Article

Prediction of Shear Strength of Reinforced Recycled Aggregate Concrete Beams without Stirrups

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Abstract: The brittle shear failure of reinforced concrete beams is complexed and unfavorable. For decades, research on the mechanical properties and durability of recycled coarse aggregate (RCA) to make recycled aggregate concrete (RAC) has been widely investigated. However, test results on the shear strength of reinforced recycled aggregate concrete beams are still limited and contradictory. This paper reports the shear strength of reinforced recycled aggregate concrete beams without stirrups. Eight RAC beams and two controlled beams with natural coarse aggregate (NCA) were tested under the four-point flexural test with the shear span-to-effective depth ratio (a/d) of 3.10. Parameters in this study were the replacement percentage of RCA (0%, 25%, 50%, 75%, and 100%) and longitudinal reinforcement ratio (◎w) of 1.16% and 1.80%. It was found that the normalized shear stresses of RAC beams with ◎w = 1.80% at all levels of replacement percentage were quite similar to that of the NAC counterparts. Normalized shear stress of the beam with 100% RCA and ◎w = 1.16% was lower than that of the NAC beam by 5%. Database of 128 RAC beams without shear reinforcement from literature was analyzed to evaluate the ability of the most recent ACI 318-19 shear provisions in shear strength prediction. A reduction factor of 0.75 is proposed to the current ACI code provision to account for the physical variations of RCA such as replacement percentage, RCA source and quality, density, amount of residual mortar, and physical irregularity.

Keywords: recycled concrete aggregate; construction waste; shear strength;

1. Introduction

With continuing economic growth, the construction of buildings and infrastructures is inevitable. Concrete is the most widely used construction material. With new construction and demolition of old buildings and structures, construction and demolition waste (CDW) are increasingly becoming a problem for the environment, leading to several pieces of research conducted to recycle this construction waste. Besides the environmental issue from concrete waste, new concrete also poses the problem of natural resource consumption because the production of cement and natural coarse aggregate has been rapidly increasing. Several researchers have conducted studies on recycling construction and demolition waste to decrease the consumption demand of natural resources and reduce construction waste [1-4]. Since the largest CDW is concrete, numerous research studies involve mechanical properties and durability of concrete materials using recycled coarse aggregate (RCA) as the coarse aggregate and compare the results to natural coarse aggregate (NCA) [5-13]. This is because the coarse aggregate is the main concrete component, constituting about 60% to 75% of concrete volume. However, several test results are not encouraging as it is found that the properties of concrete with RCA are inferior to concrete with NCA [5], [14-18].

 doi:10.20944/preprints202108.0131.v1

Further studies, therefore, focused on the quality improvement of RCA and mixture proportions [19-21]. For example, Fathifazl et al. proposed the Equivalent Mortar Volume (EMV) method to proportion concrete mixtures with RCA [22]. Katz [23] improved RCA using a silica fume solution and ultrasonically cleansing. Li et al. [24] coated RCA with pozzolanic powder to enhance the slump and strength of concrete.

The uses of RCA are generally limited to non-structural applications such as road subbase and back-fills. However, application in structural components is still limited because of the lack of guidelines and skepticism toward recycling materials. Therefore, several studies have been conducted to investigate the shear behavior of concrete made with RCA. Sogo and Sato [25] conducted tests on the shear strength of RCA beams with 100% RCA. The beams were with and without shear reinforcement. Their experimental results indicated a decrease of shear strength by 20%. Etxeberria and Vazquez [26] tested twelve beams with and without stirrups using different RCA replacement percentages of 0%, 25%, 50%, and 100%. The results indicated that the shear strength of beams was not influenced by low percentages of RCA replacement (25%). Gonzalez-Fonteboa and Martinez-Abella [27] reported the shear strength of four tested beams with 50% RCA. The shear strength of RCA beams showed an insignificant difference compared to the controlled NCA beams. Similar results were observed by Knaack and Kurama [28] and Ignjatovic et al. [29] that the effect of RCA on the shear strength was small. On the other hand, study reports by Arezoumandi et al. [30, 31], Rahal and Alrefaei [32], Etman et al. [33], and Pradhan et al. [34] suggested contradictory results. Shear strengths in those tests were less than their NCA counterparts.

The shear strength and behavior of reinforced concrete beams are so complexed that numerous tested data must be collected and analyzed. Several researchers attempted to conduct more tests on the shear strength of RCA beams and compared the results to existing design codes. Several parameters influence the shear strength, such as the percentage of RCA replacement, size of the beams, concrete compressive strength, shear span-to-depth ratio, and longitudinal reinforcement ratio [34]. In this study, the percentage of RCA replacement and longitudinal reinforcement ratios were selected as studied parameters. The levels of RCA substitution were 0%, 25%, 50%, 75% and 100%. Effects of longitudinal reinforcement ratios of 1.16% and 1.81% were evaluated. The shear strength of two NCA beams and eight RCA beams without stirrups were experimentally investigated. Test results of RCA beams without stirrups from other studies [22, 26-42] were compiled and analyzed to evaluate the applicability of an existing ACI 318-19 shear provision. The modification factor in the existing code was proposed to account for the physical variations of RCA such as replacement percentage, RCA source and quality, density, amount of residual mortar, and physical irregularity.

2. Material and Methods

The experimental program involves the testing of 10 reinforced concrete beams without shear reinforcement. Variables are the percentage of RCA replacement and longitudinal reinforcement on the shear strength of the beams. Materials, mixture proportions, and details of specimens are described below.

2.1 Materials Properties

The concrete mixtures included Type 1 Ordinary Portland Cement (OPC), natural coarse aggregate (NCA), recycled coarse aggregate (RCA), sand, water, and superplasticizer. The RCA was prepared by crushing concrete cylinder waste with a compressive strength of approximately 24-30 MPa (Figure 1a). A low cost, custom-made jaw crusher machine was developed at Walailak University which is able to crush waste concrete with the maximum feeding size of 150 mm (i.e. 150x300 mm cylinders). The jaw crusher is an AC 220 volt electric machine able to produce the adjustable discharge size of 10-40 mm with the capacity of 1-3 tons/hr [43] (Figure 1c). The properties of NCA and RCA used in this study are listed in Table 1.

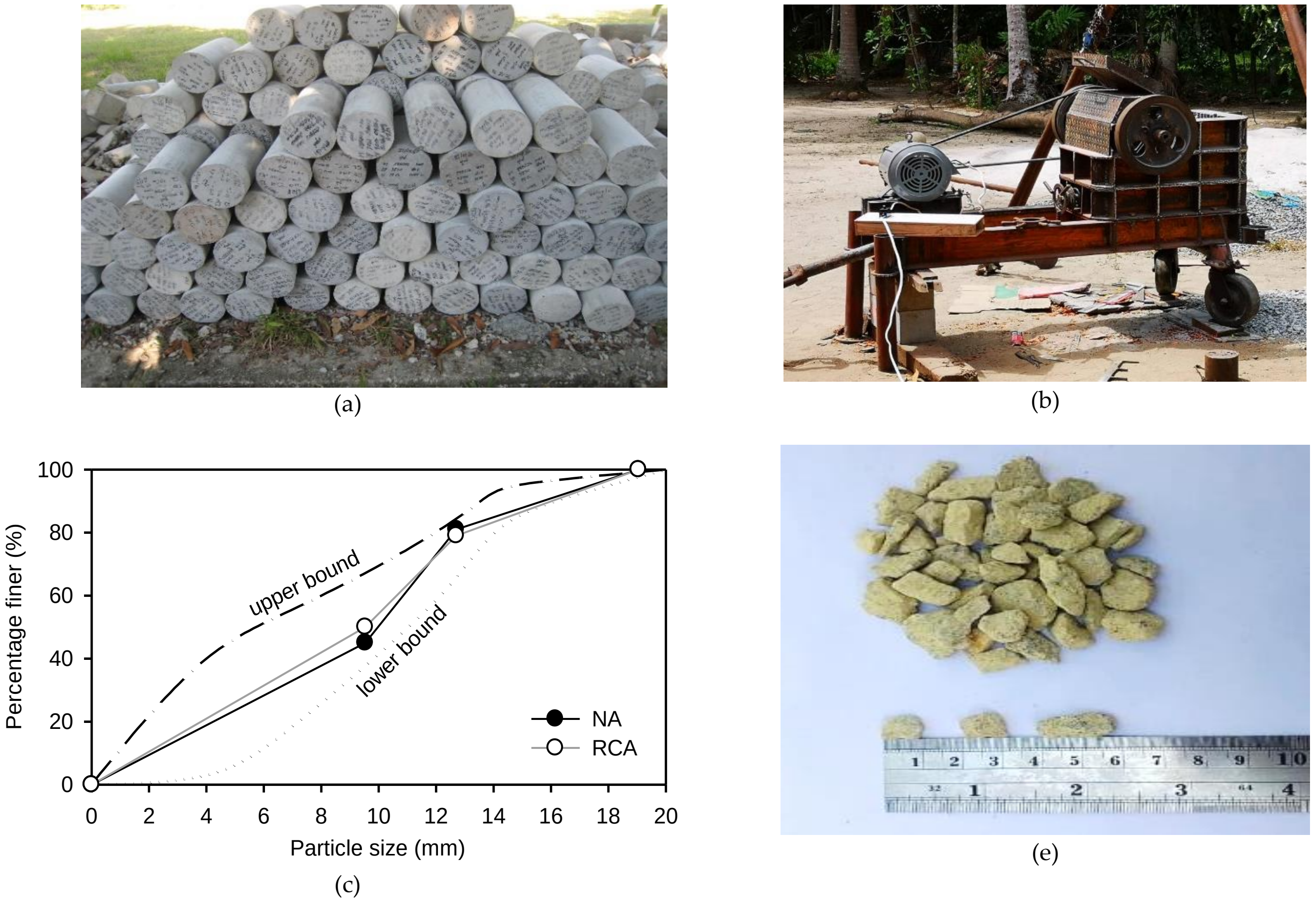

Figure 1. (a) Concrete cylinder waste (b) Custom-made crushing machine (c) Coarse aggregate %passing and, (e) Recycled coarse aggregate (RCA).

