180 part of the autogenous shrinkage does not have the potential to develop due to the 181 equilibrium between the internal driving force and the external restraint force. Therefore, the 182 stress in the restrained concrete is predominantly caused by the elastic part of the autogenous shrinkage [39]. If we assume the stress in the concrete is solely induced by the elastic part of 183 184 the autogenous shrinkage and do not consider the stress relaxation, the stress can be 185 calculated using Equations 2 and 3. A schematic representation of the calculation process is 186 shown in Figure 6.

187
$$\varepsilon_{AS} = \varepsilon_{AS,elas} + \varepsilon_{AS,creep}$$
 (2)

188 where ε_{AS} is the measured autogenous shrinkage of AAC. $\varepsilon_{AS,elas}$ and $\varepsilon_{AS,creep}$ are the elastic 189 part and creep part of the autogenous shrinkage of AAC, respectively.

190
$$\sigma_{AS,elas} = \varepsilon_{AS,elas} \times E$$
 (3)



191 where $\sigma_{AS,elas}$ is the stress induced by the elastic part of the autogenous shrinkage ($\varepsilon_{AS,elas}$).

193 Figure 6. Schematic representation of stress calculated based on the elastic part of the autogenous shrinkage.

194

192

The relationship between the creep deformation and the elastic deformation under a load canbe expressed by Equation 4 [26,40].

197
$$\varepsilon_{AS,creep}(t,\tau) = \varepsilon_{AS,elas}(\tau)\varphi(t,\tau)$$
 (4)

198 where $\varphi(t, \tau)$ is the creep coefficient. τ (days) is the time when the load is applied.

According to [39,41], the creep coefficient of OPC based concrete can be calculated by Equation 5. For AAC systems, however, no models are available yet to account for the creep 201 deformation. Therefore, Equation 5 is tentatively used in this study to calculate the creep 202 coefficient of AAC.

203
$$\varphi(t,\tau) = \left(\frac{\alpha(t)}{\alpha(\tau)} - 1\right) + 1.34 * \omega^{1.65} \tau^{-d} (t-\tau)^n \frac{\alpha(t)}{\alpha(\tau)}$$
 (5)

where α is the degree of reaction. ω is the water-to-solid ratio (0.344, see section 2.1). n is the relaxation factor whose value is empirically taken as 0.3 and *d* is the constant whose value is empirically taken as 0.35 according to [41].

207 The reaction degree α of the concrete can be calculated with Equation 6 [42–44].

$$208 \qquad \alpha(t) = \frac{Q(t)}{Q_{max}} \tag{6}$$

where Q(t) is the reaction heat and Q_{max} is the ultimate total heat at the completion of the reaction. Q(t) and Q_{max} of AAC are calculated according to the procedure described in Appendix A.

With Equations 2 and 4– 6, the elastic part and creep part of the autogenous shrinkage of AACcan be calculated.

214 2.3.3 Calculation of the stress by taking relaxation into account

Like the creep of the concrete under free condition, stress relaxation in concrete under restrained condition is another result of the viscoelasticity of the material. Due to the relaxation, the stress generated in restrained concrete would be reduced with the elapse of time. According to van Breugel [39], the stress induced by restrained elastic deformation can be calculated with Equation 7 by taking into consideration of the relaxation. A schematic representation of the calculation process of the stress induced by restrained autogenous shrinkage is shown in Figure 7.

222
$$\sigma_{relaxed}(t,\tau) = \sigma_{elas}(\tau)\psi(t,\tau)$$
 (7)



where $\psi(t, \tau)$ is the relaxation factor. τ (days) is the time when the load is applied.

Figure 7. Schematic representation of stress calculated based on the elastic part of the autogenous shrinkage by
 taking relaxation into account.

227 The relaxation coefficient of concrete can be calculated from the creep coefficient $\varphi(t,\tau)$ 228 using Equation 8 [39,45].

229
$$\psi(t,\tau) = e^{-\varphi(t,\tau)}$$
 (8)

The stress generated in the concrete after a certain curing age can be considered as the accumulations of the stresses resulting from the elastic deformations that occurred at previous time intervals, e.g. from τ_1 to τ_{n-1} , as shown in Equation 9.

233
$$\sigma_{relaxed} = \sum_{k=1}^{n-1} \Delta \sigma_{relaxed}(\tau_k)$$
(9)

where $\Delta \sigma_{relaxed}(t_k, \tau_k)$ is the increment of elastic shrinkage-induced stress after relaxation from τ_k to τ_{k+1} .

A schematic representation of the calculation process mentioned above is shown in Figure 8.



239

237

238

240 **3. Experimental results and discussion**

241 3.1 Mechanical properties

242 **3.1.1** Compressive and splitting tensile strength

The strength development of the concrete is shown in Figure 9. It can be seen that AAC generally shows high strength. At the age of 28 days, the compressive strength of AAS and AASF concretes is around 90 MPa and 75 MPa, respectively. According to EN 206 [46], the strength can be classified as C60 or higher. AAS concrete shows higher compressive and splitting strength than AASF concrete, which is consistent with the findings on the positive correlation between slag/fly ash ratio and the mechanical properties in the literature [47–49]. The lower strength of AASF concrete is due to the replacement of slag by fly ash, which has a

low reactivity at ambient temperature [50]. The dissolution of fly ash particles is slow no matter in cementitious systems or in alkali-activated systems [50,51]. Nonetheless, with the elapse of time the amorphous phases in fly ash would eventually contribute to the strength growth [51], which can be reflected by the considerable increase of the compressive strength of AASF concrete from 7 days to 28 days.



