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ORIGINAL ARTICLE

Influence of recycled concrete aggregates on the shear strength of reinforced concrete beams

Influência dos agregados reciclados de concreto na resistência ao cisalhamento de vigas de concreto armado

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Abstract: This research evaluates the influence of the replacement of natural coarse aggregates (NCA) by recycled concrete aggregates (RCA) on the shear strength of reinforced concrete beams. Experimental tests on six reinforced concrete beams with RCA replacement ratios of 0%, 30%, and 100% are presented. Furthermore, a database with results of 170 tests on beams with RCA is used to discuss adjustments in the recommendations presented by ABNT NBR 6118 to estimate the shear strength of reinforced concrete beams. According to the Demerit Points Classification (DPC) proposed by Collins, 80% of the theoretical results obtained using models I and II from the Brazilian code fall in an appropriate safety condition range, showing that the substitution of NCA by RCA has a low impact on the shear strength reinforced concrete beams.

Keywords: shear, recycled concrete aggregates, reinforced concrete.

Resumo: Esta pesquisa avaliou a influência da substituição de agregados graúdos naturais (AGN) por agregados reciclados de concreto (ARC) na resistência ao cisalhamento de vigas de concreto armado. Ensaios experimentais em seis vigas de concreto armado com taxas de substituição de ARC de 0%, 30% e 100% são apresentados. Além disso, um banco de dados com resultados de 170 ensaios em vigas com ARC é utilizado para discutir os ajustes nas recomendações apresentadas pela ABNT NBR 6118 para estimar a resistência ao cisalhamento de vigas de concreto armado. Segundo o *Demerit Points Classification* (DPC), proposto por Collins, 80% dos resultados teóricos obtidos com os modelos I e II da norma brasileira caem dentro de uma faixa considerada de segurança apropriada, mostrando que a substituição dos AGN por ARC tem baixa influência na resistência ao cisalhamento de vigas de concreto armado.

Palavras-chave: cisalhamento, agregados reciclados de concreto, concreto armado.

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1 INTRODUCTION

Civil construction is one of the leading industrial sectors and contributes significantly to economic growth and social development. In this context, the aggregate industry is an important segment, as according to Langer et al. [1], on a global level, it involves the exploration of significant quantities of non-renewable natural resources. The civil construction industry is also considered a relevant waste generating agent. Pinto [2] presents and discusses the high numbers of construction and demolition waste (CDW) production in Brazil. More recent data (see [3]), provided by the Brazilian Association of Public Cleaning and Special Waste Companies (ABRELPE), indicate that in 2018, the total

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quantity of CDW collected by the cities in the country was of 122,012 tonnes/day, resulting in a per capita generation of 0.585 kg/inhabitant/day.

The use of CDW in non-structural or structural concrete production has been the focus of intense scientific studies. This theme is also relevant considering the expected increase of concrete consumption and, therefore, of coarse aggregate, as highlighted by Arezoumandi et al. [4]. Rahal and Alrefaei [5] investigated the effect of replacing natural coarse aggregate (NCA) by RCA and did not observe any damaging effects in the shear strength of reinforced concrete beams for replacement rates until 16%. For beams with shear reinforcement, Ignjatović et al. [6] did not observe significant changes in both behaviour and shear strength for NCA to RCA replacement rates of 0, 50, and 100%.j

From a theoretical point of view, it is expected that the change of aggregates will affect the shear strength since it may alter the roughness of the failure plane, reducing the contribution of concrete through aggregate interlock. For structural elements with high flexural reinforcement ratio (ρ_l), these effects may be even more significant because crack widths are wider at failure. These variables (aggregate interlock and ρ_l) were the target of some classic shear investigations, such as those carried by Taylor [7] to Poli et al. [8], or more recent research, such as those presented by Ulaga [9] and Sagaseta and Vollum [10].

This paper presents an experimental and theoretical investigation about the influence of the replacement of NCA by RCA on the shear strength of reinforced concrete beams without and with shear reinforcement. These results are used to evaluate the need for adjustments in the current recommendations presented by ABNT NBR 6118 [11], in case it is used to predict the shear strength of reinforced concrete beams with the replacement of NCA by RCA.

2 LITERATURE REVIEW

The impact of the replacement of NCA by RCA in the compressive strength of concrete can vary significantly. Studies from [7] and [12] to [13] indicate a 30% reduction in the concrete strength due to the replacement of NCA by RCA. On the other hand, according to [14] and [15], the reduction can reach up to 76%. The loss of strength is more pronounced if recycled concrete aggregates from an unknown source are used. The use of RCA produced from the waste of high-strength concrete ($f_c \geq 50$ MPa) will result in a compressive strength comparable with those obtained with NCA, that according to Schubert et al. [16], is related to the aggregate's water absorption. Cordeiro et al. [17] describe that it is possible to optimize RCA characteristics by incorporating reactive and non-reactive fines, improving the performance of concretes made with RCA.

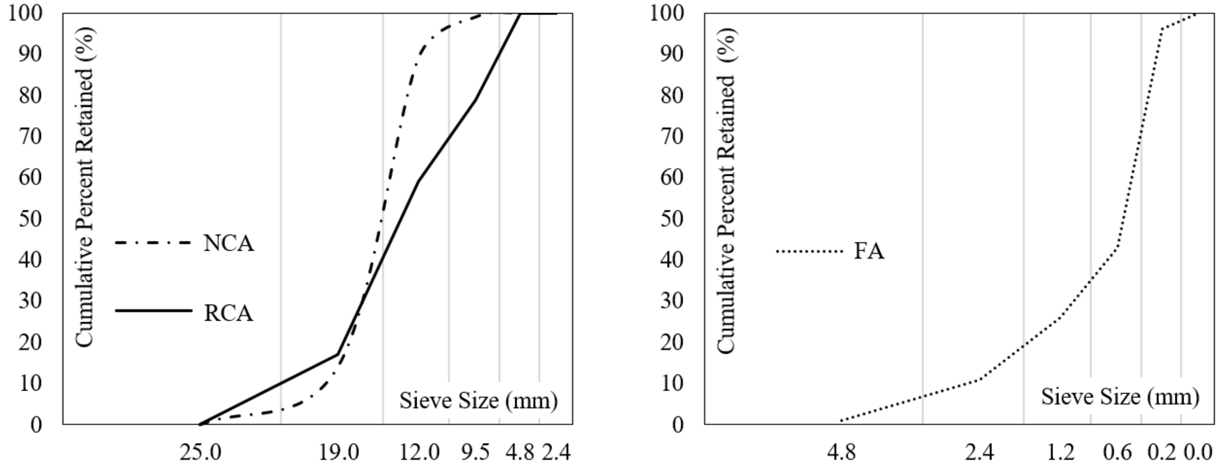
In structural terms, [6] and [18] concluded that there is a similarity in both the crack pattern and the cracking load for reinforced concrete beams without stirrups, regardless of whether the concrete was produced with NCA or RCA. [6] also reports that no significant reductions in the shear strengths were measured. However, according to Rahal and Alrefaei [5], replacements of 23% and 35% led to a shear strength reduction of 10% and 21%, respectively. In the case of beams with shear reinforcement, these authors did not measure significant reductions in the shear strength, regardless of the amount of RCA. The shear-span can be another parameter to be investigated, as Choi et al. [19] observed pronounced reductions in the shear strength due to RCA's use in beams with a lower a/d ratio. Regarding the design standards, González-Fonteboa and Martínez-Abella [20] observed a better correlation between theoretical estimates of shear strength and experimental results for beams without shear reinforcement, which is not allowed in practice, than for those with stirrups, pointing that this is a topic that deserves more scientific efforts.

3 MATERIALS AND METHODS

3.1 Experimental program

The concrete used to cast the tested beams was produced with 0, 30, and 100% replacement rates of NCA by RCA. Basaltic gravel with a maximum size of 25 mm was used as a natural coarse aggregate. As a fine aggregate, medium natural sand was used. The coarse recycled concrete aggregates were produced at the Civil Engineering Laboratory of the Federal University of Pará. For their production, the laboratory's structural concrete wastes were carefully selected to guarantee that only regular strength concrete ($f_c < 50$ MPa) was used as origin material.

