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► **To cite this version:**

Menghuan Guo, Frédéric Grondin, Ahmed Loukili. Numerical method to model the creep of recycled aggregate concrete by considering the old attached mortar. *Cement and Concrete Research*, 2019, 118, pp.14 - 24. 10.1016/j.cemconres.2019.01.008 . hal-03485901

HAL Id: hal-03485901

<https://hal.science/hal-03485901>

Submitted on 20 Dec 2021

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1 Numerical method to model the creep of recycled aggregate concrete by
2 considering the old attached mortar

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6 **Abstract**

7 Experimental creep tests **have shown** that recycled aggregate in concrete increases its displacement
8 rate. In order to explore this behavior, **multiscale modeling** has been performed. To consider the
9 old attached mortar surrounding recycled aggregates, creep parameters were calculated from the
10 cement hydrates scale. **Sixteen** ordinary mortars were calculated to have a large range of values. A
11 homogenization model was applied to calculate the viscoelastic properties. At the concrete scale, in
12 order to take into account the influence of the heterogeneity of the recycled material, a creep-damage
13 coupling model **was** used. The proposed numerical method demonstrates that the viscoelastic
14 strain of recycled aggregates is **not negligible**, and leads to a larger creep displacement and **higher**
15 **damage level**. Its influence on the time-dependent behavior of recycled aggregate concrete should
16 be considered in **modeling**.

17 *Keywords:*

18 Recycled aggregate concrete, Old mortar, Modelling, Creep, Viscoelastic

19 **1. Introduction**

20 In the 21st century, the concept of sustainable development prompts the reutilization of **cement-**
21 **based** products derived from construction and demolition waste. Concrete made **by partially or com-**
22 **pletely substituting recycled aggregates (RAs) for natural aggregates represents an environmentally**
23 **friendly material that holds the promise of recycling construction waste**. Extensive research work
24 has been carried out to study the workability and mechanical properties of recycled aggregate con-
25 crete (RAC) with the purpose of extending its application for structural use [1–7]; fruitful results

Pre-proof
Printed in the USA
Prepared for submission to Elsevier
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November 29, 2018

26 have been achieved. However, research concerning the durability-related performance of recycled
27 concrete has until now been limited. In particular, the creep behavior of structural RAC has not
28 yet been fully studied, despite the fact that creep is of considerable importance for the structural
29 design of concrete.

30 As we all know, the creep deformation of normal concrete is a complex time-dependent phe-
31 nomenon influenced by several factors, including the mix design, temperature and relative humidity,
32 load conditions and size of the structure [8, 9]. In the case of RAC, this phenomenon becomes much
33 more complicated. Generally, the incorporation of RAs leads to accelerated concrete creep, the mag-
34 nitude of which, all other things being equal, depends on the aggregates' quality and replacement
35 ratio [10, 11]. Domingo-Cabo et al. [12] reported that when 100% coarse natural aggregate was
36 replaced with RAs, there was a 51% increase in the creep deformation as compared to that expe-
37 rienced by the control concrete. According to Seara-Paz et al. [13], the specific creep of recycled
38 concrete is higher than that of conventional concrete, with the increment ranging from 51% to
39 73%. Tam et al. [14] experimentally investigated the drying creep behavior of concrete made
40 with varied RA replacement ratios of 0%, 30%, 50%, 70% and 100%, and concluded that drying
41 creep increases with the replacement ratios. Wang et al. [15] also demonstrated that the long-term
42 deflections of composite concrete slabs increase with the RA substitution ratio. Similar results are
43 reported by [16–20].

44 As observed by several researchers mentioned above, the increased deformation by creep is
45 directly related to the attached old mortar surrounding the aggregate particles. The old residual
46 mortar is usually porous and contains microcracks [21, 22], due to the prior mechanical crushing
47 process. The content, elastic modulus and creep behavior of the old adhering mortar can greatly
48 influence the RAC's creep characteristics [17, 20]. In order to account for this influence, Fan
49 et al. [17] has adapted the Neville model to predict RAC creep with high accuracy. Likewise,
50 Fathifazl and Razaqpur [23] have modified five rheological models for conventional concrete to
51 make them suitable for application to RAC, with the effects of the quantity and properties of
52 residual mortar on RAC creep delicately involved in the modification. Fathifazl et al. [24] also
53 demonstrated that by applying the proposed "residual mortar factor" to the existing ACI and CEB

54 methods for calculating creep of conventional concrete, these methods could be also applied to
55 predict RAC creep. Based on this proposed residual mortar factor, Wang et al. [15] has developed
56 a homogeneous finite-element model to account for the time-dependent behavior of composite RAC
57 slabs by including the effects of non-uniform shrinkage, creep and concrete cracking. Knaack and
58 Kurama [16] concluded that the determined adjustment factors can be applied to existing natural
59 aggregate concrete creep relationships (that is, code-based models) to predict the creep strains of
60 RAC. Some of those authors [12, 13, 25–28] also put forward a set of correction factors for the creep
61 coefficient of recycled concrete.

62 Generally, the aforementioned creep prediction methods are used either by proposing a creep
63 coefficient to account for the amplified influence of recycled concrete, or by adding a residual mortar
64 factor in existing creep models for conventional concrete. Both of these methods depend greatly
65 on the experimental outcome. Several comparative studies have also been carried out [10, 28].
66 However, owing to the high scatter of old mortar properties and content, the application of these
67 creep prediction models for RAC is quite limited. The available information on the effect of RAs on
68 concrete creep does not allow a full understanding of their influence on the deformation of concrete
69 structures. Thus the necessity of explicitly considering the effect of the properties of unknown old
70 attached mortar on the creep prediction model is quite evident. Furthermore, a recent work [29]
71 demonstrated that the resistance of recycled coarse aggregate to fragmentation is significantly lower
72 than that of natural aggregate. The experimental results of [30–32] also showed that RAs tend to
73 have higher water absorption, lower density and lower particle strength. It is vital that RA quality
74 is evaluated in order to understand its role on the creep behavior of concrete [28]. In fact, analysis
75 of the effects of all the existing phases in RAC on the creep behaviour of the material remains
76 challenging. This study attempts to tackle this problem by carrying out a mesoscopic modeling of
77 RAC. The RA mesh was generated explicitly in the finite element code Cast3M by considering the
78 old original aggregate surrounded by the old attached mortar. RAs were distributed in a random
79 way in concrete, and simulations of creep behavior were performed. A coupled creep-damage model
80 [33] was used, and numerical results were analyzed and compared to experimental measurements.
81 As the characteristics of old attached mortar are unknown, we assumed that they were equivalent

82 to those of a set of traditional mortar. The methodology for the determination of the fracture
83 properties of this set of chosen mortar has been presented in [3]. In this study, in order to characterise
84 the time-dependent viscoelastic properties of the chosen mortar, an inverse analysis that allows the
85 numerical model to upscale the phase characteristics to macroscopic properties of concrete was
86 carried out [34]. A numerical homogenization method was applied for the calculation of the creep
87 compliance between the different scales.

88 2. Experimental basis

89 In most cases, concrete structures are subjected to low load levels, which are generally inferior to
90 40 % of the material strength. It is generally accepted that in this loading range, the instantaneous
91 mechanical behavior of concrete remains elastic and creep remains linear without damage occurring.
92 It is very interesting to check whether RAC's creep compliance complies with the same practice.
93 In this work, the RAC beams at the age of three months were loaded under a stress level of 40 % of
94 the maximal resistance F_{max} for three months in order to evaluate the influence of the substitution
95 ratio of RA for natural aggregate on the kinetics and amplitude of bending creep.

96 Concrete mixtures were prepared with CEM II/A-L 42.5N from Rochefort, crushed limestone
97 aggregate and RA. RAs were obtained by crushing unknown waste concrete provided by one of
98 the largest French recycling platforms (DLB de Gonesse), while the source concrete came from
99 surrounding demolition sites. Therefore, the recycled materials used in this study represent the
100 properties of a variety of collected waste from general construction and demolition sites. Variation
101 of the RA replacement percentage (0, 30 % and 100 %) relative to the total mass of coarse aggregate
102 in concrete mixtures was studied. All the mixtures were designed to have the same compressive
103 strength in the range of 25-30 MPa and the same workability, according to the structural class
104 S4 [8]. The studied mixtures are noted respectively: C-N-N - Concrete with Natural sand and
105 Natural aggregate; C-N-30R - Concrete with Natural sand and 30 % Recycled aggregate; C-N-100R
106 - Concrete with Natural sand and 100 % Recycled aggregate.

