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Properties of early-age concrete relevant to cracking in massive concrete

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Abstract

This paper provides an overview of the early-age properties of concrete, from a materials science and concrete engineering perspective. The aim of this manuscript is to contribute to gap analysis and to improve the strategy for evaluation methods of the risk of cracking in mass concrete. Strength, elastic modulus, and volumetric stability at early ages are discussed.

1 Introduction

The temperature of mass concrete may increase substantially due to the combination of heat liberation caused by cement hydration and thermal boundaries that for very thick members may even approach adiabatic conditions. This temperature increase results in thermal deformation of structural members made of concrete and it influences both the kinetics of cement hydration and the phase assemblage of the hydration products.

25 When the temperature of mass concrete increases and then decreases again, thermal strain (expansion upon
26 heating and contraction upon cooling) occurs. Due to temperature gradients in the cross-section of a concrete
27 member, the core of the member (experiencing higher temperature) will initially expand more than the outer
28 skin (at lower temperature). Tensile stresses will consequently arise in the skin, which may cause surface
29 cracks. These cracks occur in the phase when the temperature is still increasing. They are normally relatively
30 thin and will close later when the temperature of the whole concrete member equilibrates. In the presence of
31 external restraint (e.g. any adjoining structure or the subgrade), compressive stresses will arise in the mass
32 concrete core when the temperature increases. These compressive stresses will eventually turn into tensile
33 stresses when the temperature decreases to equilibrate with the external environment. This is mainly caused
34 by the difference of Young's modulus of concrete during temperature increase and during temperature
35 decrease. The initial compressive stresses are quite low because of the low Young's modulus and high stress
36 relaxation of the concrete in the heat liberation phase. When the temperature decreases, the degree of
37 hydration of the concrete is higher and with it the Young's modulus, while relaxation/creep decreases with the
38 degree of hydration. Another reason why the tensile stresses will exceed the initial compressive stress is the
39 presence of other types of contraction, for instance autogenous shrinkage. Finally, if the temperature of the
40 fresh concrete is higher than the equilibrium temperature of the environment during the cooling phase, the
41 temperature difference during expansion will be smaller than that during cooling. Additionally, the coefficient
42 of thermal expansion (CTE) might be at a minimum at setting and then increase with self-desiccation. A
43 schematic illustration about the process of cracking in massive concrete structures is shown in Figure 1.
44 Even though, cracks will be present by design in reinforced concrete, cracking due to volume change will
45 enlarge the crack width and such cracks may jeopardize the durability of concrete. Therefore, the risk of
46 cracking needs to be evaluated and mitigation strategies need to be implemented.

47

Figure 1 Schematic figure for cracking of mass concrete

To predict the risk of thermal cracking at any age, it is necessary to calculate the self-induced stresses (stresses induced by either self-restraint or external restraint to deformations of the concrete) and compare them with the tensile strength of the concrete at that age. For a 1D configuration (which is always ideal and in practice needs to be adapted to the actual geometry), the stress can be computed as [1]:

$$\dot{\sigma} = E_c(t)(\dot{\epsilon} - \dot{\epsilon}_{sh} - \dot{\epsilon}_{cr} - \dot{\epsilon}_{th}) \quad (1)$$

58 where $E_c(t)$ is the Young's modulus at time t , the dot indicates derivative respect to time and ε is the total strain
59 defined as:

60

$$\varepsilon = \varepsilon_e + \varepsilon_{sh} + \varepsilon_{th} + \varepsilon_{cr} \quad (2)$$

62

63 Where e indicates elastic, sh shrinkage, th thermal and cr creep. The thermal strain and the shrinkage are the
64 driving forces of cracking, while the creep strains act to reduce the magnitude of self-induced stresses at early
65 ages.

66 In some cases, equation (1) is modified by introducing the effect of damage on the Young's modulus as
67 $(1 - D) \cdot E_c(t)$, where D is the damage coefficient due to cracking (see e.g. [2]); this approach is followed
68 especially when the cracking behavior is considered numerically. Equation (2) is modified by introducing the
69 plastic strain ε_{pl} , which occurs when the Young's modulus evolves (e.g., due to hydration or damage of
70 concrete) under sustained load, when the stress history results in irreversible creep, or when the thermal
71 expansion coefficient changes under temperature variation.

72 As already mentioned, Eq. 1 is valid only for the ideal 1D case. For actual, complex geometries the strains and
73 the stresses need to be calculated in 2D or 3D with a finite element model (FEM). It is remarked here that in
74 the general case, the elastic stresses will also depend on the Poisson's ratio. For typical geometries (e.g., walls,
75 piers, layer structures), the Japan Concrete Institute (JCI) Guidelines [3] and Architectural Institute of Japan
76 [4] already provide equations to evaluate the cracking risk. Also for typical geometries, reference can be made
77 to this ACI report [5]. It must be remarked that also for these simplified approaches, the knowledge of the
78 mechanical properties and of the volume changes of the concrete is essential.

79 The definition of mass concrete is complex because it depends both on the (thermal) boundary conditions
80 and on the material properties of the concrete. Mass concrete is defined in ACI 116R [6] as "any volume of
81 concrete with dimensions large enough to require that measures be taken to cope with generation of heat from
82 hydration of the cement and attendant volume change to minimize cracking." While this definition may be

83 useful in cases of litigation (e.g., if cracks in a structure are identified as thermal cracks, it may be inferred that
84 not enough measures have been taken), it cannot be used to identify dangerous situations beforehand. Ulm
85 and Coussy developed a definition of mass concrete that depends on a combination of concrete properties,
86 including heat liberation, heat transport, volume change of concrete (thermal deformation strain, shrinkage
87 strain, and creep strain), and physical property development (especially Young's modulus and tensile strength)
88 [7]. Following this approach, relatively thin elements made of high-performance concrete, with rapidly
89 developing stiffness and high heat liberation, may qualify as mass concrete just like much thicker sections of
90 normal strength concrete.

91 Due to the short period (typically a couple of days to a few days) in which the temperature of mass concrete
92 first reaches its peak and then decreases to approach the ambient temperature and the small surface to volume
93 ratio of mass concrete, the contribution of drying shrinkage to deformations and cracking can be considered
94 negligible. On the other hand, in the case of high-performance concrete, substantial autogenous shrinkage will
95 develop at the same time as the thermal deformations and the combined self-induced stresses might exceed
96 the tensile strength and result in cracking. In addition, both the autogenous shrinkage and the CTE are
97 controlled by self-desiccation, the decrease of internal relative humidity due to chemical shrinkage.