Table 1. Physical properties of fine, natural and recycled aggregates.

Properties	FA	NCA	RCA
Bulk Specific Gravity (SSD)	2.6	2.7	2.43
Unit Weight (kg/m3)	-	1730	1397
Water Absorption (%)	1.05	0.28	4.59
Moisture (%)	1.35	0.61	2.24
Fineness Modulus	2.7	-	-
Max. size (mm)	4.76	19.1	18.6
Impact value (%)	-	10.15	13.4
Crushing value (%)	-	21.77	23.12
Residual mortar (%)	-	-	32.5

The diameters of steel used for longitudinal reinforcement were 16 mm and 20 mm. The nominal diameter of shear reinforcement was 6 mm. The yield stress and ultimate stress of the reinforcing steel are shown in Table 2.

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Table 2. Mechanical properties of steel reinforcement.

Nominal size (mm)	Yield Stress (MPa)	Ultimate Stress (MPa)	Elongation (%)
20	519	668	1.5
16	561	658	1.6
6	424	639	2.1

2.2 Concrete mixture proportions

According to ACI 211.1 [17], five mix proportion was designed with a water-cement ratio of 0.5 and a targeted compressive strength of 30 MPa. As shown in Table 1, RCA had lower specific gravity and unit weight but higher water absorption and moisture. Therefore, water was adjusted to compensate for water content in recycled aggregate concrete due to the higher absorption and moisture. Superplasticizer was also utilized to increase workability in both NCA and RCA mix. Five concrete mixtures have been designed for the slump of 150 mm. The mix proportions are shown in Table 3.

Table 3. Concrete mixture proportions (in kg/m^3).

Mix Type	Cement	FA	NCA	RCA	Water	SP
NCA	357	719	1069	-	190	1.07
25% RCA	357	750	802	216	190	1.07
50% RCA	357	780	535	432	190	1.07
75% RCA	357	810	267	648	190	1.07
100% RCA	357	840	-	864	190	1.07

Note: FA is the fine aggregate, NCA is the natural coarse aggregate, RCA is the recycled concrete aggregate, SP is the superplasticizer.

2.3 Details of test beams

Beam specimens were tested to investigate the effects of RCA incorporation on the shear strength of beams without stirrups. A total of ten beams with rectangular cross-sections, 2.8 m long, 200 mm wide, 300 mm high, and 260 mm effective depth, were cast. The beams had an effective test span of 2.4 m and a shear-span-to-depth ratio of 3.1. The percentage of RCA replacement and longitudinal reinforcement ratio in beams were used as parameters in the study. Two beams were fabricated from NCA, while eight beams were cast with RCA (25%, 50%, 75%, and 100%). The specimens were organized into two groups defined by two longitudinal reinforcement ratios (ρ_w). The first group consisted of five beams reinforced with three 16-mm deformed bars in the tension zone (ρ_w = 1.61%). The second group used longitudinal reinforcement of three 20-mm deformed bars (ρ_w = 1.8%). The location of shear failure was pre-selected by installing stirrups on the other side of the beam. Figure 2. depicts the details and test setup of beam specimens.

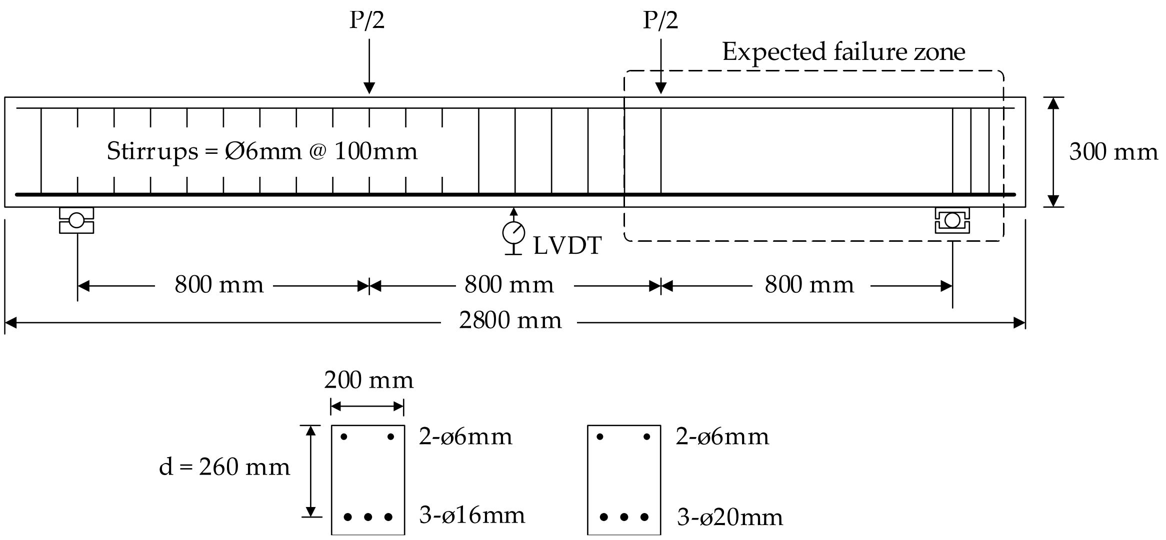

Figure 2. Test setup and details of beam specimens.

The codified names of the beam specimens begin with RCA followed by RCA percentage replacement. Bar sizes are added at the end to distinguish different longitudinal reinforcement. For instance, RCA0-DB20 means the beam contains NCA (no RCA) and three 20-mm longitudinal reinforcement with $\rho_w = 1.80\%$. The designation RCA25-DB16 indicates that the beam incorporates 25% of RCA and three 16-mm longitudinal reinforcement with $\rho_w = 1.16\%$.

2.4 Test Setup and Procedure

The beams were tested under the four-point bending test. The beams were supported on a roller and pin support, located at 200 mm from each end of the beams. Two linear variable differential transformers (LVDT) were installed at mid-span to monitor the deflection of the beams. Load and deflection were recorded by a data acquisition system. The load was slowly applied by an electrical-controlled hydraulic jack and a load cell of 1000 kN. Load cell applied the force on the transfer beam resting on two supports on the top of the tested specimens. The load was applied continuously until the beams failed. On the same testing day, six concrete cylinders were tested to determine the compressive strength of the tested beam.

3. Results and Discussion

A summary of test results is shown in Table 4. This section discusses the results observed in this study.

3.1. Ultimate capacity and failure behaviour

All tested beams failed in shear, as illustrated in Figure 3. Table 4 summarizes the cylindrical compressive strength, the ultimate load, P_u, and shear force at failure V_{test}, half of P_u. At cracking loads of approximately 40 kN and 50 kN for the beams with DB16 and DB20 series, flexural cracks developed at mid-span were very small. At ultimate load, an inclined crack suddenly appeared and caused the failure of the beams. Diagonal cracks for all beams were similar, with crack angles ranging from 30 to 40 degrees. Diagonal cracks of NAC beams were approximately 40 degrees, while crack angles of RAC beams were about 32 degrees. All of the beams failed in shear when the inclined flexural-shear crack propagated to the beam compression zone. At ultimate load, the lower tip of diagonal crack also penetrated to the support. The mode of failures of RCA beams was similar to tested beams from other researchers [27, 29, 31].