Figure 9. Compressive (a) and splitting strength (b) of AAC. For the splitting strength, the error bar is shown in
 the diagram, but it is too small to be clearly distinguished from the marker.

258

259 Despite the high compressive strength of AAC mixtures, their splitting tensile strength is not 260 remarkably high. This can be seen more clearly in Figure 10, in which the splitting tensile 261 strength -to-compressive strength ($f_{t,sp}/f_c$) ratios of the concrete are plotted. The $f_{t,sp}/f_c$ ratio 262 is an important parameter that allows the estimation of $f_{t,sp}$ by knowing f_c or vice versa [52]. 263 The ratio also provides insight into the stress type (compression or tension) to which the 264 concrete is more prone. It can be seen from Figure 10 that AAS and AASF concretes showed 265 nearly identical $f_{t,sp}/f_c$ except at the age 3 days.

266 The f_{t.sp}/f_c ratio of AAC decreases from around 0.1 at the first day to only 0.055 at 28 days. This 267 decrease might be due to the development of microcracking within the concrete resulting from the restrained autogenous shrinkage of the material [53,54]. Although the samples for 268 269 strength test were not subject to external restraint, the aggregates can act as local restraints 270 to the autogenous shrinkage of the surrounding paste [55]. Due to the large autogenous 271 shrinkage of AAC (as will be discussed in the following section), microcracks may have 272 developed within the concrete, although no visual cracks were observed on the surface of the 273 sample. The development of microcracking can harm the increase of tensile strength. 274 Although the absolute tensile strength of AAC increased with the curing age as shown in Figure 275 9, the developing rate of the tensile strength became lower than that of the compressive 276 strength as indicated by the decreasing $f_{t,sp}/f_c$.





278

Figure 10. Splitting tensile strength-to-compressive strength ($f_{t,sp}/f_c$) ratios of AAC.

279

280 3.1.2 Elastic modulus

The elastic modulus of AAC is shown in Figure 11. AAS concrete showed higher elastic modulus than AASF concrete at the early age. After 7 days, however, the elastic modulus of AAS concrete started to decrease slightly while the one of AASF concrete kept increasing. At 28 days, AAS and AASF concrete showed similar elastic modulus.

285 The decline of elastic modulus of AAS concrete with curing age has also been reported by 286 Prinsse et al. [54], in which both reduced elastic modulus and splitting tensile strength were 287 observed in AAS concrete. In that study, the concrete was cured in a climate chamber with the 288 temperature at 20°C and the relative humidity above 95%, after demoulded at 1 day. The 289 reduction was attributed to the leaching of ions from the sample and the change of curing 290 condition when the samples were taken out from the climate chamber to ambient condition 291 for testing [54]. The loss of structural ions due to leaching and the severe drop of the 292 environment relative humidity from above 95% to around 50% may induce substantial drying 293 shrinkage and related microcracking, thus damaging the microstructure of the concrete [56]. 294 This study, in contrast, applied sealed curing before the samples were tested in ambient 295 relative humidity. Therefore, no leaching occurred. The change of the environment RH and 296 the impact on the consequent drying shrinkage should be much less severe than that reported 297 in [54]. However, the autogenous shrinkage, as will be discussed in the coming section, had 298 already been developing before the exposure of the samples for strength tests, which can also 299 induce microcracking. Therefore, the reduced elastic modulus of AAS concrete herein is 300 believed to be due to the same reason for the reduced $f_{t,sp}/f_c$ of the concrete, which is the 301 development of microcracking caused by autogenous shrinkage.



- 302
- 303

Figure 11. Elastic modulus of AAC.

304

305 3.2 Autogenous shrinkage

The autogenous shrinkage of the concrete is shown in Figure 12. AASF concrete showed lower autogenous shrinkage than AAS concrete in the whole period studied. This findings are consistent with the autogenous shrinkage results on corresponding AAS and AASF pastes [37]. The autogenous shrinkage of AAS and AASF concrete develops rapidly at the first day and second day, respectively, which are in line with the accelerated reaction stages of the mixtures (see Figure A.1). At the age of 21 days, the autogenous shrinkage of AAS and AASF concrete reaches 609 and 325 μm/m, respectively.



313

314

- Figure 12. Autogenous shrinkage of AAC obtained from ADTM measurements.
- 315

316 3.3 Autogenous shrinkage-induced stress

Figure 13 shows the stress development in AAS and AASF concrete tested by TSTM. The sudden drop in the stress to around zero indicates the occurrence of cracking in concrete. It can be seen that AAS concrete showed a rapid stress development due to the high autogenous shrinkage. The stress in AASF concrete remains low in the first 1.5 days due to low shrinkage (see Figure 12). The tensile stress generated in AASF concrete was substantially lower than in
 AAS concrete. The lower autogenous shrinkage and also the lower elastic modulus of AASF
 concrete contribute to the lower stress in AASF concrete than in AAS concrete. Nonetheless,
 the tensile strength (1.4 MPa) tested by TSTM of AASF concrete was also lower than that of
 AAS concrete (2.7 MPa). Eventually, AASF concrete cracked even earlier than AAS concrete.