98 To evaluate the risk of thermal cracking, it is necessary to understand a number of interrelated phenomena,
99 including:

- 100 1) the hydration process of the cement with varying temperature (rate of hydration, activation energy of
101 hydration, phases formed at different temperatures, heat liberation, the final degree of hydration);
- 102 2) the heat capacity of the concrete, which combined with the potential heat of hydration of the cement
103 allows to calculate the adiabatic temperature rise (the theoretical maximum increase of temperature with no
104 heat losses, useful as an engineering approximation);
- 105 3) the thermal boundary conditions and the thermal conductivity of the concrete, which determine both
106 how much lower the maximum temperature in the concrete will be compared to the adiabatic temperature
107 and also how large the temperature gradients will be within the concrete member;

108 4) the CTE of the concrete, including its dependence on self-desiccation. In addition, the CTE of the different
109 components within the concrete (matrix and aggregates, and also steel reinforcement) will induce self-induced
110 stresses at the mesoscale that may result in microcracking;

111 5) the autogenous shrinkage of the concrete, which depends both on the self-desiccation (governed by
112 cement hydration) and on the temperature of the concrete;

113 6) the elastic properties of the concrete, which governs the magnitude of the self-induced stresses that may
114 induce cracking. The Young's modulus depends on the degree of hydration or the maturity of the concrete. A
115 different porosity or phase assemblage at higher temperature will affect the Young's modulus, as well as
116 microcracking arising e.g. from differential thermal expansion or autogenous shrinkage;

117 7) the stress relaxation will reduce the magnitude of the self-induced stresses in the concrete compared to
118 the elastic stresses. The stress relaxation is also affected by the degree of hydration/maturity, by the phase
119 assemblage and by the temperature;

120 8) the tensile strength of the concrete. The tensile strength is affected by the maturity and by the
121 temperature. Moreover, the tensile strength during a long-term sustained load might be different from the
122 short-term tensile strength;

123 9) Modeling and predicting boundary conditions for numerical calculation. Numerical calculation results
124 are greatly affected by the modeling and settings of boundary conditions. For the thermal problem, both the
125 temperature at the construction site and that at the production plant should be predicted; due to the
126 uncertainties of this prediction, the obtained results should be evaluated from statistical point of view;

127 10) When crack opening is of concern, early-age bond behavior (time development of bond properties, bond
128 behavior change under varying of loads, including bond creep under variation of stress, developments of
129 cracks around rebar due to autogenous shrinkage and temperature change).

130 This contribution addresses mainly the mechanical properties and the volume changes, while the hydration
131 process, the resultant heat production and the thermal properties (in particular specific heat and thermal
132 conductivity) have been covered elsewhere [8–13]. The main aims of this paper are 1) improving the current

understanding of cracking in massive concrete and 2) presenting improved strategies for numerical modeling of the risk of cracking, such as the JCI guidelines [3]. To evaluate the crack width in massive concrete, it is necessary to take into account the interaction between reinforcement bars and concrete; such structural aspects are not covered by this contribution. Drying is also an important issue for the quality of surface concrete, but this problem is also not covered here because it is not essential to understand cracking in massive concrete.

2 Strength and elastic modulus

The physical properties of concrete at early ages evolve with cement hydration. At the same time, they are also influenced by the temperature increase due to the exothermic reaction of cement with water and by temperature changes in the surrounding environment (including daily cycles) and by drying after demolding. The prediction of the properties of concrete is necessary for performance-based design of reinforced concrete [14], especially for long-term in-service performance and durability performance. As mentioned before, this contribution focuses mainly on the physical properties of mass concrete.

2.1 Strength

Hydration of cement confers strength and stiffness to concrete. As cement reacts with water to precipitate cement hydrates, van der Waals forces between the cement hydrates of increasing specific surface area lead to the development of strength and Young's modulus of hardened cement paste and concrete. Therefore, the degree of hydration of the cement dominates the strength development. In the assumption that the cement hydration process does not change with the progress of the reaction, the strength development can be modeled as a function of degree of hydration. Considering concrete as a porous medium, an universal inverse relationship exists between porosity and strength among porous materials [15]. The initial porosity in the fresh state (determined by the water-to-binder ratio, w/b) as well as the pore filling process by low-density hydrates (here, gel pore water and interlayer water are considered as a part of the calcium silicate hydrate, C-

S-H, which is the main phase of Portland cement hydrates) are the key phenomena for strength and stiffness development. Powers [16] correlated the 28-day compressive strength of mortars with the gel/space ratio, which is the volumetric ratio of hydrates to the available space in the matrix, excluding unreacted cement particles. The concept of gel/space ratio can be also applied to strength evolution in hardened cement paste at early ages [17,18]. When supplementary cementitious materials (SCM) are included in the system, the rate of reaction, the density of the C-S-H (as the Ca/Si ratio determines the density of the C-S-H [19][20]), and the other hydrates that are formed have a strong impact on the rate of strength development and on the ultimate strength. Due to the lower density of C-S-H as a consequence of lower Ca/Si atomic ratio, in cementitious systems containing pozzolanic materials the strength develops more efficiently, since more porosity is filled with the same degree of hydration of the cement [21].

For engineering purposes, the concrete strength development is expressed by a function of time (*fib*, ACI, JCI) based on the compressive strength at 28 days (or the strength class). One widely-used equation, by *fib*, is introduced below [22]:

$$f_c(t) = \beta_c \cdot f_{c,28d} \quad (3)$$

where $f_c(t)$ is the mean compressive strength at age t in days, $f_{c,28d}$ is the compressive strength at 28 days, is $\beta_c = e^{s(1-\sqrt{28d/t})}$ is a time-development function, t is the corresponding age in days, and s is a coefficient that depends on the strength class and cement type (the strength development is more rapid with smaller values of s). For mixtures with the same cement type, higher strengths at 28 days, $f_{cm,28d}$, are obtained by lowering the water-to-cement ratio (w/c). In fact, at the same degree of hydration, a higher proportion of the porosity is filled when the initial porosity is lower (lower w/c) and the strength develops more rapidly with cement hydration.

