

Early age autogenous shrinkage cracking risk of an ultra-high performance concrete (UHPC) wall: Modelling and experimental results

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Early Age Autogenous Shrinkage Cracking Risk of an Ultra-High

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Performance Concrete (UHPC) Wall: Modelling and Experimental Results

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- 4

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16 Abstract

17 Ultra-High Performance Concrete (UHPC) exhibits high autogenous shrinkage (AS) which significantly increases the risk of early age cracking. To predict the risks of early age shrinkage 18

19 cracking of environmentally friendly CEM III-based UHPC, a numerical model originally

20 developed for early age crack assessment of ordinary concrete, has been further developed and

21 applied on a demonstration wall with high risk of cracking, cast on a non-deforming slab. The

22 design of the wall was determined through numerical simulation using different parameters,

23 resulting from specific experiments performed on the desired concrete mixture. Early age crack

24 assessment parameters for the model were obtained through different tests performed using the

Temperature-Stress Testing Machine (TSTM). Finally, this UHPC wall was built, and occurring 25

26 strain deformations were recorded in real time using fiber optic (SOFO) sensors embedded in the 27 wall, and measurements taken from demountable mechanical strain gauges (DEMEC).

28 Restrained shrinkage measurements were obtained for the same mixture through ring tests. A

29 comparison between the numerical simulation results and the measurements proved that the

30 proposed model is suitable for UHPC, and the model predicts well the time of crack appearance.

Finally, it has been shown that shrinkage values along the wall height are influenced by the 31

- degree of restraint. 32
- 33

34 Keywords: Ultra high performance concrete; modelling; early age crack assessment; autogenous

35 shrinkage; large-scale test

1. Introduction

Predicting the early age cracking risk of cementitious materials is a complicated and difficult 37 task, since there are many properties involved, which develop very fast and highly influenced by 38 39 temperature (i.e. from heat of hydration) and curing, especially for Ultra High Performance 40 Concrete (UHPC) and High Performance Concrete (HPC). These concrete mixtures are known 41 for their great mechanical and potentially high durability properties that are obtained due to their 42 very low water to cement ratio, generally between 0.2 and 0.3 [1][2]. The high risk of early age 43 cracking is the challenging issue in these durable concretes. Due to the low w/c ratio, UHPCs 44 and HPCs undergo a high Autogenous Shrinkage (AS) driven by a self-desiccation that develops 45 fast during the very early age (within the first days of age). AS, also known as autogenous deformation or basic shrinkage (Eurocode 2), is defined as the external macroscopical volume 46 47 reduction that occurs under isothermal/sealed conditions, in other words it is the shrinkage that 48 occurs when there is no temperature variation, volume change due to loss or absorbance of 49 substances, or any application of external force [3]. During hardening, the internal relative 50 humidity (RH) will drop and self-desiccation will occur if no external (or internal) water source 51 is present. In mixtures with low water-to-binder ratios, water is rapidly consumed and a dense 52 matrix is formed containing small pores. Hydrostatic tension forces (capillary forces) increase 53 with the increase of smaller pores and lead to the increase in self-desiccation and autogenous 54 shrinkage.. When it comes to UHPCs, RH drops rapidly in a short time to typically reach around 73% at 8 days and 68% after 3 months, while it was found to be 91% at 3 months for ordinary 55 56 concrete [4]; these values will of course vary from one concrete to another. The previous 57 statement implies that water/vapor filled pores are gradually emptied, and capillary tensions rise 58 on the pore surface which is then perceived as autogenous deformations. As concrete is weak in 59 tension at the early age, it is vulnerable for crack development resulting from high early AS 60 [5][6]. These cracks impair the durability and the functionality of the UHPC structures by letting 61 water with chemical agents and gases infiltrate inducing carbonation, steel corrosion, chemical 62 attack and/or freeze-thaw damage [7]. Thus, in order to limit the cracking risk, it is very 63 important to correctly estimate the evolution of the autogenous shrinkage [8].

64 Autogenous shrinkage can be measured on cement pastes, mortar and concrete using direct 65 and/or indirect methods. The direct methods measure the volume or the linear autogenous deformations whereas indirect methods measure in general the relative humidity or the porosityof the cementitious material and correlations will be needed between AS and porosity or RH [9].

The buoyancy method, as developed by Loukili et al. [10], and the capillary tube method [11] are used for volumetric autogenous measurements. Linear autogenous deformations rely on contact and non-contact methods. The corrugated tubes method introduced by Jensen and Hansen [12] is a contact method that combines volumetric and linear deformations with the ability to start the measurement right after casting. Eddy current sensors and laser displacement sensors are considered as non-contact methods for measuring autogenous shrinkage.

74 Different models were used to evaluate and predict autogenous shrinkage in HPCs. Tazawa and 75 Miyazawa [13] suggested a model that includes the product of the ultimate autogenous shrinkage 76 value, ε_{∞} (w/cm), and coefficients β (t) and γ , which describe respectively the development rate 77 of AS with time and the effect of cement type. Lee et al. [14] modified the previous model to account for the AS strain that is contributing to the stress development, and applied it on 78 79 concrete containing mineral additives. Based on the pore structure of the concrete and the 80 capillary tension, Li et al. [15] created a prediction method of early AS of self-consolidating 81 concrete. Model B4 developed by Wan-Wendner et al. [16], is an improved model of the B3 one 82 [17] where prediction was expanded to modern concretes with numerous types of admixtures, 83 and mineral additions as well as the effects of various types of aggregates. Some of these 84 additions increase the shrinkage and creep and others decrease it. The important novelty of the 85 B4 model is the separation of autogenous shrinkage and drying shrinkage.

86 To predict the early age cracking behavior in massive concrete structures is still a difficult task to 87 perform, because of several complex coupling phenomena such as between the autogenous and 88 thermal strains. In a benchmark study done by Benboudjema et al. [18], different numerical 89 approaches were conducted on multi-scale models (micro- and macroscopic) to predict the early 90 age cracking behavior of a massive concrete structure in external restrained conditions. The 91 study concluded that at the macroscale, a thermo-activation method had to be used in the model 92 in order to have a satisfactory prediction of the adiabatic temperature; while a good prediction of 93 stresses can be achieved if the fast evolution of Young's modulus right after setting time can be 94 estimated. Other physically based numerical model approaches to predict shrinkage of structures 95 have been developed such as fib Model Code 2020, a developed version of fib MC2010 [19], 96 dealing with both new and existing concrete structures. Another key parameter when it comes to

97 restrained stress development and corresponding early-age cracking is the time dependent stress 98 response, hereafter denoted creep. Creep can significantly reduce AS- and temperature-induced stresses at early ages: the literature shows a reduction of restrained stresses of as much as 30 -99 100 50% due to the beneficial creep behaviour of concrete [18–21]. However, it should be noticed 101 that temperature-induced compressive creep at very early ages also can contribute to an increase 102 in the succeeding tensile stress development [22-23]. Because of the above described intricate 103 relations between early-age phenomena, an extensive knowledge is needed for the careful 104 calibration and development of models. This is particularly true regarding UHPC whose final 105 properties might considerably depend on their formulation and early age characteristics. 106 Therefore, coupled numerical and experience-based design methods would be particularly 107 beneficial in the scope of tailor-made cementitious materials and structures.