107 First, three-point bending tests were carried out on notched concrete beams with the dimensions
108 of $100 \times 200 \times 800 \text{ mm}^3$ (b×h×L) at the age of 28 days [3]. F_{max} was thus determined according

109 to fracture tests controlled by the crack mouth opening displacement (CMOD). Flexural creep tests
110 were then performed on frames with a capacity ranging from 5 to 50 kN (Figure 1) [35]. The load
111 was applied by gravity with a weight and counterweight system, and the displacement was measured
112 at midspan. All tests occurred in a climate-controlled chamber at 50% of relative humidity and a
113 temperature of 20 °C.

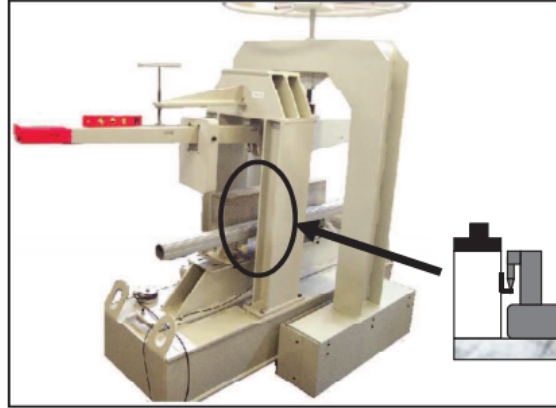


Figure 1: General view of the flexural creep frame.

114 For comparison between the three types of concrete that were studied, the specific delayed
115 displacement was defined as the amount of deflection per unit applied stress. The stress was
116 evaluated according to the following relationship:

$$\sigma = \frac{3}{4} \frac{\rho g h l^2}{(h - a_0)^2} + \frac{3}{2} \frac{F l}{b(h - a_0)^2} \quad (1)$$

117 where ρ represents the density of concrete, l the distance between the two supports, F the applied
118 force, h and b , respectively, the height and the width of the beam and a_0 the height of the notch.
119 After subtracting the instantaneous response, the specific creep displacement was obtained.

120 Figure 2 presents the specific delayed displacements for the three concrete types. Note that the
121 creep kinetics were comparable for natural aggregate concrete and 30% RAC, with an amplitude
122 1.2 times greater after three months of loading for the latter one. For concrete with 100% RA,
123 creep developed faster and the amplitude was practically 1.6 times greater than normal concrete.
124 The creep displacement of normal concrete, after three months of loading, is reached by C-N-100R

125 after only four weeks of loading. These results agree with the previous work of other researchers
126 [12–14]. The fracture analysis of these three types of concrete [3, 36] offers two assumptions: either
127 RA creeps, or microcracks initiate and weaken the concrete that is subjected to constant load. For
128 the purpose of checking these two assumptions, we analyzed the creep evolution by calculating the
129 ratio of the specific delayed displacement at time t to the final specific displacement measured at
130 the end of the test, as shown in Figure 3.

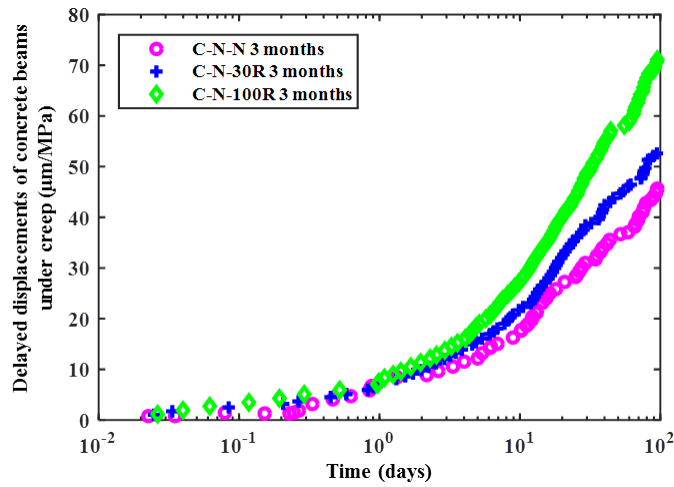


Figure 2: Delayed displacements of concrete beams under creep.

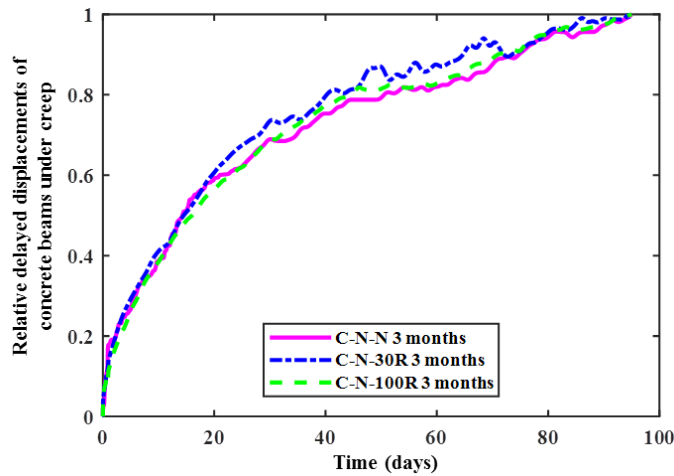


Figure 3: Relative delayed displacements of concrete beams under creep.

131 Note that the delayed displacement amplitudes were quite different for the three kinds of con-
132 crete. Nevertheless, the evolution trend of the relative delayed displacement was comparable for the
133 three concrete types, which suggests that the creep mechanism was identical for recycled concrete
134 and normal concrete. In other words, viscoelasticity and/or microcracks were of the same type in
135 the three kinds of concrete though their amplitudes were different. In brief, the experimental tests
136 performed in this work show clearly the effect of RA on the creep behavior of concrete. However,
137 these tests were time-limited. It is imperative that a creep prediction model that considers all the
138 involved mechanisms be built in order to prevent recycled concrete structures from cracking at an
139 early age as well as in the long term, and to quantify, at the first time, the degradation of their
140 properties.

141 3. Description of the representative specimens for multiscale modeling

142 The model used here is based on the assumption that the creep of concrete depends mainly
143 on that of calcium silicate hydrates (C-S-H) [34]. According to Smilauer et al. [37], C-S-H is the
144 only phase that is responsible for creep at the cement paste scale and at the concrete scale. The
145 experimental study of Torrenti et al. [38] showed that the derivative of the basic creep compliance
146 with respect to the loading time is unique whatever the loading age. Others researchers [39–41]
147 have also concluded that the creep of cement paste originates in C-S-H. All of them confirm the
148 hypothesis made in the multi-scale model used here. In this model, in order to calculate the creep
149 defined by a volume of material with a viscoelastic matrix and elastic inclusions, four scales are
150 considered:

- 151 • Sub-microscopic, corresponding to the C-S-H scale, represented by a homogeneous C-S-H
152 matrix and the pores as inclusions
- 153 • Microscopic, corresponding to the cement paste scale, represented by a homogeneous matrix
154 equivalent to the material of the sub-microscopic scale and the hydration products other than
155 C-S-H and the residual cement clinkers as inclusions

- 156 • Sub-mesoscopic, corresponding to the mortar scale, represented by a homogeneous matrix
157 equivalent to the cement paste and the grains of sand as inclusions
- 158 • Mesoscopic, corresponding to the concrete scale, represented by a homogeneous matrix equiv-
159 alent to the mortar and the aggregates as inclusions

160 In this study, an effective interphase (EI) between aggregate and cement mortar **was** added and
161 considered in the modeling [42]. **Thus at the concrete scale**, a three-point bending beam with a
162 mesoscopic mesh generated in the central part of the beam **was** simulated so as to take into account
163 the heterogeneity of the material, i.e. **the** aggregate, cement mortar and EI. For the rest, a homoge-
164 neous mesh with **a gradually larger element toward the end of the beam was incorporated in order**
165 **to avoid stress concentration**. The macroscopic properties of those two parts **were considered to be**
166 **the same as for concrete**. A constant load of 40% of the maximal strength **was** applied to the rigid
167 plate (linearly elastic law) fixed at the top middle of the beam (Figure 4). For the other two scales,
168 the simulation of a representative elementary volume (REV) in direct tension **was** carried out; a
169 uniaxial loading **was** applied to the upper side of the REV while the vertical displacement of the
170 lower side of the REV **was** constrained. In order to ensure the stability of the volume and to comply
171 with minimum dimension conditions, the size of a REV must be at least equal to four times the
172 diameter of the largest inclusion and the finite element size should be less than or equal to 0.8 times
173 the diameter of the smallest inclusion [43]. Here, at the mortar scale the REV **was** $10 \times 10 \text{ mm}^2$
174 and at the cement paste scale the REV **was** $300 \times 300 \text{ }\mu\text{m}^2$. As concrete structures are brittle in
175 tension, the tensile cracking **was** predominant in **the** flexural test. At the lower scales, constant
176 tensile loads were applied on mortar and cement paste specimens and only the effective part of the
177 compliance in the loading axis was considered. The other deformations showed negligible values.
178 Thus, the homogenization calculations **were** performed only in 2D. In the case of massive concrete
179 structures, **3D calculations** are recommended, but **they were** not studied in this work.