Concrete containing SCM generally shows larger s values (slower strength development) than concrete with ordinary Portland cement (CEM I) and a liner correlation between the content of SCM (in particular of fly ash,

FA, and ground granulated blast-furnace slag, GGBS) with the s value has been found [23]. Evidently, the slower rate of hydration of blends containing GGBS and in particular FA overcomes the better efficiency of pore filling with C-S-H of lower Ca/Si.

The type of normal weight aggregates used has a small impact on the compressive strength at standard curing conditions within the range of ordinary concrete strength (up to ~ 60 MPa). However, when the cementitious system contains both high amounts of alumina and limestone aggregates, an increase in concrete strength might be observed due to forming of the mono-carbo-aluminate phase that contributes to filling the porosity [24,25].

For problems of cracking at early ages, knowing the rate of development of the tensile strength is essential. According to Kasai, the very early tensile strength development ($\sim$ less than 12 MPa in compressive strength) is almost linear to that of the compressive strength [26,27]. During this period, the development rate of tensile strength is relatively faster than that of the compressive strength, but at later stages the trend becomes the opposite [28]. The relationship between tensile strength $f_t(t)$ and compressive strength $f_c(t)$ can be described by equation (2), which has been widely used for predicting the material properties prediction [29,30].

$$f_t(t) = k \cdot f_c(t)^m \quad (4)$$

where $f_t(t)$ is the mean tensile strength at age t ; k , m are parameters that depend on the concrete and generally m is ranging from 0.66 to 0.87 [28] and k depends on strength level and m value.

Besides reasons of opportunity (the compressive strength is often the only measured quantity), the link between the two is established by the fact that both compressive and splitting tensile strength depend on the shear failure or tension failure of hardened cement paste.

Fracture energy is also an important factor for crack propagation and localization of cracks in concrete members. Fracture energy of early age concrete [31–33] and fracture energy change after cracking [34] have

208 been reported. And a models based on such early age properties as well as plastic behavior is applied to finite
209 element analysis [35] as well as rigid body spring-network model analysis [36] for the prediction of cracks in
210 mass concrete.

211

212 2.2 Elastic modulus

213 The elastic modulus (the Young's modulus is mainly discussed in this section,) of the concrete is one of the
214 key parameters for evaluating the risk of cracking. Approaches to predict the Young's modulus based on
215 knowing the mixture proportion of the concrete and the elastic properties of its components have been
216 proposed since the 1960s [37–39]. Later, the impact of the interfacial transition zone came under the spotlight
217 since the 1990s [40–42].

218 Since the aggregates occupy about 70% of the volume of a concrete and the Young's modulus of the
219 aggregates is generally larger than that of hardened cement paste, the Young's modulus of the aggregates has
220 the largest impact on that of the concrete. Even for aggregates of nominally the same mineralogy, the Young's
221 modulus of the aggregates may differ according to area of production, porosity/density, defects (or veins), and
222 anisotropy. On the contrary, the shape of aggregates (for example, rounded or crushed) plays a minor role for
223 the Young's modulus [43].

224 Lokhorst [44] and Bentz et al. [45] expressed the Young's modulus of hardened cement paste as a function
225 of degree of hydration. Bernard and Ulm evaluated the Young's modulus of cement paste by using a self-
226 consistent scheme / Mori-Tanaka approach based on the constituent phases and their Young's modulus [46].
227 Quantitative phase information [47–49], gel/space ratio [18], and 3-D tomographic information [50]¹ have
228 recently been applied to evaluate the Young's modulus of hydrating cement pastes.

229 Characterizing the physical properties of the ITZ is very difficult. Quantitative data can be obtained by the
230 fracture energy test [51–57] and by micro- and nanoindentation [58–61]². Based on this information, three-

¹ Authors' note: Currently, X-ray and neutron tomography have insufficient resolution to be used for this purpose, especially considering the complex multi-scale structure of cement paste.

² Authors' note: Nano-indentation requires very flat surfaces for reliable and reproducible results. Interfaces

231 phase models [43,62] and models based on a homogenization scheme [63,64] can be used.

232 Air voids, including both entrained and entrapped air, have a significant influence on the Young's modulus
233 of concrete [65]. The amount of entrained air is generally controlled and entrapped air must be minimized at
234 the placing stage.

235 For engineering and design purposes, an equation similar to equation (5) is often used for expressing the
236 development in time of the Young's modulus of concrete [22]:

237

$$238 \quad E_c(t) = \beta_c^{0.5} \cdot E_{c,28d} \quad (5)$$

239

240 Where $E_c(t)$ is the mean Young's modulus at an age t in days, $E_{c,28d}$ is the compressive strength at 28 days
241 of age, and β_c is the time-development function.

242 It should be emphasized that the Young's modulus development is generally faster than both the strength
243 and the heat liberation [28]. This is clear from the equations shown above, which are consistent with a faster
244 relative development of the elastic modulus compared to the strength. Gutsch and Rostasy reported results on
245 axial tensile tests on cylinders at early ages according to which the tensile strength is approximately linear
246 with the degree of hydration after setting, while the Young's modulus develops faster and the compressive
247 strength slower [66]. Also experimental results in [67] are in agreement with this trend. A theoretical
248 explanation is that the elastic modulus depends on the establishment of a percolated solids network through
249 the microstructure [67], while the strength is known to be governed by the size of the largest defects. Cement
250 hydration products connect quickly the solids (unhydrated cement and aggregates), while filling of the pores
251 proceeds at much lower speed [67]. This suggests that at early age (low degree of hydration until setting) there
252 is no elastic modulus development, and then the elastic modulus development is almost linear to the degree
253 of hydration, while at later ages (high degree of hydration), the Young's modulus development rate becomes

between materials of different hardness, like the ITZ, are extremely difficult to polish without inducing artefacts. These issues might result in the large scatter that is currently observed in these studies.

lower than the rate of heat liberation [67]. Therefore, in mass concrete, the Young's modulus is rapidly increasing before reaching the temperature peak; consequently, high tensile stresses will develop in mass concrete due to temperature changes and temperature gradients under outer restraint conditions.

Some codes report a more convenient equation than equation (5) for calculating the elastic modulus as a function of the compressive strength (ACI report 209, JSCE):

$$E_c(t) = a \cdot \rho^n \cdot f_c(t)^n \quad (6)$$

Where a and n are coefficients and ρ is the density of the concrete. The density of the concrete is considered in equation (6) to take into account the impact of the aggregates on the Young's modulus, assuming that stiffer aggregates are also denser.