108 Various approaches at early-age cracking calculations are found in the literature [21], [26]–[33]. 109 Some design approaches are merely an estimation of whether the concrete will crack or not, while other and more complex approaches, chemomechanical [34] thermo-chemo-mechanical 110 111 [35]or hygro-thermo-chemo-mechanical approaches [36], also provide a prediction of the crack 112 development and the size and rate of occurring crack widths [37]. For all such early-age crack 113 assessments, the accuracy of the outcome is very dependent on the quality and correctness of the 114 models and material parameters used as input [23], [38]–[42]. In the current study, a pragmatic 115 "to crack, or not to crack"-approach was applied [43] by using the special-purpose program 116 CrackTeSt COIN [44]. This approach is considered accurate and advanced, but still practical and 117 easy applicable for contractors and structural designers [45]. In the current study, an early-age 118 crack assessment on a typical restrained wall was performed in CrackTeSt COIN to assess and 119 predict the cracking risk. The model was also used to further determine the conditions necessary 120 to achieve cracking at a specific point of interest in time, around 20 hours after water addition. 121 The combination of a thin wall cast on a stiff base was selected, since this would best provoke 122 cracks by early age shrinkage. The wall was kept thin to minimize the influence of temperature 123 related deformations, and the use of fibres was omitted. The same wall design could then 124 afterwards be used to test the effectiveness of internal curing agents such as superabsorbent 125 polymers to avoid early age cracks. Parameters for the model were obtained through different 126 tests including a test in a Temperature-Stress Testing Machine (TSTM). Then a wall has been 127 built in order to verify the simulated cracking risk through visual observation and deformation measurements. The real-time occurring strains were recorded automatically with the use of embedded fiber optic sensors and manually with mechanical demountable strain gauges through measuring points glued on the wall. In addition, ring tests were carried out to measure restrained shrinkage, and concrete mechanical properties such as compressive strength and E-modulus were determined.

133 **2. Materials and methods**

134 **2.1.** Concrete characterization

135 **2.1.1. Introduction**

The current sub-section describes the first part of the study: determination of parameters necessary to conduct early age crack assessment of the given concrete. The currently applied methodology for parameter characterization has been verified and thoroughly described in [43]. The results presented in the current section were used to create a material database file representing the given concrete for implementation in CrackTeSt COIN (CTC) [44], where CTC is a two-dimensional special-purpose program for early age crack assessment.

142 **2.1.2.** Concrete mix design

143 The concrete mix design was determined after various optimisation steps, starting from a 144 previously reported mix design [46]. This initial mix design was adapted to the Norwegian 145 situation and made with local Norwegian ingredients (such as aggregates of size 0-8 mm, which 146 are typical in Norway). A blast furnace slag cement CEM III/A 52.5 R (Dyckerhoff Variodur 40) 147 was used in the mix with a dry undensified silica fume (19.8% by weight of cement), and a limestone filler with a quantity of 23.8% by weight of cement. The superplasticizer ViscoCrete 148 149 UHPC-2 with 40% of active compound was used with a dosage of 1.4% by weight of cement; 150 the water to binder ratio (w/b) equaled 0.2. In the majority of UHPC applications fiber 151 reinforcement is known to be present. In this study, fibers were not used since it was not our goal 152 to increase the flexural or tensile strength. Besides, the mixture design of this study was part of a 153 project [47] where the main focus was rather on the improvement of the matrix itself by internal 154 curing agents known as SAPs (superabsorbent polymers) to achieve crack resistance properties.

155 Three batches were mixed out of the composition shown in Table 1, batches A, B and C. Batch A

156 was used to determine the hydration heat evolvement of the cement, batch B was used to assess

157 the compressive and tensile strength development, and batch C was used for tests in the Free-

158 Deformation (FD) and the Temperature-Stress Testing Machine (TSTM) systems.

159 Table 1: Concrete mix design

Materials	(kg/m³)
Sand 0/4	401.9
Basalt 4-8 mm	649.3
Silica Fume - Elkem Microsilica 940 U	153.8
Cement - Variodur 40 (CEM III/A 52.5 R)	778.2
Filler - Betofill VK50 (limestone)	185.5
Water	186.4
Superplasticizer-SIKA Viscocrete UHPC-2	11.2

160

161 **2.1.3.** Testing of Concrete

162 2.1.3.1. Hydration heat

Semi-adiabatic calorimeter tests were performed in a NTNU-box (15 L samples) in order to determine the hydration heat evolvement of the concrete. The NTNU-box comprises a plywood box insulated by 100 mm Styrofoam (with known thermal properties) on all sides. During testing, the box was stored in air at 38 °C while the air and concrete temperature were measured continuously for 4 days.

168 2.1.3.2. Strength and E-modulus

169 Compressive strength tests were performed on triplicate 100x100x100 mm³ cubes according to NS-EN 12390-3:2009 [48]. Uniaxial tensile strength tests were performed on triplicate 170 171 100x100x600 mm³ prisms (without notch) in an INSTRON 5985 electromechanical testing 172 system [49], the test set-up is described by Klausen et al. [43]. The main focus in the current 173 study has been early age cracking, and in this perspective, the elasticity modulus in tension is the 174 main parameter. The tensile E-modulus was found from the load-deformation relation measured 175 in the uniaxial tensile strength tests, where the E-modulus was deduced as the stress/strain ratio 176 between 10% and 40% of the failure load.

177 **2.1.4.** Model equations and parameters

The maturity principle was applied to describe the effect of curing temperature on the heat and property development of the given concrete. The Arrhenius equation was used as temperature function, and the rate of hydration $H(T_i)$ could consequently be expressed as given in Eq. 1.

$$H(T_i) = exp\left\{\frac{E_T(T_i)}{R} \cdot \left(\frac{1}{293} - \frac{1}{273 + T_i}\right)\right\}$$
 Eq. 1

Where: R (J/(K.mol)) is the gas constant, T_i (°C) is the temperature and E_T (J/mol) is the activation energy: $E_T = A + B(20-T_i)$, A (J/mol) and B (J/(K.mol)) are model parameters, where B = 0 for T > 20 °C and B has a given value for T < 20 °C

185

The increase in maturity within a time increment is then $H(T_i) \cdot \Delta t_i$, while the maturity time at a certain concrete age is the sum of all maturity growth increments. For the given concrete, the activation energy parameters A and B were assumed based on experiments with previous concretes having similar properties [50], see Table 2.

