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Towards a codified design of recycled aggregate concrete structures: background for the new *fib* Model Code 2020 and Eurocode 2

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Towards a codified design of recycled aggregate concrete structures: background for the new *fib* Model Code 2020 and Eurocode 2

Running Head: Recycled aggregate concrete – background for MC2020 and EC2

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ABSTRACT

The use of recycled aggregate (RA) to produce recycled aggregate concrete (RAC) is a proven way of decreasing the consumption of natural aggregate (NA) and the landfilling of construction and demolition waste. However, adoption of codes for the design of RAC and RAC structures has been lacking. Within the framework of the new *fib* Model Code 2020 and Eurocode 2, provisions for RAC can be adopted. Therefore, in this study, a comprehensive and critical review of literature on RAC is performed as well as own meta-analyses of results. Material properties of RAC and structural behaviour of reinforced and prestressed RAC members are analysed, and based on the findings, code adjustments for RAC are proposed. The results show that, in order to incorporate RAC into design codes, changes are necessary in expressions for physical–mechanical properties (volumetric mass, modulus of elasticity, tensile strength, fracture energy, peak and ultimate strains, shrinkage strain and creep coefficient), durability-related properties (minimum concrete cover for durability) and structural behaviour (shear strength of members not requiring shear reinforcement and deflections). The recommendations are formulated in terms of the total mass substitution ratio of RA that can be classified as Type A according to standard EN 206 for concrete. The results and findings presented herein can provide an important contribution towards the codification of RAC use and the wider utilization of RA in construction.

Keywords:

Recycled concrete, recycled aggregate, mechanical property, design, concrete structure, Model Code

1. Introduction

Currently, the construction industry has one of the largest environmental impacts: 40% of raw stone, gravel and sand consumption; 25% of virgin wood; 40 % of total energy and 16 % of annual water consumption [1]. Within the construction industry, concrete is the most widely produced and used material, with a global annual production of over 25 billion tons [2]. On the one hand, such a figure means an immense strain on natural resources, for example, the production of natural aggregate (NA) surpasses 40 billion tons per year [3]. On the other hand, the amount of concrete structures being built leads to large quantities of construction and demolition waste (CDW) being generated every year: in the EU, up to 850 million tons [4], in the US close to 500 million tons [5] and in China approximately 1.5 billion tons [6]. Importantly, concrete waste typically constitutes around 50% of CDW [7].

An optimal solution for managing CDW remains its recycling and the production of recycled aggregates (RA), i.e. "aggregate resulting from the processing of inorganic material previously used in construction" [8]. However, even in countries with high concrete waste recycling rates, the market uptake of RA remains low and its use is still mostly relegated to non-structural applications such as backfilling, road base or sub-base [3]. This is a sub-optimal solution since it typically leads to "down-cycling", e.g. recycling of structural concrete and the use of produced RA in lower-value, non-structural applications. This is especially important considering that close to 50% of concrete produced in the EU is reinforced, structural concrete [9]. Therefore, the objective of the construction industry and concrete community in particular, must be the wider application of RA in producing structural concrete, i.e. recycled aggregate concrete (RAC). This has led to wide initiatives in the research community, such as the RECYBETON national project in France [10,11], aimed at systematically investigating the use and properties of RAC.

One important reason for the lack of RAC utilization is the absence of regulation, and in particular design guidelines. So far, the European Committee for Standardization (CEN) has dealt with RA through standards on aggregates for concrete EN 12620 [12] and standards on concrete EN 206 [8]. Nonetheless, the effect of RA incorporation into RAC in terms of mechanical and structural behaviour has not been comprehensively taken into account. However, a significant body of literature now exists on RAC, with wide ranges of experiments on physical-mechanical, long-term, and durability-related properties, as well as structural behaviour of full-scale reinforced and prestressed RAC elements and structures. This knowledge is now being transferred into design guidelines and codes. The International Federation for Structural Concrete (*fib*) and CEN have undertaken initiatives to incorporate recommendations for structural design of RAC into their respective codes, i.e., the new Model Code 2020 (MC2020) [13] and the new revision of Eurocode 2 (new EC2) [14].

In order to facilitate the understanding and use of these new codes, this paper presents and explains the theoretical and experimental background behind a proposal of RAC code provisions that can be incorporated into MC2020 and the new EC2. The aim of both codes is to ensure a target reliability of structures designed according to them, using the partial safety factor format [15,16]. For concrete, this is achieved through the partial safety factor for concrete, γ_C, which is the product of the material partial safety factor γ_c by the model partial safety factor γ_{Rd}. Therefore, the analysis of RAC has to be done on the material and structural levels. Regarding material properties, it is necessary to assess the variability of RAC properties relative to natural aggregate concrete (NAC) and the applicability of code expressions to RAC. For structural behaviour it is necessary to assess the adequacy of different resistance models (e.g. flexure, shear).

In this study, the primary course of investigation was to conduct own meta-analyses of gathered experimental results wherever possible. In cases where this was not possible, the secondary sources of information were studies that performed probabilistic evaluations to assess material and model partial safety factors. Finally, when such studies were also lacking, general conclusions were drawn from existing individual studies.

2. Recycled aggregates for use in concrete

Generally, recycling facilities will receive and process CDW containing different materials, i.e., the composition of RA will not be uniform. Other types of waste in CDW can include asphalt, soil, bricks, glass, rocks, etc. Preferably, only RA produced from concrete waste, i.e. recycled concrete aggregate (RCA) would be used in concrete for structural applications. However, this is not realistic from an industry perspective. Therefore, other materials must be accepted in RA as well.

The CEN standard for aggregates for concrete [12] classifies RA components into unbound stone (Ru), crushed concrete (Rc), crushed brick (Rb), bituminous materials (Ra), floating materials (FL), glass (Rg) and other (X) such as gypsum, metal and wood. The standard then classifies RAs according to its composition considering its components, Table 1. For example, Rc_{90} is an RA containing more than 90% of crushed concrete (Rc). This provides a framework for classifying RAs appropriate for use in structural concrete, e.g. by prescribing which categories from Table 1 can be used in which amounts. For example, the standard EN 206 for concrete [8] classifies RA into two types:

- Type A (Rc_{90}, R_{cu95}, R_{b10-}, R_{a1-}, FL_{2-}, XR_{g1-}) and
- Type B (Rc_{50}, R_{cu70}, R_{b30-}, R_{a5-}, FL_{2-}, XR_{g2-})

Considering the above stated, within the scope of this paper, RA will refer to aggregates containing more than 90% of crushed concrete or more than 95% of crushed concrete and unbound stone, i.e. Rc_{90} and Rcu_{95}, i.e. Type A aggregate per EN 206 [8]. However, this definition is extended to both coarse RA (≥ 4 mm) and fine RA (< 4 mm) and appropriate distinctions are made where necessary.

3. Recycled aggregate concrete

3.1. Comparison with natural aggregate concrete and RA substitution ratio

The majority of research on RAC has been based on comparing the performance of RAC to that of a "reference" NAC. In most cases, a reference NAC is a concrete that has the same $(w/c)_{\mathrm{eff}}$ ratio as RAC (whether RA is pre-soaked or additional water is used). This generally leads to a reduced strength of RAC relative to NAC, and probably lower workability (unless controlled for by admixtures). Another approach can be a "performance-based" comparison in which RAC and NAC are produced to have the same compressive strength f_c (and preferably similar workability), but their $(w/c)_{\mathrm{eff}}$ ratio are different. In research, the majority of studies have adopted the approach of using the same $(w/c)_{\mathrm{eff}}$ ratio for RAC and reference NAC (but not always with a simultaneous control of workability). The question of RA water absorption has also been important in terms of mixing procedure. The main approaches so far have been the use of "pre-saturated" RA (immersed in water for a certain period or until achieving saturated surface-dry conditions) or the use of RA in their natural moisture state but with a compensation of RA water absorption through additional mixing water. Generally, using pre-saturated RA has been found to lead to a weaker interface transition zone and reduced RAC strength [17,18]. Therefore, using RA in the natural moisture state and compensating for water absorption is preferable, with this option also having the benefit of being easier to implement in practice.

Furthermore, RACs have been produced by substituting different amounts of NA with RA. These replacements can be partial (<100%) or total (=100%) and they can be by volume or by mass. In addition, replacement can be done only on fine NA, only on coarse NA or on both. Within this paper two symbols will be used for the RA content in RAC: (1) α_{RA} is the mass substitution ratio of total RA (fine + coarse) relative to the total mass of aggregates (fine + coarse NA and RA) and (2) α_{CRA} is the mass substitution ratio of coarse RA relative to the total mass of coarse aggregates (NA and RA). For example, assuming 800 and 1200 kg/m^3 of fine and coarse aggregate, if 600 kg/m^3 of coarse RA are used, then $\alpha_{CRA} = 600 / 1200 = 0.5$ and $\alpha_{RA} = 600 / (800 + 1200) = 0.3$. Therefore, if only coarse RA is used, it can be assumed that $\alpha_{RA} \approx 0.6 \cdot \alpha_{CRA}$. In this study, the choice was made for using a mass substitution ratio as it is easier to implement in practice. Nonetheless, if mix design is done starting from an NAC mix in which NA is replaced by RA by mass, a verification of the volumetric equation should be made, especially to ensure the cement content is in accordance with EN 206 [8].

In practice, which fractions will be replaced and up to which amounts, will be regulated, e.g. in Europe by the standard for concrete EN 206 [8], placing limits on percentages of fine and coarse RA that can be used (however, without considering effects on concrete mechanical properties). In the case of EN 206, maximum RA substitution ratios are (and will be in new revisions) provided in terms of separate substitution ratios of coarse and fine RA, and furthermore, separately according to environmental exposure class. For example, RECYBETON recommendations [11] stipulate that for exposure classes XC1 and XC2 60% and 20% of coarse and fine RA can be substituted, respectively, as long as there is a reduction of the maximal water-to-binder ratio of 0.05. Considering the earlier assumed amounts of 1200 and 800 kg/m^3, respectively, this results in $\alpha_{RA} = (0.6 \cdot 1200 + 0.2 \cdot 800)/2000 = 0.44$. The allowable amounts only decrease: for example, for exposure class XF2, they are 40% and 15% for coarse and fine RA, respectively, resulting in $\alpha_{RA} = 0.3$. It should also be noted that it is not recommendable to substitute only fine RA (e.g. 20% for XC2), but either only coarse or both coarse and fine at the same time, with the fine content equal to or lower than the ratio of replacement rates in EN 206 (e.g. for XC2 this ratio is 3:1 for coarse-to-fine RA substitution).

Considering the above stated, in this study, the influence of RA on RAC properties is expressed through the α_{RA} coefficient. However, the expressions proposed in this study are valid only for cases when

- only coarse RA is used and

- both coarse and fine RA are used, respecting the substitution ratios of EN 206 [8],
- both coarse and fine RA belong to RA Type A according to EN 206 [8].

The cases when only fine RA is used are not considered and are not covered by the expressions proposed in this study. This is justified by both inferior properties of fine RA relative to coarse RA (significantly larger percentage of residual mortar attached to RA particles) and the smaller amount of literature investigating the effects of fine RA on RAC.

3.2. *Volumetric mass density (specific mass)*

The specific mass of NAC typically varies from 2.25 to 2.45 t/m^3, depending on mix proportions and type of aggregates. If the specific cases of lightweight or heavyweight aggregates are not considered, the density of aggregates ρ_{ag} is between 2.55 and 2.80 t/m^3.

The standard EN 1991-1-1 [19], related to actions on structures, indicates that the specific weight of concrete should be taken equal to 24 kN/m^3 and that 1 kN/m^3 should be added when considering reinforced concrete. If we assume a mix design for 1 m^3 of concrete corresponding to this value (Table 2), then, once this concrete is crushed, the specific mass of RA will be $\rho_{\mathrm{c}} = 2.4$ t/m^3.