According to the data shown in the JCI standard, the impact of using SCM on equation (6) is not significant [3].

2.3 Temperature effect on development of properties

The *maturity method* can be conveniently used for estimating the development of the physical properties of concrete that is subjected to a temperature history during hardening. To apply the maturity concept to the prediction of the development of the physical properties in massive concrete, a non-steady state heat equation with the heat production term of concrete and appropriate boundary conditions (generally, heat transfer boundaries) should be solved to obtain the temperature history at given positions in massive concrete. While it is theoretically equivalent to a description based on the degree of hydration, the maturity method is less complex since it is based on simple temperature measurements, instead than on measurements of degree of hydration of the cement. With the words of Saul [68]: "Concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time go to make up that maturity."

279 The origin of the maturity method can be found in a series of papers from England [68–70]. This work led
 280 to an index for concrete strength development that takes into account the combined effect of time and
 281 temperature, known as the *Nurse-Saul maturity function* [71]:

$$282$$

$$283 \quad M = \sum_0^t (T - T_0) \Delta t \quad (7)$$

$$284$$

285 Where M : maturity at age t , T : average concrete temperature during the time interval Δt , T_0 : datum
 286 temperature (usually taken to be -10°C), corresponding to the temperature below which no strength
 287 development is observed, t : elapsed time, and Δt : time interval. While practical, a main drawback of this
 288 function is the lack of physical meaning.

289 Later research showed that this linear approximation (product of time and temperature) is not always
 290 accurate. In particular, the best fit with the Nurse-Saul function cannot reproduce the experimental data at
 291 early age and later age. Freiesleben Hansen and Pedersen proposed a new maturity index based on the
 292 Arrhenius equation to represent the nonlinear effect of the temperature on the rate of the reaction [72].

$$293$$

$$294 \quad t_e = \sum_0^t e^{\frac{-E_a}{R} \left(\frac{1}{T} - \frac{1}{T_r} \right)} \Delta t \quad (8)$$

$$295$$

296 Where t_e : equivalent age at the reference temperature, E_a : apparent activation energy, equal to 33.5 kJ/mol
 297 above 20°C [72], R : universal gas constant, T : average absolute temperature of the concrete during interval Δt ,
 298 and T_r : absolute reference temperature.

299

300 When examining the different reactions occurring during cement hydration, the activation energy of the
 301 hydration of alite (the main phase in Portland cement) is ranging from 26 to 42 kJ/mol and that of belite is
 302 ranging from 23 to 56 kJ/mol [73]. It has also been shown that the activation energy of cement hydration

303 depends on the degree of hydration [74] because cement is composed by multiple minerals phases and also
304 the hydration of every phase involves multiple reactions. For this reason, E_a is defined as an apparent activation
305 energy that cannot be calculated theoretically but whose approximate value can be determined experimentally.
306 According to calorimetry experiments, the apparent activation energy for heat liberation ranges from about
307 30 to 48 kJ/mol [75–78]. The *fib* model code [22] gives a value of 33.3 kJ/mol for the apparent activation energy
308 for compressive strength development. Previous data of activation energy for type I cement showed a wide
309 range of values (26 – 47 kJ/mol) [79–82]. The addition of GGBS increases the apparent activation energy
310 [23,79–81], while the addition of FA reduces the activation energy [23,83].

311 The maturity method was initially used to predict the compressive strength (especially for accelerated steam
312 curing or curing at lower temperature). Later, it has been used to evaluate other properties of concrete, such
313 as Young's modulus [30,82], (splitting) tensile strength [82], creep[84–87], and autogenous shrinkage [88–95].
314 As expected, the apparent activation energy is different for each property. It has been pointed out that the
315 application of the maturity concept may be complicated or impossible for certain properties (e.g., creep and
316 self-induced stresses development, electrical properties [96]) that depend on both the degree of hydration and
317 e.g. on the time under load or on the temperature.

318

319 2.4 Cross-over effect and other factors affecting the properties in concrete members

320 The strength development of the concrete within an actual structure is not the same as that of a concrete
321 specimen with standard curing conditions (either sealed or moist/underwater curing at constant
322 temperature). At the surface of a real structure, the concrete dries out after demolding. More important for
323 mass concrete, the concrete within a structure will be exposed to temperature variations due to the heat
324 liberation of the cement reaction and ambient temperature changes. To satisfy the required performances, the
325 compressive strength of the concrete in a real structure (generally evaluated on cores drilled from the
326 structure) should be assured by adding a strength margin to the concrete under standard curing conditions.
327 One possible approach is introduced in the Japanese Standard Specification 5 (JASS5), which is based on a

328 large database about the difference in compressive strength between mockup concrete specimens exposed to
329 the ambient in a variety of seasons and locations and the compressive strength with standard curing. These
330 differences are statistically summarized as a function of cement type, average temperature during curing and
331 member size (for mass concrete or not) [97].

332 In particular, elevated curing temperatures have a significant influence on properties of concrete
333 components and concrete. At high temperature, the development of the compressive strength exhibits what
334 has been called “crossover behavior” [98]. As the rate of cement hydration is accelerated by elevated
335 temperatures, the rate of strength development at very early ages is enhanced. However, the long-term
336 strength of concrete cured at elevated temperatures is lower than that of concrete cured at room temperature
337 (a crossover of the curves is observed at later curing ages). This behavior is rather general and was observed
338 already in the 1950’s in normal strength concrete [99,100] and later also in high-performance concrete [101].

339 First microstructural explanations of the crossover behavior have been advanced starting in the 80’s.
340 According to Bentur [102], curing at elevated temperature increases the proportion of mesopores (in the range
341 of about 130 nm in diameter obtained by mercury intrusion porosimetry) while the total porosity was not
342 affected. Scanning electron microscopy studies by Kjellsen et al. [103] and Scrivener et al. [104,105] showed
343 that the density of C-S-H is increased during curing at elevated temperature. Bahafid et al. [106] confirmed
344 these findings using mercury intrusion porosimetry and XRD/Rietveld analysis. The densification of C-S-H
345 causes the increase in the proportion of larger pores in cement paste, which in turn results in a strength
346 reduction of hardened cement paste. In addition, ettringite is unstable at high temperatures [73,107]. A
347 reduction in the amount of short ettringite needles was confirmed at elevated temperatures and this can also
348 contribute to the reduction of strength [108]. This decomposition or instability of ettringite at high
349 temperatures also introduces the risk of delayed ettringite formation when the temperature decreases again
350 [109,110].