The development of compressive strength, tensile strength and E-modulus in tension were modelled by a modified version of the CEB-FIP MC 1990 model, Eq. 2 [51]–[53]. For each property, the model was fitted to the test results by using the method of least squares. The obtained model parameters are presented in Table 2, and illustrated in Figure 3.

194

$$X(t_e) = X(28) \cdot \left\{ exp\left[s \cdot \left(1 - \sqrt{\frac{672 - t_0}{t_e - t_0}} \right) \right] \right\}^n$$
 Eq. 2

Where $X(t_e)$ is the property as a function of maturity t_e (MPa if X is equal to f_c or f_t and GPa if it is equal to E), X(28) is the property at 28 days, s and n are unitless curve-fitting parameters (the s-parameter is the same for all properties, while the n-parameter is varying), and t_0 is the starting time for stress development [maturity hours] found from the TSTM-test.

- 199
- 200

201 Table 2: Model parameters obtained by using the method of least squares for all tested ages

A	В	t_0	S	n_c	n_t	n_E	f_{c28}	f_{t28}	E_{28}
$[J \cdot mol^{-1}]$	[J·mol ⁻¹ ·K ⁻¹]	[h]					[MPa]	[MPa]	[GPa]

45000	400	8.0	0.141	1.000	0.744	0.323	151.2	6.7	42.8
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- 202
- 203

2.1.5. Testing in the FD and TSTM systems

The currently investigated concrete was tested in the Free-Deformation (FD) system and the Temperature-Stress Testing Machine (TSTM) system. The test set-ups and procedures for these systems are thoroughly described and illustrated in [43], [54], [55].

In the FD system, the autogenous shrinkage of two horizontally oriented and sealed specimens with dimensions of $100 \times 100 \times 500$ mm³ was measured. The FD system is temperature-controlled, and the specimens were exposed to 20 °C isothermal conditions during testing. The length measurements were initiated approx. 2 hours after mixing. Autogenous shrinkage was measured up until 289 hours (12 days), and all measurements were zeroed at the starting time for stress development, t₀ = 8 hours (found from the TSTM test).

The TSTM system consists of a dilation rig and a Temperature-Stress Testing Machine (TSTM). 213 The dilation rig measures the free deformation of a horizontally oriented specimen with 214 215 dimensions of 100x100x500 mm³, while the TSTM is constructed to measure the restrained 216 stress development in the hardening phase for a given degree of restraint. For both rigs, the 217 concrete specimens were carefully sealed with plastic sheets and aluminum foil to prevent any 218 water loss during testing. The TSTM system is temperature-controlled, and the specimens were 219 subjected to a realistic temperature history representing a 50 mm thick concrete wall cast under 220 Norwegian summer conditions, i.e. ambient temperatures of 20 °C.

221

Large-scale demonstration wall

222

2.2.

2.2.1. Concrete mix design and mixing procedure

223 Based on the modelling results in section 3.1, the demonstration wall was cast as seen in Figure 1 224 with the following dimensions: the slab was $2 \ge 0.95 \ge 0.2 = 0.2$ 225 $2 \ge 0.05$ m³ (h x l x t). To limit the influence of heat produced by the exothermic hydration 226 reaction of cement, the thickness of the wall was kept limited. This was done to minimize the 227 influence of thermal contraction caused by cooling beyond the hydration heat peak, and of 228 thermal gradients which could lead to thermal cracking. The slab was reinforced with two 229 reinforcement meshes (150 mm) of diameter 8 mm positioned with a minimum concrete cover of 230 25 mm, and the wall with a steel mesh of mesh size 150 mm and diameter 5 mm positioned in the middle of the wall. The slab-wall connection was created using a 1200 mm long steel mesh with a height of 60 mm (mesh size 10 mm and diameter 2 mm) introduced in the slab for 50 mm along the length of the wall and two reinforcement bars (diameter of 12 mm and total length of 300 mm), each located at 75 mm from one of the ends. The mesh and the reinforcement bars were placed in the slab by sawing a groove and drilling two holes and subsequently gluing them with epoxy.



238 Figure 1: Schematic representation of the demonstration wall and its dimensions

The mix design of the traditional concrete of the slab and mixing procedures of the concrete ofboth the slab and the wall can be found in

Table 3 and Table 4. The mix design of the concrete wall is the same as shown in Table 1. It 241 242 should be noted that the amount of superplasticizer used (1.1 m% by weight of cement) was less 243 than that used in the batches for the modelling (1.4 m% by weight of cement) to obtain the same 244 slump flow value (700 mm). The slab was cast 3 months prior to the wall and stored at ambient 245 temperature ranging between 7 °C to 22 °C, to ensure that the influence of remaining drying 246 shrinkage of the slab could be neglected in the time period considered. The concrete of the slab 247 and the wall were each mixed using a rotating pan mixer Zyklos from the company Pemat-248 Germany with a capacity of 200 L, a speed of 46 rpm for the rotation of the mixing blades and 249 189 rpm for the pan.

250 Table 3: Mix design composition of the slab

237

Slab

Materials	kg/m³	
Sand 0/4	670	_
Gravel 2/8	490	
Gravel 8/16	790	
Cement-CEM I 52.5N	300	
Water	150	
Superplasticizer BASF MasterGlenium 51	1.67	

251

Table 4: Mixing procedures for the concrete of both slab and wall

Slab - 5 mins	Wall – 10 mins		
Dry mixing for 1 min	Dry mixing for 1 min		
Add water while mixing	Add water while mixing		
Mixing for 1 min	Mixing for 1 min		
Add superplasticizer	Add superplasticizer while mixing and mix for 4 mins		
Mixing for 3 mins	Stop at 6 mins and scrape borders for 1 min		
	Mixing for 3 mins		

²⁵³

254 The concrete mixture of the wall was poured into the mould from the top with no vibration since 255 it was a self-compacting mixture. Three batches were needed to fill the formwork, with the third 256 batch being mixed 40 minutes after the first one. The pouring of the wall proceeded for 55 257 minutes and the wall was kept in a climate-controlled room for the whole testing period with a temperature of $20 \pm 2^{\circ}$ C and a relative humidity (RH) of $60 \pm 5\%$. At 19 hours after the contact 258 259 between water and cement, the formwork was removed. No curing was applied after formwork 260 removal. All of the following experiments were additionally performed using these batches 261 except for the ring test which was performed using a separate batch of concrete.