180
181 **To simplify the modeling** at the cement paste scale, only one type of particle with a single size
182 **was** used to represent the inclusions. Its elastic properties **were** calculated by averaging the elastic

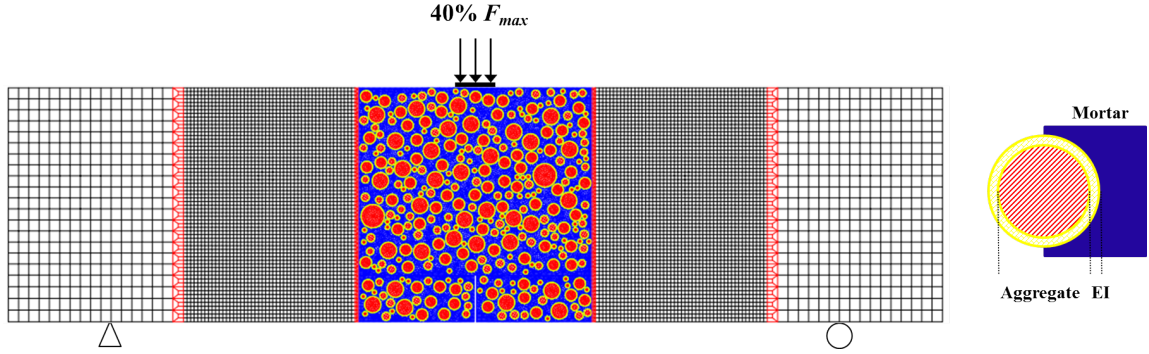


Figure 4: Meshing and boundary conditions of the concrete beam for a three-point bending creep test.

183 characteristics of the hydrates (CH, ettringite, gypsum, C_3AH_6 , FH_3) and the residual cement
 184 phases (C_3S , C_2S , C_3A , C_4AF), with the set forming a volume denoted by $V_{hy-clinker}$. For mortar
 185 scale, the experimental size distribution of sand grains was used. For concrete scale, the aggregate
 186 size distribution in the modelling was defined according to the experimental value for the three
 187 kinds of concrete (Table 1).

Table 1: Inclusion size distribution for each scale (ϕ - diameter and V_f - volume fraction).

Cement paste	ϕ (mm)	0.03							
	V_f (%)	30							
Mortar	ϕ (mm)	0.1875	0.375	0.75	1.5	2.4	3.4	4.8	
	V_f (%)	4	10	15	13	4	3	1	
Concrete (C-N-N)	ϕ (mm)	4	6.3	7.1	10	14	20		
	V_f (%)	3	5	4	16	13	2		
Concrete (C-N-100R)	ϕ (mm)	5	6.3	8	10	12.5	16	20	
	V_f (%)	2.8	4.3	2.1	9.6	10.5	8	1.8	
Concrete (C-N-30R)	ϕ (mm)	6.3	10	10	12.5	14	16	20	20
	V_f (%)	4	2	7	4	8	5	4	7

188 For each scale, the mesh generation of the mesoscopic part was conducted with the algorithm
 189 developed by Mounajed [43, 44]. It consisted of generating a volume by introducing a random dis-
 190 tribution of the various inclusions of different sizes in a solid matrix. Note that the third interphase

191 surrounding aggregate was introduced only for the concrete scale [42].

192 The distribution algorithm **generated** random positions of the inclusion gravity centres in the
 193 interested area of the volume. At this step, an inclusion i **was** formed by an aggregate with a
 194 surrounding interphase. The unit volume of this inclusion of type i **was** calculated as follows:

$$\begin{cases} V_{inclusion} = \pi R_i^2 & \text{in cement paste and mortar} \\ V_{inclusion} = \pi(R_i + \delta_{EI})^2 & \text{in concrete} \end{cases} \quad (2)$$

195 where R_i is the inclusion radius of type i and δ_{EI} the thickness of EI.

196 A test **was conducted** to check that two inclusions **did** not overlap. The distance between the
 197 gravity center of an existing inclusion with a radius of R_1 and that of a candidate inclusion with a
 198 radius of R_2 should be higher than the distance d_{min} defined as:

$$d_{min} = R_1 + R_2 \quad (3)$$

199 After the validation of the inclusions' position, the elements **were** assigned with material char-
 200 acteristics. **First**, all the elements **were defined as having the matrix properties**. Then, the model
 201 **assigned** different material properties to the elements by checking the distance d_{vef} between the
 202 gravity center of the element and that of the inclusion:

- 203 • If $d_{vef} \leq R_i$ the inclusion properties
- 204 • If $R_i < d_{vef} \leq R_i + \delta_{EI}$ the EI properties (only in concrete)
- 205 • If $d_{vef} > R_i (+\delta_{EI})$ the matrix properties

206 4. The problem formulation

207 4.1. Creep-damage coupling model

208 A volume V of concrete **was** formed by three media: a matrix defined by the medium V_m ,
 209 inclusions defined by the medium V_i and at the concrete scale EI around the inclusions defined by
 210 the medium V_{EI} . A constant force \bar{F} **was** applied on one of the surface boundary Γ_1 with the unit

211 normal vector \bar{n} . This load implied local displacement fields $\bar{u}(\bar{y})$, local strain fields $\bar{\varepsilon}(\bar{y})$ and local
 212 stress fields $\bar{\sigma}(\bar{y})$ in each point \bar{y} of V .

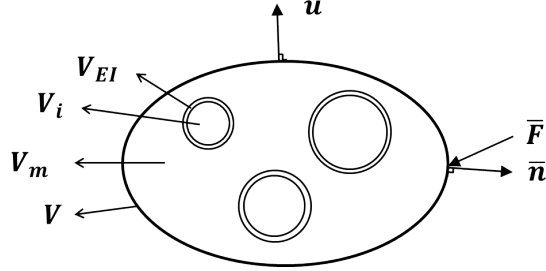


Figure 5: Illustration of the the mechanical problem.

213 The non-linear elastic problem was written as follows:

$$\overline{div} \bar{\sigma}(\bar{y}) = \bar{0} \quad \forall \bar{y} \in V \quad (4)$$

$$\bar{\sigma}(\bar{y}) = \bar{\bar{C}}(\bar{y}, \bar{\varepsilon}(\bar{y})) : (\bar{\varepsilon}(\bar{y}) - \bar{\varepsilon}^v(\bar{y})) \quad \forall \bar{y} \in V \quad (5)$$

$$\bar{\varepsilon}(\bar{y}) = \frac{1}{2} (\bar{\nabla} \bar{u}(\bar{y}) + {}^t \bar{\nabla} \bar{u}(\bar{y})) \quad \forall \bar{y} \in V \quad (6)$$

$$\bar{F} = \bar{\sigma} \cdot \bar{n} \quad \forall \bar{y} \in \Gamma_1 \quad (7)$$

$$\bar{u} = \bar{U} \quad \forall \bar{y} \in \Gamma_2 \quad (8)$$

214 where $\bar{\bar{C}}(\bar{y}, \bar{\varepsilon}(\bar{y}))$ represents the secant stiffness tensor depending on the local strains and damage,
 215 $\bar{\varepsilon}^v(\bar{y})$ the local viscoelastic strain fields and Γ_2 the boundary of V on which macroscopic displace-
 216 ments conditions were applied.