The RCA production process consisted of grinding the wastes with a jaw crusher. After this procedure, all the material was sieved, following the recommendations of ABNT NBR NM 248 [21], and separated into different granulometric ranges. The methods adopted by the Brazilian and international standards, used for natural aggregates, can be unsuitable for RCA. For this reason, the methodology proposed by Leite [22] was used to determine the aggregate specific gravity of the RCA. Figure 1 shows the granulometric composition of the aggregates and their main characteristics.



a) The granulometric curve of coarse aggregate b) The granulometric curve of fine aggregate

	FA	NCA	RCA
Fineness Modulus	2.61	7	6.96
Maximum Size	4.8 mm	25 mm	25 mm
Specific Gravity	2.61 g/cm³	2.62 g/cm³	2.66 g/cm³
Unit Weight	1.67 g/cm³	1.34 g/cm³	1.11 g/cm³

c) Properties of coarse and fine aggregates

Figure 1. Aggregates characterization.

The concrete proportioning was set to achieve a C30 strength class and was based on the design curves presented by Santos et al. [23], using high early strength Portland cement CP-V ARI RS. The concrete's workability was measured by slump tests, following the recommendations presented by ABNT NBR NM 67 [24]. The slump values were defined as 15 ± 2 cm for both concrete with NCA and RCA. The wet curing of the concrete beams and specimens was initiated after the concrete's surface hardening. The beams and the cylindric samples used to characterize the concrete's mechanical properties were wet-cured for seven days.

Six reinforced concrete beams with a 2,200 mm length and rectangular section (180 mm width and 280 mm height) were tested. Two beams served as reference and were cast with NCA concrete, one without shear reinforcement and the other with closed stirrups. The rest of the beams were cast with concrete with 30% and 100% replacement of NCA by RCA, having or not shear stirrups. All the beams had ρ_l equal to 2.13%, about half of the maximum value allowed by the ABNT NBR 6118 [11]. The beams with stirrups had a shear reinforcement ratio of 0.10%. Table 1 and Figure 2 show the main characteristics of the tested beams.

Table 1. Characteristics of the tested beams.

Specimen	% of RCA	*d* (mm)	*a* (mm)	*a*/*d*	ø$_f$ (mm)	ø$_w$ (mm)	ρ_l (%)	ρ_w (%)
V0	0	246	620	2.52	20	0	2.13	0
W0						4.2		0.10
V30	30	246		2.52	20	0	2.13	0
W30						4.2		0.10
V100	100	246		2.52	20	0	2.13	0
W100						4.2		0.10

Note: b_w = 180 mm; h = 280 mm; L = 2200 mm; c = 20 mm; f_c = 30 MPa; f_{ys} = 500 MPa; α = 90º; $\rho_w = A_{sw} / (b_w \cdot s)$; $\rho_l = A_{sw} / (b_w \cdot d)$

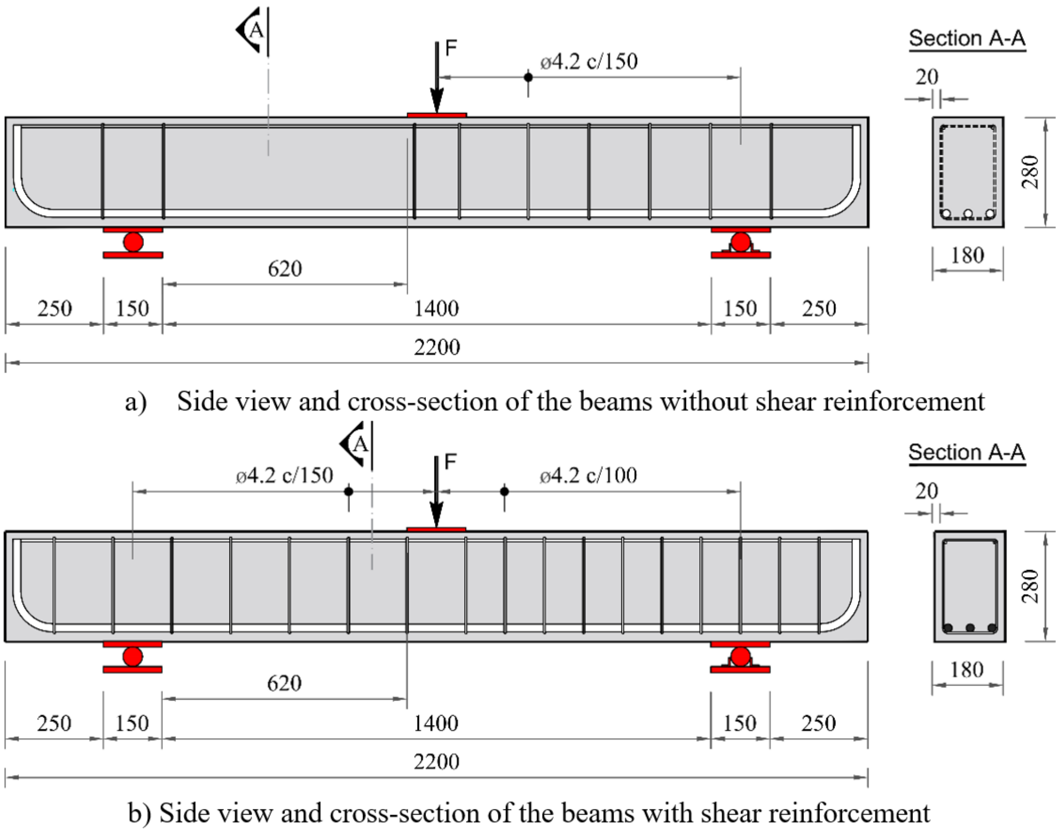


Figure 2. Flexural and shear reinforcement of the tested beams

The tested beams had two symmetrical spans with different amounts of shear reinforcement to guarantee that failure would occur in the weaker side where the strain gauges were placed. Figure 2 shows that the beams' left span is the weak side and was the focus of this experimental investigation. The beams denominated as “V” do not have stirrups on the left span and the beams denominated as “W” had closed stirrups made with 4.2 mm bars spaced at each 150 mm. For the flexural reinforcement, three steel bars with 20 mm of diameter distributed on a single layer were used, as shown in Figure 2.

Simply supported beams with three-point loading were tested. The supports were made with double “I” steel profiles in which two steel plates (150 mm wide and 20 mm thick) and one roller (40 mm diameter) were attached. The load was applied using a testing machine in 5 kN load steps, and its intensity was continuously measured by a load cell connected to an electronic acquisition data system. Figure 3 shows the testing system.

A potentiometric linear displacement transducer attached to a yoke was used to measure the vertical displacements (see Figure 4a). One pair of strain gauges was used to measure the flexural reinforcement strains, as shown in Figure 4b. Strains in the shear reinforcement were also measured by pairs of strain gauges attached to 3 layers of stirrups, as shown in Figure 4c. The results presented for the flexural and shear reinforcement strains refer to the mean values of the pairs of strain gauges.