262 **2.2.2.** Testing of fresh concrete properties for the wall

263 2.2.2.1. Slump flow, density and air content

The slump flow was measured according to NBN EN 12350-8 [54]. To determine the density of the fresh self-compacting concrete the method described in NBN EN 12350-6 [56] was followed. The air content of the fresh mixtures was determined using the method described in NBN EN 12350-7 [57].

268 2.2.2.2. Final setting time

269 For the final setting time of the mixture, the penetrometer test based on the procedure described

270 in the standard ASTM C 403 [58] was used. This test was used to determine the starting point of

- the shrinkage measurements.
- 272

2.2.3. Hardened concrete properties

273 2.2.3.1. Compressive strength and E-modulus

The E-modulus and compressive strength tests were determined on triplicate cast cylinders with a height of 300 mm and a diameter of 150 mm according to NBN EN 12390-13 and NBN EN 12390-3, respectively [59], [60]. E-modulus and compressive strength were determined at an age of seven days and 85 days for the wall, and 7 and 28 days for the slab.

278 2.2.3.2.Shrinkage

279 Real time deformation behaviour of the wall was measured through fibre optic SOFO sensors 280 (from the company SMARTEC, Switzerland [61]) based on low-coherence interferometry. 281 Sensors were embedded in the wall through attachment to the steel reinforcement mesh before 282 casting. The sensors were mounted at the following three positions: the bottom, the middle and 283 the top annotated respectively as (B), (M) and (T) at a height of 0.120 m, 0.705 m and 1.3 m 284 measured from the slab-wall interface. Five sensors were introduced in the wall, three long 285 sensors with an active length of 1 m (T, M and B2) and two small sensors with an active length 286 of 0.2 m at the bottom edges to monitor AS of the edges: B1 and B3.

After demoulding, on the outer surface of the wall, measurement points for demountable mechanical strain gauges (DEMEC) were glued at three levels following the same pattern as the SOFO positions. At the top, middle and bottom, ten measuring points were glued with a 200 mm spacing between the points, and the first and the last point were 100 mm away from the edges. The DEMEC measurements were taken manually each day for 28 days, and then one measurement at 60 days and one at 120 days. The positions of the SOFO sensors and theDEMEC points on the wall can be seen in Figure 2.







297 <u>Restrained shrinkage</u>

Restrained shrinkage was investigated according to the method described in the standard ASTM C 1581-04 [62] known as the ring test method. The deformations of the inner steel ring, which represent the strains of the deforming concrete, were measured at three locations through three strain gauges attached to the ring. Measurements were performed every ten minutes from the time of casting until the age of 20 days. The outer steel ring was removed after 19 hours (same time as formwork removal from the wall), and the rings were sealed with plastic foil to avoid water evaporation, and stored in a climate-controlled room at $20 \pm 2^{\circ}$ C and $60 \pm 5\%$ RH.

305 3. Results

306 3.1. Concrete characterization

307 3.1.1. Testing to find the input parameters for the modelling

- 308 3.1.1.1. Strength and E-modulus
- 309 The obtained compressive strengths, direct uniaxial tensile strengths and the E-moduli in tension
- 310 are the average of three tested specimens, results are presented in

- 311 Table 5. The tensile E-modulus was found from the load-deformation relation measured in the
- 312 uniaxial tensile strength tests, where the E-modulus was deduced as the stress/strain ratio 313 between 10% and 40% of the failure load.
 - **Compressive strength*** Tensile strength** E-modulus in tension** Time [h] f_c [MPa] Time [h] Time [h] f_t [MPa] E_c [GPa] 24 68.9 27 3.9 27 34.8 120 126.7 72 5.7 72 696 149.3 42.6 670 696 6.6
- 314 Table 5: Strength and E-modulus of the concrete

315 * cube strength, 100x100x100 mm³ cubes

316 ** prism strength, 100x100x600 mm³ prisms

317 3.1.1.2. Hydration heat

- The box was stored in air at 38 °C while the air and concrete temperature were measured 318
- 319 continuously for 4 days. The hydration heat evolvement results are presented in Figure 3.



39.1

Figure 3: Hydration heat evolvement (a). Laboratory test results versus model: compressive
 strength (b), tensile strength (c) and E-modulus (d)

322 3.1.2. Autogenous shrinkage measured in Free Deformation (FD) and Temperature Stress

323 **Testing Machine (TSTM)**

324 Autogenous shrinkage was measured up until 289 hours (12 days), and all measurements were

325 zeroed at the starting time for stress development, $t_0 = 8$ hours (found from the TSTM test).

326 Autogenous shrinkage measured in the FD system, i.e. under 20 °C isothermal curing conditions,

327 is provided in Figure 4 (a). The result represents the average measurements of two concrete328 specimens.

329 The TSTM system is temperature-controlled, the applied temperature history, Figure 4 (b), was

determined using the program CTC, and it was based on the calorimetric heat evolvement testresults and the geometry of a pre-defined wall.

332 The TSTM test was performed in order to determine material- and model parameters for early 333 age crack assessment in CTC. A degree of restraint of only 30% was chosen in order to minimize 334 the restrained stresses during testing, and hence to keep the test running as long as possible to 335 obtain enough data for parameter estimation. The measured stress development of the concrete 336 when subjected to realistic temperature conditions and a degree of restraint of 30% is presented 337 in Figure 4 (b). The specimen developed failure in tension at 4.1 MPa after only 26 hours (51 338 hours of maturity), which is a somewhat lower tensile strength level than found from the 339 corresponding uniaxial tensile strength tests. It should however be noticed that the specimen in 340 the TSTM test was subjected to both realistic temperature conditions and sustained loading prior 341 to failure, i.e. there were considerable variations in the test conditions between the tensile 342 strength test and the TSTM test.

In addition to autogenous shrinkage and stress development, tests in the TSTM system also provided the starting time for stress development, t_0 , the incremental E-modulus development and the coefficient of thermal expansion, CTE, for the concrete in question. The starting time for stress development t_0 was found from the restrained stress development test as the maturity time when the measured stress reached 0.1 MPa for tests performed under realistic temperature conditions. After testing, the CTE was determined by applying temperature steps of ± 3 °C around an initial temperature of 20 °C. The CTE was determined as the average value over four such temperature loops. The CTE for the current concrete was found to be $10.7 \times 10^{-6/\circ}$ C after 250 hours of maturity (10 days of maturity).