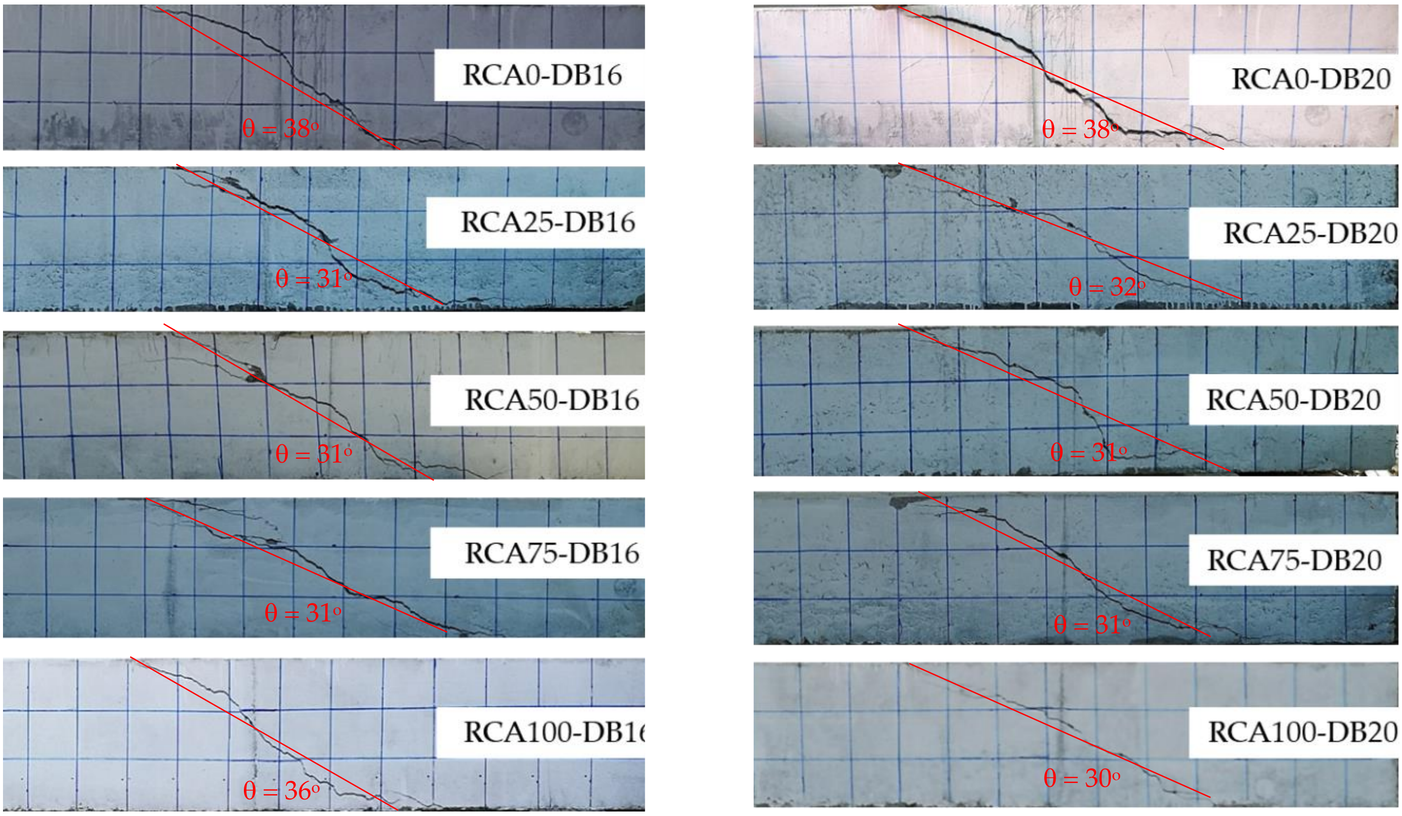

Figure 3. Failure of beams due to diagonal tension.

Table 4. Summary of the test results.

Specimen ID	f_c' (MPa)	ρ_w (%)	P_u (kN)	Δ_u (mm)	Shear crack inclination angle (Degree)	V_{test} (kN)	$v_{test}=V_{test}/bd$ (MPa)	$v_{test}/\sqrt{f_c'}$
RCA0-DB16	29.9	1.16	125	5.9	38	62.5	1.20	0.220
RCA25-DB16	35.7	1.16	135	6.5	31	67.5	1.29	0.217
RCA50-DB16	29.0	1.16	119	6.2	31	59.5	1.14	0.212
RCA75-DB16	32.9	1.16	125	5.8	31	62.5	1.20	0.209
RCA100-DB16	31.9	1.16	122	5.9	36	61.0	1.17	0.208
RCA0-DB20	29.7	1.81	166.6	6.8	38	83.3	1.60	0.294
RCA25-DB20	30.7	1.81	167.8	6.2	32	83.9	1.61	0.291
RCA50-DB20	23.1	1.81	145	5.5	31	72.5	1.39	0.290
RCA75-DB20	34.1	1.81	175	6.5	31	87.5	1.68	0.289
RCA100-DB20	29.5	1.81	162.4	6.3	30	81.2	1.56	0.287

3.2. Load-deflection Responses

The load-deflection relations of all tested beams are shown in Figure 4a and Figure 5a for the beams with ρ_w = 1.16% and 1.81%, respectively. Before the first flexural cracks, all beams behaved in a linear elastic manner. Beams in the DB16 series (ρ_w = 1.16%) lost some stiffness at the first cracking loads of approximately 40 kN. For beams in the DB20 series (ρ_w = 1.81%), the first cracking loads were about 50 kN, at which point beam stiffness began to decline. The deflections of all tested beams were limited and less than 7 mm. These results were similar to tests reported by other researchers [32], [39], [30] as the beams failed in shear before yielding of longitudinal reinforcement. It was also reported that additional deformation due to shear cracks increase overall deflection of the beam by up to 25% after the formation of shear cracks [44].

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The ultimate loads and shear forces were higher in the beams with a higher longitudinal reinforcement ratio, as expected. These results confirmed the effect of longitudinal reinforcement ratio on shear strength of concrete, which has been recently included in ACI 318-19. The difference in the deflections of RCA and NCA beams was not noticeable. Other researchers also reported inconsistencies of differences in deflections between RCA and NCA beams [32], [27], [38], [45].

Figure 4b and Figure 5b show responses of beams in terms of average shear stress (V_{test}/bd) normalized by the square root of the concrete cylinder strength, fc′. V_{test} is ultimate shear in N, fc′ is in MPa, b is the width of beam section in mm, and d is an effective depth in mm. These figures allow comparisons of shear strength with different concrete compressive strengths. From Figure 4b, the differences of normalized shear stresses between RCA and NCA beams were negligible for the beams with ρ_w = 1.16% (DB16 series). It was also observed that the amount of RCA barely affects the normalized shear strength of tested beams. Normalized shear stresses of the beams in this series were approximately 0.21. According to ACI 318-14 shear provisions, the normalized shear stress of reinforced concrete beams is 0.17. It was evident that all of the RCA beams with ρ_w = 1.16% had normalized shear stress higher than that of the ACI 318-14 shear equation. A similar trend was observed for beams with ρ_w = 1.81% (DB20 series). Normalized shear stresses of the NCA and RCA beams in this series were approximately 0.29. These results contradicted several previous tests by other researchers [32], [30], [34], [33], [31] that reported much lower shear strength of RCA beams compared to NCA beams. However, some researchers reported similar shear strength of RCA and NCA beams [29], [28]. To the author's knowledge, the disagreement of test results may be attributed to the source and quality of concrete waste used to produce RCA. In this study, RCA was obtained by crushing good quality concrete waste with approximate compressive strength 30 MPa.

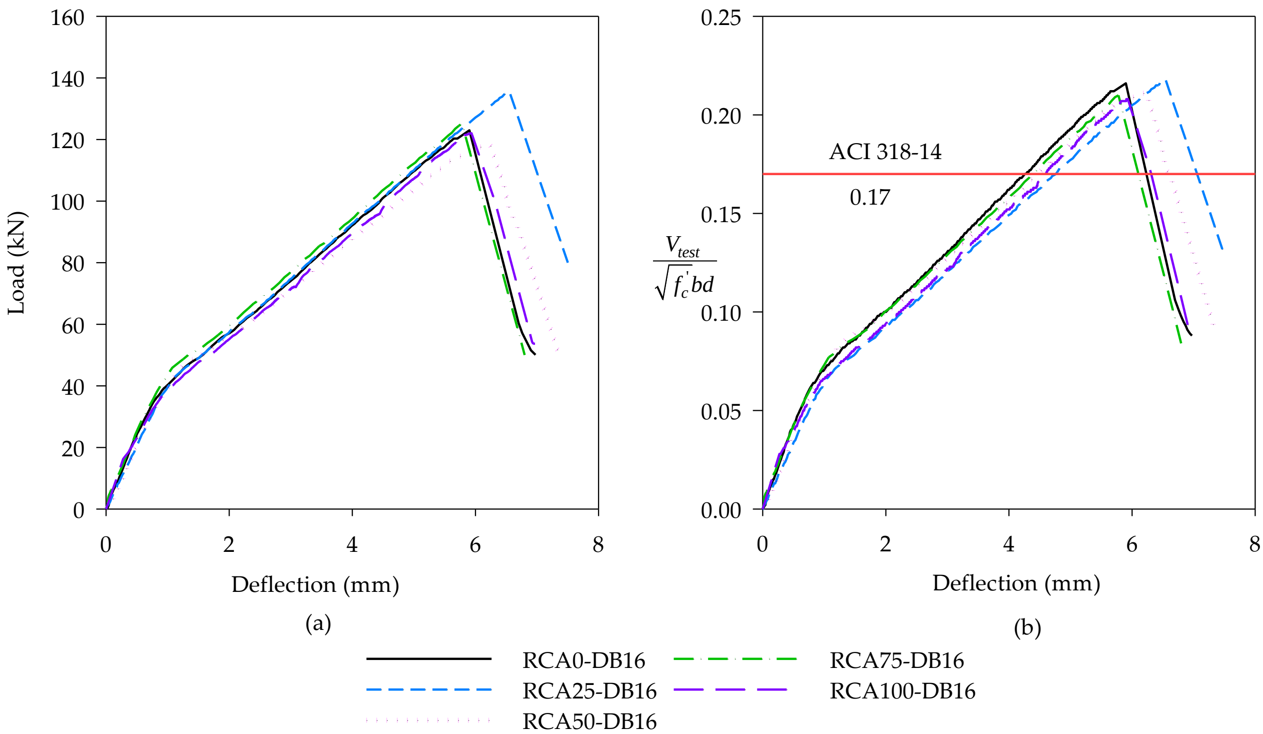

Figure 4. (**a**) Load-deflection curve for beams with reinforcement ratio, ρ_w = 1.16% ; (**b**) Normalized shear stress vs deflection for beams with reinforcement ratio, ρ_w = 1.16%.