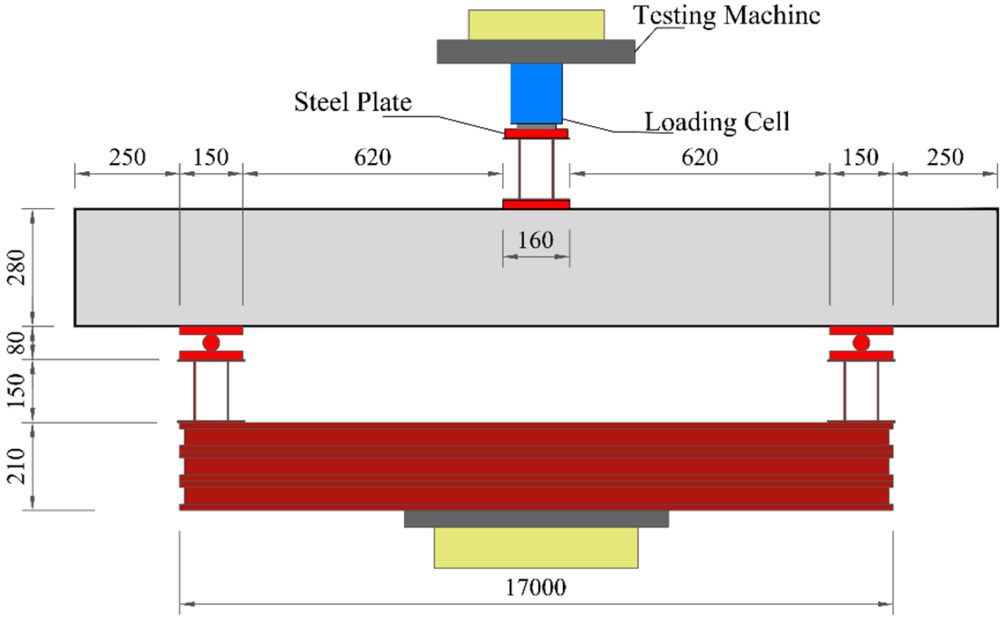


Figure 3. Testing setup

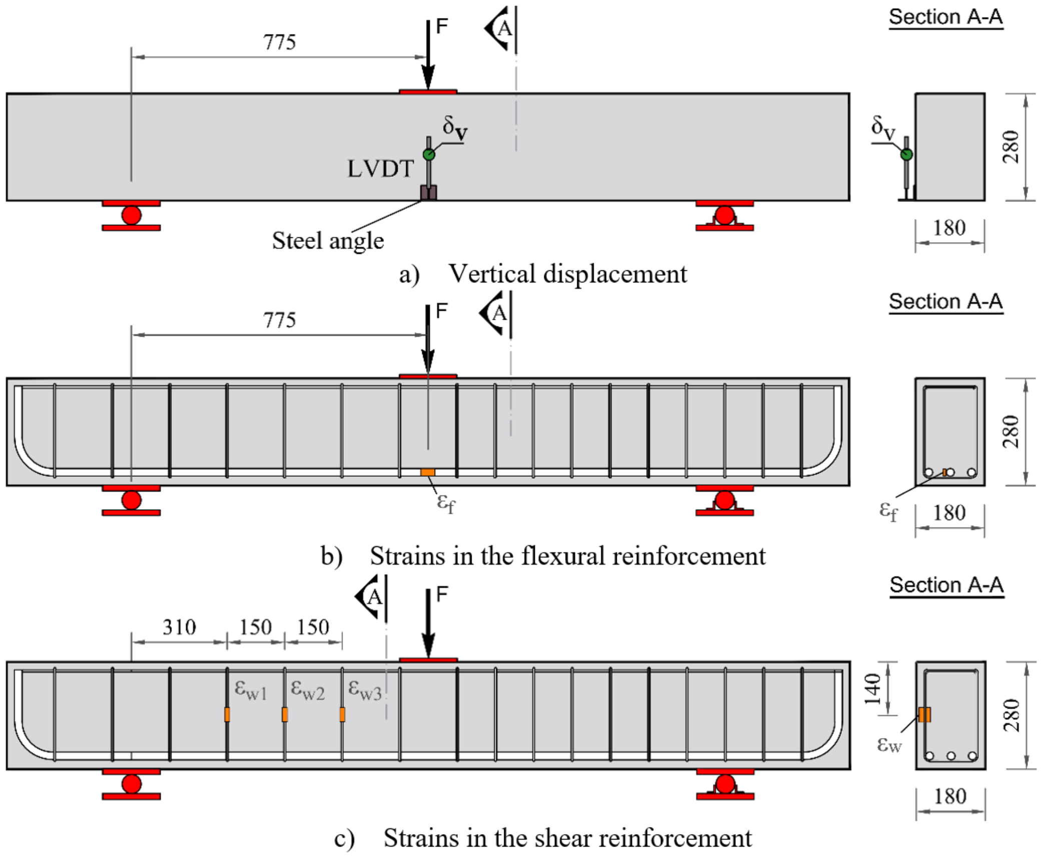


Figure 4. Instrumentation of the tested beams.

3.2 Shear strength of beams according to ABNT NBR 6118

ABNT NBR 6118 [11] considers that the shear strength of a beam (V_R) can be computed as the sum of the contributions given by the concrete shear resistant mechanisms (V_{Rc}) and the steel activated up to failure (V_{Rs}), as expressed in Equation 1. The Brazilian code presents two different models to calculate the contributions from concrete (V_{Rc}), shear reinforcement (V_{Rs}), and the maximum resistance due to concrete strut's crushing (V_{Rmax}). Model I assume that the concrete struts have an angle of θ = 45°, and the shear strength can be calculated considering Equations 1-4. Model II admits that the concrete strut angle can vary between 30º to 45º in relation to the beam's longitudinal axis. Thus, the shear strength can be obtained by using Equation 1 and Equations 5-7.

$$V_R = V_{Rc} + V_{Rs} \leq V_{R,max} \tag{1}$$

$$V_{Rc,I} = 0.6 \cdot f_{ctd} \cdot b_w \cdot d \tag{2}$$

$$V_{Rs,I} = \frac{A_{sw}}{s} 0.9 \cdot d \cdot f_{ywd} \cdot (sen\,\alpha + \cos\alpha) \tag{3}$$

$$V_{R,max\,I} = 0.27 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_{ck} \cdot b_w \cdot d \tag{4}$$

$$V_{Rc,II} = V_{Rc,I} \cdot \frac{V_{R,máxII} - V}{V_{R,máxII} - V_{Rc,I}} \leq V_{Rc,I} \tag{5}$$

$$V_{Rsw,II} = \frac{A_{sw}}{s} 0.9 \cdot d \cdot f_{yw} \cdot (\cot\alpha + \cot\theta) \cdot sen\,\alpha \tag{6}$$

$$V_{R,máxII} = 0.54 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_{ck} \cdot b_w \cdot d \cdot sen^2 \cdot (\cot\alpha + \cot\theta) \tag{7}$$

Where:

b_w is the smallest cross-sectional width; d is the effective depth of the beam; s is the spacing of the stirrups; $f_{ctd} = \frac{f_{ctk,inf}}{\gamma_c}$; $f_{ctk,inf} = 0.7 \cdot f_{ctm}$; $f_{ctm} = 0.3 \cdot f_c^{2/3}$ for concrete strength class varying from C20 to C50; $f_{ctm} = 2.12 \ln(1 + 0,11 \cdot f_{ck})$ for concrete C55 until C90; f_{yw} is the yield strength of the shear reinforcement, limited to 500 MPa in this paper.

3.3 Database

A database containing results of shear tests on reinforced concrete beams with the replacement of NCA by RCA. An extensive literature review was carried, and the following criteria were considered during the selection of the test results:

a) Only beams with shear span-to-depth ratio (a/d) greater than or equal to 2.5 were used, restricting the analyses and conclusions of this study to the case of slender beams;
b) Only beams made of concrete with compressive strength ranging from 20 MPa to 90 MPa were used to respect the scope of the ABNT NBR 6118 [11];
c) Only beams with ρl and ρw within the minimum and maximum limits prescribed by ABNT NBR 6118 [11] were used.

Following these ideas, a database with 170 beams (see Annex A) was developed. For the analyses and discussions, these beams were classified into three groups:

a) Group 1 contained beams with NCA to RCA replacement rates of less than 11%. There were 22 beams without shear reinforcement and 20 beams with closed stirrups;

b) Group 2 had beams with RCA replacement rates ranging from 11% to 50%. There were 32 beams without stirrups and 20 beams with stirrups.

c) Group 3 had beams with RCA replacement rates greater than 50%. There were 46 beams without shear reinforcement and 30 beams with stirrups.