The autogenous shrinkage for the concrete subjected to a realistic temperature history was deduced by subtracting the thermal dilation (calculated by the CTE and the measured temperature) from the total free deformation measured in the dilation rig. The autogenous shrinkage is given in Figure 4 (a), where the curve was zeroed at the starting time for stress development, $t_0 = 8$ hours. The autogenous shrinkage development was observed to be quite similar for both 20 °C isothermal and realistic temperature curing conditions, Figure 4 (a).

358 In the current study, the time-dependent behavior of the concrete has been described by 359 viscoelasticity for ageing materials, and more specific, by the double power law [63]. The creep 360 model parameters ϕ_0 , d and p were deduced by the following procedure: The restrained stress 361 development measured in the TSTM was back-calculated by a calculation routine in Excel. The 362 performed stress calculations were based on linear viscoelasticity with age adjusted effects, using the degree of restraint, the measured temperature and free deformation from the dilation rig as 363 input data. The creep parameters were estimated based on previous experience on similar 364 365 concretes, and then adjusted by fitting the calculated stress development to the measured stress 366 development, Figure 4 (b). The creep model parameters deduced for the current concrete were: $\phi_0 = 0.70$, d = 0.42 and p = 0.26. The stress development is driven by thermal dilation (TD) and 367 368 autogenous deformation (AD). Usually, the temperature increase causes an initial compression 369 and appurtenant compressive stress. In the current case, the AD is so high it outcompetes the 370 temperature effect and causes a tensile stress from the start. Between 10 and 12 hours, the AD 371 rate is somewhat lower, and the sum of AD and TD is close to zero. Hence the stress 372 development between 10 and 12 hours is very small due to the lack of volume changes in this 373 period.

374



Figure 4: Autogenous shrinkage measured in the FD (isothermal) and the TSTM-system (realistic temperature evolution), zeroed at t_0 (a); Stress development in the TSTM, realistic curing temperature, R = 30% (b)

378 3.2. Modelling

A model was established in the program CrackTeSt COIN (CTC) [44] in order to simulate the temperature-, stress- and strength development in the planned UHPC demonstration wall described in the previous section,

Figure 5 (a). The main aim was to assess and predict the cracking risk in the wall, and to determine further the conditions necessary to achieve cracking at around 20 hours after water addition. CTC is a two-dimensional special-purpose program for temperature- and stress calculations in young hardening concrete. A crack risk simulation in CTC comprises a heat flow analysis followed by a structural analysis. Stress computations in the structural analysis include stresses in the orthogonal direction according to the boundary conditions given by the user. The heat and the structural analysis use the same element mesh generated by CTC

Figure 5 (b). All in all, CTC provides a crack assessment of a given structure by simulating the temperature- and stress development, as well as the time and location where the generated stress exceeds the corresponding tensile strength.

392 The wall geometry was modelled as defined in the previous section, see

Figure 5 (a). The ambient temperature and the fresh concrete temperature were assumed to be 23

394 °C and 20 °C, respectively. The restraint conditions, which vary over the wall height, were

395 defined by CTC based on the geometry of the structure and the applied concrete. The

- 396 investigated concrete was implemented in CTC by a material database file created from the
- 397 laboratory test program presented in the previous section. It should be noticed that the program

398 uses autogenous deformation measured in the laboratory versus maturity as input, and does not 399 include the effect of drying. The reason for this is that CTC is generally used for massive 400 structures, where the effect of drying can be neglected. The variation parameters in the currently 401 performed analyses were "type of formwork" as well as "formwork removal time", which both 402 affect the temperature development, and hence the cracking risk, in the wall. Analyses were run 403 for the wall subjected to three different formwork alternatives:

- 404 20
 - 20 mm plywood removed at 18 hours
- 405 20 mm plywood removed at 72 hours

406 • 50 mm plywood removed at 72 hours

407 The CTC calculation results are presented in Figure 6, where each graph represents an average408 value over the hatched area of the wall as illustrated in

Figure 5 (a). The hatched area is 50x50 mm² and it is located 150 mm up in the wall when measured from the top of the slab. The restraint is highest close to the underlying foundation, while the temperature increase is highest further up in the wall. The highest stress development is found at the most unfortunate combination of high restraint and high temperature, which for massive walls is considered to be approximately one wall thickness up from the top of the slab [53], [64]. For the current wall, CTC found this location to be at the hatched area illustrated in Figure 5 (a).

416 Simulated temperature developments for the three formwork alternatives are presented in Figure 417 6 (a). The thicker formwork, 50 mm plywood, provided better insulation and hence a higher 418 temperature increase and decrease during curing. For the alternatives with 20 mm plywood, an 419 earlier "formwork removal time" resulted in a somewhat steeper temperature decrease during the 420 cooling period.

421 The simulated stress developments (average over the hatched area) were quite similar over the 422 first 22 hours for all three formwork alternatives, Figure 6 (b). For the given concrete, the early 423 age volume changes are strongly dominated by a very high autogenous deformation (AD). 424 Consequently, a small variation in the curing temperature will only constitute a limited influence 425 on the total early age volume changes, and hence the stress development. The difference in peak 426 temperature between the cases of 50 and 20 mm plywood formwork, is approximately 6 to 7 °C. With a CTE of around 10.7 x 10⁻⁶/°C, this causes a difference in thermal strain of about 60-70 427 428 microstrain between the different formworks. The AD at the same time is -420 microstrain. This 429 makes clear that for UHPC, AD is the main driving force for stress development, and a 430 difference of 60-70 microstrains caused by temperature does not have a big influence on the 431 stress development. The increase in tensile strength was slightly higher for the 50 mm plywood 432 alternative due to its higher temperature increase during curing, however, the correspondingly 433 higher temperature-decrease during the cooling phase also caused a somewhat higher stress 434 development over time when compared to the other alternatives, Figure 6 (b). For the wall cast 435 with 20 mm plywood formwork removed at 18 hours after water addition, the generated tensile 436 stress in the hatched area exceeded the corresponding tensile strength at approximately 18 hours 437 after water addition. The cracks will initiate in the wall center, and they have to achieve a certain 438 width to be seen with the naked eye. However, at 20 hours after water addition, it is expected 439 that cracking can be observed for the given demonstration wall.

440



Figure 5: Demonstration wall model for use in CTC (a) and generated mesh (b)

441



Figure 6: Calculated temperature history (a); Calculated stress- and corresponding strength development (b)

442 **3.3.** Large-scale demonstrator

443 **3.3.1.** Fresh concrete properties

The slump flow of the wall was equal to 706 ± 22 mm, with an air content of 2.7% and a final setting time of 5.5 hours. The values represented are the average of the results obtained from the three batches used for the wall except for the final setting time which was performed only on one batch.