217 The isotropic damage model developed by Fichant et al. [45] was chosen for this study. For the
 218 isotropic version of the model, the relationship between effective stress and total stress is given by
 219 [46]:

$$\bar{\sigma}(\bar{y}) = (1 - d) \bar{\bar{\sigma}}(\bar{y}) \quad (9)$$

220 where d represents the scalar value of the isotropic damage that depends only on the equivalent
 221 strain calculated according to the positive part $((\cdot)_+)$ of the elastic strain tensor $\bar{\varepsilon}^e(\bar{y})$:

$$\bar{\bar{\varepsilon}}_{eq} = \sqrt{(\bar{\varepsilon}^e)_+ : (\bar{\varepsilon}^e)_+} \quad (10)$$

222 The evolution of the damage variable, due to external mechanical loads, **was** an exponential
223 form:

$$d = 1 - \frac{\varepsilon_{d0}}{\varepsilon_{eq}} \exp[B_t(\varepsilon_{d0} - \varepsilon_{eq})] \quad (11)$$

224 where B_t represents a damage parameter to control the slope of the strain softening constitutive
225 relation in function of the width h of the element and ε_{d0} the strain threshold.

226 The viscoelastic strain **was** defined as the sum of the elementary strains associated with each
227 chain of the generalized Kelvin-Voigt model:

$$\bar{\varepsilon}^v(t) = \sum_{i=1}^4 \bar{\varepsilon}_v^i(t) \quad (12)$$

228 For each chain i the viscoelastic strain $\bar{\varepsilon}_v^i(t)$ **was** the solution of a first order nonlinear differential
229 equation:

$$\eta^i \dot{\bar{\varepsilon}}_v^i(t) + k^i \bar{\varepsilon}_v^i(t) = \bar{\sigma}(t) \quad (13)$$

230 where k^i **was** the stiffness and η^i the viscosity of the Kelvin-Voigt unit.

231 The total viscoelastic strain **was** calculated by summing all the elementary viscoelastic strains,
232 and the latter **were** obtained by solving analytically the differential equations:

$$\Delta \bar{\varepsilon}_v^{n+1} = \bar{\varepsilon}_v^{n+1} - \bar{\varepsilon}_v^n = a_v \bar{\varepsilon}_v^n + b_v \bar{\sigma}_n + c_v \bar{\sigma}_{n+1} \quad (14)$$

233 where $\bar{\varepsilon}_v^n$ represents the viscoelastic strain vector at the time step n and a_v, b_v, c_v are the material
234 parameters [47].

235 In fact, the viscoelastic strain field **was** related to the stress field by the creep compliance $\bar{\bar{J}}(t)$
236 such that:

$$\bar{\varepsilon}^v(t, \bar{y}) = \bar{\bar{J}}(t) : \bar{\sigma}(t, \bar{y}) \quad (15)$$

237 For a 2D model, the compliance tensor $\bar{\bar{J}}(t)$ was defined by three components: $J_{1111}(t)$, $J_{2222}(t)$
 238 and $J_{1212}(t)$, corresponding respectively to the compliance in the loading axis, in the axis perpen-
 239 dicular to the loading and in the transverse axis. According to the generalized Kelvin model $J_{ijkl}(t)$
 240 is defined as follows:

$$J_{ijkl}(t) = \frac{1}{E(t)} + \sum_{p=1}^3 \frac{1}{k_{ijkl}^p(t)} \left(1 - e^{-\frac{t}{\tau_{ijkl}^p}}\right) \quad (16)$$

241 where $E(\text{MPa})$ is the elastic modulus, $k_{ijkl}^p(\text{MPa})$ is the stiffness of the Kelvin unit and $\tau_{ijkl}^p(\text{s})$ is
 242 the characteristic time.

243 In order to realize the creep compliance transformation between the different scales, a numerical
 244 homogenization method [34] of the creep compliance was applied.

245 4.2. Numerical homogenization method

246 According to Grondin [48] for a bi-phase REV the solving of an elastic homogenization problem
 247 is equivalent to the solving of problems (equations (4)-(8)) at each time step t^p of the time dis-
 248 cretization interval $t^0 = 0, t^1, \dots, t^p, \dots, t^n = t^{max}$. The tensor relating the strain field and the stress
 249 field is constant at the computation time t^p , such that:

$$\langle \bar{\varepsilon}^v(t^p, \bar{y}) \rangle_V = \bar{\bar{J}}^{hom}(t^p) : \langle \bar{\sigma}(t^p, \bar{y}) \rangle_V \quad (17)$$

250 The final creep compliance is the integral of $\bar{\bar{J}}^{hom}(t^p)$ with respect to time. The applied stress
 251 can be described by considering only the time-dependent stresses:

$$\langle \bar{\sigma}(t, \bar{y}) \rangle_V = H(t) \langle \bar{\sigma}^0 \rangle_V \quad (18)$$

252 where $H(t)$ is the Heaviside function:

$$\begin{cases} H(t) = 0 & \text{if } t = 0 \\ H(t) = 1 & \text{if } t > 0 \end{cases} \quad (19)$$

253 By summing the components of the constant elementary tensors with respect to time, the
254 formula (17) becomes:

$$\sum_{p=0}^{p=n} \left\langle \varepsilon_{ij}^v(t^p, \bar{y}) \right\rangle_v = \sum_{p=0}^{p=n} J_{ijkl}^{hom}(t^p) \left\langle \sigma_{kl}(t^p) \right\rangle_V \quad (20)$$

255 By using the expression (18), the preceding formula is written:

$$\sum_{p=0}^{p=n} \left\langle \varepsilon_{ij}^v(t^p, \bar{y}) \right\rangle_v = \sum_{p=0}^{p=n} J_{ijkl}^{hom}(t^p) H(t^p) \left\langle \sigma_{kl}^0 \right\rangle_V \quad (21)$$

256 Through replacing $H(t^p)$ by its expression and considering that the strain is zero at $t = 0$, one
257 obtains:

$$\left\langle \varepsilon_{ij}^v(t, \bar{y}) \right\rangle_v = J_{ijkl}^{hom}(t) \left\langle \sigma_{kl}(t) \right\rangle_V \quad (22)$$

258 It should be highlighted that at the sub-microscopic scale, the effective creep compliance **was**
259 obtained by applying the Ricaud and Masson formula [49] that relates the viscoelastic coefficients
260 of a porous composite (k_{f_p}) with the volume fractions of the pores (f_p):

$$k_{f_p} = k_{car} \frac{4}{3A(f_p)} \quad (23)$$

261 where $A(f_p) = \frac{f_p}{1 - f_p}$.

262 Here, we **considered** this formula for the matrix of cement paste when k_{f_p} represents the vis-
263 coelastic coefficient of C-S-H. Thus, at the cement paste scale, by knowing the creep compliance
264 of the matrix $J_{CSHP}(t)$, the effective creep compliance of cement paste J_p^{hom} **could** be calculated
265 by the homogenization method. Then we **went** up to the mortar scale, and finally to the concrete
266 scale.

267 **5. Determination of the viscoelastic parameters**

268 *5.1. Characteristic viscoelastic coefficients of C-S-H*

269 The calibration procedure **began** by the concrete scale in order to reproduce the delayed dis-
 270 placement curves of concrete C-N-N, and then **descended** successively to the C-S-H scale. **First**, at
 271 the concrete scale, the Young’s modulus E and the tensile strength f_t of both the mortar and ag-
 272 gregate **were** already determined in Guo et al. [3]. Then, at the mortar scale, the Young’s modulus
 273 of the cement paste **was** calculated according to a hydration model [50], and the tensile strength
 274 **was calibrated by simulating uniaxial tension tests of mortar**. The Young’s modulus and the tensile
 275 strength of sand grains **were** taken from the literature [34]. At the cement paste scale, the Young’s
 276 modulus of the matrix (C-S-H + capillary pores) **was** calculated according to **Mori-Tanaka’s ho-**
 277 **mogenization method**. The tensile strength of the matrix **was calibrated by simulating uniaxial**
 278 **tension tests of cement paste**. The Young’s modulus of inclusions **was** calculated by averaging the
 279 Young’s modulus of each component of the $V_{hy-clinkers}$ set. Their tensile strength **was** derived from
 the literature [34]. All the parameters determined here are summarized in Table 2.