This database, formed by results from tests presented in references [4], [5], [6], [16], [19], [20], [25], [26], [27], [28], [29], [30], [31] and [32], allows a comprehensive evaluation of the influence of the values of ρ_l, ρ_w, and the NCA to RCA replacement on the shear strength of reinforced concrete beams.

4 RESULTS AND DISCUSSION

4.1 Experimental tests

Tests on cylindrical specimens, 100 mm of diameter and 200 mm of length, were performed to determine the compressive strength of concrete, following the recommendations of the ABNT NBR 5739 [33]. Diametral compression tests on 100 x 200 mm cylindrical specimens were carried to determine the concrete's tensile strength, following the recommendations of the ABNT NBR 7222 [34]. The modulus of elasticity of the concrete was also measured, following the recommendations of ABNT NBR 8522 [35], in tests on cylindrical specimens with 150 mm of diameter and 30 mm of height.

The concrete's compressive and tensile strength's presented values represent the highest strength measured in a pair of specimens. For the modulus of elasticity, the results presented are the mean of the results obtained in three cylindric samples. The results of the characterization tests are presented in Figure 5.

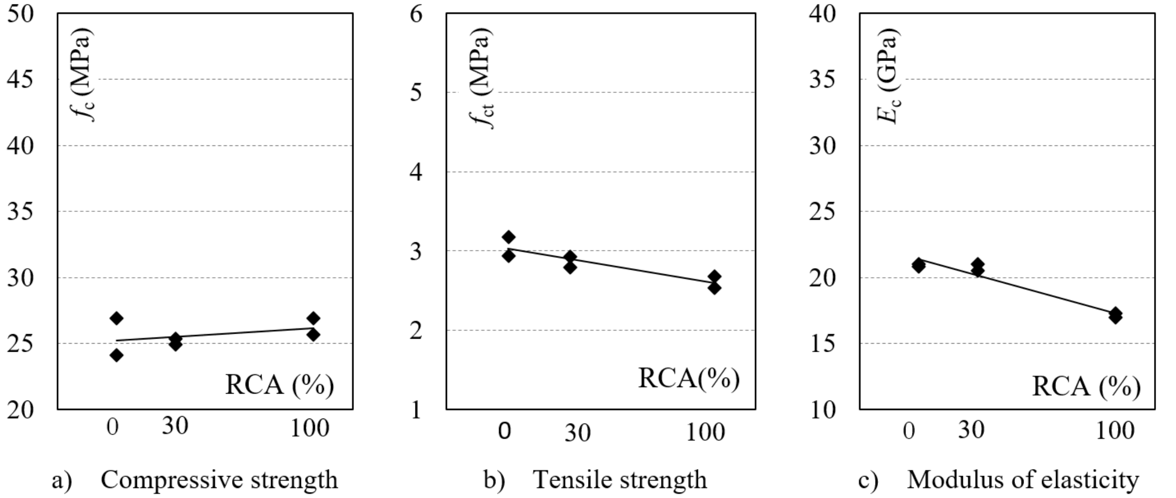


Figure 5 – Results of characterization tests.

The compressive strength results show that the concrete proportioning was successful, and the mean resistance was maintained regardless of the replacement ratio of NCA by RCA. The same was not observed for either the tensile strength or the modulus of elasticity of the concrete. Concerning the tensile strength, a reduction trend is noted for both replacement ratios of 30% and 100%. In general, an average reduction of 15% was measured in the tensile strength due to 100% replacement ratio of NCA by RCA. The substitution of 30% of RCA did not affect the modulus of elasticity results, but a replacement ratio of 100%, an average reduction of 20% was observed.

Three samples of each rebar size were submitted to axial tensile tests, following the recommendations of ABNT NBR ISO 6892 [36], to obtain the mechanical properties of the flexural and shear reinforcement. These results are summarized in Table 2 and are on the expected range of the steel bars commercialized in Brazil.

Table 2 – Mechanical properties of reinforcement bars

ø (mm)	f_{ys} (MPa)	ε_{ys} %	E_s (GPa)
4,2	610	3,02	202
20	546	2,80	194

Figure 6 presents the tested beams' response, measured by the relation between the applied shear force (V) and the vertical displacements (δ) in mid-spam. The behaviour of the beams without shear reinforcement was fragile, and the shear failure occurred abruptly, with low displacement levels. For these beams, it was not observed relevant reductions of their flexural stiffness due to the replacement of the NCA by RCA. The beams with shear reinforcement reached significantly higher loading levels before the ruin, allowing the observation of a reduction in flexural stiffness due to the replacement of NCA by RCA.

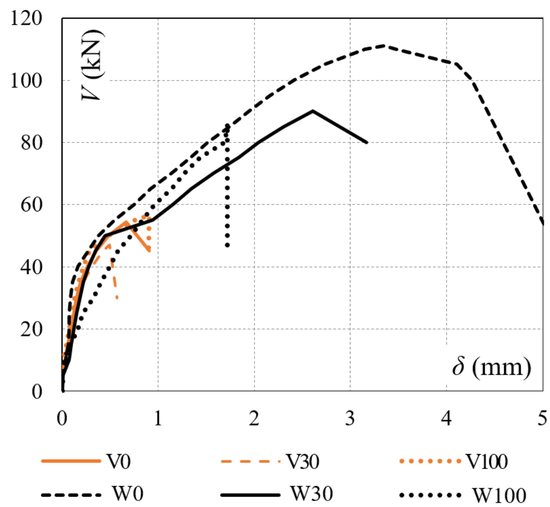


Figure 6 – V - δ response of the tested beams.

Table 3 shows the beams' measured strengths and compares the experimental results with theoretical predictions using models I and II from [11]. For beams without shear reinforcement, a significant reduction of the shear resistance was observed with the replacement of 30% of NCA by RCA. The same was not observed for a 100% replacement ratio, as V100 strength was almost the same as the reference beam (V0). For tests on beams with stirrups, a trend of strength reductions was experimentally observed. The substitution of NCA by RCA led to a 19% reduction of the shear strength of beam W30 and 22% for beam W100.

Table 3 – Experimental relations with design codes.

Experimental values		V_{Rcs} (kN)		V_u / $V_{Rcs.I}$	V_u / $V_{Rcs.II}$
Beams	V_u (KN)	I	II		
V0	54.33	53.6	53.6	1.01	1.01
V30	46.74	53.7	53.7	0.87	0.87
V100	56.57	58.6	58.6	0.96	0.96
W0	110.97	76.6	83.9	1.45	1.32
W30	90.07	72.3	79.3	1.25	1.14
W100	86.08	73.4	80.5	1.17	1.07
Mean				1.12	1.06
SD				0.21	0.16
COV (%)				19.11	14.75

Figure 7 graphically presents the ratio between the experimentally measured shear strengths (V_u) and the theoretical results obtained according to NBR 6118 [11] ($V_{Rcs,I}$, and $V_{Rcs,II}$). In Figure 7, the tests on beams without shear reinforcement reveal that the theoretical models overestimate the concrete contribution in the shear strength, regardless of the replacement ratio of NCA by RCA. On the other hand, for beams with stirrups, the theoretical models presented by NBR 6118 [11] produced conservative estimates of the shear strength, even for beams with RCA, with model II presenting a better correlation between the estimates and theoretical results. It should also be highlighted that the theoretical calculations' safety levels decreased within the increase of the RCA content.