448 **3.3.2.** Hardened concrete properties

449 3.3.2.1. Compressive strength and E-modulus

450 The mechanical properties of the concrete used for the demonstration structure (slab + wall) can 451 be found in Table 6. Note that the measured compressive strength of 87.2 MPa at 7 days of age is 452 significantly lower than that of the concrete tested for the numerical simulation input (section 2.1.3.2) being 126.7 MPa at 5 d of age. This large difference is confirmed by testing at later ages 453 454 too: at 85 d of age the cylinder strength for the demonstration wall was 121 MPa, while the cube 455 strength of the concrete used for the model input was 149 MPa at 28 days of age. Different 456 specimen geometry (150/300 mm cylinders vs 100 mm³ cubes), is probably one reason for a 457 lower value, as it is known that cubes give generally higher measured strength values, in the 458 order of 5 to 10 percent in UHPC [65]. In addition, several comprehensive studies on high 459 strength concrete [55], [66], [67] concluded that strength values of 100 mm³ cubes were on 460 average 11-12 % higher than those of 150/300 mm cylinders. Furthermore, in an old Belgian standard [68], a general formula has been used to estimate with an accuracy of 10% the ratio between the compressive strength obtained from a 150/300 mm cylinder, R_x , to the compressive strength obtained from a cube of 100 mm³, R_{w10} . The ratio corresponding to the cylinders used in this study is $\frac{R_x}{R_{w10}} = 0.75$. Therefore, if we consider the strength value of 121 MPa obtained for the wall on cylinders, it would correspond to a value of 161 MPa on 100 mm³ cubic specimens, showing that the differences between cube and cylinder strength can indeed be large.

467 Table 6: Mechanical properties of the concrete used for the demonstration wall (on cylinders, d=150 mm
468 and h=300 mm)

	Compressive	E-modulus (GPa)	Compressive	E-modulus (GPa)
	strength (MPa)	7 days	strength (MPa)	28 or 85 days
	7 days		28 or 85 days	
Slab	48.8 ± 0.7	-	57.6 ± 2.2	39 ± 8
		4	(28 d)	(28 d)
Wall	87.2 ± 5.7	48	121.1 ± 3.5	47
			(85 d)	(85 d)

469

470 3.3.2.2. Shrinkage

471 Measurements of the real-time occurring strains in the wall were obtained automatically from the 472 fibre optic SOFO sensors embedded inside the wall and are represented in Figure 7. The 473 measurements were recorded during 4 months to follow the whole period of active shrinkage (as 474 it can be seen on the shrinkage curves there is no big difference in terms of deformations 475 between the third and the fourth month), the curves represent the values recorded from the long 476 sensors with an active length of 1 m. But since the early age cracking risk is discussed in this 477 study, it is interesting to consider especially the first few days of the concrete age. Thus, 478 shrinkage values over the first 8 days obtained from the long and one short fiber optic sensor are 479 represented in Figure 7 on the right. Measurements from the second short sensor (B3 in Figure 2) 480 could not be presented as there was too much noise in the obtained results. As it can be seen 481 from the shrinkage curves, the top of the wall shrinks more than the middle and more than the 482 bottom, demonstrating the influence of the restraining condition at the bottom, i.e. the wall 483 cannot freely shrink at this location. At around one day, deformation curves transform from a

484 very high rate of shrinkage deformations to a much lower shrinkage rate. With water being 485 rapidly consumed at early age, deformations are expected to increase in low water-to-binder ratio 486 concrete. After few hours, rigidity of UHPC becomes quickly high which restrains the following 487 deformations. At the age of 8 days shrinkage values of -130μ m/m, -320μ m/m, -640μ m/m and -488 815 μ m/m were obtained for bottom 1, bottom 2, middle and top, respectively. At 120 days, 489 shrinkage values obtained from the long sensors were -540μ m/m, -910μ m/m and -1110μ m/m 490 for the bottom, the middle and the top, respectively.



491

Figure 7: Deformations of the top (T), the middle (M) and the bottom (B2) of the wall obtained from the long fiber optic sensors embedded inside starting from t0 equal to the final setting time = 5.5 hours for a period of 4 months (on the left); on the right a zoomed graph over the first 8 days of the deformations in addition to the deformation curve given by the short sensor B1

496 Shrinkage strain curves obtained from the demountable strain gauges measurements were also in 497 accordance with the results found from the fiber optic sensors as it can be seen in Figure 8. The 498 starting point of the DEMEC measurements is taken equal to 23h30 after water-cement contact, 499 since the points needed to be glued on the wall after formwork removal (the zero point on the 500 time axis is the contact moment between water and cement). Over the 4 months period of testing, 501 it was confirmed that the wall has shrunk more at the top than the bottom; a value of 360 μ m/m 502 is reached for the top, 330 μ m/m for the middle and 250 μ m/m for the bottom. In Figure 9 the 503 results from the SOFO measurements are compared with the ones obtained from the DEMEC 504 measurements over the first 28 days, during which regular DEMEC measurements were taken. 505 The curves were plotted from an age of 23h30 after water-cement contact (the time where the 506 first DEMEC measurements were obtained), and the two sensor types followed the same 507 behavior over 28 days.



- 514 Figure 9: Comparison of the occuring shrinkage strain results obtained from the fiber optic SOFO sensors 515 and the demountable strain gauges (DEMEC), starting at an age of 23.5h after water-cement contact (a) is
- 516 for the bottom, (b) for the middle and (c) for the top positions on the wall



The results of the restrained shrinkage test are presented in Figure 10, each curve represents the values obtained from the sensors attached to the ring (one sensor was not working properly therefore only two out of three are shown). Time zero is equalled to the water-cement contact and the curves are plotted from the age of one hour. The sudden jump in the curve represents the time of cracking, where the sensors measure an abrupt change. The strain values reached about -

523 100 μ m/m before the sample cracked at around 2 days.