326 The classification of cracking potential according to ASTM C1581 [36] is shown in Table 3, 327 where cracking time (in days) and average stress rate (in MPa/day) were considered. 328 Accordingly, the cracking potential of the two concrete mixtures belongs to the category 329 "moderate-low". Compared with the results obtained by TSTM on OPC based concrete in the 330 literature [31,57,58], it can be seen that AAC showed lower stress and later cracking than OPC 331 based concrete with similar strength. The lower cracking potential of AAC than OPC based 332 concrete with similar or even lower strength is very positive information for the application of 333 AAC as a building material.

- 334 The reason why AAC showed higher autogenous shrinkage but later cracking initiation than
- 335 OPC based concrete is believed to be mainly due to the stress relaxation. The evident

relaxation of AAC is in line with the large creep of AAM paste identified in [59,60]. These two

phenomena, creep and relaxation, both originate from the viscoelasticity of the material [61].

In the next section, the creep part of the autogenous shrinkage will be quantified.



- 339
- 340

Figure 13. Autogenous shrinkage-induced stress in AAC obtained by TSTM measurements.

341

342 Table 3. Classification of cracking potential according to ASTM C1581 [36].

Cracking time t _{cr} (days)	Average stress rate S (MPa/day)	Potential for cracking
$0 < t_{cr} \le 7$	S ≥ 0.34	High
$7 < t_{cr} \le 14$	0.17 ≤ S < 0.34	Moderate-High
$14 < t_{cr} \le 28$	$0.10 \le S < 0.17$	Moderate-Low
t _{cr} > 28	S < 0.10	Low

344 4. Numerical results and discussion

345 4.1 Calculated stress based on autogenous shrinkage

The calculated stress with Equation 1, based on the autogenous shrinkage (see Figure 12) and the elastic modulus (see Figure 11) of the concrete, are presented and compared with measured stress in Figure 14. It can be seen that the calculated stress in AAC is around 7-8 times higher than the experimentally measured one. These results confirm that the creep/relaxation plays an important role in the stress development in AAC. The stress is significantly overestimated when the creep part in the autogenous shrinkage and the stress relaxation are not considered.



Figure 14. Calculated stress in AAS (a) and AASF (b) concrete from the autogenous shrinkage and the elastic
 modulus of the concrete.

356 4.2 Calculated stress based on the elastic part of autogenous shrinkage

According to Equations 2 and 4 – 6, the elastic part and creep part of the autogenous shrinkage
 of AAC can be calculated, as shown in Figure 15.



359

353

Figure 15. Calculated elastic part and creep part of autogenous shrinkage of AAS (a) and AASF (b) concrete. The
 autogenous deformation curve for AASF concrete is modified during modelling in order to exclude the
 influence of expansion

The calculated stress in AAC according to Equation 3 is shown in Figure 16. It can be seen that considering only the elastic part of autogenous shrinkage in calculation gives a much better prediction of the shrinkage-induced stress than the results shown in Figure 14, where the total autogenous shrinkage was used as input. Nonetheless, the calculated stress is still two times higher than the stress measured by TSTM, indicating that the relaxation of the stress has to be considered in order to estimate the time-dependent stress.



369

Figure 16. Calculated stress in AAS (a) and AASF (b) concrete from the elastic part of the autogenous shrinkage
 and the elastic modulus of the concrete.

372

373 4.3 Calculated stress by taking relaxation into account

According to Equations 7-9, the stress in AAC by taking relaxation into account is calculated 374 375 and shown in Figure 17. It can be seen that the calculation considering the elastic part of the 376 autogenous shrinkage and the relaxation of the stress with time provides a fairly good 377 agreement between calculated and measured stress evolutions in AAC. . For AAS concrete, 378 the calculation underestimates the stress in the first 7 days while overestimates the stress at 379 20 days. For AASF concrete, an opposite trend is observed. This discrepancy is probably 380 because these two concrete mixtures have different creep compliances, but in the calculation, 381 the same model was used.

382 In Equations 4-6, the water-to-solid ratio and the reaction degree were considered for the 383 creep calculation of the paste, however, the creep behaviour of concrete depends not only on 384 the deformability of the paste but also on the restraining effect of the aggregates. The 385 restraining effect of aggregates is determined not only by the size and volume fraction of the 386 aggregates but also by the interface between paste and aggregates, viz. the interfacial 387 transition zone (ITZ) [62,63]. In this study, the size and the volume fraction of aggregates for 388 AAS and AASF concrete are the same (see Table 2), however, differences in the ITZ properties 389 of AAS concrete and AASF concrete were found in previous studies [48,54] which investigated 390 the same mixtures as this study. Generally, AASF concrete has more porous and weaker ITZ 391 than AAS concrete [48,54]. Although the influence of ITZ on the deformation of concrete has 392 not been clearly understood [64-66], it has been found that the creep of the ITZ is higher 393 compared to the bulk matrix [67]. Therefore, the creep compliance of AASF concrete is 394 supposed to be less restrained by the aggregates due to the weaker ITZ. As a result, using the same equations and empirical parameters in calculation may lead to slight overestimation and
 underestimation of the creep in AAS and AASF concrete, respectively.