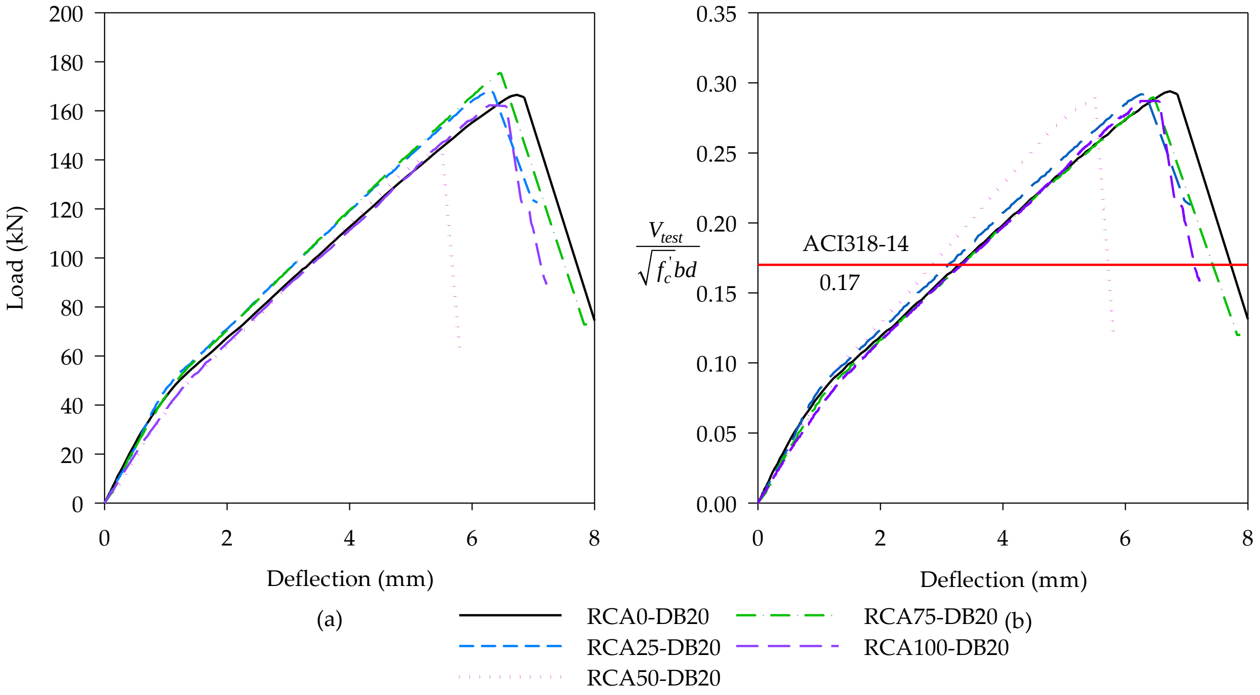

Figure 5. (a) Load-deflection curve for beams with reinforcement ratio, ρw = 1.81%; (b) Normalized shear stress vs deflection for beams with reinforcement ratio, ρw = 1.81%.

4. Prediction of concrete shear strength

On the basis of the considerations above, and to facilitate the rapid adoption of RCA in concrete construction, the current ACI equation to predict the shear strength of concrete member without stirrups is empirically investigated. In this study, the equation included in the ACI design guideline is modified using an empirical parameter based on extensive test data available in the literature. In the following, the current ACI equation to calculate concrete shear strength (v_c) is investigated. It should be noted that all of the partial safety factors for material as well as load and resistance factors adopted by the different design standards are not included in the equation below in order to allow for easier comparisons.

4.1. Ultimate concrete shear strength

Table 5 summarizes test results of 10 beams and compare to the shear strength predicted by ACI 318-14 simplified shear equation and ACI 318-19 shear provisions. Prediction of shear strength by ACI318-14 is shown in Table 5, column 6. ACI318-14 simplified shear equation is shown as the following expression.

$$V_c = 0.17\sqrt{f_c'}bd \quad (1)$$

where V_c is the shear provided by concrete (N), f_c' is the specified compressive strength of concrete cylinder (MPa), b is the width of cross-section (mm), and d is the effective depth (mm).

Recently, ACI 318-19 shear provisions have improved equations for the shear strength of reinforced concrete beams. For beams with $A_v < A_{v,min}$, the shear equation is in the new expression.

$$V_c = 0.66\lambda_s\left(\rho_w\right)^{1/3}\sqrt{f_c'}bd \quad (2)$$

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where ρ_w is the ratio of A_s to bd and λ_s is the size effect modification factor, defined as

$$\lambda_s = \sqrt{\frac{2}{1+0.004d}} \leq 1 \tag{3}$$

Shear strength prediction by ACI 318-19 expressed in equation (2) is calculated and shown in Table 5 column (8). Comparison of shear strength ratio (V_{test} / V_c) from ACI 318-14 and ACI 318-19 is shown in column (7) and column (9) of table 5. V_{test} / V_c in column (7) of Table 5 ranged from 1.22 for the 100% RCA beam to 1.29 for the NCA beam with ρ_w = 1.16%. For the beams with ρ_w = 1.81%, V_{test} / V_c were 1.73 for the NCA beam and 1.69 for the 100% RCA beam. Figure 6 compares shear strength from tests (V_{test}) with V_c predicted by ACI 318-14 simplified equation and ACI 318-19 new shear equation. Shear strength of all tested specimens was higher than ACI 318 shear prediction. Form the graph, V_c predicted by ACI 318-19 is more conservative than ACI 318-14 for ρ_w = 1.16% (DB16 series). However, for for ρ_w = 1.81% (DB20 series) ACI 318-19 shear strength is slightly higher than ACI 318-14. It is evident that the shear equation in both ACI 318-14 and ACI 318-19 provisions conservatively estimated the shear strength of RCA beams. Furthermore, these values indicated a slightly higher margin of safety for NCA over RCA beams.

Table 5. Summary of concrete and beam test results.

Specimen ID	**fc′ (MPa)**	**ρ_w (%)**	**Ultimate Load, P_u (kN)**	**V_{test} (kN)**	**V_c (kN) ACI318-14**	**V_u/V_c, ACI318-14**	**V_c (kN) ACI318-19**	**V_u/V_c, ACI318-19**
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
RCA0-DB16	29.9	1.16	125	62.5	48.35	1.29	42.07	1.49
RCA25-DB16	35.7	1.16	135	67.5	52.85	1.28	45.99	1.47
RCA50-DB16	29.0	1.16	119	59.5	47.60	1.25	41.42	1.44
RCA75-DB16	32.9	1.16	125	62.5	50.74	1.23	44.15	1.42
RCA100-DB16	31.9	1.16	122	61.0	49.95	1.22	43.46	1.40
RCA0-DB20	29.7	1.81	166.6	83.3	48.19	1.73	48.65	1.71
RCA25-DB20	30.7	1.81	167.8	83.9	48.94	1.71	49.41	1.70
RCA50-DB20	23.1	1.81	145	72.5	42.49	1.71	42.89	1.69
RCA75-DB20	34.1	1.81	175	87.5	51.55	1.70	52.04	1.68
RCA100-DB20	29.5	1.81	162.4	81.2	48.01	1.69	48.47	1.67

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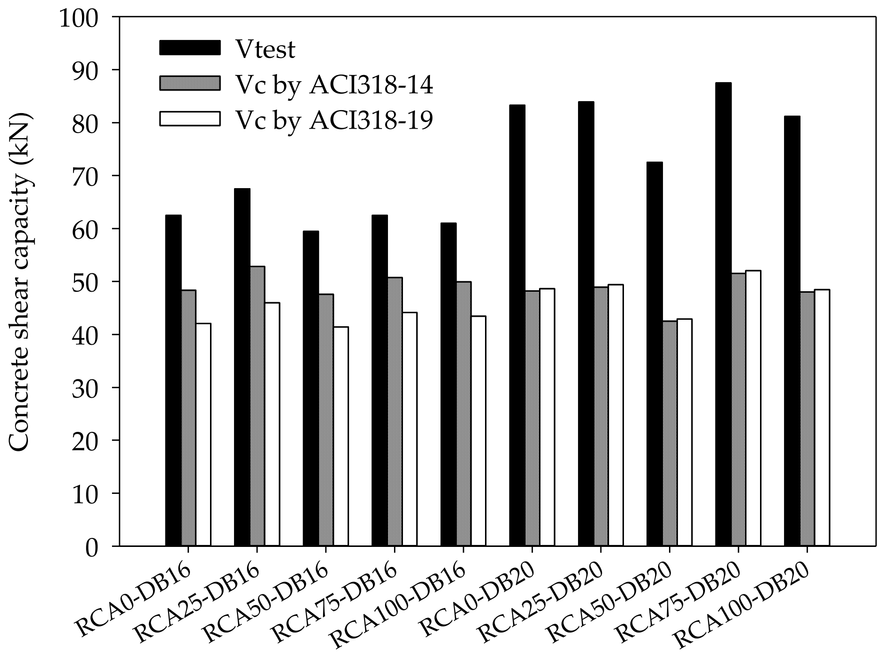

Figure 6. Comparisons of V_{test} with V_c from ACI 318-14 and ACI 318-19 predictions.

