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Technical Paper

New Eurocode 2 provisions for recycled aggregate concrete and their implications for the design of one-way slabs

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ABSTRACT

A significant amount of research has been performed on recycled aggregate concrete (RAC), both on the material and structural level. This has enabled the formulation of material and structural resistance models that can be safely and reliably used for the structural design of RAC members and the new Eurocode 2 (EC2) will contain an informative annex detailing provisions for the design of RAC. Thus, an increased market uptake of recycled aggregate (RA) can be achieved, leading to potential sustainability improvements of concrete structures. In order to familiarize designers with the new provisions for RAC, this paper presents an example of one-way slab design using varying RA substitution ratios, as well as a parametric study on the implications of RAC provisions on slab slenderness. The results of this study show that RAC one way slabs can be successfully designed using EC2. Although such slabs might require larger depths than natural aggregate concrete slabs, their applicability in the typical slenderness range is possible.

1 Introduction

The movement towards increasing sustainability of all aspects of human activity is gaining momentum, from the UN 2030 Agenda [1] to the European Green Deal [2]. Among these initiatives, achieving the sustainability of the construction industry is critical, considering its global environmental impact and social importance [3,4]. This goal can be reached by transforming the construction sector into full circularity [5].

Within these plans, the “greening” of concrete is crucial considering its production of over 25 billion tons per year [6]. One important aspect is the use of aggregates in concrete production: on the one side, over 40 billion tons of natural aggregate (NA) are produced annually [7] and, on the other side, construction and demolition waste leave behind immense quantities of waste [8]. Therefore, an option that has gained importance and helps address both challenges is the recycling of construction and demolition waste (CDW) to produce recycled aggregate (RA) which can later be used to replace NA and produce recycled aggregate concrete (RAC).

The use of RA to produce structural RAC has been heavily researched over the past decades and significant knowledge of material and structural behavior of RAC has been generated. In order to leverage this knowledge and

increase the market uptake and implementation of RAC, new structural design standards that provide design guidelines for RAC are necessary.

In Europe, the European Standardization Organization (CEN) has the mandate to develop structural design codes (Eurocodes). Currently, a new version of Eurocode 2 prEN1992-1-1 (EC2) for the design of concrete structures is being developed and will contain an annex with special provisions for the structural design of RAC members [9]. Such an innovation will enable designers to safely and reliably design and construct RAC structures.

Therefore, it is first necessary to familiarize the users of EC2 with these new provisions and assess their implications on the design of RAC members. For this purpose, a calculation example and parametric analysis of one-way reinforced RAC slab design is presented. One-way slabs are chosen as very common elements in building structures, that are typically governed by serviceability criteria which are of great importance in RAC design. Hence, first the provisions of EC2 for RAC are presented, after which one-way slab ultimate limit state (ULS) and serviceability limit state (SLS) design is presented and discussed, followed by concluding remarks.

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2 Annex N of prEN1992-1-1

The provisions for RAC in EC2 are given in Annex N. An extensive background to Annex N can be found in the study by Tošić et al. [10], whereas herein it will be only briefly presented. Namely, as a CEN standard, EC2 is normatively linked to other standards, in this case the standard for concrete EN 206 [11] and for aggregates for concrete EN 12620 [12].

Firstly, EN 12620 presents a composition-based classification of coarse RA (particle size > 4 mm) into aggregates composed of crushed concrete (R_c), unbound stone (R_u), crushed brick (R_b), bituminous material (R_a), glass (R_g), floating material (FL) and other (X). The standard also provides a nomenclature for RA based on the percentages of each component type. For example, R_{c90} is RA that contains $\geq 90\%$ of crushed concrete (R_c), whereas R_{b30-} is RA that contains $< 30\%$ of crushed brick (R_b).