351 Since concrete is a composite material made of aggregates and cement paste matrix, a mismatch in the
352 volume changes of the components may result in cracking and loss of strength. In general, the (instantaneous)

CTE of hardening cement paste ranges from 8 to $28 \times 10^{-6}/^{\circ}\text{C}$ (before setting, the value is much higher, and after setting, the thermal expansion coefficient increases due to self-desiccation.) [111–116] and that of the aggregate is ranging from 3 to $13 \times 10^{-6}/^{\circ}\text{C}$ [117–120]. Consequently, an increase of temperature may cause detachment between the aggregate and the cement paste as well as cracking within the aggregates due to their inhomogeneity. In addition, due to the delayed response of hardened cement paste to changes of temperature [115], the shrinkage of the cement paste is restrained by the aggregates and this may results in damage in hardened cement paste. Son and Hosoda [121] monitored the cracking in concrete due to elevated temperature at early age by using acoustic emission (AE) sensors, observing that the tensile strength of the concrete decreased with the number of AE counts (which indicate cracking events in the concrete). Reducing the aggregate size as well as partial replacement with saturated lightweight aggregates could mitigate the number of cracks and the reduction of the tensile strength of the concrete [121]. In case of mature concrete, it has been experimentally confirmed that changes of compressive strength, splitting tensile strength and Young's modulus of concrete under drying and/or heating are determined by the mismatch of volume changes between aggregates and the cement paste that exhibits colloidal nature. Shrinkage and CTE of coarse aggregates have a large influence on the variation of concrete properties under drying and/or heating [122,123]. For early age concrete, it was confirmed by numerical simulations that cracks are initiated at the interface between aggregate and cement paste when the temperature increases due to the difference in CTE of concrete components [124–126]. This was experimentally confirmed by Zunino et al, who clearly showed that the lower CTE of coarse aggregate produces damage (detaching or shrinkage-restraint cracking) around aggregates and the resultant reduction of Young's modulus and compressive strength of early age concrete exposed to cyclic temperature changes ranging from 31.8 to 75.3°C [127]. Similar trends were found in tension, but there is no information on tensile strength of concrete with direct comparison to compressive strength [128].

Sustained load has also impact on the strength. In case of compressive load, generally it brings an increase of compressive strength [129–131]. While early age studies in tension are extremely complex and very rare, one paper showed that the direct tensile strength of concrete under sustained load (caused by autogenous

378 shrinkage) is decreased [132].

379 The aggregate type is another important factor for the formation of visible cracks in members under tension.
380 When the bond strength between cement paste and aggregate is high and the fracture energy is large, multiple
381 small cracks will form around aggregates under sustained tensile stress instead of a localized visible crack.
382 Such process has been experimentally and numerically confirmed in the case of drying shrinkage-induced
383 cracking in concrete [133]. An example of engineering modeling of these behaviors is reported in [134].

384

385 3 Volumetric stability

386 3.1 Autogenous deformation

387 In High-Performance Concrete at early ages, due to the low w/b (typically lower than 0.40) a substantial
388 decrease in the internal relative humidity (RH) occurs without any external drying. For very low w/b, the
389 internal RH can even drop to about 80-75% within a few days [135].

390 When cement hydrates in concrete, the capillary pores are progressively refined. At the same time, chemical
391 shrinkage empties the pores and causes a drop of internal RH, also called self-desiccation [136]. According to
392 most authors, the driving force of autogenous shrinkage is the change in pore pressure induced by self-
393 desiccation [90,137].

394 In case of ordinary strength concrete, (say above w/b of 0.40), gradual development of autogenous shrinkage
395 is observed [138,139] and this continuous development of autogenous shrinkage, even after the strength is
396 well developed, will induce tensile stresses in concrete if the target member is restrained. Therefore, the JCI
397 guideline takes into account the autogenous shrinkage for the whole strength range to evaluate the risk of
398 cracking in massive concrete [3].

399 The historical database about autogenous shrinkage regards mostly concrete mixtures with Portland cement
400 or also with silica fume. Jensen and Hansen [140] showed that silica fume, especially in high amounts (10-20%
401 by mass of cement), considerably increases self-desiccation and autogenous shrinkage. When FA substitutes a
402 part of the cement (which increases the w/c), a reduction of the autogenous shrinkage is observed, depending

on the reactivity and the fineness of the fly-ash [141,142]. However, at later ages, the shrinkage rate may become higher in FA systems compared to a Portland cement reference [142,143]. These results can be explained by the restraint offered by portlandite against C-S-H shrinkage [140], according to which a lower amount of portlandite results in larger shrinkage. This is also confirmed by the difference in the drying shrinkage of ordinary Portland cement paste and low heat (belite rich) Portland cement paste [144]. Several authors found that GGBS increases the autogenous shrinkage [101,145,146], starting at already at early ages due to the higher reactivity of GGBS compared to FA. GGBS could also produce a microstructure with finer pores, which would increase self-desiccation [101].

While autogenous shrinkage is an important factor that might contribute to early-age cracking in high-performance concrete, its development at different temperatures and in particular during semi-adiabatic temperature histories is still poorly understood [89,101,147–149].

Several models for predicting autogenous shrinkage can be found in the literature, e.g. [91,93,145]. In addition, some codes suggest formulas to calculate the autogenous shrinkage, e.g. fib, JSCE, ACI. In particular, the formula given by the CEB-FIP Model Code 2010 [150], equation 9, was recently upgraded to better cover HPC, characterized by rapid hydration and high autogenous shrinkage [151]:

$$\varepsilon_{cas}(t) = \varepsilon_{cas0}(f_{cm}) \cdot \beta_{as}(t) \quad (9)$$

Where ε_{cas0} is the final autogenous shrinkage at time infinite, $\varepsilon_{cas0}(f_{cm}) = -\alpha_{as} \left(\frac{f_{cm}/10}{6+f_{cm}/10} \right)^{2.5} \cdot 10^{-6}$; α_{as} is a coefficient dependent on the type of cement; $\beta_{as}(t) = 1 - \exp(-0.2 \cdot \sqrt{t})$ is a coefficient that describes the time development of autogenous shrinkage; f_{cm} is the mean compressive strength at age 28 days. It must be remarked that, in a recent publication [152], equation 9 was found to underestimated the autogenous shrinkage of a HPC mixture.