Figure 17. Calculated stress in AAS (a) and AASF (b) concrete from the elastic part of the autogenous shrinkage
 and the elastic modulus of the concrete, with the stress relaxation taken into account.

400

397

401 Despite the reason for the small discrepancies exhibited in Figure 17, the calculations in this 402 section have clearly shown the important roles played by relaxation in the stress evolution of 403 restrained AAC. Neglecting the creep and relaxation behavious would lead to an 404 overestimation of the self-induced stress in AAC.

405 4.4 Estimation of the cracking initiation of AAC

406 With the calculated stress and tensile strength, the cracking initiation of the concrete can be 407 estimated. Due to the difficulty in conducting the uniaxial tensile test on concrete, the tensile 408 strength measured by the splitting test on concrete cubes or cylinders is usually used to 409 estimate the cracking potential [68]. However, it should be noted that the maximum stress of 410 concrete at failure is normally lower than the splitting tensile strength. The first reason is that 411 the tensile strength of concrete measured uniaxially is normally lower than the tensile 412 strength measured by splitting tests [69]. The second reason is the reducing effect of 413 relaxation on the tensile strength. While the relaxation can reduce the stress, it can also 414 aggravate the development of microcracking, which is detrimental to the tensile strength 415 [70,71]. Due to the local creep, viz. change of the internal geometric constitution of the paste, 416 the internal stress is redistributed with time, causing a relief of stress in higher stressed zones 417 but new stress concentration in other zones [35]. As a result, additional local failure within the 418 concrete can be induced. This is probably the reason why the tensile strength of concrete 419 which experiences creep/relaxation under restrained condition is normally lower than that of 420 concrete which is free from loading [70,72]. Therefore, the influence of relaxation on the 421 tensile strength needs also a consideration when estimating the cracking time of the concrete. 422 The third reason is that the cube under splitting stress has to fail in the middle, while the beam 423 under uniaxial tensile stress will fail at the weakest cross-section. Due to the larger size of the 424 beam than the cube, the tensile strength of the weakest cross-section should be statistically 425 lower than that of the middle cross-section of the cube. Hence, a reduced tensile strength is 426 normally considered, as shown in Figure 18.



427

428 Figure 18. A schematic diagram of the stress development and the resultant cracking of concrete due to 429 restrained shrinkage.

430

431 For OPC based concrete, the failure stress-to-splitting strength ratios were in the range of 0.7 432 - 0.8 [35,73,74]. Based on experiments on a dozen specimens, Lokhorst [35] found OPC based 433 concrete, on average, failed at 75% of the actual tensile splitting strength irrespective of the 434 age. However, the failure stress-to-splitting strength ratios for AAS concrete and AASF 435 concrete in this study were only 0.56 and 0.37, respectively. These ratios were lower than 436 those for OPC concretes, possibly because of the more evident creep/relaxation in AAC. 437 Besides, it should be noted that only a limited number of samples were tested in this study. 438 To obtain a representative reduced factor of the tensile strength of AAC, intensive 439 experimental work on numerous samples and mixtures is needed.

440 **5. Concluding remarks**

In this study, the cracking potential of AAS and AASF concrete subjected to restrained
 autogenous shrinkage is evaluated. Based on the experimental and numerical results, the
 following remarks can be made:

- 444 1. AAC concrete shows generally high compressive strength. The $f_{t,sp}/f_c$ and the elastic 445 modulus of AAC decrease with the curing age, which may be due to the development 446 of microcracking resulting from the continuous autogenous shrinkage.
- TSTM is utilized for the first time to track the stress evolution and cracking initiation of
 AAC. AAC is found to show moderate-low cracking potential, despite its high
 autogenous shrinkage. The low autogenous shrinkage-induced stress in AAC is mainly
 attributed to the pronounced relaxation of the concrete.
- With the elastic part of the autogenous shrinkage and the stress relaxation taken into
 account, a very good prediction of the stress evolution in AAC is obtained. In contrast,
 calculations without considering the creep and relaxation would lead to a significant
 overestimation of the stress in AAC.

4. A reducing factor of the splitting tensile strength needs to be considered when
estimating the cracking time of AAC, since the failure stress of the restrained concrete
beam is lower than the splitting tensile strength of cubes that are free from load.

458

459 Acknowledgment

Zhenming Li and Xuhui Liang would like to acknowledge the funding supported by the China
Scholarship Council (CSC) under grant No. 201506120072 and No. 201806050051. This work
is supported also by the grant from the Netherlands Organisation for Scientific Research
(NWO). Prof. Klaas van Breugel is acknowledged for the discussion on the model. The two
anonymous reviewers are appreciated for their few but valuable comments.