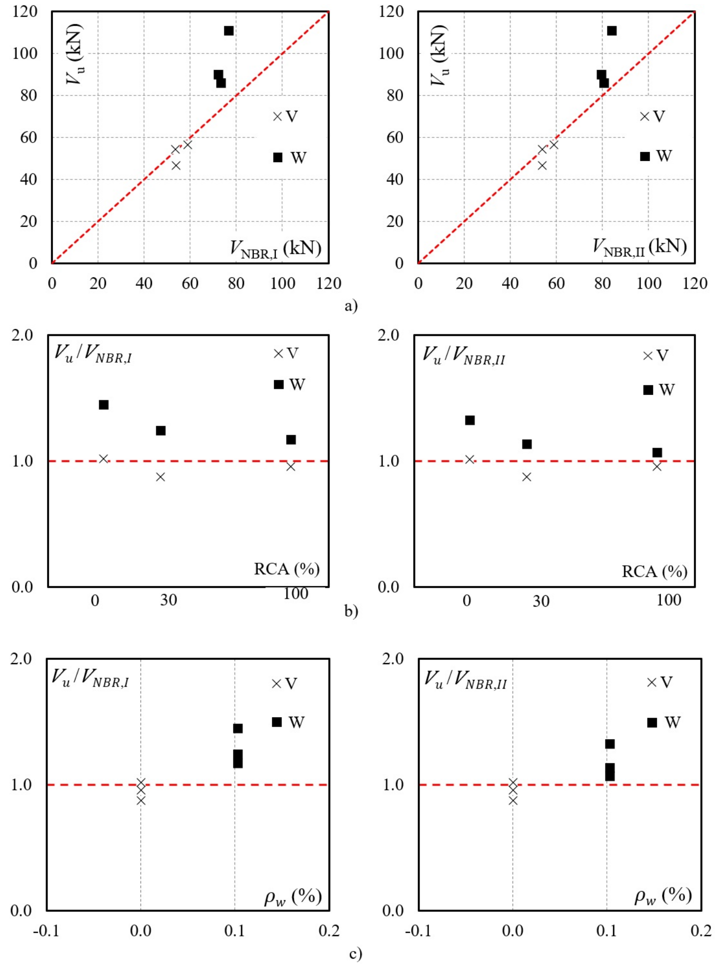


Figure 7 – Comparison between the experimental and theoretical for the tested beams.

4.2 Database

Figure 8 presents comparisons between the experimental shear strengths (V_u) and the theoretical estimates obtained using models I and II ($V_{NBR,I}$, and $V_{NBR,II}$) from [11] for the beams with shear reinforcement. These analyses are carried in general terms in Figure 8a and as a function of the percentage of replacement of NCA by RCA (Figure 8b) and the flexural and shear reinforcement ratio (Figures 8c and 8d). Figure 8a shows that, for both models, most of the theoretical estimates were conservative.

Figures 8b, 8c, and 8d evidence that, regardless of the replacement percentage of NCA by RCA, the safety of the theoretical estimates, measured by the ratio between experimental and theoretical results, is mainly affected by the flexural reinforcement ratio of the beams. In this context, if any adjustments in NBR 6118 [11] would have to be recommended, they would not be motivated by using recycled concrete aggregates instead of natural coarse aggregates.

Similar analyses are carried in Figure 9 but considering the results of tests on beams without shear reinforcement. These analyses are carried to check the performance of the theoretical models presented by NBR 6118 [11] to account for the contribution of concrete to the shear resistance of reinforced concrete beams and if this performance is affected by the replacement of natural coarse aggregates by recycled concrete aggregates. In general, the dispersion between experimental and theoretical estimates is significantly smaller in this situation (see Figure 9a). However, for a significant number of results, the theoretical models overestimate the shear resistance of the tested beams. No correlation between the replacement ratio of NCA by RCA is observed (see Figure 9b), and the safety level of the theoretical estimates slightly increases with the growth of the flexural reinforcement ratio of the beams, regardless of the use or not of recycled concrete aggregates.

Table 4 evaluates the performances of theoretical models according to the Demerit Points Classification (DPC) proposed by Collins [37]. For beams with shear reinforcement (see Figure 10a), the model I from [11] showed better performance, with 80% of results falling into the appropriate safety and conservative classes. The worst performance from model II resulted from 18% of their results being classified in the dangerous class. Considering the results of beams without shear reinforcement, both models had similar performance, as shown in Figure 10b.

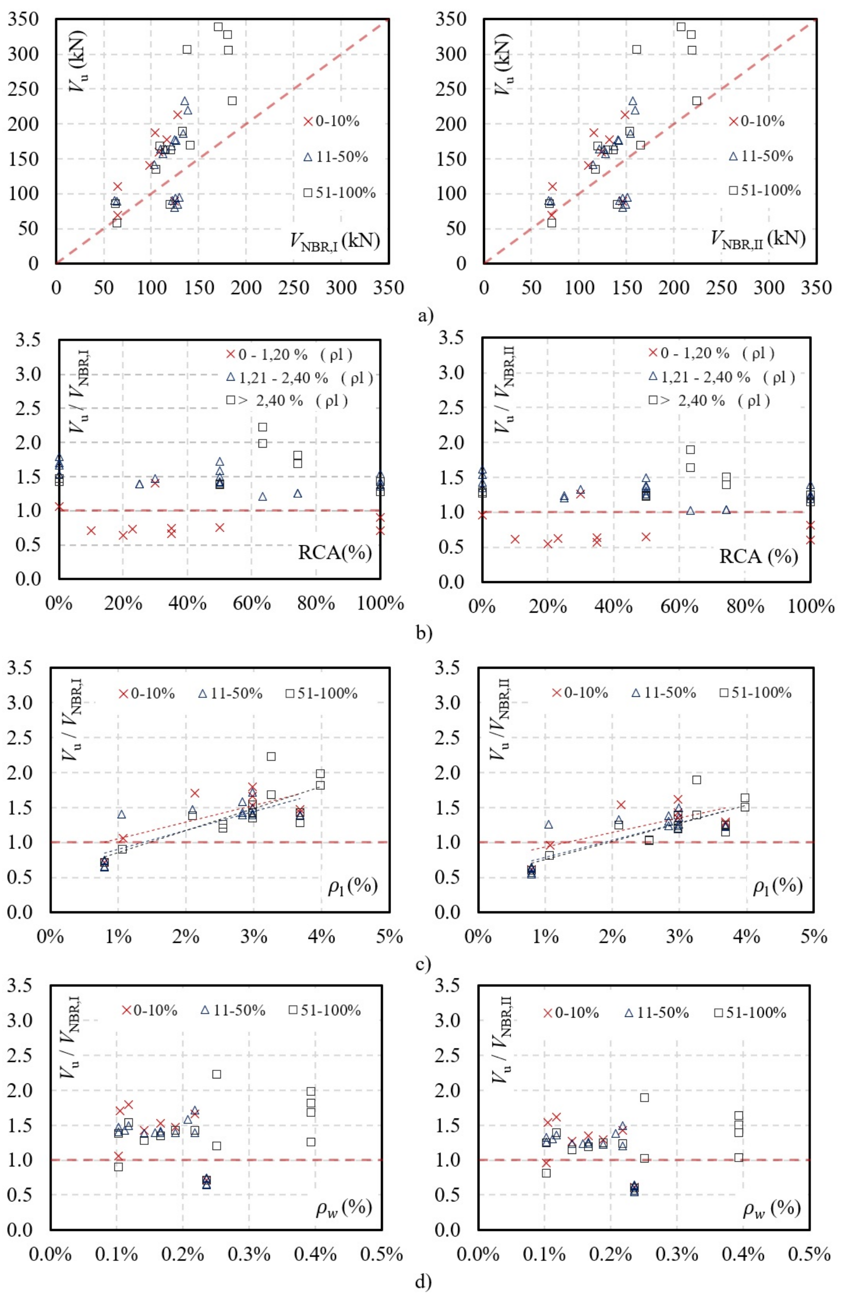


Figure 8. Comparison between the experimental and theoretical for the database of beams with stirrups