Table 2: Young’s modulus and tensile strength of the constituents of each scale.

		f_t (MPa)	E (GPa)
Concrete (C-N-N)	Mortar	1.6	27.45
	Aggregate	6	78
Mortar	Cement paste	1.5	14.34
	Sand grain	8	80
Cement paste	Matrix of the paste	1.4	8.18
	Inclusion of the paste	3	51.6

280
 281 Considering the creep-loading period (**three months**), four chains of Kelvin-Voigt **were** taken
 282 into account, corresponding respectively to the characteristic time of 0.1, 1, 10 and 100 days.
 283 Then, at the concrete scale, the viscoelastic coefficients of mortar **were** identified by minimizing the
 284 difference between the calculated specific delayed displacement and the measured one (Figure 6).
 285 The identification of these parameters made it possible to calculate the creep compliance of mortar
 286 $J_m(t)$ by using the expression of $J_{ijkl}(t)$ (equation (16)).

287 At the mortar scale, the viscoelastic parameters of the cement paste were adjusted to obtain the
 288 compliance of mortar $J_m^{hom}(t)$, which was equivalent to $J_m(t)$ as determined in the previous step
 289 (Figure 6). The identified parameters were used to calculate the creep compliance of cement paste
 290 $J_p(t)$.

291 At the cement paste scale, the viscoelastic parameters of the paste's matrix $J_{CSHP}(t)$ were
 292 calibrated to get the compliance of the paste $J_p^{hom}(t)$, identical to $J_p(t)$ as defined in the last step
 293 (Figure 6). All the identified parameters are recapitulated in Table 3. Note that creep deformation
 294 for inclusions at each scale was considered as equal to zero, which corresponds to a very high
 295 coefficient value.

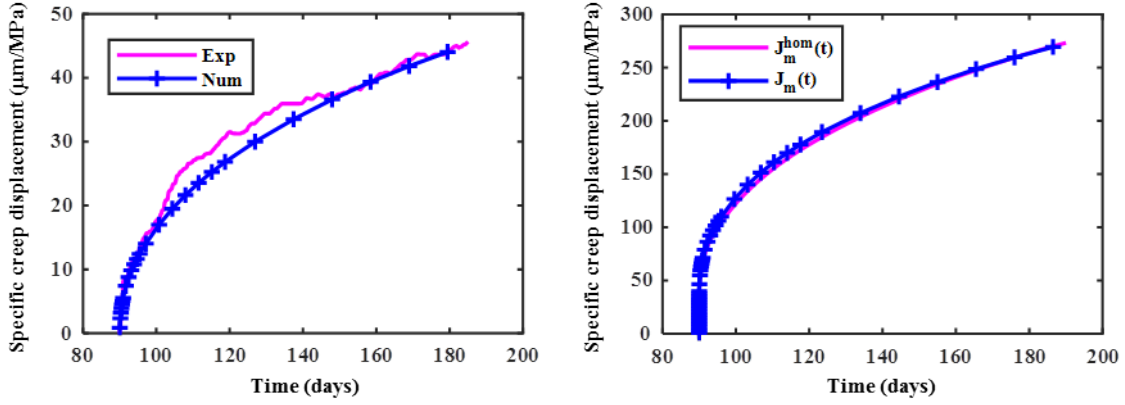
Table 3: Viscoelastic parameters of the constituents of each scale.

		k_v^1 (GPa)	k_v^2 (GPa)	k_v^3 (GPa)	k_v^4 (GPa)
Concrete (C-N-N)	Mortar	90	45	22	4
	Aggregate	1×10^{30}	1×10^{30}	1×10^{30}	1×10^{30}
Mortar	Cement paste	48	19	8	1.7
	Sand grain	1×10^{30}	1×10^{30}	1×10^{30}	1×10^{30}
Cement paste	Paste matrix	25	9	4	0.8
	Inclusion of paste	1×10^{30}	1×10^{30}	1×10^{30}	1×10^{30}

296 The viscoelastic parameters of the cement paste matrix identified above were dependent on the
 297 porosity of concrete. The coefficients $k_{f_p}^i$ are associated with the i chain of the Kelvin model.
 298 According to Ricaud and Masson [49], the viscoelastic coefficients of a porous medium are related
 299 to the characteristic coefficients which are independent of the porosity and represent only the solid
 300 phase (equation 23). For concrete C-N-N, the volume fraction of pores calculated by the hydration
 301 model is 34% of the cement paste volume; thus f_p is equal to 46.9% at the C-S-H scale. The
 302 values of the Kelvin characteristic coefficients of C-S-H (the cement paste matrix) were calculated
 303 according to the Ricaud and Masson formula and the corresponding results are presented in Table
 304 4.

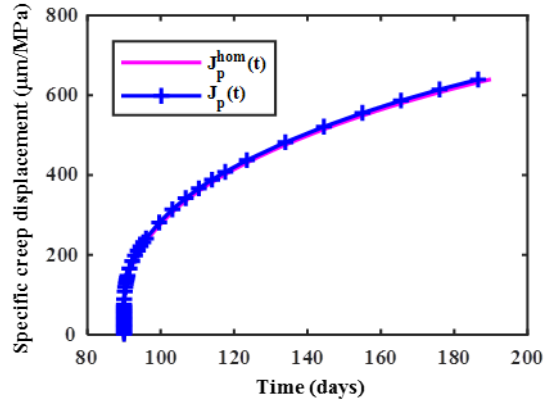
305 5.2. Calculation of the viscoelastic parameters of EI

306 In the case of RAC, the EI around recycled aggregate is composed of not only the new interfacial
 307 transition zone (ITZ) but also the old ITZ and the old residual mortar. However, it is usually



(a)

(b)



(c)

Figure 6: (a) Comparison between the experimental delayed specific displacement of concrete C-N-N and the identified numerical one, (b) comparison between the compliance $J_m(t)$ and the identified compliance $J_m^{hom}(t)$ at the mortar scale, (c) comparison between the compliance $J_p(t)$ and the identified compliance $J_p^{hom}(t)$ at the scale of cement paste.

Table 4: Characteristic viscoelastic coefficients of Kelvin for the matrix of cement paste.

k_{car}^1 (GPa)	k_{car}^2 (GPa)	k_{car}^3 (GPa)	k_{car}^4 (GPa)
16.56	5.96	2.65	0.53

308 observed that with the aging of concrete, the ITZ tends to have the same properties as the cement
309 mortar near this zone; the differences in mechanical strength also reduce gradually with time. The
310 microcracking origins identified from the multiparametric clustering of acoustic emission signals
311 recorded during flexural tests were aggregate fracture, cracking in the old residual mortar and
312 cracking in the mortar matrix [3], which confirms the observation mentioned above. Thus in our
313 modeling, we chose to not distinguish the old ITZ within the old attached mortar by assigning them
314 the same properties. Furthermore, in order to limit the uncertainties about the local properties, we
315 did not adjust for the new aggregate-matrix ITZ. Consequently, in this work the EI surrounding
316 recycled aggregate is considered to be composed of a layer of old mortar and a layer of new mortar.
317 The thickness of the EI is chosen to be 100 μm so as to complete the simulation within an acceptable
318 time [42].

319 As demonstrated in the previous work of the authors [36], the properties of old attached mortar
320 have a non-negligible effect on the macroscopic mechanical behavior of recycled concrete and need
321 to be properly defined. Since it is very difficult to define the real content of old mortar in EI, the
322 authors considered five composition possibilities of EI [51, 52], i.e., 10 %, 20 %, 30 %, 40 % and 50 %
323 old mortar with the corresponding 90 %, 80 %, 70 %, 60 % and 50 % new mortar, respectively, to
324 characterize the influence of old mortar content on the global mechanical behavior of concrete. The
325 modeling results of fracture tests showed that the most reasonable component of EI for our case of
326 study was 20 % old mortar and 80 % new mortar [36]. Certainly the old adhered mortar content
327 varies with the waste material origin. However, the same methodology can be used with minor
328 adjustments to determine the EI fractional component for other types of RAC, by calibrating with
329 certain experimental tests.

330 In order to represent the properties of unknown old attached mortar that is composed of un-
331 known cement paste as matrix and sand as inclusion, four types of ordinary cement paste ($w/c=0.4$,
332 0.5, 0.6 and 0.65) were studied. First, at the cement paste scale, the viscoelastic parameters of the
333 four types of cement paste were determined by calculating the volume fraction of the pores f_p in the
334 cement paste's matrix according to the hydration model. Simultaneously, for each type of paste, the
335 Young's modulus of its matrix (C-S-H + capillary pores) was calculated based on Mori-Tanaka's

336 **homogenization method**. The Young's modulus of the inclusions **was** obtained by averaging that of
 337 each component of the set $V_{hy-clinkers}$. All the results are summarized in Table 5.