Secondly, EN 206 classifies coarse RA into two types: Type A and Type B, based on their composition:

- Type A (R_{c90} , R_{cu95} , R_{b10-} , R_{a1-} , FL_{2-} , XR_{g1-}) and
- Type B (R_{c50} , R_{cu70} , R_{b30-} , R_{a5-} , FL_{2-} , XR_{g2-})

Further, EN 206 provides certain limits for coarse RA incorporation based on environmental exposure classes (e.g. 50% for X0). However, these limits are imposed so that no change of RAC properties is assumed relative to a reference natural aggregate concrete (NAC). Therefore, what is needed are limits on RA substitution that are accompanied by material and structural models that take into account the effects of RA incorporation. In this way, the provisions of Annex N were formulated.

The basic variable that defines the effect of RA on RAC properties is the RA substitution ratio α_{RA} defined as the “quantity of fine and coarse recycled aggregate/total quantity of aggregate” (i.e. varying from 0 to 1). This way, the incorporation of fine RA (particle size ≤ 4 mm) with future revisions of EN 206 and EN 12620 is also facilitated.

For lower RA substitution ratios ($\alpha_{RA} \leq 0.2$ for reinforced concrete, RC) it is assumed that there are no changes to RAC properties. For higher RA substitution ratios ($0.2 < \alpha_{RA} \leq 0.4$ for RC, $\alpha_{RA} \leq 0.2$ for prestressed concrete, PC), special provisions given in Table 1 may be applied. For even higher substitution ratios ($\alpha_{RA} > 0.4$ for RC and $\alpha_{RA} > 0.2$ for PC) properties of RAC should be measured. All of the above stated is valid for Type A RA (according to EN 206 [11]), whereas for Type B, the limiting substitution ratios should be decreased by 50%.

As can be seen from Table 1, the largest impact of RA incorporation is on elastic and long-term RAC properties, i.e. modulus of elasticity, shrinkage and creep. On the structural level, beside an effect on the reduction of the roughness zone in shear cracks, there is a reduction of tension stiffening that impacts deflection control. Finally, for durability, the new EC2 allows either the use of current environmental exposure class-based concrete covers or the use of a new performance-based classification based on “exposure resistance classes” (ERC). The use of ERCs means that

concrete cover is directly determined based on the concrete resistance class (determined by testing or deemed-to-satisfy requirements on concrete mix design). Therefore, in this case, no changes in cover are necessary between NAC and RAC that belong to the same ERC. However, as countries will be able to opt out of the use of ERCs, provisions for increasing the minimum concrete cover due to durability are provided for exposures to carbonation and chloride ingress.

3 Design of one-way reinforced RAC slabs according to Annex N

3.1 Description of one-way reinforced RAC slab parametric study

To analyze the implications of Annex N provisions on RAC design, a simply supported one-way slab was selected (considering a 1-m wide strip, i.e. $b = 1000$ mm). The span of the slab L was constant and equal to 6 m. The effective depth d was determined by considering L/d ratios from 15 to 25. In this way, the range of slenderness limits prescribed by prEN1992-1-1 for one-way elements was covered. In this way, effective depths ranged from 240 to 400 mm.

The slab was considered as an indoor element in a building structure. Hence, an environmental exposure class XC3 was assumed and a nominal cover $c_{nom} = c_{min,dur}$ equal to 20 and 25 mm for NAC and RAC, respectively. Then, the reinforcement center of gravity d_1 was adopted as 30 and 35 mm for NAC and RAC, respectively and the overall height $h = d + d_1$. A relative humidity of 50% was adopted.

Two concrete classes were considered: C25/30 and C50/60 to cover both extremes of RAC applicability according to Annex N. Finally, three concretes were considered for each class: $\alpha_{RA} = 0, 0.2$ and 0.4 , i.e. NAC and RAC at the lower and upper limit of RA substitution ratios compatible with provisions in Table 1. Herein, the concretes were denominated NAC, RAC 0.2 and RAC 0.4.

The load on each element consisted of self-weight (determined from slab thickness and considering the adjustment for RAC in Table 1), additional dead load of 1.5 kN/m² and a live load of 3.0 kN/m², with a quasi-permanent combination coefficient $\psi_2 = 0.3$. Reinforcement B500B was considered and a service life of 50 years. In total, 66 cases were generated (11 L/d ratios per concrete, 2 concrete classes and 3 RA substitution ratios).