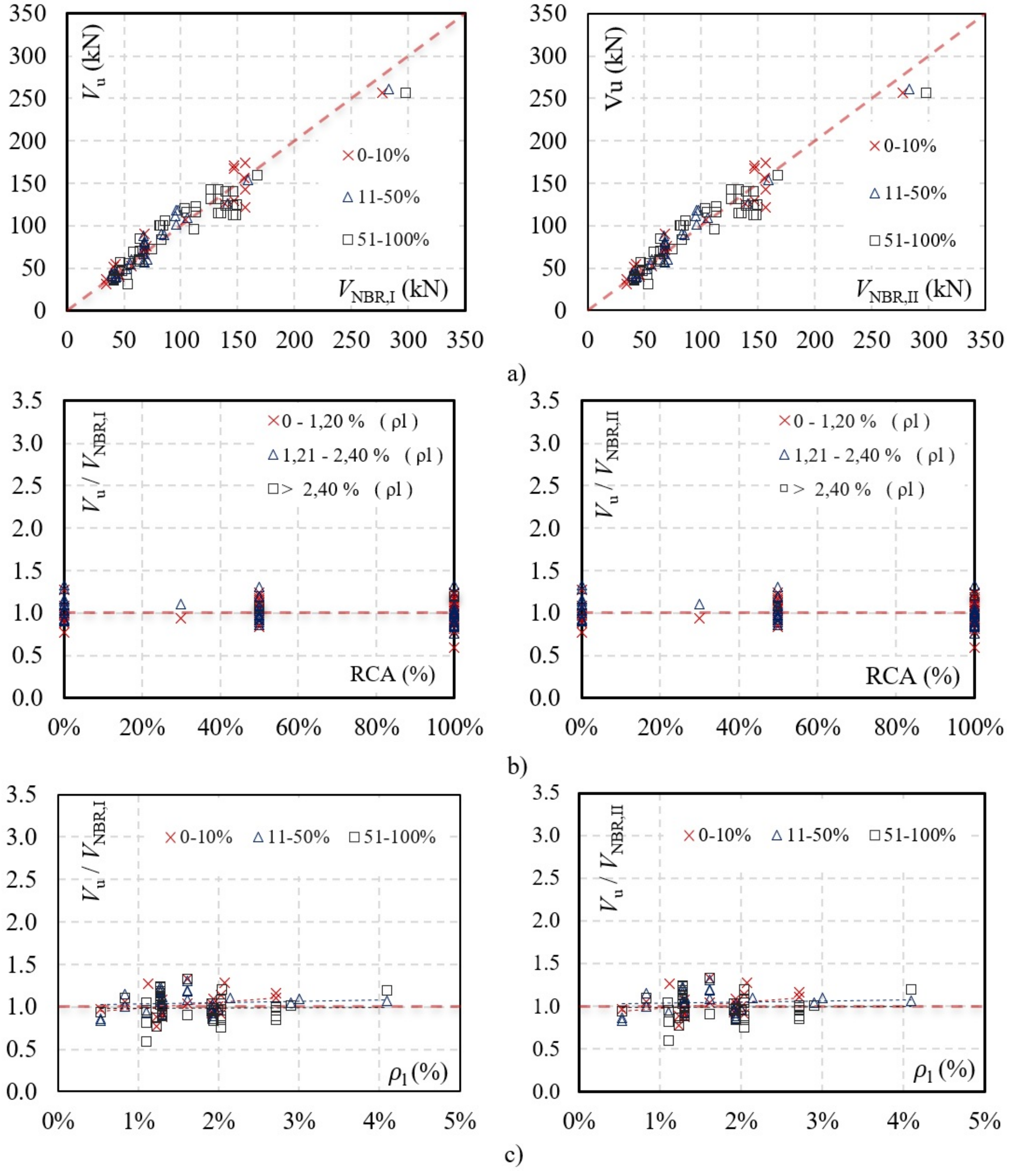


Figure 9. Comparison between the experimental and theoretical for the database of beams without stirrups

Table 4. Performance of theoretical estimates as proposed by Collins [37].

V_u/V_{teo}	Score	Classification
< 0.50	10	Extremely dangerous
[0.50 - 0.65[	5	Dangerous
[0.65 - 0.85[	2	Low safety
[0.85 - 1.30[	0	Appropriate safety
[1.30 - 2.00[	1	Conservative
≥ 2.00	2	Overconservative

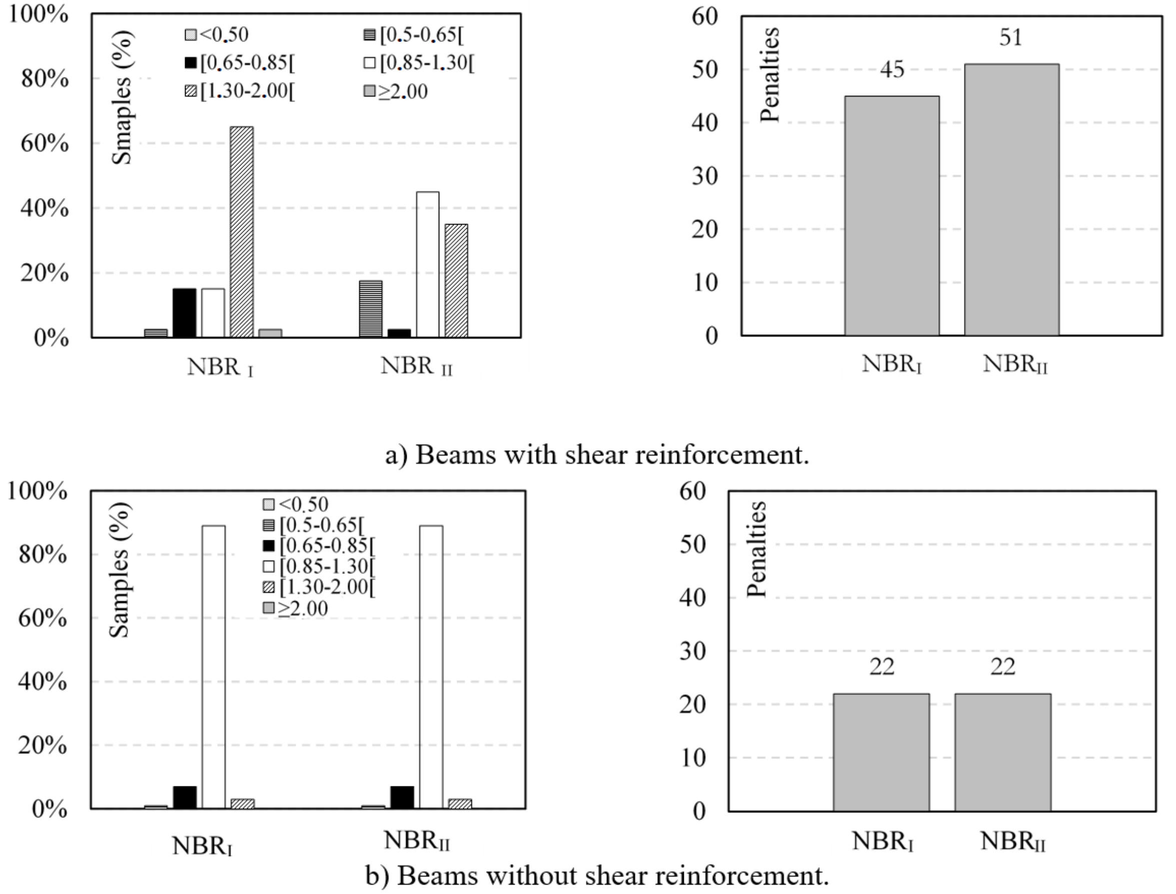


Figure 10. Result of the DPC for the beams within the database.