Table 5: Elastic parameters for the four types of cement paste.

w/c		0.4	0.5	0.6	0.65
f_p		0.14	0.313	0.426	0.469
Matrix of cement paste	E (GPa)	16.45	11.61	9.05	8.18
	ν	0.26	0.28	0.29	0.3
Inclusion of cement paste	E (GPa)	53.09	52.9	52.19	51.6
	ν	0.31	0.31	0.31	0.31

338 **Note that the parameters vary with the w/c ratio**, except the Poisson's ratio of the inclusion.
 339 **For the purpose of representing the range of the elastic parameters, the cement pastes with w/c**
 340 **equal to 0.4 (minimum) and 0.65 (maximum) were** taken into account in the following simulation.
 341 Moreover, the viscoelastic parameters of the maximum group **had already been determined, as**
 342 **discussed** in the previous section. Thus, only the properties of the minimum group **remained** to be
 343 determined.

344 As mentioned above, the viscoelastic parameters of the cement paste's matrix **were** calculated
 345 according to the equation (23). Then the compliance of **the cement paste J_p^{hom} was** determined
 346 by the numerical homogenization method (Figure 7) as a constant load of 40% of the maximal
 347 strength **was** applied at the age of three months. The instantaneous part **was** removed when the
 348 desired force **was** reached. By minimizing the difference between the homogenized compliance and
 349 the calculated one (formula (16)), the viscoelastic parameters of Kelvin model k_v^i for cement paste
 350 at different ages **were** identified according to the least square method.

351 The identification of k_v^i for cement pastes **made** it possible to calculate the homogenized creep
 352 compliance J_m^{hom} at the mortar scale (Figure 7). Then the viscoelastic parameters k_v^i of the mortar
 353 **were** determined. Table 6 shows the calculated and retained values for scales from C-S-H to mortar.
 354 The inclusions' effect on the viscosity of the material **is noted here** since, for example, the compliance
 355 of mortar is inferior to that of the cement paste, due to the presence of sand grains.

356 As the viscoelastic parameters of new mortar **were** determined in the previous section, the

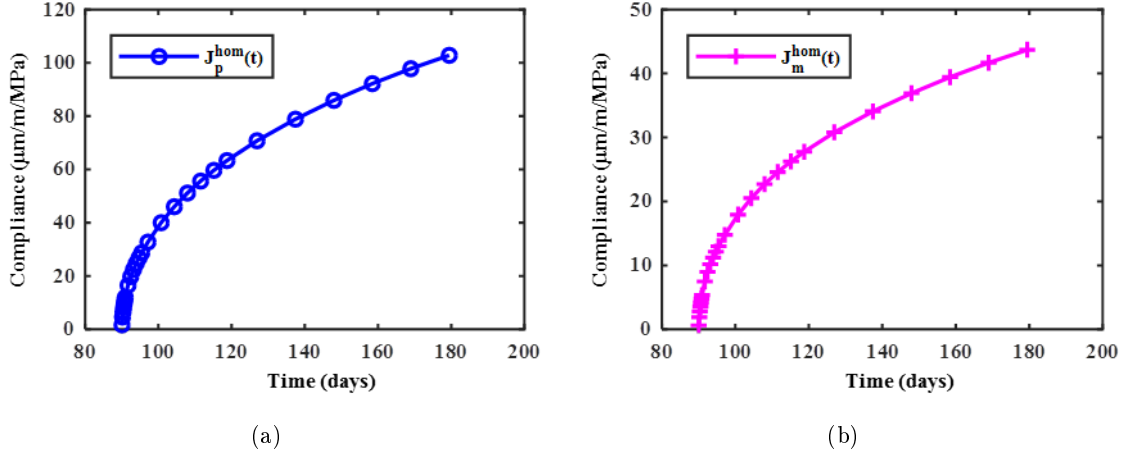


Figure 7: (a) Creep compliance of cement paste, (b) creep compliance of mortar.

Table 6: Viscoelastic parameters of the matrices for the three scales.

	k_v^1 (GPa)	k_v^2 (GPa)	k_v^3 (GPa)	k_v^4 (GPa)
Matrix of cement paste	135.64	48.83	21.70	4.34
Cement paste	369.98	86.84	39.62	9.31
Mortar	892.43	203.62	97.88	21.61

viscoelasticity of the EI (both the minimum and the maximum groups) was obtained by averaging the viscoelastic parameters of its components ($k_v^{au} = 0.2k_v^{old} + 0.8k_v^{new}$). The retained values are listed in Table 7. **The viscoelasticity of the homogeneous parts of the beam was considered identical to that of new mortar**, since the stiffness and the viscoelasticity of the extreme zones do not have a great influence on the overall creep displacement. Thus the parameters in these zones can be **arbitrarily defined as** close to those of a viscoelastic material, which is very similar to concrete. Here we **took** directly the viscoelastic parameters of the mortar; **it can be verified** that the results do not change with higher stiffness.

Table 7: Viscoelastic parameters of all the considered phases in beam C-N-100R.

	k_v^1 (GPa)	k_v^2 (GPa)	k_v^3 (GPa)	k_v^4 (GPa)
Mortar	90	45	22	4
Recycled aggregate	1×10^{30}	1×10^{30}	1×10^{30}	1×10^{30}
Interphase (min)	250.49	76.72	37.18	7.52
Interphase (max)	90	45	22	4
Homogeneous concrete	90	45	22	4

Based on all the available parameters, the numerical simulation **was** carried out. The obtained specific delayed displacements are shown in Figure 8 in comparison with the experimental result. **Note** that the numerical values are **less than** those of the experiment. In order to understand this difference, a review of the whole modeling process **was** conducted. Until now, the influence of 100% RA on the creep displacement of concrete has been taken into account by the identification of viscoelastic parameters of the interphase between RA and the new forming mortar. These parameters were calculated from the characteristic coefficients of C-S-H by passing through the scales of cement paste and mortar. The only difference of the assumed old cement paste and the new one is the volume fraction of the pores f_p in the cement paste’s matrix. In this work, f_p of the old cement paste **was** less than that of the new cement paste, which is consistent with the findings of Li et al. [53] that the rehydration of old cement paste reduces its porosity. Actually, the difference of f_p is not sufficient to explain the increment of **the RAC’s creep deformation**. Furthermore, several studies [54, 55] have shown that the **RAs themselves have a higher porosity as well as** a higher water

378 absorption ratio, and are less resistant to shock and friction [29]. The aggregates' porosity affects
 379 the elastic modulus and thus indirectly affects concrete creep. As the aggregates' elastic moduli
 380 decrease, there is greater stress on the cement paste and thus greater creep [28]. So the viscoelastic
 381 strain of RAs should be taken into account in the simulation. After being verified several times,
 382 $k_v^{ra} = 23$ GPa was assigned to represent the viscoelasticity of RA.

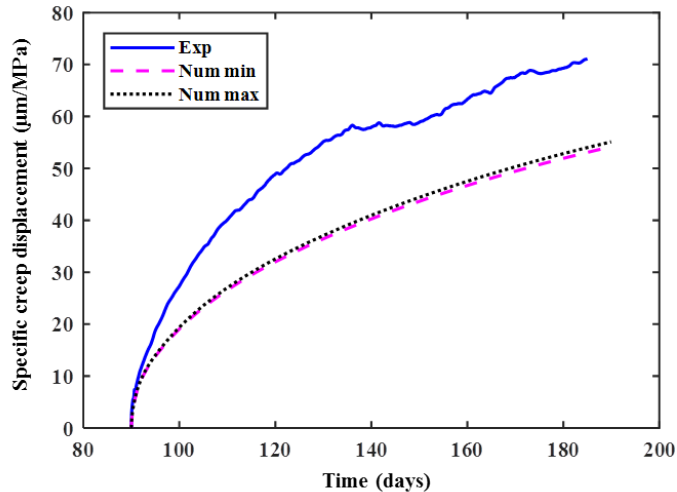


Figure 8: Comparison of the numerical specific creep displacement with the experimental one for beam C-N-100R.