3.2 Comparison of the design of RAC and NAC one-way slabs

First, ULS design was performed on each slab by calculating the necessary longitudinal reinforcement for ensuring flexural resistance, $A_{s,ULS}$. In this part of design, there are practically no differences between NAC and RAC. Although RAC slabs are 5 mm thicker than NAC ones (for a given L/d), this increase is offset by a reduced self-weight so that the differences in bending moments and $A_{s,ULS}$ reinforcement do not exceed 1%.

Table 1. Proposed expressions for RAC design properties.

RAC property	Correction for RAC
Density	$\rho_{\text{RAC}} = 2.50 - 0.22 \cdot \alpha_{\text{RA}}$
Modulus of elasticity	$E_{\text{cm}} = k_E \cdot (1 - 0.25 \cdot \alpha_{\text{RA}}) \cdot f_{\text{cm}}^{1/3}$
Shrinkage strain	$\varepsilon_{\text{cs,RAC}} = (1 + 0.8 \cdot \alpha_{\text{RA}}) \cdot \varepsilon_{\text{cs}}$
Creep coefficient	$\varphi_{\text{RAC}} = (1 + 0.6 \cdot \alpha_{\text{RA}}) \cdot \varphi$
Peak strain	$\varepsilon_{\text{c1}} = (1 + 0.33 \cdot \alpha_{\text{RA}}) \cdot 0.7 \cdot f_{\text{cm}}^{1/3} \leq 2.8\text{‰}$
Ultimate strain	$\varepsilon_{\text{cu1}} = (1 + 0.33 \cdot \alpha_{\text{RA}}) \cdot [2.8 + 14 \cdot (1 - f_{\text{cm}}/108)^4] \leq 3.5\text{‰}$
Shear strength	$\tau_{\text{Rd,c}} = (1 - 0.2 \cdot \alpha_{\text{RA}}) \cdot \frac{0.66}{\gamma_V} \cdot \left(100 \cdot \rho_l \cdot f_{\text{ck}} \cdot \frac{d_{\text{dg}}}{d_v} \right)^{1/3}$
	$\tau_{\text{Rd,c,min}} = (1 - 0.2 \cdot \alpha_{\text{RA}}) \cdot \frac{11}{\gamma_V} \cdot \sqrt{\frac{f_{\text{ck}} d_{\text{dg}}}{f_{\text{yd}} d_v}}$ The parameter d_{dg} taking account of concrete type and its aggregate properties shall be assumed as $d_{\text{dg}} = 16 \text{ mm}$.
Deflection control	$\zeta = 1 - \beta_{\text{tRA}} \cdot \left(\frac{\sigma_{\text{sr}}}{\sigma_s} \right)^2$
	where $\beta_{\text{tRA}} = 1.0$ for single, short-term loading $\beta_{\text{tRA}} = 0.25$ 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 $c_{\text{min,dur}}$ 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 $\alpha_{\text{RA}} > 0$, the values of $c_{\text{min,dur}}$ should be increased by 5 mm for exposure to carbonation and 10 mm for exposure to chloride ingress

ρ_{RAC} – density of RAC; E_{cm} – modulus of elasticity; k_E – factor dependent on the type of NA (can be taken as 9500);
 f_{cm} – mean compressive strength of concrete; $\varepsilon_{\text{cs,RAC}}$ – shrinkage strain of RAC; ε_{cs} – shrinkage strain of NAC;
 φ_{RAC} – creep coefficient of RAC; φ – creep coefficient of NAC; $\tau_{\text{Rd,c}}$ – shear stress resistance of members without shear reinforcement;
 γ_V – partial factor for shear resistance; ρ_l – longitudinal reinforcement ratio; f_{ck} – characteristic compressive strength of concrete;
 d_{dg} – size parameter describing shear failure zone roughness; d_v – shear-resisting effective depth;
 $\tau_{\text{Rd,c,min}}$ – minimum shear stress resistance; ζ – tension stiffening distribution coefficient; σ_{sr} – stress in tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking; σ_s – stress in tension reinforcement;
 $c_{\text{min,dur}}$ – minimum concrete cover due to durability requirements