If a new concrete is produced with this RA, and assuming that the other components in the mix design are unchanged (or at least that the changes are limited), if we note $\alpha_{\mathrm{V,RA}}$ the volumetric substitution ratio, the variation of the specific mass ρ_{RAC} of the concrete made with the RA will be

$$\Delta\rho_{\mathrm{RAC}} = \rho_{\mathrm{ag}} \cdot V_{\mathrm{ag}} - [\rho_{\mathrm{ag}} \cdot V_{\mathrm{ag}} \cdot (1 - \alpha_{\mathrm{V,RA}}) + \rho_{\mathrm{c}} \cdot V_{\mathrm{ag}} \cdot \alpha_{\mathrm{V,RA}}] = (\rho_{\mathrm{c}} - \rho_{\mathrm{ag}}) \cdot V_{\mathrm{ag}} \cdot \alpha_{\mathrm{V,RA}} \quad (1)$$

Considering values from Table 2, the following is obtained:

$$\Delta\rho_{\mathrm{RAC}} = (2.40 - 2.73) \cdot 0.684\, \alpha_{V,RA} \approx -0.22\, \alpha_{V,RA} \quad (2)$$

The variation is expressed here as a function of the volumetric substitution ratio $\alpha_{\mathrm{V,RA}}$. The mass substitution ratio α_{RA} could be expressed as a function of $\alpha_{\mathrm{V,RA}}$:

$$\alpha_{\mathrm{RA}} = \frac{\rho_{\mathrm{c}} \cdot V_{\mathrm{ag}} \cdot \alpha_{\mathrm{V,RA}}}{(\rho_{\mathrm{ag}} \cdot V_{\mathrm{ag}}(1 - \alpha_{\mathrm{V,RA}}) + \rho_{\mathrm{c}} \cdot V_{\mathrm{ag}} \cdot \alpha_{\mathrm{V,RA}})} = \frac{\rho_{\mathrm{c}} \cdot \alpha_{\mathrm{V,RA}}}{(\rho_{\mathrm{ag}} \cdot (1 - \alpha_{\mathrm{V,RA}}) + \rho_{\mathrm{c}} \cdot \alpha_{\mathrm{V,RA}})} \quad (3)$$

Therefore, $\alpha_{\mathrm{V,RA}}$ in Equation (2) can be replaced by

$$\alpha_{V,RA} = \frac{\rho_{ag} \cdot \alpha_{RA}}{\rho_c + (\rho_{ag} - \rho_c) \cdot \alpha_{RA}} \tag{4}$$

The comparison of the evolution of $\Delta\rho_{RAC}$ as a function of α_{RA} or $\alpha_{V,RA}$ (Figure 1) shows that the difference between the two possible expressions is very small and allows adopting the simpler expression [11] for reinforced concrete:

$$\rho_{RAC} = 2.50 - 0.22 \cdot \alpha_{RA} \tag{5}$$

A comparison with experimental results from a real site [20] and from a study by Adessina et al. [21] demonstrates good agreement. Of course, if the volumetric mass of the initial concrete is lower, the variation of volumetric mass of the concretes made with RA from such a concrete will be higher.

3.3. Mechanical properties

3.3.1. Compressive strength

In structural design of concrete, the most important mechanical property is compressive strength; for typical structures, once a concrete class (and consequently, compressive strength) is selected, all other properties are determined based on it. As expected, a significant body of literature has been dedicated to this topic.

Researchers have paid a lot of attention to the compressive strength of RAC relative to reference NAC. In this regard, one of the most comprehensive studies is a meta-analysis by Silva et al. [22] in which the authors analysed 119 publications from the period 1978–2014, studying the effects of RA content, mix design approach, adding admixture, etc. on compressive strength of RAC relative to reference NAC. The authors show that for equal $(w/c)_{eff}$:

- the compressive strength of RAC, $f_{c,RAC}$, is generally lower than the compressive strength of reference NAC, $f_{c,NAC}$;
- the ratio of RAC-to-reference NAC compressive strength, $f_{c,RAC}/f_{c,NAC}$, will decrease with increasing RA substitution ratio (α_{RA});
- the "strength loss" of RAC relative to reference NAC is greater when using pre-soaked RA than when using water absorption compensation;

- using fine RA leads to a greater strength loss of RAC relative to reference NAC.

Results show that, when using only coarse RA and the water absorption compensation method, the expected values of $f_{c,RAC}/f_{c,NAC}$ are approximately 0.97, 0.93 and 0.84 for 20%, 50% and 100% of coarse RA, respectively [23]. Expectedly, the use of fine RA leads to larger strength loss of RAC [22]. Importantly, all meta-analyses of RAC compressive strength [22,23] find that a low substitution ratio of RA (whether coarse or fine), typically below 20–30%, does not have detrimental effects of RAC compressive strength, or if there is any strength loss, it is such that it can effectively be disregarded.

It should also be noted, that in the majority of research, RAC was produced in the range of normal-strength concretes, i.e. with compressive strengths below 60 MPa. Although high-strength strength RAC can be produced [21,24–26], the amount of research is not enough to draw conclusive remarks. Therefore, a codified design of RAC can be proposed only for RAC belonging to a maximum concrete strength class of C50/60.

In structural design, because the compressive strength is specified, the interest is not in the relationship between RAC and NAC compressive strength, but in the *variability* of RAC compressive strength. The statistical distribution of concrete compressive strength is what determines the material partial safety factor γ_c. A detailed study by Pacheco et al. [27] using coarse RA and controlling for inter- and intra-batch variability found no significant effect of incorporating coarse RA on the variability of compressive strength, modulus of elasticity and tensile splitting strength of RAC and no dependence on the RA substitution ratio could be detected. The tested properties were also found to be normally distributed [27], Figure 2. Therefore, the material partial safety factor for RAC compressive strength can be retained as equal to that of NAC, i.e. $\gamma_{c,RAC} = \gamma_{c,NAC}$. As for the relation between the characteristic and mean compressive strengths, f_{ck} and f_{cm}, respectively, the relation $f_{cm} = f_{ck} + 8$ MPa was shown to be a valid assumption for NAC [28] and the same can be considered for RAC as well.

Regarding the development of RAC compressive strength over time, Omary et al. [29] found that it is independent of the RA substitution ratio and can be predicted equally well as for NAC using code expression from the *fib* Model Code 2010 and the current Eurocode 2 [15,30].

3.3.2. Modulus of elasticity

For concrete with only NA, the new EC2 expresses the modulus of elasticity E_{cm} in terms of the compressive strength f_{cm} using the following equation:

$$E_{cm} = k_E \cdot f_{cm}^{1/3} \tag{6}$$

where, E_{cm} and f_{cm} are in MPa. The coefficient k_E account for the type of aggregate and is 9500 for siliceous aggregates or quartzite, with values that can range between 5000 and 13000 for other types of aggregates.

The relationships between compressive and modulus of elasticity of 425 concrete mixes from 24 studies [17,18,21,31–51] were analysed. When compressive strength was measured on cubes, the corresponding values on cylinders were calculated using the following equation proposed by Neville [52]:

$$f_{cm} = f_{cm,cyl} = \left(0.76 + 0.2 \cdot \log\left(145 \cdot \frac{f_{cm,cu}}{2840}\right)\right) \cdot f_{cm,cu} \tag{7}$$

where $f_{cm,cyl}$ and $f_{cm,cu}$ are mean compressive strengths measured on cylinders and cubes, respectively.

The results are displayed in Figure 3. It can clearly be seen that the percentage of RA has an influence on this relationship.

Hence, the results were analysed with the following type of equation:

$$E_{cm} = (k_E - (k_E - k_{RA}) \cdot \alpha_{RA}) \cdot f_{cm}^{1/3} \tag{8}$$

The coefficients k_E and k_{RA} represent the effect on E_{cm} by NA and RA, respectively. For each independent set of data, coefficients k_E and k_{RA} were fitted. This approach gave the following mean values: k_E = 9900 and k_{RA} = 7100 with an overall mean error (the average of the absolute differences between the predictions and measurements) of 1100 MPa on the prediction of the modulus of elasticity, while the mean error was 1200 MPa when only RAC are considered. Thus, RA do not degrade the precision of the model. Among the different k_{RA} values obtained, 17% range between 4000 and 6000, 60% between 6000 and 8000 and 23% are higher than 8000. Therefore, by choosing a fixed value k_{RA} = 7100 one may be optimistic in only 17% of the cases. It is then suggested to adopt the following equation for the new MC2020:

$$E_{cm} = k_E \cdot (1 - (1 - 7100/k_E) \cdot \alpha_{RA}) \cdot f_{cm}^{1/3} \tag{9}$$

This equation reflects the fact that RA are generally more flexible than natural aggregates due to the presence of the residual cement paste and that the reduction in modulus for the same compressive strength with the recycling rate is all the greater the stiffer the natural aggregate. With k_{RA} = 7100, RA behaves as medium limestone. It can be noted that if NA display a k_E value of 5000 (e.g., a weak sandstone), the modulus may increase with the introduction of RA. Figure 4 displays the quality of the fitting.

If the value k_E = 9500 is used for the estimation of the ratio $7100/k_E = 7100/9500 = 0.75$, as proposed by [44] Equation (9) could be simplified in Equation (10):

$$E_{cm} = k_E \cdot (1 - 0.25 \cdot \alpha_{RA}) \cdot f_{cm}^{1/3} \tag{10}$$

Such an equation fits the experimental data with the same level of error (1200 MPa for concrete with α_{RA} less than 40%) as Equation (9). Nevertheless, for high levels of recycling, this equation loses its physical meaning. Particularly for concrete with 100% of RA (i.e., with $\alpha_{RA} = 1$), E_{cm} depends on k_E according to Equation (10) even though no NA is included in the concrete.

3.3.3. Tensile strength

Considering the composite nature of cracking and the fracture mechanics associated with it, the tensile strength of concrete is a property with a much higher variability than compressive strength [53]. The most common way of testing for tensile strength is using splitting cylinder tests; then, the mean axial tensile strength f_{ctm} can be adopted as 0.9–1.0 of the measured splitting tensile strength $f_{ct,sp}$ [15,30]. As for the variability of tensile strength, Pacheco et al. [27] did not find any discernible effect of RA incorporation on the variability of RAC tensile strength. Therefore, its statistical distribution can be assumed identical to that of NAC, with lower and upper characteristic values equal to $0.7 \cdot f_{ctm}$ and $1.3 \cdot f_{ctm}$, respectively [30].

In actual design codes, tensile strength is calculated from compressive strength. Eurocode 2 and the *fib* Model Code 2010 both use the following expressions (in [MPa]):

$$f_{ctm} = 0.3 \cdot f_{ck}^{2/3} = 0.3 \cdot (f_{cm} - 8)^{2/3}; \text{for concrete strength class} \leq \text{C50/60} \tag{11}$$

$$f_{ctm} = 2.12 \cdot \ln\left(1 + \frac{f_{ck} + 8}{10}\right) = 2.12 \cdot \ln\left(1 + \frac{f_{cm}}{10}\right); \text{for concrete strength class} > \text{C50/60} \tag{12}$$

To assess the appropriateness of these expressions, the relationship between compressive and splitting tensile strength of 393 concrete mixes from [10,17,18,21,31–34,44–51,54–62], was analysed. When compressive strength was measured on cubes, the corresponding value on a cylinder was determined according to Equation (7) and it was assumed that $f_{ctm} = 0.9 \cdot f_{ctm,sp}$ for all concretes. Figure 5 summarizes the collected data.