4.2 Effect of Longitudinal Reinforcement

The previous ACI 318 simplified shear equation does not consider the effect of longitudinal reinforcement ratio in shear strength prediction. However, this shorcoming has been improved in ACI 318-19 by introducing ρ_w in the shear expression, as indicated in equation 2. The difference between normalized shear stress predicted by ACI 318-14 and ACI 318-19 shear provisions at different longitudinal reinforcement ratios is ploted in Figure 7. It is evident that for ρ_w higher than 1.8% shear prediction by ACI 318-14 is lower. However, for low ρ_w ACI 318-19 yields more conservative predictions. In both shear equations, test results from this study showed higher normalized shear stresses for all tested beams.

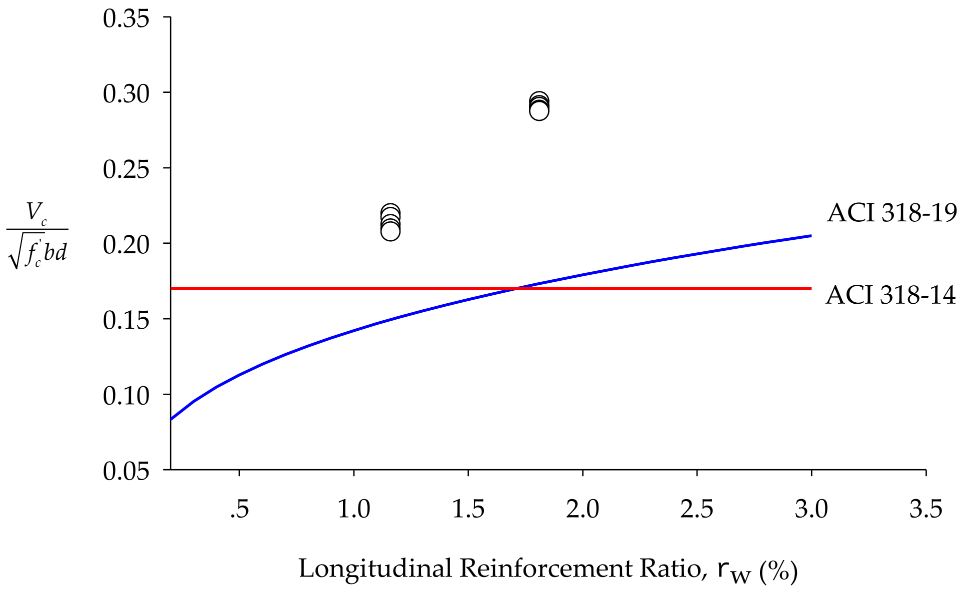

Figure 7. Comparisons of the normalized shear stress from experiments, ACI 318-14, and ACI 318-19

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4.3 Effect of RCA Replacement Ratio

Figure 8 shows the comparison of V_{test}/V_c predicted by ACI 314-14 and ACI 318-19 for different RCA replacement percentages and longitudinal reinforcement ratios. ACI 318-19 shear provisions consider size effect from the effective depth of beam section. It also includes the longitudinal reinforcement ratio (ρ_w) in the shear equation. For beams with ρ_w = 1.16% (3DB16), V_c from ACI 318-19 was lower than ACI 318-14. For the beams with ρ_w = 1.81%, V_c from both ACI 318-14 and ACI 318-19 were quite similar. However, higher V_c was expected from ACI 318-19 shear provisions when ρ_w was greater than 1.8, as discussed in previous section. The decreasing trends of V_{test} / V_c was observed when the amount of RCA increased, particularly in the case of lower ρ_w. However, the decrease rate was low, where V_{test} / V_c of RCA100 was only 5% and 2.2% lower than NCA beams with ρ_w = 1.16% and 1.81%, respectively.

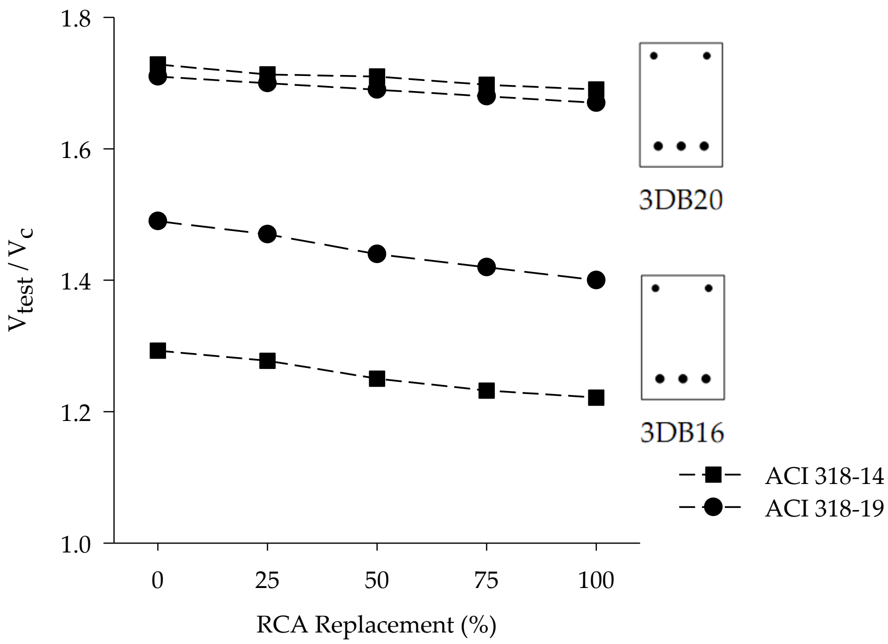

Figure 8. Ratios between ultimate shear from test and concrete shear prediction from ACI318 shear provisions.

5. Modifications to code equation to allow for the use of RCA

5.1 Proposed RCA uncertainty factor to existing design equation

The shear database was developed to evaluate the applicability of ACI 318-19 to predict the shear strength of RCA beams. The database included a total of 128 RCA beam tested results from other researchers [22, 26-42, 46] and this study. Details of 128 beam tested results is given in Appendix A. Parameters in this study were RCA replacement ratio, compressive strength (f_c'), effective depth (d), shear span-to-effective depth ratio (a/d), and longitudinal reinforcement ratio (ρ_w). Shear force at failure in all of the tested beams was denoted V_{test}. V_c was calculated according to ACI 318-19 shear provisions expressed in equations (2) and equation (3).

Figure 9a shows the ratios of tested to calculated shear strength compared to the RCA replacement percentage. ACI 318-19 shear equation yielded conservative results when the percentage of RCA was below 75%, in which only two tested results failed below 1. For the beams where RCA completely substituted NCA, eight test results were unconservative.

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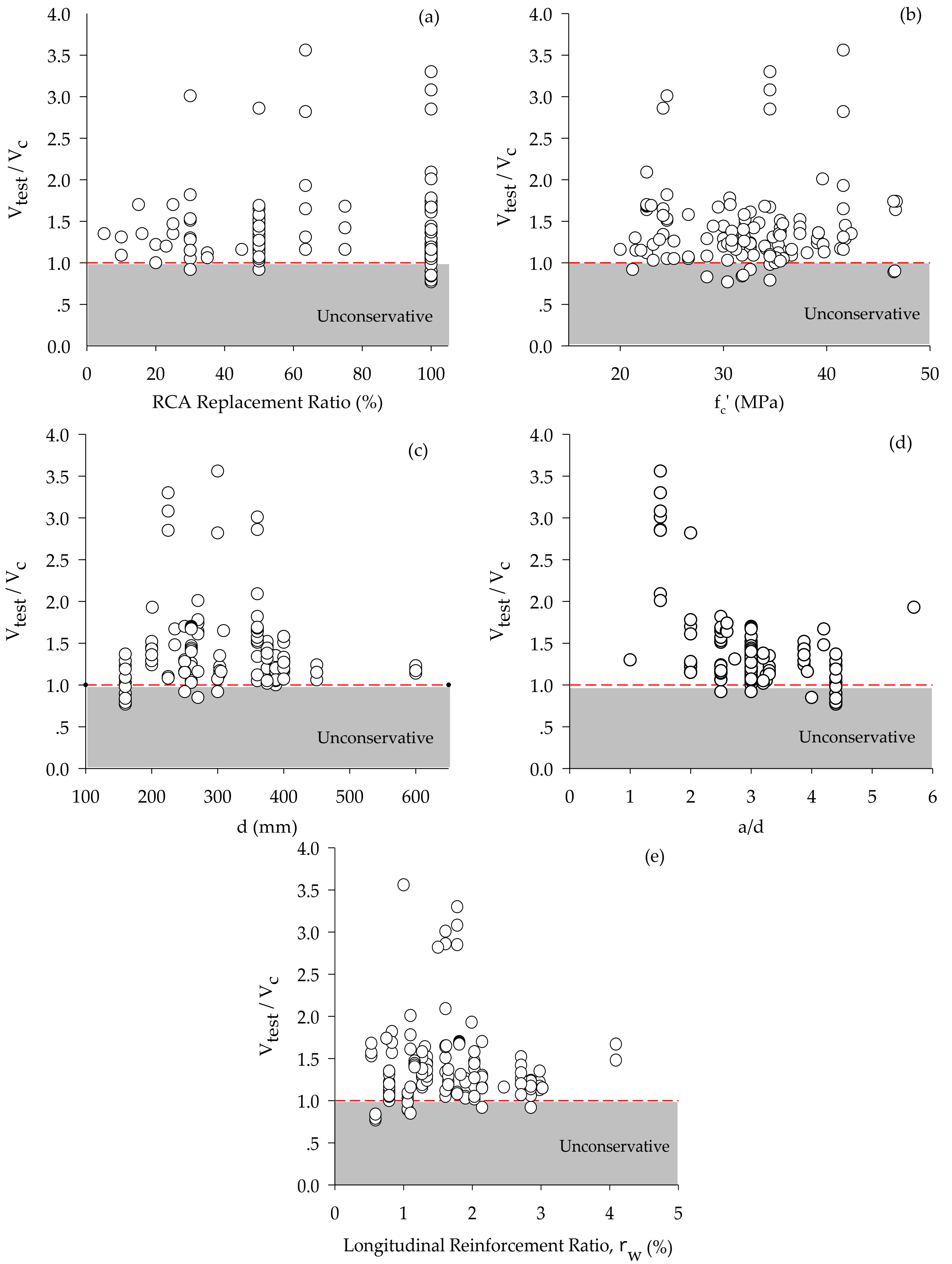

Figure 9. Comparison between V_{test} / V_c using ACI 318-19 shear provisions for different parameters.