465

466 Appendix A

467 In section 3.4, Q(t) and Q_{max} of AAC was used to calculate the reaction degree and the creep 468 coefficient.

469 In this study, a TAM Air isothermal calorimeter (Thermometric) was used to measure the 470 reaction heat of the pastes. Before measurements, the calorimeter was calibrated at 20 °C. 471 The temperature of the measuring channels of the calorimeter was controlled at 20 ± 0.01 °C. 472 Approximately 5 g of paste were cast into each glass vial and were immediately loaded into 473 the measuring channels. The internal diameter of the glass vial was 24.5 mm. The mixing and 474 loading procedures lasted about 15min from the moment of adding activator. The data was 475 recorded every 1 min to 7 days. The calorimetry results were normalized by the weight of the 476 paste. The heat flow of the paste is shown in Figure A.1 (a) and the Q(t) of AAS and AASF pastes during the first week of reaction was shown in Figure A.1 (b). Due to the same curing 477 478 condition (iso-thermal) of the paste and concrete samples, the reaction degrees of the paste 479 and the concrete are assumed identical at all time.







482

For the Q(t) after the first week and the Q_{max} of the paste/concrete, the method of curve fitting is needed. According to [44,75,76], the exponential model shown in Equation A.1 can provide a good Q_{max} prediction for AAMs.

486
$$Q(t) = Q_{max} \exp\left[-\left(\frac{\tau}{t}\right)^{\beta}\right]$$
(A.1)

487 where τ and β are the fitting parameters associated with the time and the shape of the 488 exponential model.

489 Due to the inability of Equation A.1 to fit multi-curvature evolutions, a piecewise 490 approximation by two functions is needed for the fitting of the heat flow curves of AAMs 491 paste. The fitted curves of the heat flow of AAS and AASF pastes are presented in Figure A.2, 492 in comparison with the experimental data. The fitting parameters and the accuracy of the 493 fitting (indicated by the adjusted R-square) are shown in Table A.1.



494

Figure A.2. Fitted heat flow of AAS paste (a) and AASF paste (b), in comparison with the experimentally
 measured heat flow.

497

498 Table A.1. Fitted parameters of the exponential model for the heat flow of AAS and AASF paste.

Mixtures	Q _{max}	τ	β	R-square
AAS curve 1	-	1.98	0.33	0.998
AAS curve 2	155.04	23.68	0.49	0.998
AASF curve 1	-	44.02	0.22	0.999
AASF curve 2	139.13	46.04	0.47	0.999

499

Figure A.2 and Table A.1 indicate a very good fitting of the curves. It can be seen in Table A.1 that AAS paste has a higher Q_{max} than AASF paste, which is reasonable. With the fitted parameters, the Q(t) and the reaction degree α of AAS and AASF concrete at different curing ages can be then calculated, as shown in Table A.2. The results were used to in section 3.4 to calculate the creep coefficient. 505 Table A.2. Q(t) and α of AAS and AASF concrete at different ages.

AAS concrete	8h	1d	2d	3d	7d	14d	20d
Q(t)	30.53	78.90	102.09	113.70	135.98	148.63	153.70
α	0.20	0.51	0.66	0.73	0.88	0.96	0.99
AASF concrete	11h	1d	2d	3d	7d	16.6d	-
Q(t)	29.77	53.67	77.03	88.11	110.65	127.00	-
α	0.19	0.39	0.55	0.63	0.80	0.91	-

506

507 **Reference**

- 508 [1] G. Habert, C. Ouellet-Plamondon, Recent update on the environmental impact of 509 geopolymers, RILEM Tech. Lett. 1 (2016) 17–23.
- 510 [2] J.L. Provis, S. a Bernal, Geopolymers and Related Alkali-Activated Materials, Annu. Rev.
 511 Mater. Res. 44 (2014) 299–327. doi:doi:10.1146/annurev-matsci-070813-113515.
- 512 [3]J.L. Provis, Alkali-activated materials, Cem. Concr. Res. 114 (2018) 40–48.513doi:10.1016/j.cemconres.2017.02.009.
- 514 [4] C. Shi, A.F. Jiménez, A. Palomo, New cements for the 21st century: The pursuit of an
 515 alternative to Portland cement, Cem. Concr. Res. 41 (2011) 750–763.
 516 doi:10.1016/j.cemconres.2011.03.016.
- 517 [5] ASTM C618 19, Standard specification for coal fly ash and raw or calcined natural 518 pozzolan for use in concrete, (2008). doi:10.1520/C0618-19.2.
- 519 [6] N. Gamage, K. Liyanage, S. Fragomeni, S. Setunge, Overview of different types of fly ash
 520 and their use as a building and construction material, (2013).
- 521 [7] J.L. Provis, Activating solution chemistry for geopolymers, in: Geopolymers, Elsevier, 522 2009: pp. 50–71.
- 523 [8] F. Collins, J.G. Sanjayan, Early age strength and workability of slag pastes activated by 524 NaOH and Na2CO3, Cem. Concr. Res. 28 (1998) 655–664.
- H. Tan, X. Deng, X. He, J. Zhang, X. Zhang, Y. Su, J. Yang, Compressive strength and hydration process of wet-grinded granulated blast-furnace slag activated by sodium sulfate and sodium carbonate, Cem. Concr. Compos. 97 (2019) 387–398. doi:https://doi.org/10.1016/j.cemconcomp.2019.01.012.
- 529 [10] J.L. Provis, J.S.J. Van Deventer, Alkali Activated Materials, 2014. doi:10.1007/978-94-530 007-7672-2.
- 531 [11] B.S. Gebregziabiher, R.J. Thomas, S. Peethamparan, Temperature and activator effect
 532 on early-age reaction kinetics of alkali-activated slag binders, Constr. Build. Mater. 113
 533 (2016) 783–793. doi:10.1016/j.conbuildmat.2016.03.098.