5 CONCLUSIONS

The main conclusions obtained were:

- The replacement of natural coarse aggregates by recycled concrete aggregates was not associated with reductions in concrete's compressive strength, considering the materials and the concrete proportioning used in the experimental program.
- The concrete's tensile strength and modulus of elasticity were slightly affected by the substitution of natural aggregates by recycled concrete aggregates. For the tensile strength, an average reduction of 15% was measured, and for the modulus of elasticity, mean reductions of 20% were observed, both for 100% replacement of NCA by RCA.
- Small reductions in the flexural stiffness and the shear resistance were observed in the tested beams due to the replacement of NCA by RCA. Those with shear reinforcement showed a conservative correlation between experimental and theoretical resistance estimates. On the other hand, those without shear reinforcement presented ultimate shear capacity below the theoretical expectation, regardless of the replacement of NCA by RCA.
- The extensive analyses allowed by the database with tests on beams without and with shear reinforcement do not show significant shear strength reductions associated with the substitution of natural coarse aggregates by recycled concrete aggregates. Considering the safety levels of the theoretical shear strengths obtained by following models I and II from the Brazilian code, these analyses show that the unsafe predictions are not associated with the use of recycled concrete aggregates but with the consideration of the contribution given by the flexural reinforcement ratio of the reinforced concrete beams.

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ANNEX A

Table A1. Database of tests on reinforced concrete beams with shear reinforcement.

Author	Beam	RCA (%)	b_w (mm)	d (mm)	a (mm)	a/d	A_s (mm^2)	A_{sw} (mm^2)	ρ_l (%)	ρ_w (%)	V_u (kN)	V_{flex} (kN)	f_{ck} (MPa)	f_{ywk} (MPa)
Larrañaga [25]	HC-2	0	200	304	1.000	3,3	1.810	57	3,0	0,2	213	214	32	495
	HC-3	0	200	304	1.000	3,3	1.810	57	3,0	0,2	177	214	32	495
	HC-4	0	200	304	1.000	3,3	1.810	57	3,0	0,1	188	214	32	495
Cardoso [26]	VAW0	0	180	254	620	2,4	491	28	1,1	0,1	69	98	23	554
	VBW0	0	180	255	620	2,5	942	28	2,1	0,1	111	166	23	554
Ignjatović et al. [6]	NAC-2	0	200	261	1.000	3,8	1.923	57	3,7	0,1	141	214	35	391
	NAC-3	0	200	261	1.000	3,8	1.923	57	3,7	0,2	160	214	35	391
Rahal and Alrefaei [5]	35-A-0-10	10	150	388	1.162	3,0	462	57	0,8	0,2	89	94	30	455
	35-A-0-20	20	150	388	1.162	3,0	462	57	0,8	0,2	80	94	30	455
	35-S-0-23(50)	23	150	388	1.162	3,0	462	57	0,8	0,2	95	95	33	455
Larrañaga [25]	HR25-2	25	200	304	1.000	3,3	1.810	57	3,0	0,2	187	220	35	495
	HR25-3	25	200	304	1.000	3,3	1.810	57	3,0	0,2	169	220	35	495
Cardoso [26]	VAW30	30	180	260	620	2,4	491	28	1,0	0,1	89	99	21	554
	VBW30	30	180	250	620	2,5	942	28	2,1	0,1	90	158	21	554
Rahal and Alrefaei [5]	35-A-0-35	35	150	388	1.162	3,0	462	57	0,8	0,2	90	94	29	455
	35-S-0-35(75)	35	150	388	1.162	3,0	462	57	0,8	0,2	85	94	32	455
	35-A-0-50	50	150	388	1.162	3,0	462	57	0,8	0,2	94	94	30	455
González-Fonteboa and Martínez-Abella [20]	V24RC	50	200	303	1.000	3,3	1.810	57	3,0	0,1	164	241	35	455
	V17RC	50	200	303	1.000	3,3	1.810	57	3,0	0,2	177	245	37	455
	V13RC	50	200	303	1.000	3,3	1.810	57	3,0	0,2	234	244	36	455
Larrañaga [25]	HR50-2	50	210	304	1.000	3,3	1.810	57	2,8	0,2	220	223	35	495
	HR50-3	50	210	304	1.000	3,3	1.810	57	2,8	0,2	176	223	35	495
	HR50-4	50	210	304	1.000	3,3	1.810	57	2,8	0,1	164	223	35	495
Ignjatović et al. [6]	RAC50-2	50	200	261	1.000	3,8	1.923	57	3,7	0,1	142	222	38	391
	RAC50-3	50	200	261	1.000	3,8	1.923	57	3,7	0,2	157	222	38	391
Fathifazl et al. [27]	EM-3S-R	63,5	200	309	800	2,6	1.571	101	2,5	0,3	170	235	33	482
	EM-6S-R	63,5	200	302	800	2,6	1.963	101	3,3	0,3	307	342	33	482
	EM-6S-D	63,5	200	301	800	2,7	2.395	157	4,0	0,4	339	374	33	482
	EV-3S-R	74,3	200	309	800	2,6	1.571	157	2,5	0,4	233	242	40	482
	EV-3S-R*	74,3	200	309	800	2,6	1.571	157	2,5	0,4	233	242	40	482
	EV-6S-R	74,3	200	302	800	2,6	1.963	157	3,3	0,4	306	365	40	482
	EV-6S-D	74,3	200	301	800	2,7	2.395	157	4,0	0,4	328	409	40	482
Rahal and Alrefaei [5]	35-A-0-100	100	150	388	1.162	3,0	462	57	0,8	0,2	85	93	27	455
Larrañaga [25]	HR100-2	100	200	304	1.000	3,3	1.810	57	3,0	0,2	190	219	34	495
	HR100-3	100	200	304	1.000	3,3	1.810	57	3,0	0,2	163	219	34	495
	HR100-4	100	200	304	1.000	3,3	1.810	57	3,0	0,1	168	219	34	495
Cardoso [26]	VAW100	100	180	258	620	2,4	491	28	1,1	0,1	58	99	22	554
	VBW100	100	180	250	620	2,5	942	28	2,1	0,1	86	160	22	554
Ignjatović et al. [6]	RAC100-2	100	200	261	1.000	3,8	1.923	57	3,7	0,1	135	225	40	391
	RAC100-3	100	200	261	1.000	3,8	1.923	57	3,7	0,2	163	225	40	391

Table A2. Database of tests on reinforced concrete beams without shear reinforcement.