A comparison between the ratios of shear strength and concrete compressive strength is plotted in Figure 9b. The concrete compressive strength ranged from 20 MPa

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to 47 MPa. Eight out of ten beams that failed below the shear strength predictions had compressive strength below 35 MPa.

Figures 9c compares the effects of an effective depth on the V_{test} / V_c. All unconservative results were from beams with an effective depth below 300 mm. Sixty-nine beams with an effective depth greater than 300 mm yielded conservative shear predictions.

One of the most important factors influencing the shear strength of reinforced concrete beams is the shear span-to-effective depth ratio (a/d). Figure 9d shows the effects of an a/d on the V_{test} / V_c. It is obvious that the ratios of shear strength were high for a/d = 1.5, where an arch action played an important role in providing shear resistance. For slender beams where a/d is at least 2.5 or higher, 10 unconservative results are observed out of 109 tests. Figure 9e shows the effect of ρ_w on the ratios of shear strength. Longitudinal reinforcement ratio (ρ_w) has been recently introduced to ACI 318-19 shear provisions. Before 2019, shear provisions of ACI 318 offered a simplified equation of shear strength, as shown in equation (1). Using this equation, other studies on the shear strength of RCA beams with a low longitudinal reinforcement ratio ($\rho_w < 1\%$) exhibited 18 unconservative results out of 128 tests [32], [42], [36] (Figure 10). On the other hand, ACI 318-19 predicted lower shear strength V_c for a low longitudinal reinforcement ratio, thus reducing unconservative V_{test} / V_c to only four tested beams for $\rho_w < 1\%$ (Figure 9e).

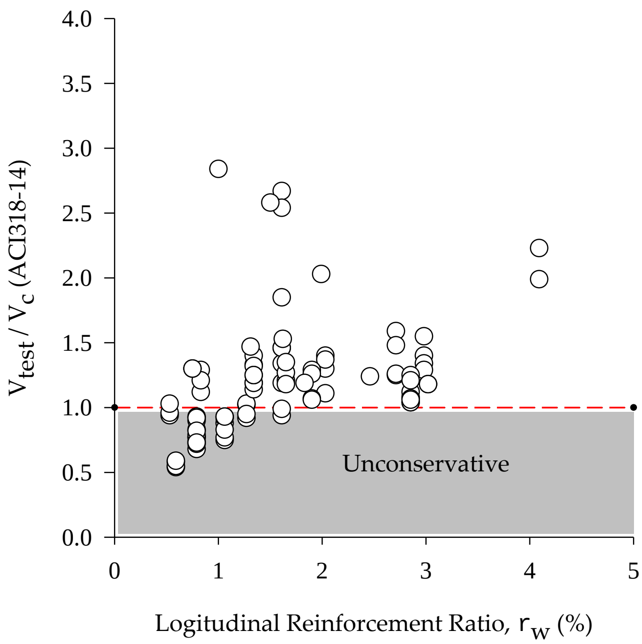

Figure 10. V_{test} / V_c using ACI 318-14 simplified shear equation for different ρ_w.

Table 6 shows the average, least conservative value, and coefficient of variation for V_{test} / V_c of beams with a/d greater than 2.5, in which ACI 318 shear equations are applicable for slender beams. ACI 318-14 yielded some unconservative results, while ACI 318-19 provided a higher average shear strength ratio.

Table 6. Summary of statistical values of shear strength ratio.

Code provision	**Ratios of V_{test} / V_c**		
	Average	Least conservative value	COV
ACI 318-14	1.15	0.54	0.25
ACI 318-19	1.28	0.77	0.30

Rahal and Alrefaei [32] studied shear strength of RCA beam by analyzing the database of 49 tested beams with a/d ≥ 3 and compared test results with vc from ACI 318-14. They found some unconservative results and recommended a reduction factor to account

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for the detrimental effect of using RCA. Their suggested modification of simplified ACI 318-14 shear equation was as follows:

$$v_c = 0.17\lambda_d \lambda_R \sqrt{f_c^{'}} \tag{4}$$

where v_c is the shear strength (MPa), f_c' is concrete compressive strength (MPa), λ_d is the reduction factor for lightweight aggregate, and λ_R is the reduction factor for RCA inclusion. Rahal and Alrefaei [32] recommended λ_R = 0.8 in concrete with RCA and 1.0 in concrete with NCA.

In this research study, a reduction factor for RCA incorporation, β_r, was proposed, as discussed in the following section.

5.2 *Model Validation and Compared to Existing Test Data*

Table 7 shows the impact reduction factor β_r on the number of unconservative results predicted by ACI 318-19 and ACI 318-14 when a/d is greater than 2.5, which is common for slender beams. It should be noted that unconserative V_{test}/V_c for the ACI 318-14 simplified shear equation are from the tested beams with ρ_w less than 1%.

Table 7. Number of unconservative values of shear strength ratio at different proposed λ_R.

β_r	Number of unconservative (V_{test} / V_c) by modified ACI 318-19	Number of unconservative (V_{test} / V_c) by ACI 318-14
0.90	6	7
0.85	4	7
0.80	2	7
0.75	0	4

Figure 11 compares V_{test} / V_c predicted by ACI 318-19 shear equations at different RCA levels. Based on the results from this plot, two β_r was proposed depending on the level of RCA replacement. For RCA replacement between 0% to 50%, β_r = 0.9 was proposed and for RCA beyond 50% replacement ratio, β_r = 0.75 was recommended so that ACI 318-19 shear equation yields unconservative shear strength ratio. Thus, the ACI 318-19 shear strength equation can be modified as follows:

$$V_c = 0.66\lambda_s \lambda \beta_r \left(\rho_w\right)^{1/3} \sqrt{f_c^{'}}bd \tag{5}$$

where ρ_w is the ratio of A_s to bd and λ_s is the size effect modification factor, defined in equation (3), λ is the reduction factor for lightweight aggregate, and β_r is the reduction factor for RCA incorporation: β_r = 0.75 for RCA replacement ratio between 0% to 50%, otherwise β_r = 0.9.

This β_r reduction factor accounts for the physical variations such as %replacement, source, density, % residual mortar and physical irregularity of RCA, as reported by several researchers [5], [14-18], and lower shear strength of reinforced concrete beams compared to NCA beams [30-34]. Prediction of shear strength in the database using equation (4) and (5) is shown in Appendix A. Equations (1), (2), (4), (5) yield average V_{test}/V_c equal to 1.27, 1.40, 1.59, and 1.72 respectively. It is evident that V_{test} / V_c is the highest at 1.72 yields conservative prediction for all range of RCA replacements when using equation 5, which includes β_r proposed in this study.

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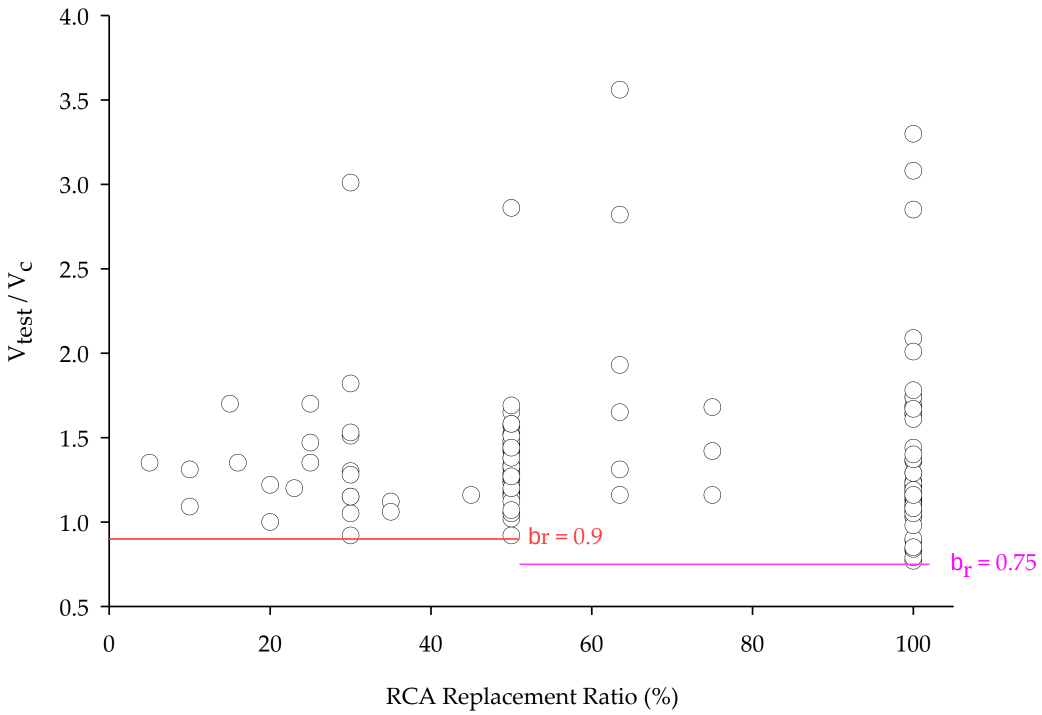

Figure 11. The proposed reduction factor for RCA to shear strength predicted by ACI 318-19 shear equation

5.3 Design recommendations

Based on the tested results of 128 RCA beams without stirrups found in literature, the reduction factor $\lambda_r = 0.75$ is suggested for ACI 318-19 shear equation to yield the best results without unconservative shear strength ratio. Normal aggregate sizes below 25 mm is recommended as the tested beams in the databased used RCA sizes less than 25 mm. However, the nature of shear behavior of reinforced concrete beams is complexed and more tests on shear strength of RCA beams is required for future work.