426 Within mass concrete, the curing conditions can be considered to be sealed³ but certainly not isothermal. A
 427 first step to study how autogenous shrinkage develops under semiadiabatic conditions is measuring and
 428 understanding the development at different isothermal conditions.

429 While the rate of autogenous shrinkage depends on the rate of cement hydration and is therefore accelerated
 430 by a temperature increase [89], there is no agreement on the effect of temperature on the magnitude of
 431 autogenous shrinkage at early ages. In cementitious materials, an increase in temperature leads to an increase
 432 in internal RH [116,153–156]. This will further accelerate cement hydration, which proceeds faster at high RH
 433 [135]. In mass concrete, when the temperature decreases again, this will lead to a reduction in internal RH
 434 compared to isothermal curing at this lower temperature, which may result in increased self-desiccation
 435 shrinkage. On the other hand, the coarsening of the mesopores happening at higher curing temperatures (see
 436 section 2.4) might end up reducing self-desiccation and autogenous shrinkage. Based on the available results,
 437 it is at this stage impossible to assess which mechanism might be dominant.

438 Most available prediction models for autogenous shrinkage do not consider the effect of curing temperature.
 439 Only a few studies, modelled the temperature dependency of autogenous shrinkage following the maturity
 440 concept with an Arrhenius equation (equation 8).

441 The JCI Guidelines [3] already incorporate the effect of temperature into the prediction of autogenous
 442 deformation, by considering the autogenous shrinkage as dependent on equivalent time instead of real time
 443 for curing at temperatures between 20 and 70 °C:

$$\varepsilon_{sh}(t_e) = \eta_c \cdot \varepsilon_{sh,\infty} \cdot \beta_{sh}(t_e) \quad (10)$$

³ Authors' note: in theory, temperature changes within the cross-section would induce moisture transport within the concrete [200], but there is no quantitative data on this issue. Therefore, currently, the impact is considered to be minor until further evidence is available.

Where $\eta_c = 1$ for Portland cement and GGBS cement, $\varepsilon_{sh,\infty}$ is the final autogenous shrinkage and $\beta_{sh}(t_e) = 1 - \exp\{-a_{sh}(t_e - t_{e,set})^{b_{sh}}\}$ describes the shape of the curve.

It is interesting to observe that both the final shrinkage and the shape of the shrinkage curve depend on the equivalent time t_e but also on the temperature history, in particular on the maximum temperature T_{max} . The coefficients a_{sh} and b_{sh} depend on the w/c only for Portland cement, while they also depend on T_{max} for GGBS cement.

In addition, the expression for the final autogenous shrinkage is:

$$\varepsilon_{sh,\infty} = \begin{cases} 3070 \exp\left\{-7.2\left(\frac{w}{c}\right)\right\} + 50 \cdot [1 - \exp\{-1.2 \cdot 10^{-6} \cdot (T_{max} - 20)^4\}] \\ 2350 \exp\left\{-5.8\left(\frac{w}{c}\right)\right\} + 80 \cdot [1 - \exp\{-1.2 \cdot 10^{-6} \cdot (T_{max} - 20)^4\}] \end{cases} \quad (11)$$

Where the above expression is valid for Portland cement and the one below for GGBS cement.

It should be noted that in the JCI guidelines the temperature-dependent final autogenous shrinkage takes into account of the impacts of time-dependent CTE, non-linear acceleration of reaction, and the difference of phase composition due to temperature change. While the JCI guidelines incorporate in a straightforward manner the effects of temperature both on the magnitude and on the shape of the autogenous shrinkage, these issues are still very much an object of debate in the literature.

According to some authors, a simple maturity transformation was rather successful [157,158] while others state that this approach is inaccurate or theoretically doubtful. In particular, in [147] it was concluded that it is impossible to predict autogenous deformation under realistic (semiadiabatic) temperature histories from isothermal tests; in particular, this method overestimates the autogenous shrinkage under semiadiabatic conditions. A possible issue with this approach is that a temperature change might induce delayed thermal deformations in addition to the instantaneous thermal expansion/contraction [115,159]. Jensen and Hansen [89] stated that a simple maturity transformation is inadequate since the internal RH depends both on the

470 degree of hydration and on the instant temperature. On the other hand, the results by Lura et al. [101] can be
471 rather accurately captured by a maturity transformation, except for the initial expansion at very early ages.
472 Summing up all the evidence presented in this section, when no experimental results are available, the
473 autogenous shrinkage at ambient conditions can be calculated using an equation provided in a design code
474 (like equation 9) and then a maturity transformation (in the form of equation 8) should be applied to model
475 the autogenous shrinkage at different temperatures. Alternatively, in equation 10 autogenous shrinkage is
476 already expressed as a function of equivalent time. While predictions of autogenous shrinkage during a
477 realistic (semiadiabatic) temperature history might not be completely accurate, these are the best approaches
478 available at the moment.

479 It has been argued that the bulk modulus that resists the driving force of shrinkage (such as capillary tension)
480 is smaller than the bulk modulus measured by ultrasonic pulse velocity and quasi-static loading [137,140,160].
481 Drying and autogenous shrinkage have been indeed modelled considering creep/viscoelastic behavior of the
482 solid skeleton [161,162]. Recent work [163], based on the analysis of a large database of autogenous shrinkage
483 and relative humidity results, is providing further evidence that long-term autogenous shrinkage may be the
484 visco-elastic deformation due to sustained pore pressure induced by self-desiccation. While this phenomenon
485 may be underestimated in current codes, this should not be an issue for the evaluation of the risk of cracking
486 in mass-concrete, which is a relatively short-term problem.

487

488 3.2 Thermal expansion and contraction

489 When the temperature of mass concrete increases and then decreases again, thermal strain (expansion upon
490 heating and contraction upon cooling) occurs. Calculations of the thermal strain requires precise knowledge
491 of the CTE, which is defined as the change in strain per unit change of temperature. However, quantitative
492 measurements of CTE need precise measuring equipment and sample conditioning. In addition, temperature
493 cycles require some time, during which cement hydration proceeds. For these reasons, CTE measurements are
494 especially challenging at early ages, in particular due to the ongoing hydration of cement. Previous studies

495 focused mainly on CTE of cement pastes or mortars, while comparatively few measurements have been
 496 performed on early-age concrete [164,165]. The CTE of concrete is significantly lower than the CTE of cement
 497 paste and mortar, due to the lower CTE of the stiff aggregates, which consequently restrain the larger thermal
 498 deformation of the cement paste.