Figure 6 shows the comparisons between experimentally measured and calculated values of splitting tensile strength $f_{ctm,sp}$ for the 393 mixes analysed. The precision of the prediction of tensile strength from compressive strength by Equations (11) and (12) remains globally unaffected by RA incorporation. These results confirm the meta-analysis performed by Silva et al. [63].

In the new EC2, Equation (11) is maintained for concrete strength classes ≤ C50/60, whereas Equation (13) is proposed for classes > C50/60. As for the new draft version of the *fib* Model Code 2020, Equation (14) is proposed for all strength classes. The mean errors of these models for concrete with only NA, are the same as with the previous one (0.4 MPa). Moreover, for both models, the conclusion is the same as previously concerning the introduction of RA: it has no influence on their predictive ability.

$$f_{\mathrm{ctm}} = 1.1 \cdot f_{\mathrm{ck}}^{1/3}; \text{for concrete strength class} > \mathrm{C50/60} \tag{13}$$

$$f_{\mathrm{ctm}} = 1.8 \cdot \ln(f_{\mathrm{ck}}) - 3.1 = 1.8 \cdot \ln(f_{\mathrm{cm}} - 8) - 3.1; \text{for all strength class} \tag{14}$$

A new analysis was then performed on the dataset comparing, this time, the reference concrete (i.e. reference NAC) from each study to the derived concrete including RA. Thus, for each study, the experimental values of $f_{ctm,sp}$ were first fitted for NAC by fitting the constant a in the following expression:

$$f_{\mathrm{ctm}} = a \cdot f_{\mathrm{ck}}^{2/3} \tag{15}$$

Once a was fitted, the experimental values of $f_{ctm,sp}$ were fitted for concrete including NA (of the same type as in reference NAC) and RA, by fitting the b value in the following expression:

$$f_{\mathrm{ctm}} = a \cdot (1 - \left(1 - \frac{b}{a}\right) \cdot \alpha_{RA}) \cdot f_{\mathrm{ck}}^{2/3} \tag{16}$$

This analysis gives a mean error of 0.2 MPa and it confirms that, generally, replacement of NA by RA has no influence on the f_{cm}–f_{ctm} relationship. Only five series involving 74 concrete from [21,45,50,60] among 393 display a negative effect of RA, with a fitted term $(1 - b/a)$ ranging from 0.13 to 0.51 (mean value 0.32). Only one series involving four concretes from [33] among 393, displays a positive effect of RA, with a fitted term $(1 - b/a)$ equal to –0.24. For replacement rates α_{RA} lower than 0.4, the negative effect, if any, is typically around 13% (0.32·0.4). This effect is probably masked in Equations (11) and (12) (or Equation (13) and (14)) by the uncertainty associated with NA performance. Hence, these equations can be used for concrete with $\alpha_{RA} \leq 0.4$, but it could be wise to verify tensile strength for higher replacement rates, as in some cases the f_{cm}–f_{ctm} relationship may be affected negatively (and more rarely positively) in a significant way by RA.

3.3.4. Stress–strain relationship

In order to conduct analyses of concrete structures, the stress–strain relationship of concrete must be known. In current standards, such as Eurocode 2, the stress–strain relationship is defined by the maximum stress (= compressive strength), modulus of elasticity, peak strain (corresponding to maximum stress, dependent on the strength class), and ultimate strain (typically adopted as 3.5‰ and corresponding to a strength of $0.6{\cdot}f_{cm}$ on the descending branch) [30]. The differences between NAC and RAC in terms of compressive strength and modulus of elasticity have been presented earlier.

Regarding peak and ultimate strains, several researchers have tested the stress–strain relationship of RAC [17,64], Generally, the results show similar shapes of the ascending and descending branches of the stress–strain curves, irrespective of the RA substitution ratio with a similar increase of the peak and ultimate strains, ε_{c1} and ε_{cu1}, respectively, with increasing RA content [17,65,66].

The general form of the stress–strain curve defined by the new EC2 is

$$\frac{\sigma_c}{f_{cm}} = \frac{k \cdot \eta - \eta^2}{1 + (k-2) \cdot \eta} \tag{17}$$

where σ_c is the compressive stress, $\eta = \varepsilon_c/\varepsilon_{c1}$, with ε_c being the compression strain, and $k = 1.05{\cdot}E_{cm}{\cdot}|\varepsilon_{c1}|/f_{cm}$. The curve is limited by the ultimate compression strain ε_{cu1}.

$$\varepsilon_{c1} = 0.7 \cdot f_{cm}^{1/3} \leq 2.8‰ \tag{18}$$

$$\varepsilon_{cu1} = 2.8 + 14 \cdot \left(1 - f_{cm}/108\right)^4 \leq 3.5‰ \tag{19}$$

Analysing the stress–strain formulation using Equation (17) and RAC with coarse RA, González Fonteboa et al. [17] proposed coefficients for a linear increase of peak and ultimate strains with increasing coarse RA substitution ratio. The maximum increase in peak and ultimate was proposed as 21% and 22%, respectively, for full coarse RA substitution. These results are in line with above-cited research on similar increases in peak and ultimate strains. Considering the total RA substitution ratio α_{RA} formulated herein (and the fact that only coarse RA was used in [17]), the following corrections of Equations (18) and (19) are proposed:

$$\varepsilon_{c1} = (1 + 0.33 \cdot \alpha_{RA}) \cdot 0.7 \cdot f_{cm}^{1/3} \leq 2.8‰ \quad (20)$$

$$\varepsilon_{cu1} = (1 + 0.33 \cdot \alpha_{RA}) \cdot \left[2.8 + 14 \cdot \left(1 - f_{cm}/108\right)^4\right] \leq 3.5‰ \quad (21)$$

As an illustration, the stress strain curves of an RAC with α_{RA} = 0.6 and reference NAC are shown in Figure 7. Both concretes were assumed as C30/37 and considering the modifications of the modulus of elasticity given in Equation (10). The increase in peak strain can be seen from the figure. However, both stress–strain curves are limited to an ultimate strain of 3.5‰: the upper limits on peak and ultimate strains remain identical to those for NAC; they are on the safe side and there are insufficient experimental results to fully support increasing these values for RAC.

3.3.5. Fracture energy

Another important parameter for nonlinear analyses of concrete structures is fracture energy, G_F. Although it is not treated under Eurocode 2, it is covered by the *fib* Model Code 2010 and can be calculated using the following expression:

$$G_F = 73 \cdot f_{cm}^{0.18} \quad (22)$$

where f_{cm} is entered in MPa, and G_F is obtained in N/m. However, an alternative expression—yielding practically identical results—is proposed for the new *fib* Model Code 2020, using a more consistent formulation based on characteristic compressive strength f_{ck} instead of the mean compressive strength f_{cm} [67]:

$$G_F = 85 \cdot f_{ck}^{0.15} \quad (23)$$

Li [68] reported that different researchers in China had found a decrease in fracture energy with 100% of coarse RA incorporation, but no results were available for intermediate RA substitution ratios. Other researchers also tested RAC of different quality, different target strengths, and characterisation tests (beam tests and wedge splitting tests) and reported a general decrease of fracture energy with increasing RA substitution ratio—between 20% and 30% for 100% of coarse RA [47,69,70]. However, the need for carrying out more tests in order to draw conclusive results was also pointed out [71]. Importantly, researchers noted a similar dependence of RAC and reference NAC fracture energy on compressive strength [47]. Therefore, a global correction of Equation (23) can be proposed, considering a decrease of G_F by 25% for 100% of coarse RA (i.e., $\alpha_{\mathrm{RA}} = 0.6$):

$$G_F = (1 - 0.4 \cdot \alpha_{\mathrm{RA}}) \cdot 85 \cdot f_{\mathrm{ck}}^{0.15} \qquad (24)$$

The expression given by Equation (24) was tested on experimental results from the three available studies [47,69,70], i.e. fracture energy was predicted for a total of 10 NAC mixes and 26 RAC mixtures (with coarse RA replacement from 20% to 100%). The values were compared with experimentally measured fracture energy and the statistical descriptors of the calculated-to-measured fracture energy ratio are given in Table 3. It can be seen that using the proposed adjustment for RAC, the mean value (μ) and coefficient of variation (CoV) of the $G_{F,calc}/G_{F,exp}$ ratio are very similar.

3.4. Time-dependent properties

3.4.1. Shrinkage

Besides the modulus of elasticity, long-term properties of RAC, i.e. shrinkage and creep, are those for which it shows the largest difference relative to NAC. This is expected as RA is generally less stiff than NA (due to residual mortar and higher porosity), offering less restraint to shrinkage; at the same time, due to the residual mortar, RAC has a larger total mortar volume than a reference NAC, meaning that a larger volume of the concrete is subjected to time-dependent changes. There are even indications that shrinkage-reducing admixtures, effective in the case of NAC, do not have the desired effect in the case of RAC [72]. Furthermore, due to the time-consuming nature of shrinkage experiments, less research has been performed testing time-dependent properties than instantaneous ones. A particular problem has been the fact that researchers have not separated shrinkage into basic and drying components, with rare exceptions [73].

Nonetheless, there have been several meta-analyses performed so far [74–76]. For example, Lye et al. [74] analysed differences in shrinkage between RAC and reference NAC mixes from 118 studies. The authors found that, on average, RAC has greater shrinkage than reference NAC and the difference increases with increasing RCA substitution ratio and with decreasing compressive strength. The differences are such that for a 100% substitution ratio of coarse RA, the shrinkage strain of RAC is 30–40% larger than that of NAC. It should be noted that the review was done only on RAC with coarse RA and a very high scatter of results is noted along with the heterogeneousness of experiment duration (biased towards shorter times, less than 180 days).

As for code predictions of RAC shrinkage, Tošić et al. [76] analysed a database of experimental results from 19 studies with 125 shrinkage time curves (i.e. with measurements for each mix for more than one point in time). The authors tested the applicability of the *fib* Model Code 2010 shrinkage prediction model [15]. Using statistical analyses and a time-weighted least-squares regression, the authors proposed a global correction coefficient for the mean shrinkage strain calculated according to the *fib* Model Code 2010:

$$\varepsilon_{\mathrm{cs,RAC}}(t,t_s) = \xi_{\mathrm{cs,RAC}} \cdot \varepsilon_{\mathrm{cs}}(t,t_s) = \left(\frac{100 \cdot \alpha_{\mathrm{CRA}}}{f_{cm}}\right)^{0.30} \cdot \varepsilon_{\mathrm{cs}}(t,t_s) \geq \varepsilon_{\mathrm{cs}}(t,t_s) \tag{25}$$

where $\varepsilon_{cs}(t,t_s)$ is the mean shrinkage strain calculated according to the *fib* Model Code 2010 shrinkage prediction model.