524

525 Figure 10: Restrained shrinkage results obtained from the ring method; the zero point on the time axis 526 represents the moment of water-cement contact

527 **4. Discussion**

528 4.1. Comparison of stress development and time of cracking

529 The work comprises three different approaches to determine stress development and subsequent 530 cracking assessment:

- a) TSTM test with 30% restraint
- b) Numerical simulation of the wall
- c) Strain measurements and visual observation of the wall

534

535 The TSTM test measured the stress development in the hardening phase for a specimen subjected 536 to a degree of restraint of 30%. This restraint did not represent the given demonstration wall, but 537 it was applied in order to minimize the restrained stress during testing, i.e. to keep the test 538 running as long as possible to obtain enough data for parameter estimation. Despite the low 539 restraint, the TSTM specimen developed failure in tension as early as 26 hours after water 540 addition, confirming that the investigated concrete is very prone to early age cracking. 541 The degree of restraint in a wall varies over the wall height and length, and it depends on several 542 parameters: the area, the geometry and the stiffness of both the wall and the base slab, the elastic 543 properties of the ground beneath, the length to height ratio of the wall, as well as possible slip 544 failures between the wall and the base slab. The wall deformation measurements in Figure 7 and 545 illustrate how the restraint in the wall decreases with increasing distance from the joint, i.e. 546 increasing deformations with increasing distance from the base slab; this represents the very 547 typical relation between restraint and the distance from the base as reported by Schleeh [69], 548 Erreur ! Source du renvoi introuvable.where several tests were conducted on micro-concrete walls with different length-to-height ratios cast on a steel slab. For a wall cast on a massive 549 550 concrete base it is suggested by BS 8110 2 [70] that the restraint factor at the joint lies between 551 0.6 and 0.8. This corresponds well with a simplified calculation method presented by CIRIA 552 C766 [33], which estimated the given wall to have a degree of restraint at the joint of 0.78, i.e. 553 78%. The same calculation estimated a degree of restraint of 30% to occur approx. 550 mm 554 above the joint. In the TSTM test, the specimen reached a tensile stress of 4.1 MPa after 26 555 hours, Figure 5. Correspondingly, the CTC simulations showed a restrained stress of approx. 4.8 556 MPa after 26 hours in an area 550 mm above the joint. Although these approaches were not fully 557 comparable due to variations in their temperature histories as well as the restraint considerations 558 discussed in the following section, they did provide restrained stresses in the same order of 559 magnitude, indicating rather good agreement.

560 The first crack in the demonstration wall was reported two hours after formwork removal (21 561 hours after water-cement contact). This was a through-going crack that reached the middle height 562 of the wall, followed by other non through-going cracks (except at the edges) as depicted in 563 Figure 11. The CTC stress simulation estimated the generated tensile stress in the critical part of 564 the wall to exceed the corresponding tensile strength somewhat earlier, at approx. 18 hours after water addition. However, the given demonstration wall has a low length to height ratio L/H = 1.3565 566 < 3, meaning that the Bernoulli-Euler assumption of "plane sections remain plane when 567 deforming" cannot be applied. This may cause 3D effects in terms of a reduction of restraint and 568 hence stresses in the wall, and these effects may not be captured by the two-dimensional CTC 569 simulation. Consequently, due to the low length to height ratio of the wall, the CTC simulation 570 was expected to provide a somewhat higher restraint and stress development than the actual wall. 571 In addition, the demonstration wall seemed to experience some slip failure between the wall and 572 the base slab, Figure 11, which also reduces the degree of restraint. Summarized, the degree of 573 restraint in the demonstration wall was expected to be somewhat lower than the degree of 574 restraint determined by CTC in the simulations, i.e. the stress development and cracking of the 575 wall was expected to occur somewhat later than estimated by the simulation. The program CTC, 576 which is intended for massive concrete structures, does not include the effect of drying. Drying 577 shrinkage is generally not considered to have a big influence on UHPC/HPC structures, and in 578 addition, the analysis expects the first crack to appear at approx. 18 hours after water addition, 579 which is the same time as formwork removal. Before 18 hours, the formwork will prevent 580 drying, and hence the possible drying-induced inaccuracy of the simulation prior to the first 581 crack can be neglected. All things considered, the simulated stress development and crack 582 assessment showed good agreement with the observations of the demonstration wall.



584 585

583

Figure 11: Crack development on the wall with the approximated crack width at each position

In addition, regarding the ring test, the observed jumps in Figure 10 represent cracking of the rings that took place at around 2 days after water-cement contact, a cracking time that was later than that for the wall. The main reason is the huge difference in the experimental set-up between the ring and the wall, e.g. the geometries and restraining conditions, that highly influence the induced stresses. In addition, the wall also experienced larger additional thermal contractionfollowing the hydration heat peak.

592

4.2. Shrinkage measurements

- 593 The work comprises four different deformation measurements:
- a) Free deformation measured in the FD-system (isothermal)
- b) Free deformation measured in the TSTM-system (realistic temperature)
- 596 c) Shrinkage strains occurring in the wall under various restraint conditions, measured with
 597 two different systems (realistic temperature)
- d) Restrained shrinkage deformations measured in the ring test (isothermal)
- 599

AS was measured under 20 °C isothermal conditions in the FD-system, and under realistic 600 601 temperature conditions in the TSTM-system. Both measurements were zeroed at the starting time 602 for stress development, $t_0 = 8$ hours, as determined from the TSTM test. The AS development was observed to be quite similar for both 20 °C isothermal and realistic temperature curing 603 604 conditions, Figure 4 (a), as opposed to other concretes where realistic curing temperature regimes 605 have been found to initiate fundamental differences in the AS development when compared to 20 606 °C isothermal curing [71]. The free deformation measurements in the TSTM-system include both 607 autogenous deformation and thermal dilation. The demonstration wall was rather thin, with a 608 thickness of only 50 mm, however, due to the high cement content and corresponding heat 609 evolvement, the given wall experienced a temperature increase of as much as 20 °C during 610 curing. This caused a thermal expansion and contraction of about 200 microstrains, which still 611 was far less than the deduced AS which reached a level of 700 microstrains after approximately 612 one week. It is thus evident that the considerable amount of AS found for the investigated 613 concrete constitutes the main mechanism when it comes to early age volume changes and hence 614 the cracking risk of the demonstration wall.