Next, the shear resistance of the slabs were calculated according to the expressions in Table 1. According to prEN1992-1-1, $\gamma_V = 1.4$ and $d_{\text{dg}} = 16 \text{ mm} + D_{\text{lower}} \leq 40 \text{ mm}$, where D_{lower} is the “smallest value of D for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete” [11]. In this case, D_{lower} was adopted as 8 mm (considering the smallest coarsest fraction to be 4/8 mm) so that d_{dg} was 24 mm for NAC and—due to the limitation in Table 1—16 mm for RAC. The design strength of reinforcement f_{yd} was obtained as $f_{\text{yk}}/1.15$.

Figure 1 presents the ratio between shear resistance ($\tau_{\text{Rd,c}} \cdot b \cdot d$) and shear action (calculated as $q_{\text{Ed}} \cdot L/2$, where q_{Ed} is the design load), plotted against the L/d ratio.

It can be seen that shear strength is satisfied for all concrete with a significant margin, although there is a notable reduction between NAC and RAC due to the limitation of d_{dg} ; the effect of α_{RA} is minor as can be seen from the differences between RAC 0.2 and RAC 0.4. The difference between NAC and RAC is approximately 25% for both concrete classes.

In the second stage deflections a were calculated and compared with permissible deflections $a_{\text{lim}} = L/250$. For this purpose, the ζ -method of interpolating deflections was applied:

$$a = \zeta \cdot a_2 + (1 - \zeta) \cdot a_1 \quad (1)$$

Figure 1. Ratio of shear resistance to shear action vs. L/d for concrete C25/30 (left) and C50/60 (right)

where a_1 and a_2 are deflections in the uncracked and fully-cracked state, respectively. The deflections in states 1 and 2 are composed of a component due to load a_{load} and a component due to shrinkage a_{cs} and are calculated as

$$a_{load,i} = K \cdot \frac{M \cdot L^2}{E_{c,eff} \cdot I_{i,i}} \quad (2)$$

$$a_{cs,i} = \delta_{cs} \cdot \varepsilon_{cs}(t, t_s) \cdot \frac{S_{i,i} \cdot L^2}{I_{i,i} \cdot 8} \quad (3)$$

where i can take values 1 or 2; K depends on the statical system (e.g., 5/48 for a simply supported beam under uniformly distributed load); $I_{i,i}$ is the moment of inertia of the transformed section; $S_{i,i}$ is the first moment of area of the reinforcement about the transformed section's centroid; $\varepsilon_{cs}(t, t_s)$ is the concrete shrinkage strain at time t with drying initiation at time t_s ; δ_{cs} depends on the statical system (e.g., 1 for a simply supported beam); and the effect of creep is

taken into account using the effective modulus of elasticity $E_{c,eff}$:

$$E_{c,eff} = \frac{1.05 \cdot E_{cm}}{1 + \varphi(t, t_0)} \quad (4)$$

where $\varphi(t, t_0)$ is the creep coefficient at time t for concrete loaded at time t_0 . For simplicity, in this study, a single load application time was adopted for all loads, i.e. $t_0 = 28$ days. Both shrinkage strain and creep coefficient were first calculated using the prEN1992-1-1 models for NAC and then, for RAC, corrected using adjustments provided in Table 1.

The results are shown in Figure 2 in terms of the a/a_{lim} ratio vs. the L/d ratio. This representation allows easy detection of L/d ratios above which permissible deflections are exceeded (shown by the dotted horizontal line). It should be noted that for all slabs, only the ULS-necessary reinforcement $A_{s,ULS}$ was adopted.

Figure 2. Ratio of deflection to permissible deflection vs. L/d for concrete C25/30 (left) and C50/60 (right)

From the figure, it can easily be seen that the deformability of RAC slabs is predicted as being larger than for NAC slabs, with only minor differences between RAC slabs themselves. Namely, for concrete C25/30 the lines for all three concretes stay parallel, with a constant “absolute” difference $((a/a_{lim})_{RAC} - (a/a_{lim})_{NAC})$ of the a/a_{lim} ratio between RAC and NAC: for RAC 0.2 this difference is 0.34–0.42 and for RAC 0.4 it is 0.46–0.50. The figure also demonstrates that NAC slabs can satisfy deflections up to $L/d = 20$, whereas this limit is around 16.5–17.0 for RAC 0.2 and 0.4.