The correction factor depends on the RAC compressive strength $f_{cm} = f_{ck} + 8$ MPa and the coarse RA substitution ratio. The proposed coefficient leads to increases of RAC shrinkage of 30–45% in the compressive strength range of 30–40 MPa, as observed in experiments [76]. At the same time, within the French national project RECYBETON, based on own experimental results, the following expression was proposed [11]:

$$\varepsilon_{\mathrm{cs,RAC}}(t,t_s) = (1 + 0.82 \cdot \alpha_{\mathrm{RA}}) \cdot \varepsilon_{\mathrm{cs}}(t,t_s) \tag{26}$$

Considering a lower boundary of RAC strength class of C20/25 for structural applications, Equation (26) was slightly adjusted to agree with Equation (25) over the coarse RA substitution ratio range above 30%. Hence, the following expression is proposed for the MC2020 and the new EC2:

$$\varepsilon_{\mathrm{cs,RAC}}(t,t_s) = (1 + 0.8 \cdot \alpha_{\mathrm{RA}}) \cdot \varepsilon_{\mathrm{cs}}(t,t_s) \tag{27}$$

A comparison of corrections coefficients $\xi_{cs,RAC}$, obtained using Equations (25) and (27) is shown in Figure 8, considering that $\alpha_{RA} \approx 0.6 \cdot \alpha_{CRA}$ and assuming the validity of Equation (25) for RAC with fine RA. It can be seen that for higher strength classes, Equation (27) is on the safe side, approaching Equation (25) in the range of α_{RA} between 0.25 and 0.60 (corresponding to a 0.4–1.0 substitution ratio of coarse RA). This choice of Equation (27) is further justified when existing standards and recommendations for the use of RAC are considered. For example, the Spanish standard for concrete structures [77] requires a global correction factor of 1.5 for the shrinkage strain of RAC with 100% of coarse RA. According to Equation (27), the correction factor is 1 + 0.8·0.6 = 1.48. Furthermore, according to Dutch recommendation [78], the shrinkage strain of

RAC with over 50% of coarse RA should be multiplied by 1.4. Using Equation (27), for 50% and 100% of RA in RAC, correction factors 1.24 and 1.48 are obtained, respectively.

3.4.2. Creep

In terms of creep, the differences between RAC and NAC are similar to the case of shrinkage, although a higher scatter is associated with the results due to more influencing factors. Several reviews were published with similar findings [75,79,80]. In the case of RAC with coarse RA, relative to NAC, creep of RAC increases with RA substitution ratio and decreases with increasing compressive strength. As in the case of shrinkage, only rare studies separate creep into basic and drying [81], results are reported in different formats (creep coefficient, creep strain, specific creep), and there is a bias towards shorter testing times. Nonetheless, on average, RAC with 100% of coarse RA can be expected to experience close to 30% more creep than a reference NAC [79].

In terms of code predictions of the RAC creep coefficient, Tošić et al. [80] formed a database of experimental results from 10 studies with 46 creep time curves (i.e. with measurements for each mix for more than one point in time). The authors analysed the performance of the *fib* Model Code 2010 creep prediction model [15]. Using statistical analyses and a time-weighted least-squares regression, the authors proposed a global correction coefficient for the mean creep coefficient according to the *fib* Model Code 2010:

$$\varphi_{\mathrm{RAC}}(t,t_0) = \xi_{\mathrm{cc,RAC}} \cdot \varphi(t,t_0) = 1.12 \cdot \left(\frac{100 \cdot \alpha_{\mathrm{CRA}}}{f_{\mathrm{cm}}}\right)^{0.15} \cdot \varphi(t,t_0) \geq \varphi(t,t_0) \qquad (28)$$

where $\varphi(t,t_s)$ is the mean creep coefficient calculated according to the *fib* Model Code 2010 creep model.

As in the case of shrinkage, the correction factor depends on the RAC compressive strength $f_{cm} = f_{ck} + 8$ MPa and the coarse RA substitution ratio. The proposed coefficient leads to increases of the RAC creep coefficient of 20–45% in the compressive strength range of 20–60 MPa [80]. Within the scope of the French national project RECYBETON, based on own experimental results, the following expression was proposed [11]:

$$\varphi_{\mathrm{RAC}}(t,t_0) = (1 + 0.9 \cdot \alpha_{\mathrm{RA}}) \cdot \varphi(t,t_0) \qquad (29)$$

Considering a lower boundary of RAC strength class in structural applications of C25/30, Equation (29) was adjusted to agree with Equation (28) over the coarse RA substitution ratio range between 30% and 100%. Hence, the following expression is proposed for the MC2020 and the new EC2:

$$\varphi_{\mathrm{RAC}}(t,t_0) = (1 + 0.6 \cdot \alpha_{\mathrm{RA}}) \cdot \varphi(t,t_0) \tag{30}$$

A comparison of corrections coefficients $\xi_{\mathrm{cc,RAC}}$, obtained using Equations (28) and (30) is shown in Figure 9, considering that $\alpha_{\mathrm{RA}} \approx 0.6 \cdot \alpha_{\mathrm{CRA}}$ and assuming the validity of Equation (28) for RAC with fine RA. The formulation of Equation (30) is also justified by considering existing standards and recommendations for the use of RAC. In the Spanish standard for concrete structures [77], a global correction factor of 1.25 for the creep coefficient of RAC with 100% of coarse RA is prescribed. Using Equation (30), the correction factor is calculated as $1 + 0.6 \cdot 0.6 = 1.36$. According to Dutch recommendation [78], the creep coefficient of RAC with over 50% of coarse RA should be multiplied by 1.1. Using Equation (30), for 50% and 100% of RA in RAC, correction factors 1.18 and 1.36 are obtained, respectively.

Besides this approach, the new EC2 and MC2020 shrinkage and creep models will explicitly allow the modification of parameters that can enable curve fitting based on experimental results. Hence, even using relatively short-term measurements (3–6 months) of creep and shrinkage, by fitting these short-term results, a relatively precise extrapolation to longer times will be possible.

3.5. Durability-related properties

The durability of reinforced concrete structures built either from NAC or RAC is related to several deterioration mechanisms that may damage the concrete matrix and the passivation layer of reinforcements within the concrete, leading to corrosion under certain conditions. Among a number of mechanisms, the most common problems in concrete structures are caused by carbonation and chloride penetration.

Replacement of NA with RA influences physical, mechanical and durability properties of concrete. RA is more porous than NA due to its residual mortar content. Concrete made with RA actually has two types of interfacial transition zones (ITZs) [82]. The volume of porous and loose ITZs in concrete increases with the use of RA in the mixture, increasing the total porosity of concrete and resulting in increased permeability and water absorption [83]. This leads to a negative impact on the resistance to both carbonation and chloride ingress. At the same time, the increasing incorporation of RA can introduce additional alkalis (from an increased amount of cement used in RAC relative NAC to achieve the same concrete class, or from the residual mortar of RA), which can potentially have a positive impact on RAC durability. Therefore, depending on the RAC mix design, RA properties and exposure conditions, one effect will be more dominant over the other, determining RAC performance.

3.5.1. Carbonation resistance

Most researchers have found that the impact of coarse RA on carbonation resistance of concrete is negative and that RAC is more vulnerable to carbonation compared with a reference NAC, when considering concretes produced with the same amount of cement [84,85], effective *w*/*c* ratio [21,57] or total *w*/*c* ratio [49,86].

With an increase of the coarse RA substitution ratio, an increase in the carbonation depth is observed [21,85,87–89]. Results from 16 investigations summarized in Ma et al. [90], prove an increase in relative carbonation depth (C_{RAC}/C_{NAC}) with the rise of RA content. This relationship is linear only up to 70% of coarse NA replacement when the carbonation depth reaches its maximal value [88]. With a further increase of coarse NA replacement level, the carbonation depth remains relatively constant. This is explained by the two mutually opposing effects that influence the carbonation behaviour of RAC: when the replacement percentage is lower, the negative effect of increased porosity overcomes the positive effect of increased content of alkalis.

However, when RAC and NAC of similar compressive strength are compared, RAC typically shows the same or only slightly higher carbonation depths compared with NAC of similar strength [89,91–93]. The reason for this lies in the fact that, usually, more cement is needed for producing RAC with the same compressive strength as the reference NAC. The higher amount of cement leads to a higher amount of alkalis that can be carbonated in the concrete cover, thus preventing the increase of carbonation depth. This is also supported by a literature review by Silva and de Brito [94].

In terms of concrete cover for durability, only a few researchers performed investigations of service life of RAC exposed to carbonation using a probabilistic approach [85, 91]. Calculations showed that for RAC with 10% to 50% of coarse RA there is no need to increase concrete cover to secure a designed service life of 50 years with the same reliability as a reference NAC of the same concrete strength class. In case of 100% coarse RA in concrete, an increase of 1 to 5 mm is needed to reach the same goal but only for lower concrete classes (up to C25/30). However, considering that none of these studies analysed the potential incorporation of fine RA, a conservative conclusion can be the adoption of a 5-mm increase in concrete cover for RAC exposed to carbonation.

3.5.2. Chloride ingress

Chloride ingress is the most important deterioration mechanism that leads to depassivation and corrosion of steel bars in reinforced concrete structures. For practical engineering applications, it is usually quantified as a measure of the chloride diffusion coefficient which can be obtained by Fick's second law of diffusion [95,96] . Therefore, the majority of research is focused on determining this parameter as well as the comparison between the values of diffusion coefficients for RAC and NAC.

Results of numerous investigations conducted in the previous decade in China and summarized by Ma et al. [90] show that the relation between chloride diffusion coefficients of RAC (D_{RAC}) and reference NAC (D_{NAC}), i.e., the relative diffusion coefficient, $D_{rd} = D_{RAC}/D_{NAC}$, increases linearly with the increase of RA content. For a substitution ratio of coarse aggregates of 100%, the range of values for this relation is between 1.2 and 2.1, with an average of 1.5. It was confirmed by Bao et al. [97] who found that the chloride diffusion coefficient of specimens with 100% of coarse RA increased by approximately 1.56 times to about 2.10 times compared with a reference NAC, depending on the quality, i.e. water absorption of RA.

The conclusion from other investigations that also considered a larger number of results—31 publications and 1115 measurements [98]—is in line with the previous one stating that when using 100% coarse or fine RA there is a probability of 95% (upper confidence level, UCL) that the chloride migration coefficient (D_{nssm}) may increase up to 1.65 and 2.95 times with respect to a reference NAC, respectively. For replacement levels close to the limits permitted by EN 206 [8], e.g. 50% of coarse RA or 20% of fine RA, the UCL values of D_{rd} are 1.32 and 1.39, respectively. Having in mind the square root law between the diffusion coefficient and concrete cover depth according to the *fib* Model Code 2010 [15], the effect of 50% coarse or 20% fine RA in RAC is estimated as 15% and 18% increase in concrete cover depth.

Taking into account the values of minimal concrete covers for structural elements in XD or XS exposure classes that usually come out from the durability design of NAC ($c_{min,dur} = 45$ mm), the final result of 50% coarse RA or 20% fine RA use would be increase in concrete cover of 8 mm compared with a reference NAC. Considering the fact that these results do not take into account the potential simultaneous replacement of coarse and fine RA and the compounding effect on durability, a reasonable conclusion can be that concrete cover for RAC exposed to chloride ingress should be increased by 10 mm.

3.5.3. Recommendations for durability

In the future, durability design of concrete structures should be dominantly based on performance-based approaches, such as exposure resistance classes (ERC) in Europe that encompass structures containing RA [14]. The straightforward relation between ERC, determined by concrete testing or by requirements on mix design (from EN206), and the thickness of concrete cover can then be established and tabulated. However, in cases in which RAC is not classified into ERCs, based on the previous analysis, the recommendation is to increase the cover prescribed for the appropriate concrete strength class, $c_{\mathrm{min,dur}}$, by 5 and 10 mm for exposure to carbonation and chloride ingress, respectively.

4. Recycled aggregate concrete structures

4.1. Flexural and shear strength

In the previous section it was shown that the variability of the main mechanical properties of RAC was not different from NAC and that identical material partial safety factors can be used ($\gamma_{c,RAC} = \gamma_{c,NAC}$). As stated earlier, the material partial safety factor is only one part of the partial safety factor for concrete ($\gamma_C = 1.5$), together with the model partial safety factor (γ_{Rd}). In order to assess whether $\gamma_C = 1.5$ is also valid for RAC, failure mechanisms of RAC structural members need to be investigated in order to assess whether they are the same as for NAC members. From such an analysis, the model uncertainties of resistance models for reinforced and prestressed RAC members can be determined and any differences in γ_{Rd} identified.