534[12]M. Ben Haha, G. Le Saout, F. Winnefeld, B. Lothenbach, Influence of activator type on535hydration kinetics, hydrate assemblage and microstructural development of alkali536activated blast-furnace slags, Cem. Concr. Res. 41 (2011) 301–310.

- 537 doi:10.1016/j.cemconres.2010.11.016.
- F. Pacheco-Torgal, Z. Abdollahnejad, S. Miraldo, M. Kheradmand, Alkali-activated cement-based binders (AACBs) as durable and cost-competitive low-CO2 binder materials: some shortcomings that need to be addressed, Butterworth-Heinemann, Oxford, UK, 2017.
- 542 [14] F. Pacheco-Torgal, Z. Abdollahnejad, A.F. Camões, M. Jamshidi, Y. Ding, Durability of
 543 alkali-activated binders: A clear advantage over Portland cement or an unproven issue?,
 544 Constr. Build. Mater. 30 (2012) 400–405. doi:10.1016/j.conbuildmat.2011.12.017.
- 545 [15] M. Mastali, P. Kinnunen, A. Dalvand, R. Mohammadi Firouz, M. Illikainen, Drying
 546 shrinkage in alkali-activated binders A critical review, Constr. Build. Mater. 190 (2018)
 547 533–550. doi:10.1016/j.conbuildmat.2018.09.125.
- 548 [16] J. Ma, F. Dehn, Shrinkage and creep behavior of an alkali-activated slag concrete, Struct.
 549 Concr. 18 (2017) 801–810. doi:10.1002/suco.201600147.
- 550 [17] C. Cartwright, F. Rajabipour, A. Radli, Shrinkage Characteristics of Alkali-Activated Slag 551 Cements, J. Mater. Civ. Eng. 27 (2014) 1–9. doi:10.1061/(ASCE)MT.1943-5533.0001058.
- 552 [18] B.D. Kumarappa, S. Peethamparan, M. Ngami, Autogenous shrinkage of alkali activated
 553 slag mortars: Basic mechanisms and mitigation methods, Cem. Concr. Res. 109 (2018)
 554 1–9. doi:10.1016/j.cemconres.2018.04.004.
- 555 [19] S. Uppalapati, Ö. Cizer, Assessing the autogenous shrinkage of alkali- activated slag/fly 556 ash mortar blends, Am. Concr. Institute, ACI Spec. Publ. 2017-Janua (2017).
- 557 [20] G. Fang, W. Tu, Y. Zhu, M. Zhang, AUTOGENOUS SHRINKAGE OF ALKALI-ACTIVATED FLY
 558 ASH-SLAG PASTES WITH AND WITHOUT SAP, in: 4th Int. Conf. Serv. Life Des.
 559 Infrastructures, 2018: pp. 449–455.
- 560 [21] M. Nedeljković, Z. Li, G. Ye, Setting, Strength, and Autogenous Shrinkage of Alkali561 Activated Fly Ash and Slag Pastes: Effect of Slag Content, Materials (Basel). 11 (2018)
 562 2121. doi:10.3390/ma1112121.
- 563 [22] Z. Li, S. Zhang, X. Liang, G. Ye, Internal curing of alkali-activated slag-fly ash paste with 564 superabsorbent polymers, Constr. Build. Mater. (2020) (under review).
- 565 [23] Z. Li, M. Wyrzykowski, H. Dong, J. Granja, M. Azenha, P. Lura, G. Ye, Internal curing by
 566 superabsorbent polymers in alkali-activated slag, Cem. Concr. Res. 135 (2020) 106123.
 567 doi:10.1016/j.cemconres.2020.106123.
- 568 [24] Z. Li, M. Nedeljkovic, Y. Zuo, G. Ye, Autogenous shrinkage of alkali-activated slag-fly ash
 569 pastes, in: 5th Int. Slag Valoris. Symp., Leuven, 2017: pp. 369–372.
- 570 [25] T. Lu, Z. Li, H. Huang, Effect of Supplementary Materials on the Autogenous Shrinkage 571 of Cement Paste, Materials (Basel). 13 (2020) 3367.
- 572 [26] A.M. Neville, Properties of Concrete, 2011.
- 573 [27] E. Tazawa, Autogenous shrinkage of concrete, CRC Press, 1998.
- 574 [28] M.S. Sule, Effect of reinforcement on early-age cracking in high strength concrete, Delft
 575 University of Technology, 2003.
- 576 [29] P. Lura, Autogenous Deformation and Internal Curing of Concrete, Delft University of