6. Conclusions and future works

This article investigates the shear strength of concrete beams with RCA as coarse aggregate. Different RCA replacements and amount of longitudinal reinforcement were used in the concrete beams. Predictions of concrete shear strength from the current ACI provisions were evaluated together with the existing test data available in the literature. The modified current ACI equation to predict concrete shear strength was proposed for the different RCA replacement.

Based on the test results of ten beams and analysis of the 128 RAC beams from literature, the following conclusions can be drawn.

- For beams with a longitudinal reinforcement ratio of 1.16%, normalized shear stress of 100% RAC beam was lower than that of NAC counterpart by 5%.
- Beams with a higher longitudinal reinforcement ratio of 1.81% showed a minimal difference between the normalized shear stress of RAC and NAC beams.
- Shear failure modes of RAC beams were similar to those of NAC beams. However, crack inclination angles of NAC beams were slightly higher.
- Current ACI 318-19 shear equation conservatively estimated the shear strength of RAC beams when the replacement percentage was less than 75%.
- For longitudinal reinforcement ratio less than 1.8%, ACI 318-19 shear equation yielded lower shear strength, V_c, than the ACI 318-14 simplified equation, thus increasing the safety factor of shear stress ratios found in previous tests in the literature.

- A reduction factor of 0.75 is proposed to the current ACI code provision to account for the physical variations of RCA such as % replacement, source, density, % residual mortar and physical irregularity.
- The modified ACI equation for predicting concrete shear strength of RAC is calibrated using 8 test data carried out by the authors, and then further verified and calibrated against 120 test data from the literature.
- It was found that the use of modified ACI equation as a design recommendation for predicting concrete shear strength of RAC gives conservative predictions for all levels of RCA up to 100%replacement.

It is noted that the design recommendation to include the reduction factor to the modified version of ACI code provision, is empirically derived which based on the 128-test data available in the literature. More rational shear tests on RAC members are required in order to fully assess physical variations of recycled concrete to be used as coarse aggregate replacement.

Author Contributions: Conceptualization, M.S. and S.L[a].; methodology, M.S. and S.L[a].; formal analysis, M.S.; investigation, S.L[a]. and T.I.; writing—original draft preparation, M.S. and S.L[a].; writing—review and editing, M.S., T.I. and R.G.; supervision, S.L[b]. and T.I.

Funding: This research was funded by Walailak University research grant, grant number WU63217. This project was also funded by National Research Council of Thailand (NRCT5-RSA63019-04).

Data Availability Statement: Not applicable

Acknowledgments: The authors acknowledge the financial support provided by Walailak University research grant (contract no. WU63217). The authors thankfully acknowledge Sanhanut Sombatthong, Thiraphongsa Thamsiriphaiboon, Phongsakorn Srisiri, Pattarapol Meeprapai, Jenwit Chookaew, and Pattarawan Thavorn for their contribution during the experimental program.

Conflicts of Interest: The authors declare no conflict of interest.

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Appendix A : Comparison of concrete shear strength of RCA in concrete beams without shear reinforcement by current ACI codes and modfied equations.

Reference	ID	% RCA	fc′ (MPa)	b (mm)	d (mm)	a/d	ρ_w (%)	V_{test} (kN)	Eq. (1)	Eq. (2)	Eq. (4)	Eq. (5)
This study (2021)	1	25	35.7	200	260	3.08	1.16	67.50	52.9	46.0	42.3	41.4
	2	50	29.0	200	260	3.08	1.16	59.50	47.6	41.4	38.1	37.3
	3	75	32.9	200	260	3.08	1.16	62.50	50.7	44.2	40.6	33.1
	4	100	31.9	200	260	3.08	1.16	61.00	50.0	43.5	40.0	32.6
	5	25	30.7	200	260	3.08	1.81	83.85	48.9	49.4	39.2	44.5
	6	50	23.0	200	260	3.08	1.81	72.50	42.4	42.8	33.9	38.5
	7	75	34.0	200	260	3.08	1.81	87.50	51.6	52.0	41.2	39.0
	8	100	29.5	200	260	3.08	1.81	81.15	48.0	48.5	38.4	36.3
Wardeh and Ghorbel (2019) [35]	9	100	34.5	200	225	1.5	1.78	130.00	44.9	45.6	35.9	34.2
	10	100	34.5	200	225	1.5	1.78	150.30	44.9	45.6	35.9	34.2
	11	100	34.5	200	225	1.5	1.78	140.40	44.9	45.6	35.9	34.2
	12	100	34.5	200	225	3	1.78	50.20	44.9	45.6	35.9	34.2
	13	100	34.5	200	225	3	1.78	49.00	44.9	45.6	35.9	34.2
Pradhan et al. (2018) [34]	14	100	46.7	200	270	2.6	1.31	92.28	62.7	56.3	50.2	42.2
	15	100	46.8	200	270	2.6	0.75	81.29	62.8	46.8	50.2	35.1
	16	100	46.5	200	270	2.6	0.75	81.10	62.6	46.6	50.1	35.0
Etman et al. (2018) [33]	17	15	22.6	150	250	2	2.14	55.50	30.3	32.6	24.2	29.4
	18	30	21.5	150	250	2	2.14	36.50	29.6	31.9	23.7	28.7
	19	45	20.0	150	250	2	2.14	35.50	28.5	30.7	22.8	27.7
	20	30	21.4	150	250	1	2.14	41.50	29.5	31.8	23.6	28.6
	21	30	21.2	150	250	3	2.14	29.00	29.4	31.6	23.5	28.5
	22	30	23.8	150	250	2	2.14	43.00	31.1	33.5	24.9	30.2
	23	30	22.0	150	250	2	2.14	37.00	29.9	32.2	23.9	29.0
Ignjatović et al. (2017) [29]	24	50	33.4	200	235	4.2	4.09	91.75	46.2	61.8	37.0	55.6
	25	100	34.5	200	235	4.2	4.09	104.75	46.9	62.8	37.5	47.1
	26	10	36.6	150	388	3	0.79	44.50	59.9	41.0	47.9	36.9

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Reference	ID	% RCA	fc′ (MPa)	b (mm)	d (mm)	a/d	ρ_w (%)	V_{test} (kN)	Eq. (1)	Eq. (2)	Eq. (4)	Eq. (5)
Rahal and Al-refaei (2017) [32]	27	20	35.0	150	388	3	0.79	40.05	58.5	40.1	46.8	36.1
	28	20	35.3	150	388	3	0.79	48.90	58.8	40.2	47.0	36.2
	29	35	35.3	150	388	3	0.79	45.05	58.8	40.2	47.0	36.2
	30	50	38.1	150	388	3	0.79	46.95	61.1	41.8	48.9	37.6
	31	75	36.6	150	388	3	0.79	47.40	59.9	41.0	47.9	30.7
	32	100	35.8	150	388	3	0.79	42.50	59.2	40.5	47.4	30.4
	33	5	37.4	150	388	3	0.79	56.00	60.5	41.4	48.4	37.3
	34	10	34.8	150	388	3	0.79	52.50	58.4	40.0	46.7	36.0
	35	16	35.4	150	388	3	0.79	54.20	58.9	40.3	47.1	36.3
	36	23	34.0	150	388	3	0.79	47.25	57.7	39.5	46.2	35.5
	37	35	35.1	150	388	3	0.79	42.50	58.6	40.1	46.9	36.1
Katkhuda and Shatarat (2016) [39]	38	50	25.2	206	260	2	1.90	58.94	45.7	46.9	36.6	42.2
	39	50	25.2	206	260	3	1.90	49.07	45.7	46.9	36.6	42.2
	40	100	23.2	206	260	2	1.90	55.04	43.9	45.0	35.1	33.7
	41	100	23.2	206	260	3	1.90	46.45	43.9	45.0	35.1	33.7
Sadati et al. (2016) [38]	42	50	32.0	305	375	3.2	1.27	117.40	110.0	89.1	88.0	80.2
	43	50	35.5	305	375	3.2	2.03	111.60	115.9	109.7	92.7	98.8
	44	50	32.0	305	400	3	2.71	151.20	117.3	120.0	93.9	108.0
	45	50	35.5	305	400	3	1.27	148.60	123.6	98.2	98.9	88.4
	46	50	32.0	305	400	3	2.03	171.70	117.3	109.0	93.9	98.1
	47	50	35.5	305	400	3	2.71	168.60	123.6	126.4	98.9	113.8
	48	50	30.8	305	375	3.2	1.27	120.50	107.9	87.4	86.3	78.7
	49	50	26.6	305	375	3.2	2.03	99.90	100.3	95.0	80.2	85.5
	50	50	30.8	305	400	3	2.71	140.80	115.1	117.7	92.1	105.9
	51	50	26.6	305	400	3	1.27	134.60	107.0	85.0	85.6	76.5
	52	50	30.8	305	400	3	2.03	136.30	115.1	106.9	92.1	96.2
	53	50	26.6	305	400	3	2.71	116.80	107.0	109.4	85.6	98.5
	54	50	32.1	300	400	3	1.27	117.50	115.6	91.8	92.5	82.6