499 The magnitude of CTE of concrete recommended by the ASTM International [166] is between 5.6 and 13×10^{-6}
 500 $^{\circ}\text{C}$, whereas the recommended range of CTE in the CEB-FIP 2010 [167] is between 6 and 15×10^{-6} $^{\circ}\text{C}$. It is
 501 evident that neither standard can be used in practical applications to suggest the actual CTE to be used in
 502 calculations of the self-induced stresses, since the indicated ranges cover a factor of two to almost three. In
 503 practice, the CTE of concrete depends significantly on concrete composition (particularly aggregate type and
 504 volume fraction). Wyrzykowski et al. [120] present a series of CTE values for concrete compiled from the
 505 literature, which mostly fall in the range 7 - 11×10^{-6} $^{\circ}\text{C}$. According to Mindess et al. [168], the CTE of aggregates
 506 falls in the range 6 - 13×10^{-6} $^{\circ}\text{C}$, with limestone and basalt in the lower end and sandstone and quartzite in the
 507 higher end. Nevertheless, it must be pointed out that the CTE of aggregates with the same mineralogy might
 508 vary substantially (even by 30-50%). As mentioned in a previous section, the CTE of cement paste might range
 509 from 8 to 28×10^{-6} $^{\circ}\text{C}$ [111–116], depending on w/c, age, and moisture state. Effective medium approaches
 510 (e.g., the Rosen-Hashin bounds [169]) are quite precise if one wants to calculate the CTE of a concrete based
 511 on the CTE of its paste and aggregates [170]. However, in practical applications the CTE of the constituents are
 512 rarely known with sufficient precision. As the CTE of concrete is mostly determined by the aggregate, the effect
 513 of SCM is expected to be small. Nevertheless, the JCI guideline [3] recommended to use 20% larger CTE for
 514 GGBS cement concrete than for Portland cement concrete, based on 8 references and 19 data.

515 Most of CTE measurements show a strong dependence on the moisture state [116,153,154]. Initially, the CTE
 516 is very high (for cement pastes, close to the CTE of water) and then it drops to a minimum at the time of set
 517 [91,114]. While according to Cusson and Hoogeveen [158], the CTE is almost constant from one day, a number
 518 of studies [91,111,114,149] find an overall increase of CTE at early ages. The CTE at the time of set is supposed
 519 to be governed by the solid skeleton and this value is not expected to change considerably due to hydration. In

fact, Sellevold and Bjøntegaard [153] showed that hydration has a limited net impact on the development of the CTE, because when concrete was rewetted after a couple of weeks of hardening, the CTE decreased to the values measured around the time of set. According to [116,153,154], a mechanism linking the change in internal relative humidity (ΔRH) to the temperature change (ΔT) can explain the CTE increase after setting (the same mechanism was already mentioned in the section about autogenous shrinkage). The increase of CTE at early ages should be more important at low w/b, where self-desiccation is prevalent. However, a substantial increase was also measured for cement pastes with w/c up to 0.45 [170]. Even if the CTE increase due to self-desiccation is less pronounced in concrete than in pastes or mortars (due to the restraint offered by the aggregates), increases between 35 and 55% have been reported for concrete by different authors [153,171]. In practice, the CTE of concrete has often been regarded as constant in numerical calculations. An increase of CTE of the proportions mentioned before will however have a substantial impact on the risk of thermal cracking [172].

According to [165,170], the CTE depends on the water-to binder ratio, while in [111] the CTE was not sensitive to this parameter.

All this section has been assuming that a concrete element deforms instantaneously responding to a temperature change. In reality, some delayed deformations may occur also after temperature equilibrium has been reached. According to a number of authors [115,153,172,173], delayed thermal deformations in early-age concrete may be caused by changes of the moisture state or by moisture transport. Delayed deformations are at this point still a subject of active research and no suggestion can be made to incorporate them into a calculation of the risk of thermal cracking.

540

541 3.3 Stress relaxation and creep

542 The extent to which elastic stresses (in a simplified approach, the product of the restrained deformations and
543 the elastic modulus of the concrete) are reduced by stress relaxation in early-age concrete is fundamental for
544 predicting stress development and cracking in mass concrete [174]. *Stress relaxation* is defined as the time-

545 dependent change in stress when the strain remains constant. The other main manifestation of time-
546 dependent behavior under stress is *creep*, defined as the time-dependent change in strain under constant
547 stress. The experimental equipment needed to determine the relaxation properties of young concrete, and also
548 the testing procedures, are extremely complex (see e.g. [175]). Creep tests are comparatively easier; moreover,
549 creep is extensively dealt with in codes, because of its importance in the calculation of deformation of concrete
550 structures. For these reasons, it is convenient to calculate the relaxation curves, needed to determine the actual
551 stresses in mass concrete, from the experimental creep results [176–178]. In the case of applications to mass
552 concrete, as mentioned previously, drying is considered to be of minor impact to the concrete structure at early
553 ages. For this reason, reference is made here only to *basic creep*, measured in sealed conditions.

554 Creep and relaxation of concrete are consequences of viscoelastic/viscoplastic behavior of the cement paste
555 restrained by elastic aggregates. C-S-H is the main phase in cement paste that manifests viscoelastic behavior
556 [179–182]. In general, creep increases with the w/c or with the porosity of the matrix in the cementitious
557 material and it decreases with the age of loading. At early ages, creep and relaxation are very significant and
558 they progressively decrease with degree of hydration or maturity. The high creep at early ages and its decrease
559 with time (aging) can be described by a number of mechanisms that are still debated in the literature (for a
560 recent discussion, see [183]). In particular, it is interesting to observe that the process of hydration, in
561 particular the dissolution of clinker, can explain at least part of the high creep measured at early ages [184].

562 The effect of SCM on early age creep and relaxation has not been studied systematically; most studies
563 concentrate on silica fume, FA and GGBS. Silica fume was found to increase early-age creep of concrete [185].
564 In [186,187], the same was found for FA, especially at high substitution levels. However, according to [185],
565 addition of FA and GGBS reduced the basic creep compliance. Also the effect of GGBS appears to be
566 controversial: in [186], GGBS did not influence the development of the creep compliance in compression, while
567 in [188] higher creep was found. To a certain extent, the contradicting trends regarding the influence of SCM
568 on creep may be due to the different reactivity of the SCM used in different studies [189].