615 Under surrounding temperature $(20 \pm 2^{\circ}C \text{ and } 60 \pm 5\% \text{ RH})$, real-time deformations of the wall 616 were measured automatically by the fiber optic SOFO sensors and manually by the demountable 617 DEMEC strain gauges. In Figure 7, results obtained from the SOFOs are plotted where the 618 influence of the restraining condition at the bottom is clearly shown. The wall undergoes a lot of 619 shrinkage strains at the top where it is more free to shrink with less stresses. Then, these strains 620 gradually decrease when approaching the restraining condition at the bottom near the slab-wall 621 connection. On the left of Figure 7, a big difference is seen between the shrinkage values of the 622 bottom sensor and the middle/top sensors, as the shrinkage strain difference between the top and 623 the middle is around 200 μ m/m whereas the one between the middle and the bottom is around 624 $400 \,\mu\text{m/m}$. The behavior of the bottom curve was studied at the very early age (0-24h) as seen in 625 Figure 12, and a change of the slope of the shrinkage strain curve was seen at around 19 hours 626 which equals to 21 hours after water-cement contact (taking into account a gap of 2 hours that 627 represents the time to cast the wall and to put the sensors into operation). The abrupt change in 628 Figure 12 represents the formation of the cracks, which was seen with the naked eye 2 hours after 629 formwork removal (formwork was removed at 19 hours after water-cement contact). The longest and first crack passed through the bottom sensor, for that reason there was a relaxation of the 630 631 stresses at that location which explains the big difference between the bottom and the middle-top 632 strain curves. At the final measurement (120 days after casting of the wall) this crack had a 633 length of 850 mm and a width of 180 µm at the bottom of the wall. Halfway the crack height the 634 width was 80 µm and at the top of the crack the width was 40 µm. Using the same interpretation, 635 the shrinkage values of the bottom sensor B1 near the edge of the wall were 23% lower than for the long bottom sensor 2 in the middle, due to the presence of many cracks that were clearly 636 637 apparent at the edges of the wall. Since B1 had more cracks over its smaller length, it measured less strain deformations than B2. 638



639

Figure 12: Representation of the shrinkage measurements obtained from the long bottom fiber optic sensor embedded inside of the wall at the early age (24 h)

642 Shrinkage strains of the wall were also manually measured with demountable strain gauges.643 Values are shown in Figure 8 over a period of 4 months (same as for the SOFOs). These curves

644 follow the same trend as the SOFOs and also show the influence of the restraining condition at 645 the bottom. The ranges of the DEMEC values at the end of the testing period are lower than the 646 ranges of strains of the SOFOs (Figure 7), which is due to the difference in the starting point of 647 the measurements. For the DEMEC measurements, points need to be glued on the wall after 648 formwork removal and first measurements are taken after the glue dried which was roughly 649 around 3.5 hours after formwork removal. Therefore, measurements started after 23.5 hours 650 when a big part of the occurring strains had already happened. A comparison between the SOFO 651 and DEMEC measurements for the first 28 days of age was performed in Figure 9. The starting 652 point of the measurements in this figure was taken to be 23.5 hours after water-cement contact 653 for both techniques. The SOFO measurements showed slightly higher strain values at the 654 different levels. Drying shrinkage is normally not considered to have a big influence on 655 UHPC/HPC structures. Zhang et al. found that most of the total shrinkage undergone by high 656 strength concrete (containing silica fume) could be attributed to AS rather than drying shrinkage 657 due to the dense and impermeable matrix [72]. Xie et al. performed autogenous and free total 658 shrinkage on UHPC prisms and showed that drying shrinkage comprises approximately 20% of 659 the total measured shrinkage. In addition, during early concrete ages (until 7 days after water to 660 cement contact), drying shrinkage is not considered since the difference between the free total 661 and autogenous shrinkage curves is negligible [73]. Nevertheless, water evaporation near the 662 surface of the wall could have been faster than deeper inside the matrix, which could have led to 663 a slightly lower level of cement hydration and therefore slightly lower shrinkage strains obtained 664 from DEMEC measurements. In addition, the rapid water evaporation at the surface will enhance 665 the formation of microcracks which leads to a relaxation of stresses around the crack itself. This also justifies the lower DEMEC shrinkage strain curves. 666

The ring test method was used to evaluate the shrinkage under restrained conditions. As it can be seen in Figure 10, samples reached a strain of -120 μ m/m before cracking after 2 days. Under realistic temperature and RH conditions ($20 \pm 2^{\circ}$ C and $60 \pm 5\%$ RH), the wall cracked two hours after formwork removal at around 21 hours after water-cement contact while rings cracked at a later age. This could be justified by the fact that the wall experienced larger additional thermal contraction following the hydration heat peak in addition to the differences in geometry and restraining conditions existing between the wall and the ring.

674 **5.** Conclusion

To predict the risks of early age shrinkage cracking for UHPC, a numerical model, which has 675 been established primarily for normal concrete, has been further developed and applied on a 676 677 demonstration wall with high risk of cracking, cast on a non-deforming slab. Thus, an early age 678 crack assessment was performed in this study using a numerical simulation model applied on a 679 typical restrained wall. A demonstration structure (a wall cast on a non-deforming slab) has then 680 been built in order to verify the simulated cracking risk through visual observation of cracking 681 and deformation measurements of the wall. A comparison between the numerical simulation 682 results and the actual measurements was presented. The currently used UHPC showed a 683 considerable autogenous shrinkage, constituting the main driving mechanism to the early age 684 volume changes and the corresponding high cracking risk of the investigated wall.

685 The following conclusions can be drawn:

686

687 • The comparison between the numerical simulation results and the actual measurements 688 proved that the proposed model is suitable for UHPC-concrete. A good fit between the 689 model and experimental results was found for the development of compressive strength, 690 tensile strength and E-modulus in tension. In addition, the performed tests and analyses 691 confirmed that UHPCs are very prone to early age cracking due to the high AS 692 development as well as the high temperatures generated during curing even though the 693 wall was rather thin. Indeed, the model highlighted the increased risk of cracking due to 694 the increased degree of restraint associated with the small wall thickness.

695 The numerical simulation gave a good prediction of the cracking risk in the wall: 696 cracking was predicted to appear at approximately 20 hours after water addition, and the 697 first crack was observed in the wall at 21 hours after contact between cement and water. 698 Hence, the novelty of the model remains in the time prediction of crack development rather than the crack development itself. The through-going crack observed in the 699 700 demonstration wall reached the mid height of the wall with a width of 180 µm at its 701 bottom, 80 µm at its middle and 40 µm at its top. The crack was followed by other 702 similar small cracks with a width of 50 µm.

Autogenous shrinkage results under isothermal and realistic conditions followed the same
 behavior and experimental and modelled results were in the same range. Under realistic

conditions the occurring strains in the wall were recorded automatically using fiber optic
sensors embedded inside the wall and manually with demountable strain gauges through
points glued on the outer surface of the wall. Another key point to stress on, AS
deformations can be measured in different ways for big/massive structures where it was
shown that AS is very dependent on the zeroing time of measurements as well.

Restrained shrinkage tests were performed on the cast concrete using the ring test
 method. Rings cracked at 2 days, a later time than the wall, which is probably because the
 wall experienced larger additional thermal contraction following the hydration heat peak
 in addition to the effect of a difference in geometry and restraining conditions that exists
 between the ring and the wall.

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