383 Remind that the viscoelastic parameters in the maximum group of the interphase were chosen
 384 for the following simulation, which implies that the maximum creep displacement was considered in
 385 this study. The new numerical curve calculated by considering k_v^{ra} is presented in Figure 9. Further
 386 verification by other types of concrete with different RA replacement ratios is needed in order to
 387 verify the introduction of the viscoelasticity of RA.

388 6. Verification of the proposed model

389 All the identified viscoelastic parameters and the corresponding methods used in the previ-
 390 ous section are summarized in Table 8. The verification of these parameters, without any other
 391 adjustment, was carried out by performing the simulation of the creep test of C-N-30R.

392 The obtained results are presented in Figure 10. We notice that the numerical curve is very close
 393 to the experimental one, which implies that the inverse analysis of the viscoelastic parameters is

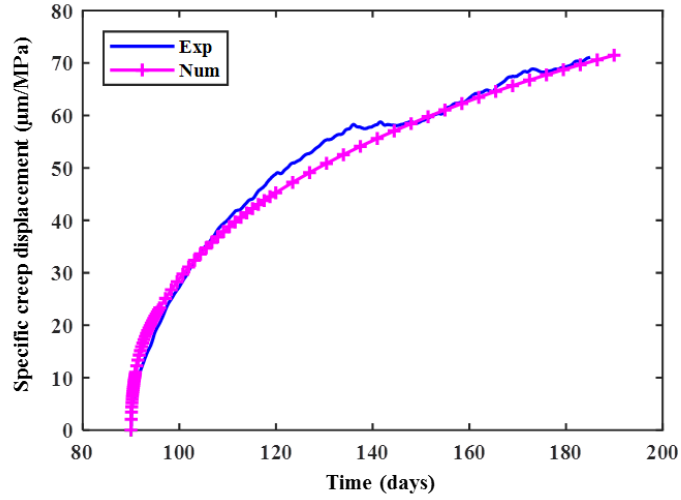


Figure 9: Specific creep displacement of beam C-N-100R by considering the creep of recycled aggregate.

Table 8: Viscoelastic parameters determined for the three scales.

Concrete type	Scale	Constituent	Identification method
C-N-N	Concrete	Mortar	Calibrated
		Aggregate	Imposed
	Mortar	Cement paste	Calibrated
		Sand grain	Imposed
Cement paste	Matrix	Calibrated	
	Inclusion	Imposed	
C-N-100R	Concrete	Mortar	Calibrated
		Aggregate	Imposed
	Mortar	Interphase	Multi-scale model
		Cement paste	Multi-scale model
	Cement paste	Sand grain	Imposed
		Matrix	Multi-scale model
Inclusion	Imposed		

394 reliable and the determined parameters can be used to estimate the actual creep evolution of RAC.
 395 In particular, the viscoelasticity of RA was an influential factor in reproducing the experimental
 396 creep behavior of RAC. Thus, the validated model can be exploited to identify the cracking risk
 397 during creep.

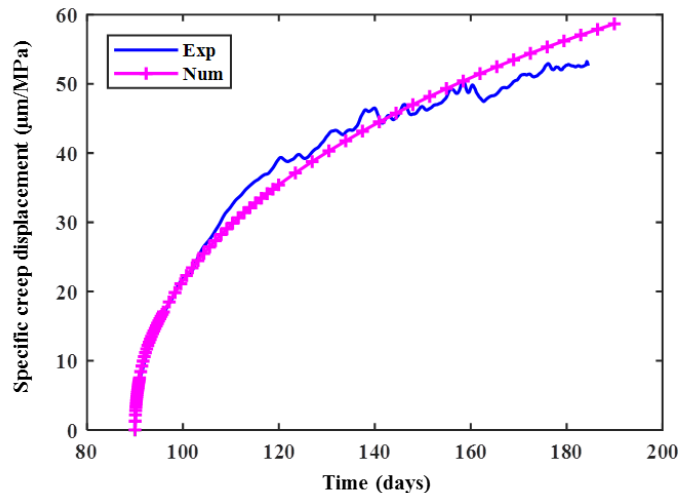


Figure 10: Comparison of the numerical specific creep displacement curve with that of the experiment for C-N-30R.

398 7. Damage evolution in concrete microstructure

399 After three months of loading at a stress level of 40% of the maximal strength, the damage
 400 localization field of beam C-N-100R was calculated (Figure 11). For comparison, the damage
 401 localization map at the 40% pre-peak loading step during the fracture test is also presented in
 402 Figure 11. Note that the damage localized in front of the notch, and developed mainly in the
 403 mortar matrix as well as in the interphase. As the RAs are less resistant, cracks were found to
 404 propagate through the aggregate. This phenomenon corresponds to our hypothesis that the effect
 405 of RA on the creep evolution of RAC is not negligible. In fact, after three months of constant
 406 loading, the failure behavior of C-N-100R was found to be more fragile during the post-peak region,
 407 though the flexural strength varied very little after creep [36] (Figure 12).

408 Similarly, the damage localization field of beam C-N-30R after three months of loading was also
 409 calculated and compared with the damage state before the constant loading of creep (Figure 11).

410 We observed that the cracks were located above the notch and propagated toward the top of the
 411 beam by passing around the aggregate that hindered them. Damage was principally found in the
 412 matrix and in the interphase around the RAs. For the fracture behavior of concrete with 30%
 413 recycled aggregate after three months of loading, similar trends were observed by comparison with
 414 that before creep [36] (Figure 12).

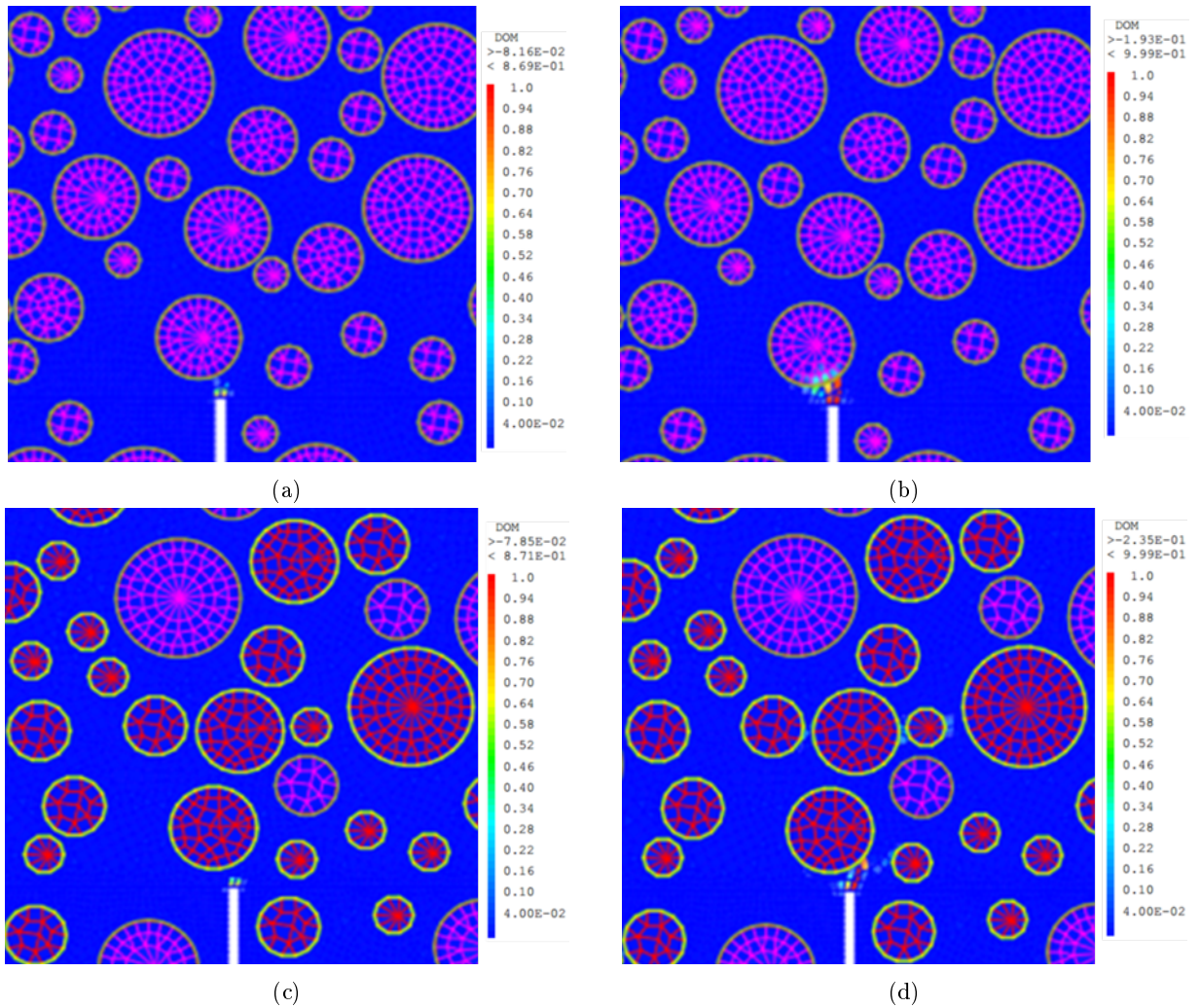


Figure 11: Damage localization fields of beams (a) C-N-100R at 40% pre-peak loading step during fracture, (b) C-N-100R after three months of loading at a stress level of 40% F_{max} , (c) C-N-30R at 40% pre-peak loading step during fracture, (d) C-N-30R after three months of loading at a stress level of 40% F_{max} .