For C50/60, because of its higher tensile strength, all slabs remain uncracked for certain L/d ratios (until 22 for NAC and until 16 for RAC). The earlier cracking of RAC is due to the decreased tension stiffening effect (Table 1). From $L/d = 22$, the lines for RAC and NAC become parallel with an absolute difference of 0.55–0.60 for RAC 0.2 and 0.60–0.70 for RAC 0.4. Finally, it can be seen that NAC can satisfy deflections for all considered L/d values, whereas for RAC, the limit is at approximately $L/d = 22.5$ –23.0.

Although the deformability of one-way reinforced RAC slabs seems significantly larger than that of NAC slabs, it should be noted that the results demonstrate their applicability in building structures. Namely, even for a very

low concrete strength class C25/30 and using only the ULS-required reinforcement, one-way RAC slabs satisfy deflections up to $L/d = 17$ and for C50/60 this increases up to 22.

3.3 Strategies for compliance with deflection limits for RAC and NAC one-way slabs

In order to further analyze and quantify the differences in RAC and NAC one-way slab behavior, two strategies were investigated in this section. The objective was to determine necessary changes to RAC and NAC one-way slabs in order for them to comply with deflection limits (i.e., achieving $a/a_{lim} \leq 1.0$).

First, it was investigated how much longitudinal tensile reinforcement needs to be increased in order for each slab to comply with deflection limits. If the original value of the a/a_{lim} ratio was already below 1, no changes were needed. The necessary multiplication factors for longitudinal reinforcement are presented in Table 2. For certain cases, deflection compliance cannot be achieved by increasing reinforcement and these cases are marked as “n/a”.

Table 2. Necessary increases in tensile reinforcement ($A_{s,prov}/A_{s,ULS}$) in order to achieve $a/a_{lim} \leq 1.0$.

L/d	C25/30			C50/60		
	NAC	RAC 0.2	RAC 0.4	NAC	RAC 0.2	RAC 0.4
15	1.00	1.00	1.00	1.00	1.00	1.00
16	1.00	1.00	1.00	1.00	1.00	1.00
17	1.00	1.00	1.10	1.00	1.00	1.00
18	1.00	1.22	1.65	1.00	1.00	1.00
19	1.00	1.61	2.58	1.00	1.00	1.00
20	1.00	2.69	10.39	1.00	1.00	1.00
21	1.16	3.93	<i>n/a</i>	1.00	1.00	1.00
22	1.71	<i>n/a</i>	<i>n/a</i>	1.00	1.00	1.00
23	2.48	<i>n/a</i>	<i>n/a</i>	1.00	1.00	1.23
24	7.82	<i>n/a</i>	<i>n/a</i>	1.00	1.48	2.22
25	<i>n/a</i>	<i>n/a</i>	<i>n/a</i>	1.00	2.07	<i>n/a</i>

It can be considered that increases of reinforcement up to 100% (i.e. a multiplication factor of 2.0) are acceptable in these cases as these slabs have very low reinforcement ratios when $A_{s,ULS}$ is considered (0.15%–0.30%). In this way, the use of RAC slabs can be extended up to L/d of 19 and 18 for RAC 0.2 and RAC 0.4, respectively, for C25/30 and up to L/d of 24 and 23, respectively, for C50/60.

Another option for complying with deflections is the increase of slab thickness, preferably in combination with increasing reinforcement. For this purpose, all cases from Table 2 that could not comply with deflections with an increase in reinforcement of 100% (given in italics in Table 2), were limited to $A_{s,prov}/A_{s,ULS} = 2.0$, after which effective depth d was increased until deflection limits were satisfied.