577 Technology, 2003.

- 578 [30] Z. Li, A. Kostiuchenko, G. Ye, Autogenous shrinkage-induced stress of alkali-activated
 579 slag and fly ash concrete under restraint condition, in: ECI (Ed.), Alkali Act. Mater.
 580 Geopolymers Versatile Mater. Offer. High Perform. Low Emiss., Tomar, 2018: p. 24.
- 581 [31] S.I. Igarashi, A. Bentur, K. Kovler, Autogenous shrinkage and induced restraining
 582 stresses in high-strength concretes, Cem. Concr. Res. 30 (2000) 1701–1707.
 583 doi:10.1016/S0008-8846(00)00399-9.
- 584 [32] F. Collins, J.G. Sanjayan, cracking tendency of alkali-activated slag concrete subjected
 585 to restrained shrinkage, Cem. Concr. Res. 30 (2000) 791–798. doi:10.1016/S0008586 8846(00)00243-X.
- 587 [33] NEN-EN 12390-3, Testing hardened concrete Part 3: Compressive strength of test 588 specimens, (2009).
- [34] B. Delsaute, C. Boulay, J. Granja, J. Carette, M. Azenha, C. Dumoulin, G. Karaiskos, A.
 Deraemaeker, S. Staquet, Testing Concrete E-modulus at Very Early Ages Through
 Several Techniques: An Inter-laboratory Comparison, Strain. (2016) 91–109.
 doi:10.1111/str.12172.
- 593 [35] S.J. Lokhorst, Deformational behaviour of concrete influenced by hydration related 594 changes of the microstructure, Delft University of Technology, 2001.
- [36] ASTM C 1581, Standard Test Method for Determining Age at Cracking and Induced
 Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage,
 ASTM Int. (2009) 1–7. doi:10.1520/C1581.
- 598 [37] Z. Li, T. Lu, X. Liang, H. Dong, J. Granja, M. Azenha, G. Ye, Mechanisms of autogenous
 599 shrinkage of alkali-activated slag and fly ash pastes, Cem. Concr. Res. 135 (2020)
 600 106107. doi:10.1016/j.cemconres.2020.106107.
- [38] T. Lu, Autogenous shrinkage of early age cement paste and mortar, Delft University of
 Technology, 2019.
- 603 [39] K. Van Breugel, Relaxation of young concrete, (1980) 144.
- [40] Z.P. BAZANT, Prediction of concrete creep effects using age-adjusted effective modulus
 method, J. Am. Concr. Inst. 69 (1972) 212–217.
- 606 [41] H. van der Ham, E. Koenders, K. van Breugel, Creep model uncertainties in early-age
 607 concrete simulations, Proc. Concreep. 8 (2008) 431–436.
- K.A. Riding, J.L. Poole, K.J. Folliard, M.C.G. Juenger, A.K. Schindler, Modeling hydration
 of cementitious systems, ACI Mater. J. 109 (2012) 225–234.
- 610 [43] A.K. Schindler, K.J. Folliard, Heat of hydration models for cementitious materials, ACI
 611 Mater. J. 102 (2005) 24.
- 612 [44] D. Ravikumar, N. Neithalath, Reaction kinetics in sodium silicate powder and liquid
 613 activated slag binders evaluated using isothermal calorimetry, Thermochim. Acta. 546
 614 (2012) 32–43. doi:10.1016/j.tca.2012.07.010.
- 615 [45] F. Wittmann, Bestimmung physikalischer Eigenschaften des Zementsteins, 1974.
- 616 [46] NEN-EN 206-1, Concrete Part 1: Specification, performance, production and