Author	Beam	RCA (%)	b_w (mm)	d (mm)	a (mm)	a/d	A_s (mm^2)	ρ_l (%)	V_u (kN)	V_{flex} (kN)	f_{ck} (MPa)
Choi et al. [19]	**NANAC-L2.5**	0	200	360	900	2,5	382	0,5	66	75	21
	NANAC-M2.5	0	200	360	900	2,5	598	0,8	72	106	21
	NANAC-H2.5	0	200	360	900	2,5	1.159	1,6	91	178	21
	NANAC-H3.5	0	200	360	1.170	3,3	1.159	1,6	71	137	21
Knaackand Kurama [28]	**S0-1a**	0	150	200	760	3,8	390	1,3	31	51	27
	S0-1b	0	150	200	760	3,8	390	1,3	37	51	27
	S0-2a	0	150	200	760	3,8	390	1,3	40	54	42
	S0-2b	0	150	200	760	3,8	390	1,3	42	54	42
Katkhuda [29]	**NC-3**	0	200	267	800	3,0	1.018	1,9	53	130	24
Arezoumandi et al. [4]	**CC-NS-4.1**	0	300	400	1.200	3,0	1.810	1,2	121	228	33
	CC-NS-4.2	0	300	400	1.200	3,0	1.810	1,2	130	226	30
	CC-NS-6.1	0	300	400	1.200	3,0	1.810	2,0	143	228	33
	CC-NS-6.2	0	300	400	1.200	3,0	1.810	2,0	167	226	30
	CC-NS-8.1	0	300	400	1.200	3,0	1.810	2,7	174	228	33
	CC-NS-8.2	0	300	400	1.200	3,0	1.810	2,7	171	226	30
Kim et al. [30]	**NA-S2**	0	200	300	750	2,5	1.161	1,9	76	204	28
	NA-M2	0	200	450	1.125	2,5	1.734	1,9	107	305	28
	NA-L2	0	200	600	1.500	2,5	2.323	1,9	126	408	28
	NA-M3	0	300	450	1.125	2,5	2.694	2,0	157	471	28
	NA-L4	0	400	600	1.500	2,5	4.645	1,9	256	816	28
Cardoso [26]	**VA0**	0	180	244	620	2,5	491	1,1	52	92	20
	VB0	0	180	253	620	2,5	942	2,1	54	159	20
	VA30	30	180	248	620	2,5	491	1,1	41	95	21
	VB30	30	180	245	620	2,5	942	2,1	47	155	21
Choi et al. [19]	**RARAC30-L2.5**	50	200	360	900	2,5	382	0,5	57	75	21
	RARAC30-M2.5	50	200	360	900	2,5	598	0,8	78	106	21
	RARAC30-H2.5	50	200	360	900	2,5	1.159	1,6	81	178	21
	RARAC30-H3.5	50	200	360	1.170	3,3	1.159	1,6	81	137	21
	RARAC50-L2.5	50	200	360	900	2,5	382	0,5	58	75	20
	RARAC50-M2.5	50	200	360	900	2,5	598	0,8	67	106	20
	RARAC50-H2.5	50	200	360	900	2,5	1.159	1,6	88	178	20
	RARAC50-H3.5	50	200	360	1.170	3,3	1.159	1,6	73	137	20
Ignjatovic (2013) [32]	**RAC50-1b**	50	200	235	1.000	4,3	1.922	4,1	60	160	29
Knaackand Kurama [28]	**S50-1a**	50	150	200	760	3,8	390	1,3	44	53	38
	S50-1b	50	150	200	760	3,8	390	1,3	39	53	38
	S50-2a	50	150	200	760	3,8	390	1,3	44	53	33
	S50-2b	50	150	200	760	3,8	390	1,3	41	53	33
Katkhuda [29]	**R50-3**	50	200	267	800	3,0	1.018	1,9	49	127	21
	T50-3	50	200	267	800	3,0	1.018	1,9	55	130	24
Sadati et al. [31]	**RAC50-1**	50	200	303	1.000	3,3	1.818	3,0	91	237	36
	RAC50-2	50	200	303	1.000	3,3	1.757	2,9	89	233	37
	RAC50-3	50	150	200	760	3,8	390	1,3	44	51	38
	RAC50-4	50	150	200	760	3,8	390	1,3	39	51	38
	RAC50-5	50	150	200	760	3,8	390	1,3	44	51	33
	RAC50-6	50	150	200	760	3,8	390	1,3	41	51	33
Kim et al. [30]	**RH-S2**	50	200	300	750	2,5	1.161	1,9	61	205	29
	RH-M2	50	200	450	1.125	2,5	1.734	1,9	109	307	29
	RH-L2	50	200	600	1.500	2,5	2.323	1,9	126	410	29
	RH-M3	50	300	450	1.125	2,5	2.694	2,0	154	473	29
	RH-L4	50	400	600	1.500	2,5	4.645	1,9	262	820	29

Table A2. Database of tests on reinforced concrete beams without shear reinforcement (cont.).

Author	Beam	RCA (%)	b_w (mm)	d (mm)	a (mm)	a/d	A_s (mm^2)	ρ_l (%)	V_u (kN)	V_{flex} (kN)	f_{ck} (MPa)
Schubert et al. [16]	**RC-M-1 Q1**	50	500	170	560	3,3	1.080	1,3	118	159	28
	RC-M-1 Q2	50	500	170	560	3,3	1.080	1,3	118	159	27
	RC-M-2 Q1	50	500	170	560	3,3	1.080	1,3	112	159	27
	RC-M-2 Q2	50	500	170	560	3,3	1.080	1,3	102	159	27
	RC-C-1	100	500	170	560	3,3	1.080	1,3	116	161	31
	RC-C-2 Q1	100	500	170	560	3,3	1.080	1,3	123	163	34
	RC-C-2 Q2	100	500	170	560	3,3	1.080	1,3	116	163	34
	RC-C-3	100	500	170	560	3,3	1.080	1,3	121	161	30
	RC-M-3 Q1	100	500	170	560	3,3	1.080	1,3	101	154	21
	RC-M-3 Q2	100	500	170	560	3,3	1.080	1,3	101	154	21
	RC-M-4 Q1	100	500	170	560	3,3	1.080	1,3	100	155	22
	RC-M-4 Q2	100	500	170	560	3,3	1.080	1,3	106	156	23
Cardoso [26]	**VA100**	100	180	250	620	2,5	491	1,1	48	96	23
	VB100	100	180	257	620	2,4	942	2,0	57	168	23
Arezoumandi et al. [4].	**RAC-NS-4.1**	100	300	400	1.200	3,0	1.810	1,2	115	222	26
	RAC-NS-4.2	100	300	400	1.200	3,0	1.810	1,2	113	226	30
	RAC-NS-6.1	100	300	400	1.200	3,0	1.810	2,0	143	222	26
	RAC-NS-6.2	100	300	400	1.200	3,0	1.810	2,0	124	226	30
	RAC-NS-8.1	100	300	400	1.200	3,0	1.810	2,7	131	222	26
	RAC-NS-8.2	100	300	400	1.200	3,0	1.810	2,7	140	226	30
Kim et al. [30]	**RF-S2**	100	200	300	750	2,5	1.161	1,9	73	208	31
	RF-M2	100	200	450	1.125	2,5	1.734	1,9	96	311	31
	RF-L2	100	200	600	1.500	2,5	2.323	1,9	125	416	31
	RF-M3	100	300	450	1.125	2,5	2.694	2,0	160	479	31
	RF-L4	100	400	600	1.500	2,5	4.645	1,9	257	831	31
Sadati et al. [31]	**RAC100-1**	100	170	270	594	2,2	505	1,1	60	116	36
	RAC100-2	100	170	270	810	3,0	505	1,1	43	83	27
	RAC100-3	100	305	400	1.240	3,1	3.306	2,7	115	433	26
	RAC100-4	100	305	400	1.240	3,1	2.477	2,0	113	364	30
	RAC100-5	100	305	375	1.219	3,3	1.453	1,3	143	216	26
	RAC100-6	100	305	375	1.219	3,3	2.322	2,0	131	316	26
	RAC100-7	100	305	375	1.219	3,3	3.100	2,7	124	404	30
	RAC100-8	100	305	375	1.219	3,3	3.100	2,7	140	404	30
	RAC100-9	100	200	303	1.000	3,3	1.757	2,9	84	231	36
	RAC100-10	100	150	200	760	3,8	390	1,3	36	51	35
	RAC100-11	100	150	200	760	3,8	390	1,3	38	51	35
	RAC100-12	100	150	200	760	3,8	390	1,3	40	51	35
	RAC100-13	100	150	200	760	3,8	609	2,0	36	75	35
	RAC100-14	100	170	270	1.080	4,0	505	1,1	32	62	28
Katkhuda [29]	**R100-3**	100	200	267	800	3,0	1.018	1,9	46	125	19
	T100-3	100	200	267	800	3,0	1.018	1,9	56	128	23
Knaackand Kurama [28]	**S100-1a**	100	150	200	760	3,8	390	1,3	36	53	35
	S100-1b	100	150	200	760	3,8	390	1,3	38	53	35
	S100-2a	100	150	200	760	3,8	390	1,3	40	53	35
	S100-2b	100	150	200	760	3,8	390	1,3	36	53	35
Ignjatovic (2013) [32]	**RAC1000-1b**	100	200	235	1.000	4,3	1.922	4,1	69	163	31
Choi et al. [19]	**RARAC100-L2.5**	100	200	360	900	2,5	382	0,5	60	75	19
	RARAC100-M2.5	100	200	360	900	2,5	598	0,8	70	105	19
	RARAC100-H2.5	100	200	360	900	2,5	1.159	1,6	85	175	19
	RARAC100-H3.5	100	200	360	1.170	3,3	1.159	1,6	58	135	19