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Reference	ID	% RCA	fc′ (MPa)	b (mm)	d (mm)	a/d	ρ_w (%)	V_{test} (kN)	Eq. (1)	Eq. (2)	Eq. (4)	Eq. (5)
Arezoumandi (2014 & 2015) [30, 31]	55	50	32.1	300	375	3	2.03	151.30	108.4	102.6	86.7	92.4
	56	50	32.1	300	375	3	2.71	171.80	108.4	113.0	86.7	101.7
	57	50	35.5	300	400	3	1.27	111.70	121.6	96.6	97.2	86.9
	58	50	35.5	300	375	3	2.03	148.60	114.0	107.9	91.2	97.1
	59	50	35.5	300	375	3	2.71	168.70	114.0	118.9	91.2	107.0
	60	100	30.0	300	400	3	1.27	114.80	111.7	88.8	89.4	66.6
	61	100	30.0	300	375	3	2.03	143.20	104.8	99.2	83.8	74.4
	62	100	30.0	300	375	3	2.71	131.40	104.8	109.3	83.8	81.9
	63	100	34.1	300	400	3	1.27	113.00	119.1	94.6	95.3	71.0
	64	100	34.1	300	375	3	2.03	124.10	111.7	105.8	89.3	79.3
	65	100	34.1	300	375	3	2.71	140.30	111.7	116.5	89.3	87.4
Knaack and Kurama (2014) [28]	66	50	41.8	150	200	3.875	1.34	44.00	33.0	30.4	26.4	27.4
	67	50	41.8	150	200	3.875	1.34	39.10	33.0	30.4	26.4	27.4
	68	50	37.4	150	200	3.875	1.34	43.70	31.2	28.8	25.0	25.9
	69	50	37.4	150	200	3.875	1.34	41.20	31.2	28.8	25.0	25.9
	70	100	39.1	150	200	3.875	1.34	36.40	31.9	29.4	25.5	22.1
	71	100	39.1	150	200	3.875	1.34	38.00	31.9	29.4	25.5	22.1
	72	100	39.2	150	200	3.875	1.34	39.90	31.9	29.4	25.5	22.1
	73	100	39.2	150	200	3.875	1.34	39.90	31.9	29.4	25.5	22.1
Kim et al. (2013) [40]	74	50	32.6	200	300	2.5	2.85	60.60	58.2	65.9	46.6	59.3
	75	50	32.6	200	450	2.5	2.85	108.90	87.4	87.6	69.9	78.8
	76	50	32.6	200	600	2.5	2.85	126.10	116.5	105.9	93.2	95.3
	77	50	32.6	300	450	2.5	3.02	154.20	131.0	133.9	104.8	120.5
	78	50	32.6	400	600	2.5	2.85	261.50	233.0	211.9	186.4	190.7
	79	100	34.9	200	300	2.5	2.85	72.90	60.3	68.1	48.2	51.1
	80	100	34.9	200	450	2.5	2.85	96.40	90.4	90.6	72.3	67.9
	81	100	34.9	200	600	2.5	2.85	125.10	120.5	109.6	96.4	82.2
	82	100	34.9	300	450	2.5	3.02	159.80	135.6	138.5	108.5	103.9

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Reference	ID	% RCA	fc′ (MPa)	b (mm)	d (mm)	a/d	ρ_w (%)	V_{test} (kN)	Eq. (1)	Eq. (2)	Eq. (4)	Eq. (5)
	83	100	34.9	400	600	2.5	2.85	256.60	241.0	219.2	192.8	164.4
Fathifazl et al. (2011) [22]	84	63.5	41.6	200	300	1.5	1.00	186.70	65.8	52.5	52.6	39.4
	85	63.5	41.6	200	300	2	1.50	169.50	65.8	60.1	52.6	45.0
	86	63.5	41.6	200	309	2.59	1.62	103.90	67.8	63.0	54.2	47.2
	87	63.5	41.6	200	201	5.69	1.99	89.30	44.1	46.4	35.3	34.8
	88	63.5	41.6	200	305	3.93	2.46	83.20	66.9	71.7	53.5	53.8
	89	63.5	41.6	200	381	2.73	1.83	99.50	83.6	76.1	66.8	57.1
Choi et al. (2010) [41]	90	30	24.5	200	360	1.5	1.61	161.70	60.6	53.8	48.5	48.4
	91	30	24.5	200	360	2.5	1.61	81.34	60.6	53.8	48.5	48.4
	92	30	24.5	200	360	3.25	1.61	56.70	60.6	53.8	48.5	48.4
	93	30	24.5	200	360	2.5	0.53	56.70	60.6	37.1	48.5	33.4
	94	30	24.5	200	360	2.5	0.83	78.40	60.6	43.1	48.5	38.8
	95	50	24.2	200	360	1.5	1.61	152.88	60.2	53.4	48.1	48.1
	96	50	24.2	200	360	2.5	1.61	87.90	60.2	53.4	48.1	48.1
	97	50	24.2	200	360	3.25	1.61	71.54	60.2	53.4	48.1	48.1
	98	50	24.2	200	360	2.5	0.53	57.82	60.2	36.9	48.1	33.2
	99	50	24.2	200	360	2.5	0.83	67.13	60.2	42.8	48.1	38.5
	100	100	22.6	200	360	1.5	1.61	107.80	58.1	51.6	46.5	38.7
	101	100	22.6	200	360	2.5	1.61	84.77	58.1	51.6	46.5	38.7
	102	100	22.6	200	360	3.25	1.61	57.77	58.1	51.6	46.5	38.7
	103	100	22.6	200	360	2.5	0.53	59.78	58.1	35.6	46.5	26.7
	104	100	22.6	200	360	2.5	0.83	70.07	58.1	41.4	46.5	31.0
González-Fonteboa and Martínez-Abella (2007) [27]	105	100	39.7	200	303	3.3	2.98	90.64	64.9	74.2	51.9	55.7
Etxeberria et al. (2007) [26]	106	25	42.4	200	303	3.3	2.98	104.00	67.1	76.8	53.7	69.1
	107	50	41.3	200	303	3.3	2.98	89.00	66.2	75.8	53.0	68.2

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Reference	ID	% RCA	fc′ (MPa)	b (mm)	d (mm)	a/d	ρ_w (%)	V_{test} (kN)	Eq. (1)	Eq. (2)	Eq. (4)	Eq. (5)
	108	100	39.8	200	303	3.3	2.98	84.00	65.0	74.3	52.0	55.8
Sato et al. (2007) [42]	109	100	46.5	150	160	4.4	1.06	21.00	27.8	23.7	22.3	17.8
	110	100	32.9	150	160	4.4	1.06	21.70	23.4	20.0	18.7	15.0
	111	100	46.6	150	160	4.4	1.06	21.40	27.9	23.8	22.3	17.8
	112	100	30.4	150	160	4.4	0.59	12.10	22.5	15.8	18.0	11.8
	113	100	28.4	150	160	4.4	0.59	12.60	21.7	15.3	17.4	11.4
	114	100	34.5	150	160	4.4	0.59	13.20	24.0	16.8	19.2	12.6
	115	100	31.8	150	160	4.4	0.59	13.50	23.0	16.1	18.4	12.1
	116	100	30.4	150	160	4.4	1.06	19.70	22.5	19.2	18.0	14.4
	117	100	28.4	150	160	4.4	1.06	20.00	21.7	18.5	17.4	13.9
	118	100	34.5	150	160	4.4	1.06	20.00	24.0	20.4	19.2	15.3
	119	100	31.8	150	160	4.4	1.06	21.40	23.0	19.6	18.4	14.7
	120	100	30.4	150	160	4.4	1.65	27.30	22.5	22.2	18.0	16.7
	121	100	28.4	150	160	4.4	1.65	27.70	21.7	21.5	17.4	16.1
	122	100	34.5	150	160	4.4	1.65	28.30	24.0	23.7	19.2	17.8
	123	100	31.8	150	160	4.4	1.65	31.10	23.0	22.7	18.4	17.1
Han et al. (2001) [46]	124	100	39.6	170	270	1.5	1.10	83.50	49.1	41.6	39.3	31.2
	125	100	30.6	170	270	2	1.10	65.20	43.2	36.6	34.5	27.4
	126	100	32.6	170	270	2	1.10	60.60	44.6	37.7	35.6	28.3
	127	100	31.2	170	270	3	1.10	42.70	43.6	36.9	34.9	27.7
	128	100	31.9	170	270	4	1.10	31.70	44.1	37.3	35.3	28.0

Note: b = the width of cross-section, d = effective depth, a = shear span, ρ_w = longitudinal reinforcement ratio

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