- 617 conformity, Eur. Comm. Stand. (2001).
- 618 [47] A. Fernández-Jiménez, J.G. Palomo, F. Puertas, Alkali-activated slag mortars:
 619 mechanical strength behaviour, Cem. Concr. Res. 29 (1999) 1313–1321.
- 620 [48] M. Nedeljković, Carbonation mechanism of alkali-activated fly ash and slag materials:
 621 In view of long-term performance predictions, Delft University of Technology, 2019.
- F. Puertas, S. Martínez-Ramírez, S. Alonso, T. Vázquez, Alkali-activated fly ash/slag
 cements. Strength behaviour and hydration products, Cem. Concr. Res. 30 (2000) 1625–
 1632. doi:10.1016/S0008-8846(00)00298-2.
- [50] Y. Ma, Microstructure and Engineering Properties of Alkali Activated Fly Ash -as anenvironment friendly alternative to Portland cement, 2013.
- 627 [51] Z. Yu, Microstructure Development and Transport Properties of Portland Cement-fly628 Ash Binary Systems, 2015.
- [52] N. Arioglu, Z. Canan Girgin, E. Arioglu, Evaluation of ratio between splitting tensile
 strength and compressive strength for concretes up to 120 MPa and its application in
 strength criterion, ACI Mater. J. 103 (2006) 18–24. doi:10.14359/15123.
- [53] Z. Li, M. Nedeljković, B. Chen, G. Ye, Mitigating the autogenous shrinkage of alkaliactivated slag by metakaolin, Cem. Concr. Res. 122 (2019) 30–41.
 doi:10.1016/j.cemconres.2019.04.016.
- 635 [54] S. Prinsse, D.A. Hordijk, G. Ye, P. Lagendijk, M. Luković, Time-dependent material
 636 properties and reinforced beams behavior of two alkali-activated types of concrete,
 637 Struct. Concr. 21 (2020) 642–658. doi:10.1002/suco.201900235.
- 638 [55] P. Lura, O.M. Jensen, J. Weiss, Cracking in cement paste induced by autogenous
 639 shrinkage, Mater. Struct. 42 (2009) 1089–1099. doi:10.1617/s11527-008-9445-z.
- F. Collins, J.G. Sanjayan, Microcracking and strength development of alkali activated
 slag concrete, Cem. Concr. Compos. 23 (2001) 345–352. doi:10.1016/S09589465(01)00003-8.
- 643 [57] P. Lura, K. Van Breugel, I. Maruyama, Effect of curing temperature and type of cement
 644 on early-age shrinkage of high-performance concrete, Cem. Concr. Res. 31 (2001)
 645 1867–1872. doi:10.1016/S0008-8846(01)00601-9.
- 646 [58] A. Darquennes, S. Staquet, M.P. Delplancke-Ogletree, B. Espion, Effect of autogenous
 647 deformation on the cracking risk of slag cement concretes, Cem. Concr. Compos. 33
 648 (2011) 368–379. doi:10.1016/j.cemconcomp.2010.12.003.
- 649 [59] H. Ye, A. Radlińska, Shrinkage mechanisms of alkali-activated slag, Cem. Concr. Res. 88
 650 (2016) 126–135. doi:10.1016/j.cemconres.2016.07.001.
- [60] A. Kostiuchenko, J. Liu, Z. Aldin, Mechanical properties and creep behaivor of an alkaliactivated concretemechanical properties and creep behaivor of an alkali-activated concrete, in: Alkali Act. Mater. Geopolymers Versatile Mater. Offer. High Perform. Low Emiss., 2018: p. 9.
- [61] T.C. Hansen, Creep and stress relaxation of concrete: a theoretical and experimental
 investigation, Svenska forskningsinstitutet för cement och betong vid Kungl. Tekniska
 högskolan, 1960.

- 658 [62] R.L. Al-Mufti, A.N. Fried, Pulse velocity assessment of early age creep of concrete,
 659 Constr. Build. Mater. 121 (2016) 622–628.
- 660 [63] O. Bernard, F.-J. Ulm, J.T. Germaine, Volume and deviator creep of calcium-leached 661 cement-based materials, Cem. Concr. Res. 33 (2003) 1127–1136.
- M. Briffaut, F. Benboudjema, C. Laborderie, J.-M. Torrenti, Creep consideration effect
 on meso-scale modeling of concrete hydration process and consequences on the
 mechanical behavior, J. Eng. Mech. 139 (2013) 1808–1817.
- 665 [65] C. Pichler, R. Lackner, A multiscale creep model as basis for simulation of early-age 666 concrete behavior, Comput. Concr. 5 (2008) 295–328.
- 667 [66] P. Lura, M. Wyrzykowski, Influence of Aggregate Restraint on Volume Changes:
 668 Experiments and Modelling, in: Concreep, 2015: pp. 17–23.
 669 doi:doi:10.1061/9780784479346.
- [67] V. Zacharda, J. Němeček, H. Šimonová, B. Kucharczyková, M. Vyhl\'\idal, Z. Keršner,
 [67] Influence of Interfacial Transition Zone on Local and Overall Fracture Response of
 [67] Cementitious Composites, in: Key Eng. Mater., 2018: pp. 97–102.
- 673 [68] NEN-EN 12390-3, Testing hardened concrete Part 6: Tensile splitting strength of test
 674 specimens, (2009).
- 675 [69] British Standards Institution, Eurocode 2: Design of concrete structures: Part 1-1:
 676 General rules and rules for buildings, British Standards Institution, 2004.
- 677 [70] D.J. Cook, P. Chindaprasirt, Influence of loading history upon the tensile properties of
 678 concrete, Mag. Concr. Res. 33 (1981) 154–160. doi:10.1680/macr.1981.33.116.154.
- 679 [71] A.D. Ross, Creep of concrete under variable stress, in: J. Proc., 1958: pp. 739–758.
- [72] D.J. Cook, P. Chindaprasirt, Influence of loading history upon the compressive
 properties of concrete, Mag. Concr. Res. 32 (1980) 89–100.
- 682 [73] H.G. Heilmann, H. Hilsdorf, K. Finsterwalder, Strength and Deformation of Concrete
 683 Under Tensile Stress, Bulletin. (1969) 94.
- 684 [74] J. Byfors, Plain concrete at early ages, Cement-och betonginst., 1980.
- 685 [75] S. Chithiraputhiran, N. Neithalath, Isothermal reaction kinetics and temperature
 686 dependence of alkali activation of slag, fly ash and their blends, Constr. Build. Mater.
 687 45 (2013) 233–242. doi:10.1016/J.CONBUILDMAT.2013.03.061.
- [76] S. Zhang, Y. Zuo, Z. Li, G. Ye, Isothermal calorimetric study on heat evolution and
 apparent activation energy of alkali-activated slag/fly ash paste, in: 2nd Int. Conf.
 Sustain. Build. Mater., Eindhoven, 2019: pp. 1–8.

691