In the case of reinforced RAC members, ultimate flexural and shear strength were the most investigated properties. A wide range of studies analysed different RAC with different substitution ratios, reinforcement ratios, and cross-section size; however, almost exclusively, simply supported beams in four-point bending were tested and almost exclusively only coarse RA was used [50,51,99–103]. The general observation from all studies was that the presence of RA does not significantly affect the ultimate or shear strengths of reinforced RA beams; however, it was noted that the amount of damage (concrete crushing) and deflections at failure were typically larger for RAC beams.

Based on a database of experimental studies on RAC and reference NAC beams, Pacheco et al. [104] assessed the model uncertainty of the Eurocode 2 [30] flexural model for RAC produced with 50% and 100% coarse RA. The authors found no statistically significant differences in the distribution of the bias factor (ratio of experimental-to-predicted values) for RAC and NAC beams. Therefore, for flexural strength, the same partial factors for NAC and RAC can be used, i.e. $\gamma_{C,RAC} = \gamma_{C,NAC} = 1.5$, to achieve the same target reliability.

In the case of shear resistance, Pacheco et al. [105] performed another probabilistic study on the uncertainty of shear resistance models for RAC, using a database of RAC (produced with coarse RA) and reference NAC beams. For the case of shear resistance, the new EC2 has a new formulation for elements not requiring shear reinforcement [14]:

$$\tau_{Rd,c} \geq \tau_{Rdc,min} \Longrightarrow \frac{0.66}{\gamma_C} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{1/3} \geq \frac{11}{\gamma_C} \cdot \sqrt{\frac{f_{ck} d_{dg}}{f_{yd}\ d}} \tag{31}$$

where $\tau_{Rd,c}$ and $\tau_{Rdc,min}$ are the concrete shear resistance and concrete minimum shear resistance, respectively, ρ_l is the longitudinal reinforcement ratio in the control cross-section, f_{ck} is the characteristic compressive strength of concrete, d_{dg} is a size parameter describing the failure zone roughness (dependent on aggregate size and f_{ck}), d is the effective depth, and f_{yd} is the design yield strength of reinforcement. It should be noted, that his formulation is similar to the existing Eurocode 2 [30] shear resistance model for beams without shear reinforcement, differing only in the formulation of the size effect.

Through a FORM analysis, Pacheco et al. [105] determined that an increase in the shear resistance partial factor for concrete was needed: instead of 1.5 for NAC, the authors suggested values of 1.6 and 1.7 for 50% and 100% of coarse RA substitution. In effect, this means that shear resistance for RAC with 50% and 100% of coarse RA is 1.5/1.6 = 0.94 and 1.5/1.7 = 0.88 of the NAC shear resistance [105]. In order not to introduce changes in partial safety factors and since the decrease in shear resistance is linearly dependent on the RA substitution ratio (and extrapolating to the total RA substitution ratio α_{RA}), the modification given in Equation (32) is adopted. In other words, a reduction of $1 - 0.2 \cdot \alpha_{RA}$ for the shear resistance is adopted, maintaining $\gamma_{C,RAC} = \gamma_{C,NAC} = 1.5$.

$$(1 - 0.2 \cdot \alpha_{RA}) \cdot \frac{0.66}{\gamma_C} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{1/3} \geq (1 - 0.2 \cdot \alpha_{RA}) \cdot \frac{11}{\gamma_C} \cdot \sqrt{\frac{f_{ck} d_{dg}}{f_{yd}\ d}} \tag{32}$$

In the case of RAC members with shear reinforcement, the authors concluded that additional tests are necessary to accurately evaluate the influence of RA [105]. However, since the shear strength of NAC and RAC beams with shear reinforcement was overestimated—and to a similar degree—a conservative assumption can be the retention of $\gamma_{C,RAC} = \gamma_{C,NAC} = 1.5$.

As for prestressed RAC members, tests are very scarce. Brandes and Kurama (2018) tested shear-critical prestressed RAC beams with 50% and 100% of coarse RA substitution and two levels of prestressing. The authors found that RA did not have a significant effect on the load–deflection response of the beams and their ultimate strength, and the behaviour of the beams was successfully numerically modelled.

4.2. Axial strength

In terms of axial strength of reinforced RAC members, several experimental programs were performed on columns [50,107–109]. Using different specimen size, longitudinal and shear reinforcement ratios and RA substitution ratios, all of the authors conclude that RA does not significantly affect the ultimate axial compressive strength of RAC columns. However, as with beams in flexure and shear, at column failure, the level of damage (concrete crushing) is larger for RAC elements.

Different authors have also studied the behaviour of confined RAC under compression, in steel and GFRP tubes [110,111]. The studies found that, relative to NAC, RA substitution adversely affected ultimate strength only through the reduction of RAC compressive strength whereas the failure mechanism and the stress–strain curve of confined RAC were the same as for NAC [110].

Considering the above-presented, the design of reinforced RAC columns under axial compression can be assumed to be identical to that of NAC with $\gamma_{C,RAC} = \gamma_{C,NAC} = 1.5$.

4.3. Deflections

If the largest differences between RAC and NAC, at the material level, exist in the modulus of elasticity, shrinkage and creep, then it is expectable that in terms of structural behaviour, the largest differences will be observed in serviceability, and especially in deflections. Long-term tests are rare even for NAC members due to being complex to carry out and requiring the control of many parameters [112], and this is also valid for RAC. So far, several studies have been performed [113–119] on reinforced RAC and NAC beams, all simply supported and under four-point bending or uniformly distributed load. As expected, the studies found larger deflections of RAC beams, relative to NAC, although strictly controlling for all parameters is not possible (differences can appear both at the material and structural level). Because of this, Tošić et al. [120] performed an analysis of 30 beams (10 NAC and 20 RAC) from three studies [117–119] in which the RAC had 50% and 100% of coarse RA, complemented with existing databases on long-term studies of NAC [112]. The authors calculated deflections of the beams using the ζ-method of interpolating curvatures between the uncracked and fully-cracked state, and numerical integration, as a general method proposed in the *fib* Model Code 2010 [15].

The authors calculated deflections using only reported compressive strength of RAC and NAC and determining all other properties from f_{cm} (modulus of elasticity, tensile strength, shrinkage, creep) using *fib* Model Code 2010 expressions. With this approach, a significant underestimation of RAC deflections was observed [120]. Even after including corrections for the modulus of elasticity, shrinkage and creep, using own proposals for adjustments [120], RAC deflections were still underestimated. Therefore, it was concluded that tension stiffening is weaker in RAC, which can be explained by the weaker aggregate and potential greater creep in tension. Hence, the authors proposed to modify the original ζ interpolation coefficient

$$\zeta = 1 - \beta_{\mathrm{tRA}} \cdot \left(\frac{\sigma_{\mathrm{sr}}}{\sigma_{\mathrm{s}}}\right)^2 \tag{33}$$

where σ_{sr} is the stress in reinforcement under the cracking load (calculated on the basis of a cracked section) and σ_s the stress in reinforcement under the considered load combination, and β_{tRA} is a coefficient accounting for the influence of the duration of loading or repeated loading.

However, since this paper justifies different proposal for the modulus of elasticity, shrinkage strain and creep coefficient—given by Equations (10), (27) and (30), respectively—a recalculation of the deflection in the database in [120] was required. Following the procedure in [120], first only corrections for the modulus of elasticity, shrinkage and creep were applied and the calculated-to-experimental deflection ratio, a_{calc}/a_{exp}, was calculated. The results are shown in Table 4. It can be seen that only for these corrections (E_{cm}, ε_{cs}, φ) agreement between RAC and NAC is good for initial deflections but long-term deflections are still underestimated. Therefore, the following is proposed for the β_{tRA} coefficient:

$$\beta_{\mathrm{tRA}} = 1.0 \quad \text{for single, short-term loading} \tag{34}$$

$$\beta_{\mathrm{tRA}} = 0.25 \quad \text{for sustained or repeated loading}$$

With this correction, the results shown in the bottom row of Table 4 demonstrate good agreement between RAC and NAC $a_{\mathrm{calc}}/a_{\mathrm{exp}}$ ratios for both initial and long-term deflections.

Furthermore, Tošić and Kurama [121] conducted a numerical parametric study on the deflections of one-way RAC and NAC slabs using a newly-developed material model for the time-dependent behaviour of concrete in OpenSees [122]. Considering different support conditions, strength classes, ambient conditions and load intensities, the authors showed that RAC with 25% of coarse RA does not exhibit any differences in deflection behaviour relative to NAC. For RAC with 50% of coarse RA, the authors found greater deflections of RAC one-way slabs, but for span-effective depth ratios L/d recommended by Eurocode 2 [30] deflection control is still generally satisfied.

As for prestressed RAC members, Brandes and Kurama [123] tested 18 pretensioned NAC and RAC beams with 50% and 100% coarse RA. While only the results of this study are available, the effect of RA incorporation on the time-dependent behaviour of the beams was modest and could be predicted using the Branson Multiplier method [124].

4.4. Cracking

Cracking of RAC beams in tension and flexure was not the topic of many research studies. Santana Rangel et al. [125] performed short-term tension stiffening tests on NAC and RAC with 25% and 50% of coarse RA specimens with compressive strengths of 25 and 65 MPa. The authors noted that RA "does not interfere" with the cracking pattern of the NAC concrete (both 25 and 65 MPa) and that RA caused only a minor increase in crack spacing and crack width. As for members in bending, studies that tested long-term deflections of reinforced NAC and RAC beams typically also reported the crack patterns and their development over time. Mercado-Mendoza et al. [126] tested simply supported beams under sustained service-load, produced from NAC and three RAC (with 30% of fine and coarse RA, with 100% of coarse RA and with 100% of fine and coarse RA). The authors reported smaller crack spacing and crack width for RAC beams (with crack spacing and crack width decreasing with increasing RA content). Tošić et al. [119] reported that under similar tensile steel stress an RAC beam with 100% coarse RA had slightly smaller crack spacing and crack width then an accompanying NAC beam.

No study yet attempted to adjust existing code models for crack control to RAC; hence, no particular provisions can be adopted. Nonetheless, the cited findings of smaller or similar crack spacing and crack width between NAC and RAC are beneficial from the aspect of durability and point to cracking not being an issue of particular concern in the design of reinforced RAC members.

4.5. Bond and anchorage length

Bond is an important property for the design of reinforced concrete structures. Several authors have shown that the bond strength of RAC decreases with the increasing RA mass substitution ratio α_{RA} [127–132]. However, several authors also indicate that the variation is in line with the variation of the compressive strength [130–132]. Indeed, if the experimental results of the bond strength are divided by a function of $(f_{cm})^{2/3}$ and normalized in comparison with the bond of the reference NAC, compared to the variability of the results, the relation between bond and compressive strength is not affected by the use of RA, as shown in Figure 10.

As bond is generally expressed as a function of the tensile strength, the consequence for the codes is that if there is no change of tensile strength with α_{RA}, the expressions for bond and anchorage length remain unchanged.

5. Scope of application and code proposal

The RAC recommendations and design guidelines proposed in this study have been made based on a large number of experimental studies from many different countries, using different RA sources, mixing procedures, etc. However, the proposals have been made exclusively based on experimental and laboratory work. Practical applications are still scarce but existing results are encouraging [133]. Therefore, the limits of application of the proposals made in this study must be made transparent.

The first thing to consider is that the vast majority of cited research has dealt with coarse RA substitution. In other words, the effects of fine RA incorporation are under-represented in the literature. However, in our approach we have opted for using the total RA substitution ratio α_{RA} considering the justification presented in section 3.1, i.e. the proposed equations are valid for cases when only coarse RA is used or both coarse and fine RA, but not when only fine RA is used. Secondly, in the case of the new EC2, other standards must be adhered to, most importantly EN 206 [8] in its current version and any future revision.