415 Based on the established model, we also calculated the creep displacement for other types

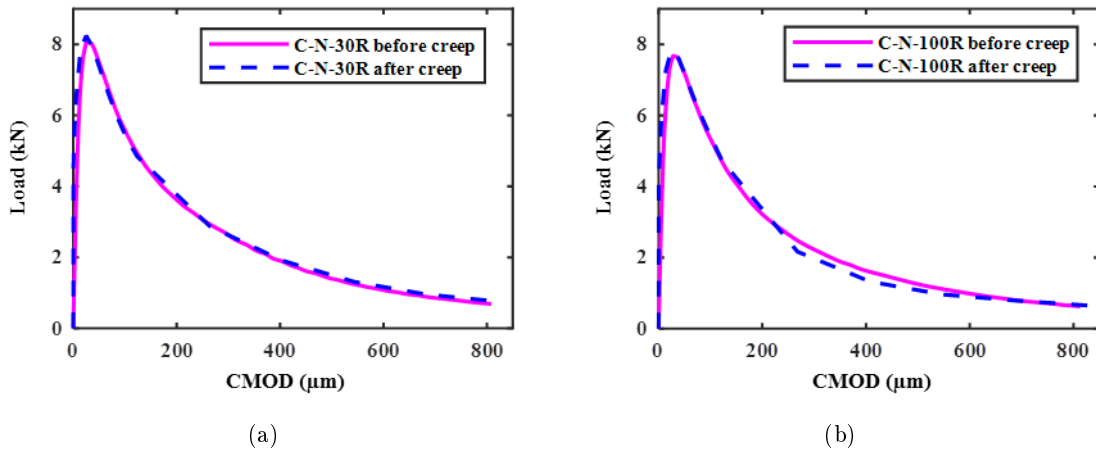


Figure 12: Comparison of the Load-CMOD curves before and after three months of constant loading for (a) C-N-30R, (b) C-N-100R.

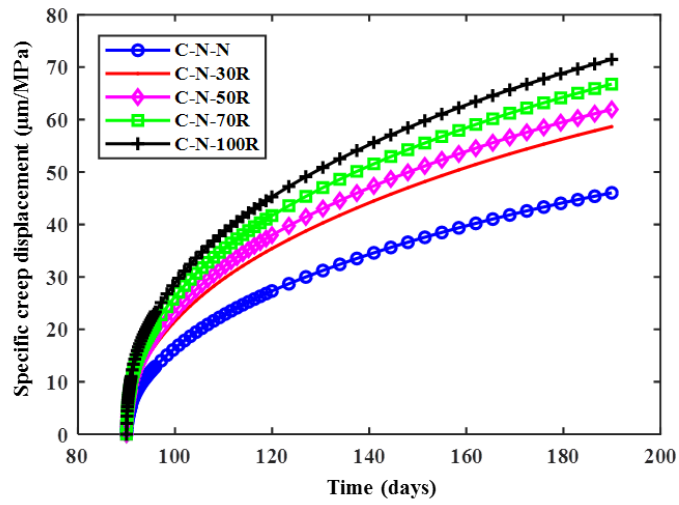


Figure 13: Identified numerical creep displacement curves for different types of recycled aggregate concrete.

416 of RAC, for example C-N-50R and C-N-70R, by applying the virtual aggregate size distribution
417 respectively (Figure 13). We noted that the creep displacement increased with the substitution
418 ratio of natural aggregate by RAs, as demonstrated in the literature [12, 14]. We concluded that
419 the proposed model has a certain capacity to predict the creep behavior of RAC on the basis of
420 the identified viscoelastic parameters for new mortar, old attached mortar and RAs. In addition,
421 by using the creep-damage coupling model, the microcracks generated during the constant loading
422 period that were almost impossible to measure by experimental methods could be detected. The
423 influence of the constituent phases on the delayed behavior of concrete could be explicitly illustrated.

424 8. General remarks

425 Normally, it takes at least six months to complete the creep test, whereas with the proposed
426 creep model we were able to obtain RAC creep evolution information after only a few hours of
427 computation. The efficiency of numerical modeling is evident in the study of long-term RAC
428 mechanical behavior. Furthermore, we were able to calculate the viscoelastic properties of any type
429 of RAC at different ages by applying the cement hydration model [50]. Creep tests of any type of
430 RAC specimens subjected to different stress levels can also be modeled for a desired loading period.

431 Generally, the creep behavior of concrete is considered to be linear under a stress level of 40 % of
432 the maximal strength. However, that is not necessarily the case for C-N-100R. As shown in Figure
433 11, the damage developed in front of the notch during creep. Further research work is needed in
434 order to study and evaluate the creep-damage coupling of RAC in the long term.

435 Due to the fact that the creep behavior of RAC is highly influenced by origin of RAs, mixture
436 design of new concrete, specimen size, loading mode, temperature and relative humidity during the
437 test, certain adjustment is surely needed and could be realized for the actual application of this
438 numerical method in other creep-related studies. The methodology proposed here could serve as a
439 useful reference for the prediction of RAC creep.

440 9. Conclusions

441 The creep behavior of three types of recycled aggregate concrete was studied on three-point
442 bending beams subjected to a stress level of 40 % of the maximal strength over a period of three

443 months. The delayed displacement of concrete **increased** with the replacement ratio of natural
444 aggregate by recycled aggregate. A micro-macro modeling of the creep test **was** carried out to
445 investigate the roles of the different phases on the creep behavior of recycled aggregate concrete.

446 **The following** conclusions **were** drawn:

447 • The creep kinetics of 30 % recycled aggregate concrete **were** comparable to those of normal
448 concrete with an amplitude 1.2 times greater after three months of loading. **For** concrete with
449 100 % recycled aggregate, **the creep developed faster and the amplitude was** 1.6 times larger
450 than that of conventional concrete.

451 • **The simulation results showed that the time-dependent behavior of recycled aggregates them-**
452 **selves was not negligible and should be taken into account in numerical modeling. The greater**
453 **contribution of recycled aggregate on the development of the delayed displacement of concrete**
454 **needs to be considered in the future application of recycled concrete structures.**

455 • According to the damage localization maps of recycled concrete, the microcracks **developed**
456 not only in the mortar matrix and **the interphase** but also in **the** recycled aggregate, due to
457 **its** higher porosity and lower particle strength. This implies that the commonly considered
458 linear creep of concrete under a stress level of 40 % of the maximal strength is not absolutely
459 the case for recycled aggregate concrete. Further studies are **essential** in order to facilitate
460 the wider application of recycled aggregate in structural concrete.

461 In addition, a similar work to model the creep behavior of recycled aggregate by **using** the
462 downscaling analysis is currently under way. The difference lies at the cement paste scale where,
463 instead of homogenizing the heterogeneous matrix and inclusions of the cement paste, the real
464 microstructure, composed of all the existing clinker minerals phases and the hydration products,
465 is considered. The influence of the microscopic components on the creep behavior of recycled
466 aggregate concrete will be presented in the next paper.

467 Acknowledgments

468 Support from the French National Research Agency (ANR: Agence Nationale de la Recherche)
469 is acknowledged (ECOREB project). The first author would like to thank the China Scholarship
470 Council for the financial support (PhD grant).

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