During the hydration process three different rate-determining processes [190] have been identified in creep experiments (without fully clarifying the mechanisms): 1) long-term power law creep, 2) short-term logarithmic creep and 3) creep due to the solidification process, whose kinetics are constant and whose amplitude depends on the time derivative of the elastic modulus at the age of loading. Different codes have models to calculate concrete creep. For example, whereas ACI [191] does not distinguish between drying and basic creep, the fib Model Code 2010 [22] gives this expression for basic creep:

$$\varepsilon_{cc}(t, t_0) = \frac{\sigma_c(t_0)}{E_{ci}} \cdot \varphi(t, t_0) \quad (12)$$

with $E_{ci} = E_{c0} \cdot \alpha_E \cdot \left(\frac{f_{cm}}{10}\right)^{1/3}$, $\varphi(t, t_0) = \varphi_{bc}(t, t_0)$, $\varepsilon_{cc}(t, t_0)$ creep strain at time $t > t_0$; $\sigma_c(t_0)$ applied constant stress at time t_0 ; E_{ci} dynamic modulus of elasticity at age 28 days; $\varphi(t, t_0)$ creep coefficient; E_{c0} assumed equal to $21.5 \cdot 10^3$; α_E constant, assumed to be 1 for quartzite aggregates; f_{cm} mean compressive strength at age 28 days; $\varphi_{bc}(t, t_0)$ basic creep coefficient, equal to $\varphi_{bc}(t, t_0) = \beta_{bc}(f_{cm}) \cdot \beta_{bc}(t, t_0)$ with

$$\beta_{bc}(f_{cm}) = \frac{1.8}{(f_{cm})^{0.7}}$$

$$\beta_{bc}(t, t_0) = \ln \left[\left(\frac{30}{t_{0,adj}} + 0.035 \right)^2 \cdot (t - t_0) + 1 \right] \quad , \quad t_{0,adj} = t_{0,T} \cdot \left[\frac{9}{2 + t_{0,T}^{1.2}} + 1 \right]^\alpha \geq 0.5 \text{ days} \quad , \quad t_{0,T} =$$

$$\sum_{i=1}^n \Delta t_i \exp \left[13.65 - \frac{4000}{273 + T(\Delta t_i)} \right]$$

Where $t_{0,adj}$ modified age of loading, taking into account the type of cement and the temperature; α coefficient that depends on the type of cement; $t_{0,T}$ temperature-adjusted concrete age; Δt_i number of days where a temperature T prevails; $T(\Delta t_i)$ temperature in °C during the time period Δt_i .

In principle, equation 12 (or any similar equation from codes) could be used to calculate the creep strains that can be directly input into equation 1 to calculate the self-induced stresses in mass concrete. However, there are two main issues with this approach:

- 1) Equation 12 and similar ones are derived from fitting a large number of measurements of creep in compression, whereas creep in tension must be known for calculating restraint stresses that lead to cracking. There are indications that creep in tension and compression might differ, especially at high

levels of load when damage occurs [192,193]. At early ages, basic creep in compression may be higher than in tension [194]. However, as summarized in [195], the ratios between compressive and tensile creep found in the literature vary widely, from up to 4 to about 0.2.

- 2) The creep equations reported in the codes are mainly concerned with long-term creep, which is used to calculate the deformation of concrete structure under sustained loads. For calculation of self-induced stresses in mass concrete, the short-term creep is determinant [196,197]. In particular, equations that fit the long-term creep behaviour of concrete might not be accurate in predicting the whole creep development, in particular at early ages.

An alternative to equation 12 suggested in the JCI Guidelines [3] uses an effective elastic modulus to take into account the effect of creep. In particular, the Young's modulus is multiplied by a reduction factor:

$$E_e(t_e) = \varphi(t_e) \cdot E_c(t_e) \quad (13)$$

where $E_e(t_e)$ is the effective modulus of elasticity at equivalent time t_e ; $E_c(t_e)$ is the actual modulus of elasticity at t_e ; $\varphi(t_e)$ is a reduction constant that takes into account creep/relaxation especially at early ages. The JCI Guidelines suggest to use $\varphi(t_e) = 0.42$ until the maximum temperature is reached, while this value increases to 0.65 at 1 day equivalent time after the temperature peak (with linear interpolation in between). The approach suggested by equation 13 would lead to restraint stresses that are about half of the elastic stresses. While this approach might be a bit conservative, it captures the order of magnitude of relaxation that might occur in concrete at early ages.

4 Conclusions

The properties of concrete related to crack formation in mass concrete have been discussed in this paper. The main conclusions are listed below:

1. To obtain a comprehensive collection of data directly useful for engineers, the concrete properties need to be related to the degree of hydration of the cement. Convenient experimental methods for determining the hydration degree of the concrete are desired. The relationships between cement

hydration as well as phase composition and physical properties of hardened cement paste and concrete should be the object of intense study in the future. Multi-scale investigation is promising way for resolving the gaps. Aggregate properties are also very important and should be studied deeper to obtain a better understanding of concrete behaviour.

2. Based on this review, the available data on concrete containing SCM is not sufficient for a full evaluation of their impact in mass concrete applications. This despite the fact that SCM have a high potential in mass concrete because of lower heat liberation compared to Portland cement. More attention should be paid to physical properties in tension for the evaluation of the risk of cracking. In particular, Young's modulus, creep/relaxation, and crack propagation data in tension are required.
3. For the prediction and evaluation of the risk of cracking, modeling of concrete properties and numerical methods are necessary. From a viewpoint of numerical calculation, the accumulation of benchmark experiments on massive concrete with history and distribution of temperature, relative humidity, evaporable water, strain, and cracks, coupling with basic concrete properties development and volume changes under different temperature conditions are desired. In addition, the data of real structures and precise investigation of real-size concrete members are necessary to fill the gaps between laboratory data and construction site. The impact of bleeding, placing joints (including impact of temperature history of overall member), reinforcement-concrete bond behavior [198,199] including plastic deformation as well as changes in stress field are necessary to evaluate the risk of cracking as well as the crack width.

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Declaration on conflict of interest

642 The authors declare that they have no conflict of interest.

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