Since such standards will allow for nationally-determined parameters, universal agreement between the presented proposals and all other standards will not be possible and a compromise must be found.

With the aim of stimulating the use of RA, but at the same time, not conflicting with EN 206, for RAC used in reinforced concrete applications, an upper limit is adopted on the applicability of proposed expressions, equal to $\alpha_{RA} = 0.4$. In the context of this study, such a replacement percentage can mean either 67% of only coarse RA substitution or a combination of coarse and fine RA (respecting limits of EN 206), e.g. 50% of coarse and 20% of fine RA. While we are confident that expressions presented herein could be used for higher replacement ratios when only coarse RA is used, for design code purposes, we believe this upper limit should be imposed. Any use of RA in a larger substitution ratio than this should be accompanied by testing to determine the relevant properties of the produced RAC. At the same time, research has unequivocally shown that low substitution ratios of RA do not alter RAC properties relative to a reference NAC. Therefore, we propose that for $\alpha_{RA} \leq 0.2$ no change in code expressions relative to NAC is needed when RAC is used in reinforced concrete applications, with the exception of durability. Namely, if the ERC is assessed, then no further adjustment for RAC is required; however, when ERC is not assessed, then the concrete cover should be increased for all RA substitution ratios. The lower limit of $\alpha_{RA} \leq 0.2$ in the context of this study can mean either 33% of only coarse RA substitution or a combination of coarse and fine RA of, e.g., 25% of coarse and 10% of fine RA.

For RAC used in prestressed concrete elements, research is much scarcer. For this case, an upper limit on the applicability of the proposal in this study is set at $\alpha_{RA} \leq 0.2$. Any use of RA in a larger substitution ratio than this should be accompanied by testing to determine the relevant properties of the produced RAC to be used as prestressed concrete. For ease of use, the limits for application are summarized in Table 5, whereas the proposed code adjustments are summarized in Table 6.

As stated earlier, the above presented proposals are valid for RA complying with classification Type A of EN 206 [8], i.e. Rc_{90} and Rcu_{95} per EN 12620 [12]. RA Type B per EN 206 EN 206 [8], i.e. Rc_{50} and Rcu_{70} are not sufficiently represented in research and not directly covered by this study. However, in order to stimulate the use of RA in practice, as a precautionary conservative measure, the limits prescribed in this section could be decreased by 50% for Type B RA.

6. Conclusions

Research on RAC from the material to the structural level has reached a sufficient level of maturity to enable the formulation of code provisions for the structural design of reinforced and prestressed RAC structures. To facilitate the adoption of such codes, this paper presents a comprehensive review of the state-of-art in RAC research, covering mechanical, durability and structural properties. Through a critical assessment of the results in literature, specific proposals are put forward for code adjustments of the *fib* Model Code 2020 and new Eurocode 2. These proposals are formulated based on a total mass substitution ratio of RA (α_{RA}) that can be classified as Type A per EN 206 [8]. Based on our findings the following is concluded:

- Within the scope of existing standards for aggregates and concrete, coarse and fine RA can be successfully used for the substitution of NA. Nonetheless, conservative limits on the substitution ratio of RA are placed when using adjusted code expressions, whereas for higher substitution ratios, tests are necessary. These limits are proposed as 40% and 20% of total aggregate replacement for reinforced and prestressed RAC, respectively. These limits are valid for cases when only coarse RA is used or both coarse and fine RA are used (respecting limits of standards for concrete composition such as EN 206). The substitution of only fine RA is not covered by this study.
- Concerning physical–mechanical properties code expressions should be modified for density, modulus of elasticity, peak and ultimate strains, fracture energy, shrinkage strain, and creep coefficient.
- Concerning durability–related properties, specifically resistance to carbonation and chloride ingress, if exposure resistance is not determined, the minimum cover for durability should be increased by 5 and 10 mm for exposure to carbonation and chloride ingress, respectively.
- Concerning structural behaviour, code expressions should be modified for shear strength of members not requiring shear reinforcement and deflections.

This review has aimed at incorporating all relevant literature into its conclusions. Even so, the conclusions presented herein, are dependent on the considered properties, tests, and parameters; and these proposals can be improved in the future. Areas of particular interest for future research are the fire resistance of RAC and new RA improvement techniques such as accelerated carbonation. Nonetheless, the results of this study can provide an important contribution towards a codified design of RAC structures and the wider use of RA in the construction sector.

Notation

α_{CRA}	mass substitution ratio of coarse recycled aggregate relative to total mass of coarse aggregates
α_{RA}	mass substitution ratio of total recycled aggregate relative to total mass of aggregates
γ_c	material resistance partial safety factor for concrete
ε_{c1}	peak strain of concrete
ε_{cs}	shrinkage strain of concrete
ε_{cu}	ultimate strain of concrete
ρ	volumetric mass of concrete
σ_c	stress in concrete
φ	creep coefficient
E_{cm}	modulus of elasticity of concrete
f_c	compressive strength of concrete
f_{ck}	characteristic compressive strength of concrete
f_{cm}	mean compressive strength of concrete
f_{ctm}	mean axial tensile strength of concrete
$f_{ctm,sp}$	mean splitting tensile strength of concrete
G_F	fracture energy
w/c	water-cement ratio
$(w/c)_{eff}$	effective water-cement ratio
CDW	construction and demolition waste
CEN	European Committee for Standardization
EC2	Eurocode 2
EU	European Union
MC2020	Model Code 2020
NA	natural aggregate
NAC	natural aggregate concrete
RA	recycled aggregate

RAC recycled aggregate concrete

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Table 1. RA categories based on components materials as per EN 12620 [12].

Category	Component	Content
Rc_{90}		≥90%
Rc_{80}		≥80%
Rc_{70}	Rc[1]	≥70%
Rc_{50}		≥50%
$Rc_{Declared}$		<50%
Rc_{NR}		No requirement
Rcu_{95}		≥95%
Rcu_{90}		≥90%
Rcu_{70}	Rc + Ru[2]	≥70%
Rcu_{50}		≥50%
$Rcu_{Declared}$		<50%
Rcu_{NR}		No requirement
Rb_{10-}		≤10%
Rb_{30-}		≤30%
Rb_{50-}	Rb[3]	≤50%
$Rb_{Declared}$		>50%
Rb_{NR}		No requirement
Ra_{1-}		≤1%
Ra_{5-}	Ra[4]	≤5%
Ra_{10-}		≤10%
$XRg_{0.5-}$		≤0.5%
XRg_{1-}	X[5] + Rg[6]	≤1%
XRg_{2-}		≤2%
$FL_{0.2-}$		≤0.2 cm^3/kg
FL_{2-}	FL[7]	≤2 cm^3/kg
FL_{5-}		≤5 cm^3/kg

[1] crushed concrete; [2] unbound stone;
[3] crushed brick; [4] bituminous materials;
[5] other; [6] glass; floating material

1 Table 2. Mix design of a concrete with $\rho_c = 2.4$ t/m^3.

	Quantity (t)	Specific mass (t/m^3)	Volume (m^3)
Aggregates	$M_{ag} = 1.865$	$\rho_{ag} = 2.730$	$V_{ag} = 0.684$
Cement	$M_c = 0.350$	$\rho_c = 3.150$	$V_c = 0.111$
Water	$M_w = 0.185$	$\rho_w = 1.000$	$V_w = 0.185$
Air	–	–	$V_{air} = 0.020$
Total	$\rho_c = 2.400$		1.000

2

Table 3. Statistical descriptors of the calculated-to-measured fracture energy ratio for NAC and RAC mixes from [47,69,70] obtained using Equation (24).

	n	μ	CoV (%)
NAC	10	1.12	1.14
RAC	26	21.9	19.9

Table 4. Statistical descriptors of the a_{calc}/a_{exp} ratios for the NAC and RAC databases from [120], with corrections in deflection control.

Database	Corrections	Deflections	No. of beams	Mean	CoV (%)
NAC	–	Initial	62	1.191	30.1
		Long-term	62	1.035	15.3
RAC	E_{cm}, ε_{cs}, φ	Initial	18	1.171	19.1
		Long-term	16	0.939	23.9
	E_{cm}, ε_{cs}, φ, β	Initial	18	1.171	19.1
		Long-term	16	1.062	18.2

1 Table 5. Limits of applicability of expressions proposed for RAC design properties.

RAC application	RA substitution ratio*	Applicability
Reinforced concrete	$\alpha_{RA} \leq 0.2$	No changes needed (for durability see Table 6)
	$0.2 < \alpha_{RA} \leq 0.4$	Expressions in Table 6 may be applied
	$\alpha_{RA} > 0.4$	Properties shall be determined by testing
Prestressed concrete	$\alpha_{RA} \leq 0.2$	Expressions in Table 6 may be applied
	$\alpha_{RA} > 0.2$	Properties shall be determined by testing

*substitution of only coarse RA or both coarse and fine RA;
substitution of only fine RA is not covered;
valid for RA Type A per EN 206[8]; for Type B, limits could be decreased by 50%

2

3

Table 6. Summary of proposed expressions for RAC design properties.

RAC property	Correction for RAC
Density	$\rho_{\mathrm{RAC}} = 2.50 - 0.22 \cdot \alpha_{\mathrm{RA}}$
Compressive strength	The relationship between the mean and characteristic compressive strength (f_{cm}–f_{ck}) remains unchanged.
Modulus of elasticity	$E_{\mathrm{cm,RAC}} = k_{\mathrm{E}} \cdot (1 - (1 - 7100/k_{\mathrm{E}}) \cdot \alpha_{\mathrm{RA}}) \cdot f_{\mathrm{cm}}^{1/3}$ or $E_{\mathrm{cm}} = k_{\mathrm{E}} \cdot (1 - 0.25 \cdot \alpha_{\mathrm{RA}}) \cdot f_{\mathrm{cm}}^{1/3}$
Tensile strength	The relationship between the mean and characteristic compressive and tensile strength (f_{cm}–f_{ctm} and f_{ck}–f_{ctm}) remains unchanged.
Shrinkage strain	$\varepsilon_{\mathrm{cs,RAC}} = (1 + 0.8 \cdot \alpha_{\mathrm{RA}}) \cdot \varepsilon_{\mathrm{cs}}$
Creep coefficient	$\varphi_{\mathrm{RAC}} = (1 + 0.6 \cdot \alpha_{\mathrm{RA}}) \cdot \varphi$
Peak strain	$\varepsilon_{\mathrm{c1}} = (1 + 0.33 \cdot \alpha_{\mathrm{RA}}) \cdot 0.7 \cdot f_{\mathrm{cm}}^{1/3} \leq 2.8‰$
Ultimate strain	$\varepsilon_{\mathrm{cu1}} = (1 + 0.33 \cdot \alpha_{\mathrm{RA}}) \cdot \left[2.8 + 14 \cdot (1 - f_{\mathrm{cm}}/108)^4\right] \leq 3.5‰$
Fracture energy	$G_F = (1 - 0.4 \cdot \alpha_{\mathrm{RA}}) \cdot 85 \cdot f_{\mathrm{ck}}^{0.15}$
Shear strength	$\tau_{\mathrm{Rd,c}} = (1 - 0.2 \cdot \alpha_{\mathrm{RA}}) \cdot \dfrac{0.66}{\gamma_{\mathrm{C}}} \cdot \left(100 \cdot \rho_{\mathrm{l}} \cdot f_{\mathrm{ck}} \cdot \dfrac{d_{\mathrm{dg}}}{d}\right)^{1/3}$ $\tau_{\mathrm{Rdc,min}} = (1 - 0.2 \cdot \alpha_{\mathrm{RA}}) \cdot \dfrac{11}{\gamma_{\mathrm{C}}} \cdot \sqrt{\dfrac{f_{\mathrm{ck}} d_{\mathrm{dg}}}{f_{\mathrm{yd}}\ d}}$
Deflection control	$\zeta = 1 - \beta_{\mathrm{tRA}} \cdot \left(\dfrac{\sigma_{\mathrm{sr}}}{\sigma_{\mathrm{s}}}\right)^2$ where $\beta_{\mathrm{tRA}} = 1.0$ for single, short-term loading $\beta_{\mathrm{tRA}} = 0.25$ for sustained or repeated loading
Concrete cover for durability	Determine exposure resistance by testing if relevant. For concrete including recycled aggregate, the same minimum cover depth for durability $c_{\mathrm{min,dur}}$ applies provided the material pertains the same exposure resistance class as concrete including natural aggregate only. If exposure resistance is not determined, for reinforced concrete and for prestressed concrete when $\alpha_{\mathrm{RA}} > 0$, the values of $c_{\mathrm{min,dur}}$ should be increased by 5 mm for exposure to carbonation and 10 mm for exposure to chloride ingress