For C25/30 and NAC, the cases with $L/d > 23$ required a slab effective depth of 275 mm. For C25/30 and RAC 0.2 and the cases with $L/d > 20$, $d = 315$ mm was required, whereas

for C25/30 and RAC 0.4 with $L/d > 19$, $d = 330$ mm was required. For NAC and C50/60, no increase in d was necessary, for the same concrete class and RAC 0.2 with $L/d = 25$, $d = 245$ mm was needed and for RAC 0.4 and $L/d > 24$, $d = 255$ mm. Finally, the required ratios d_{RAC}/d_{NAC} for achieving compliance with deflections are shown in Table 3. Table 3. Increases in RAC slab effective depth relative to NAC (d_{RAC}/d_{NAC}) in order to achieve $a/a_{lim} \leq 1.0$.

It can be seen from the table that for the lowest concrete class C25/30, the necessary increase in effective depth is limited to 22% for RAC 0.4, whereas this is only 6% for C50/60. Therefore, it can be concluded that when using RAC slabs, higher strength classes should be aimed for, as well as lower RA substitution ratios and applications requiring lower L/d ratios.

Table 3. Increases in RAC slab effective depth relative to NAC ($d_{\text{RAC}}/d_{\text{NAC}}$) in order to achieve $a/a_{\text{lim}} \leq 1.0$.

L/d	C25/30		C50/60	
	RAC 0.2	RAC 0.4	RAC 0.2	RAC 0.4
15	1.00	1.00	1.00	1.00
16	1.00	1.00	1.00	1.00
17	1.00	1.00 ^a	1.00	1.00
18	1.00 ^a	1.00 ^a	1.00	1.00
19	1.00 ^a	1.03 ^b	1.00	1.00
20	1.05 ^b	1.10 ^b	1.00	1.00
21	1.09 ^b	1.14 ^b	1.00	1.00
22	1.15 ^b	1.20 ^b	1.00	1.00
23	1.15 ^b	1.22 ^b	1.00	1.00 ^a
24	1.15 ^b	1.22 ^b	1.00 ^a	1.02 ^b
25	1.15 ^b	1.22 ^b	1.02 ^b	1.06 ^b

^adeflections satisfied only with increased reinforcement

($A_{s,\text{prov}}/A_{s,\text{ULS}} < 2.0$)

^b $A_{s,\text{prov}}/A_{s,\text{ULS}} = 2.0$

4 Conclusions

This study presents the new provisions of EC2 for the design of RAC structures, as well as the study of the implications of these provisions on the design of one-way slabs. For this purpose, a parametric study of RAC one-way slabs is performed with ULS and SLS design carried out. Additionally, the higher deformability of RAC one-way slabs in terms of deflections is discussed together with strategies for satisfying deflection criteria. Based on the findings of the study, the following can be concluded:

- The provisions of Annex N of the new EC2 provide a detailed framework for the structural design of RAC members, considering differences in both material and structural properties between RAC and NAC.
- The ULS design of RAC simply-supported one-way slabs does not provide any substantial differences compared with NAC design. The only difference exists in shear resistance calculation; however, for the range of L/d values considered in this study, and the adopted loading, there are no implications of these differences.
- Significant differences exist in the deformability (i.e., deflection behavior) between RAC and NAC one-way simply supported slabs, particularly for the low concrete strength class C25/30. Nonetheless, RAC slabs remain applicable in the typical slenderness ranges for this type of element.
- Increasing reinforcement above the ULS-necessary amount can in a limited number of cases lead to satisfying deflection limits. However, in the majority of cases, this measure needs to be combined with an increase in effective depth. For concrete class C25/30, the increase of RAC effective depth, relative to NAC can be up to 22%, whereas it is only 6% for C50/60. Therefore, RAC one-way slabs should be used in higher strength classes whenever possible, as well as with lower RA substitution ratios ($\alpha_{\text{RA}} = 0.2$).

While indicative, the conclusions of this study are not conclusive and cannot be extrapolated beyond the range of parameter values adopted herein. Further and more detailed studies are needed to cover a wider range of statical systems, member types, loading arrangements, etc. Nonetheless, the study provides a first step towards familiarizing designers with the new provisions of EC2 for the structural design of RAC.

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