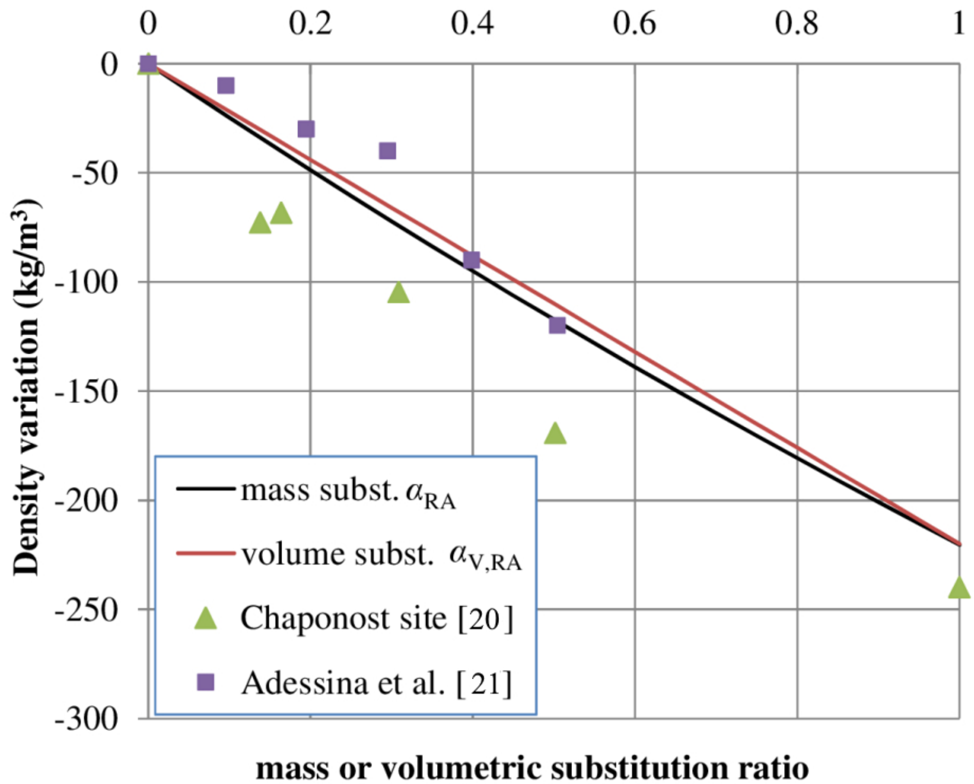

Figure 1. Comparison of the evolution of the density using the mass or the volume substitution rate. The relations are compared with the results from the Chaponost site [20] and from the work by Adessina et al. [21].

80x65mm (300 x 300 DPI)

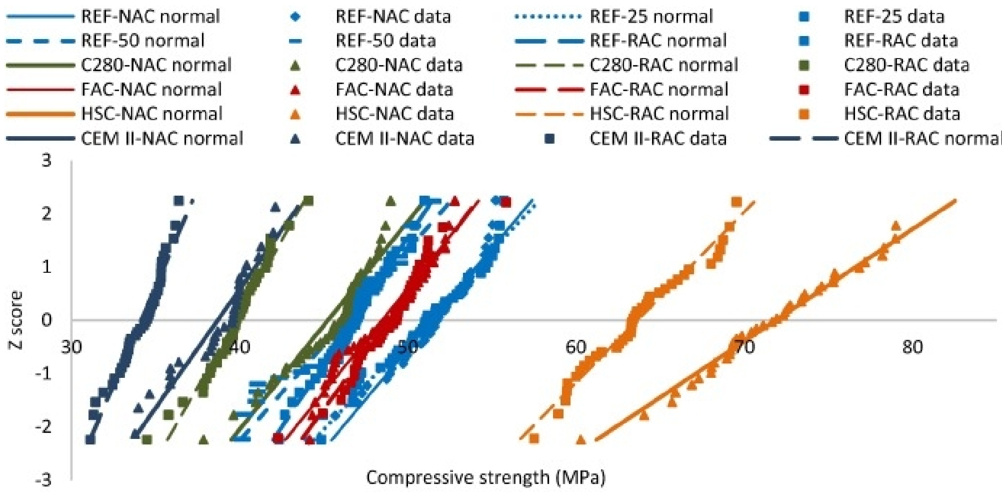

Figure 2. Normal distribution plot: compressive strength. 28-day tests except FAC (91-day). Here: REF – reference mix composition; NAC – natural aggregate concrete; 20 – RAC with 20% coarse RA; 50 – RAC with 50% coarse RA; RAC – RAC with 100% coarse RA; C280 – concrete with reduced binder content; FAC – concrete with 25% volume replacement of cement by fly ash; HSC – high-strength concrete; CEM II – concrete with a CEM II/B-L 32.5N cement. Originally published in [25].

154x75mm (113 x 113 DPI)

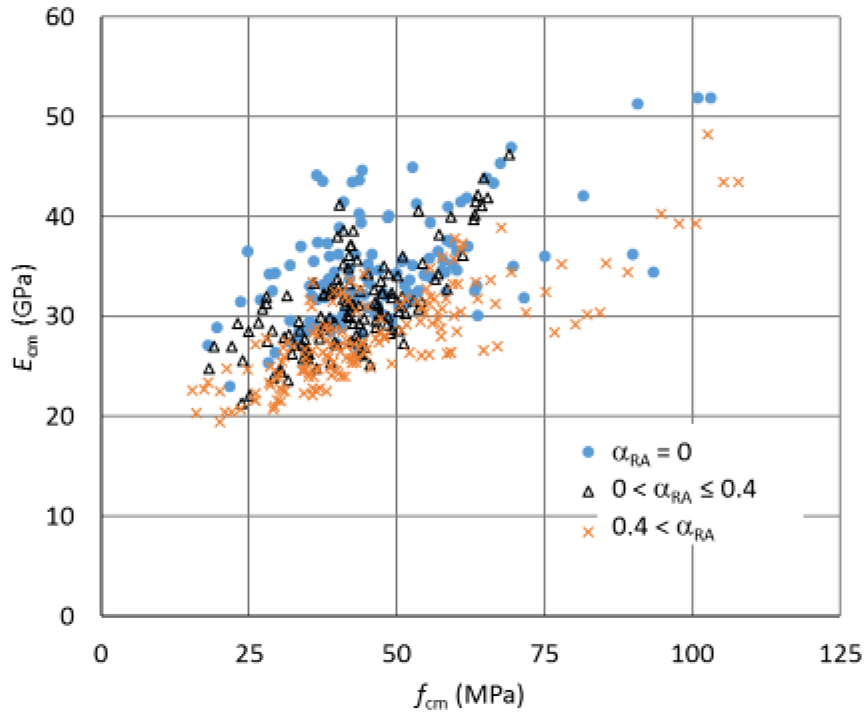

Figure 3. Relationship between the modulus of elasticity and compressive strength for 425 concretes covering a range of f_{cm} between 15 and 110 MPa, with different amounts of RA (130 with $\alpha_{RA} = 0$, 125 with $0 < \alpha_{RA} \leq 0.4$ and 170 with $0.4 < \alpha_{RA} \leq 1$).

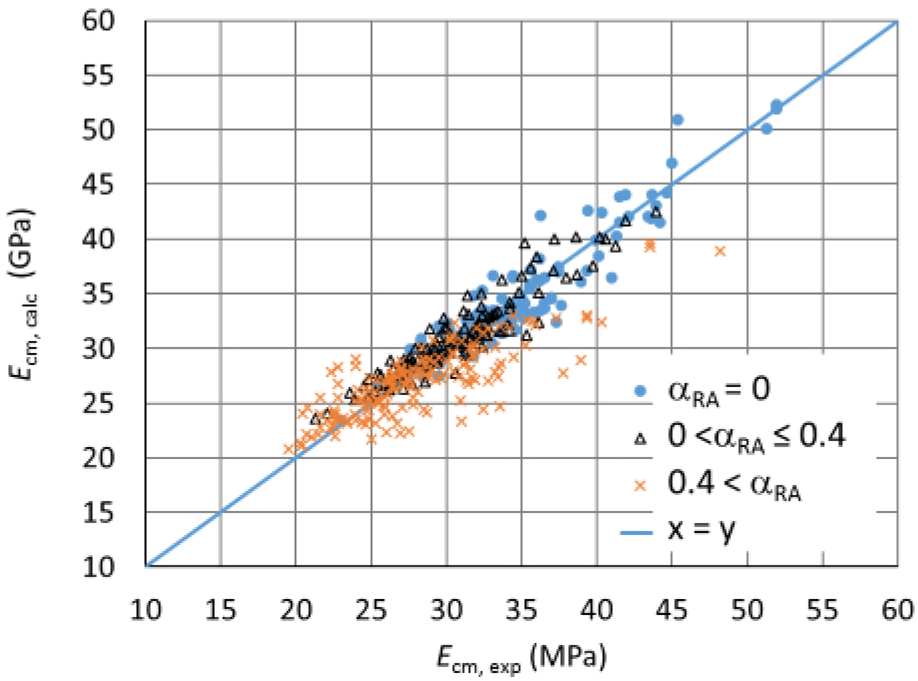

Figure 4. Relationship between experimental and calculated values of the modulus of elasticity calculated using Equation (9) for 425 concretes with different amounts of RA. Mean error 1.7 GPa (reduced to 1.3 GPa if α_{RA} is less than 0.4 and 1.2 GPa if $\alpha_{RA} = 0$).

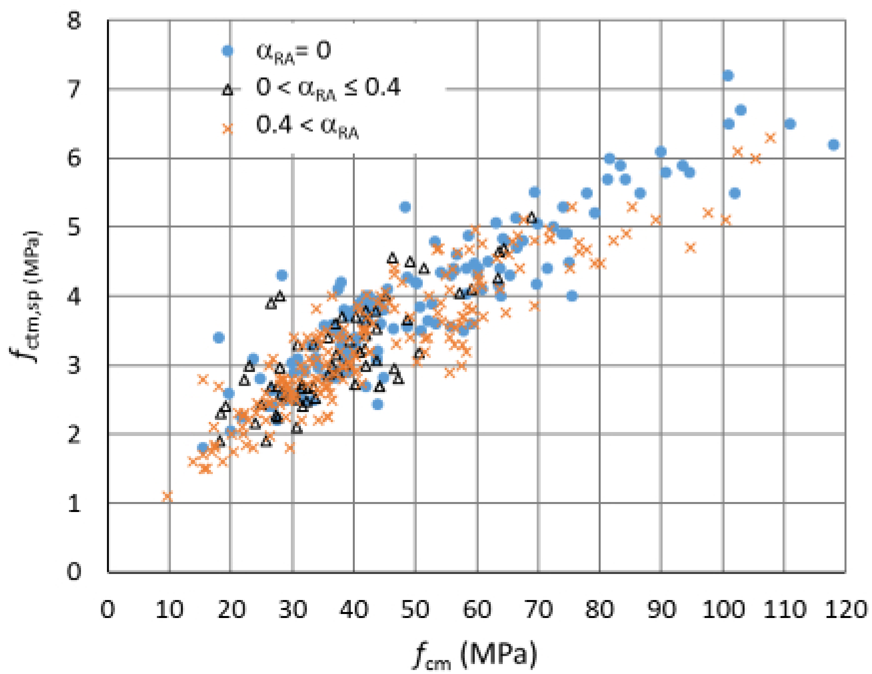

Figure 5. Relationship between tensile splitting strength and compressive strength for 393 concretes covering a range of f_{cm} between 10 and 118 MPa (94 with $f_{ck} \geq 50$ MPa), with different amounts of RA (137 with $\alpha_{RA} = 0$, 62 with $0 < \alpha_{RA} \leq 0.4$ and 194 with $0.4 < \alpha_{RA} \leq 1$).

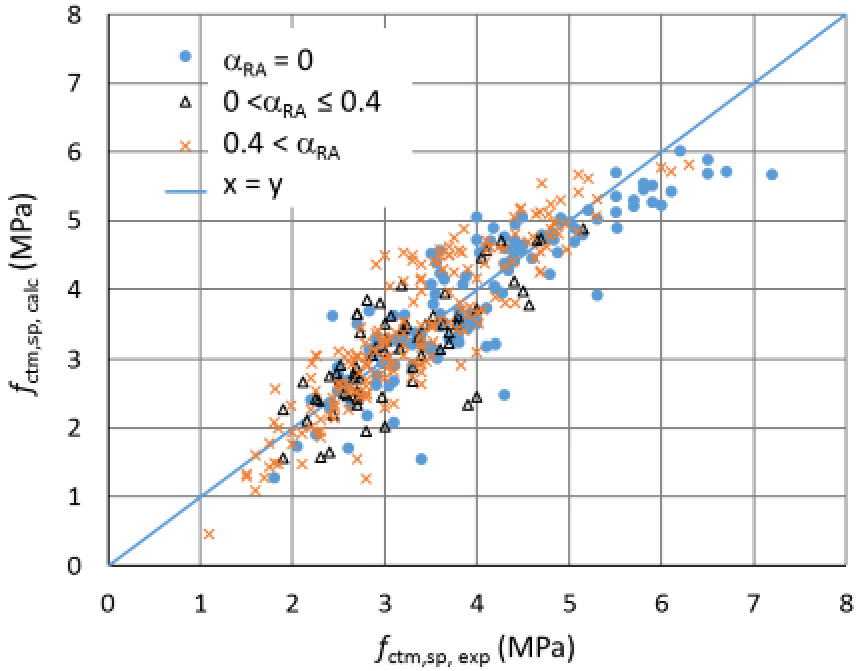

Figure 6. Comparison between experimental and theoretical values of splitting tensile strength calculated from Equations (11) and (12). The mean error for mixes with NA is 0.4 MPa (10.7%), whereas it is 0.38 MPa (12.6%) when $0 < \alpha_{RA} \leq 0.4$ and 0.41 (13.1%) when $0.4 < \alpha_{RA}$. The global mean error is 0.4 MPa.

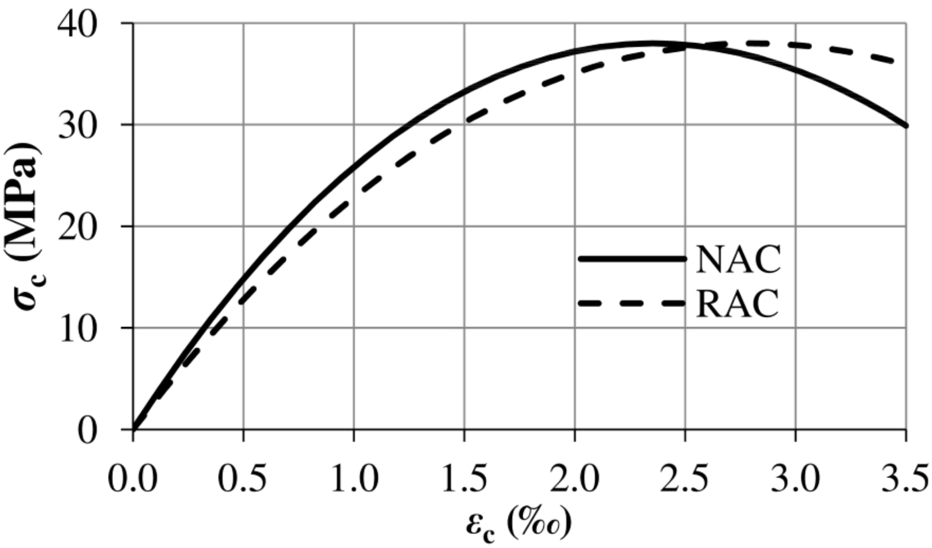

Figure 7. Stress–strain relationship for a C30/37 NAC and RAC with α_{RA} = 0.6.

80x47mm (300 x 300 DPI)

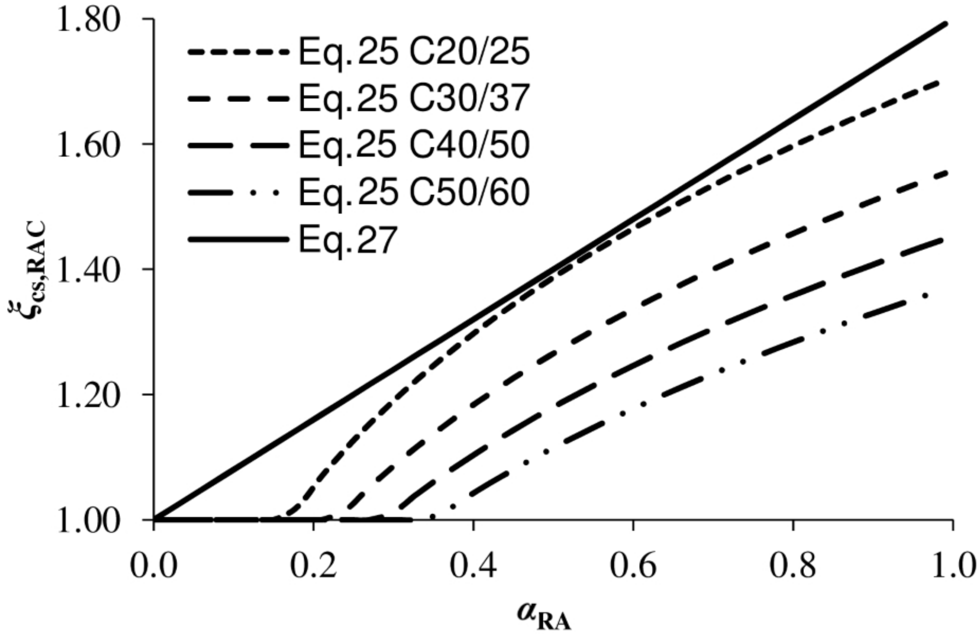

Figure 8. Comparison of shrinkage correction coefficients $\xi_{cs,RAC}$ using Equations (25) and (27).

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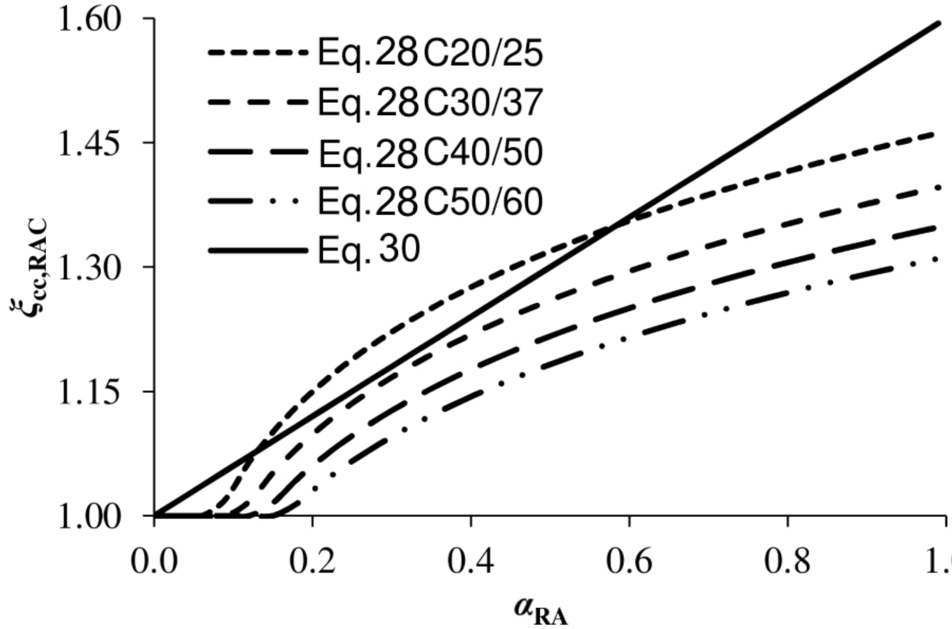

Figure 9. Comparison of creep correction coefficients $\xi_{cc,RAC}$ using Equations (28) and (30).

80x55mm (300 x 300 DPI)

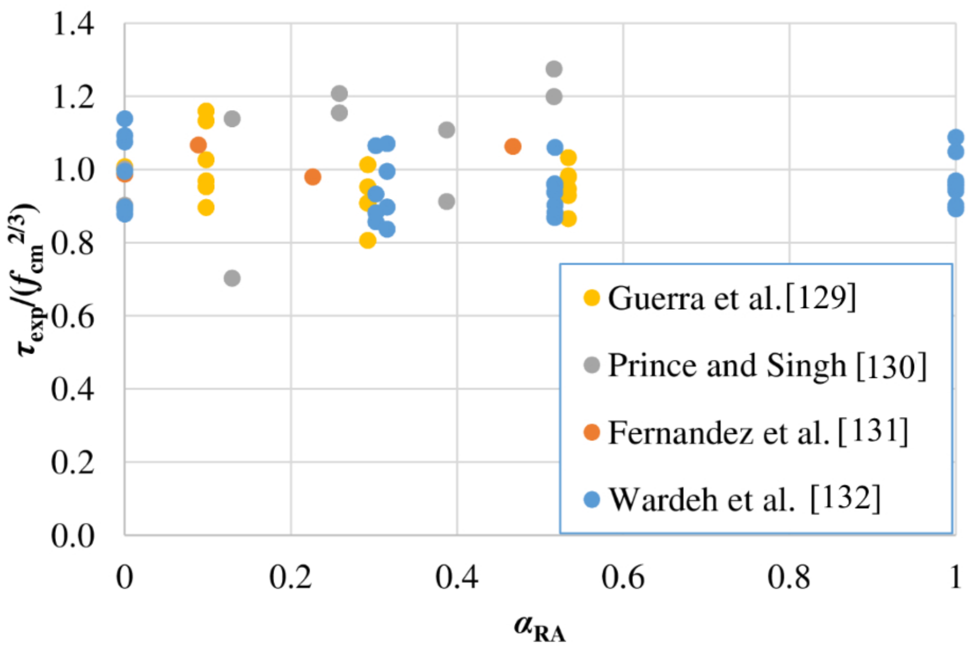

Figure 10. Evolution of the normalized ratio $\tau_{exp}/(f_{cm})2/3$ with α_{RA}.

80x52mm (300 x 300 DPI)