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ORIGINAL PAPER

Shear strength of self-compacting concrete and recycled aggregate concrete beams: an appraisal of design codes

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Abstract
Provisions are given in design codes for the calculation of shear strength of conventional concrete (CC) beams. In this paper, a database is generated for self-compacting concrete (SCC) and recycled aggregate concrete (RAC) slender beams with and without shear reinforcement which were investigated for shear strength. Shear capacities of 103 SCC beams and 109 RAC beams with and without shear reinforcement are calculated using the provisions of ACI 318-14, JSCE-2007, NZS 3101-2006 and AS 3600-2009. Calculated nominal shear strengths (V_n) are compared with the experimental shear strengths (V_{exp}) and statistical parameters are obtained for each code. It was found that all the four codes yielded unconservative estimates of the shear capacities for SCC and RAC beams without shear reinforcement having longitudinal reinforcement less than 1% and depth greater than 450 mm. All the four codes produced reasonable and conservative estimates of the shear capacities of SCC and RAC beams with shear reinforcement. AS 3600-2009 produced minimum average of V_{exp}/V_n with least scatter but at the same time it yielded maximum unconservative results. A modification in the depth factor of AS 3600-2009 reduced the percentage of unconservative results from 18.67 to 7.8% for SCC beams and 24.67 to 8% for RAC beams without any increase in coefficient of variation (COV).

Keywords Shear strength · Self-compacting concrete · Recycled aggregate concrete · Statistical analysis

Introduction

Quantity and size of coarse aggregate in self-compacting concrete (SCC) is generally lower than that of CC of the same strength class. Recycled aggregate concrete (RAC) has two interfacial transition zones, one is the old one between mortar and coarse aggregate and the other is between the recycled coarse aggregate (RCA) and mortar. The old ITZ is generally considered as the weak link in RAC.

Shear strength is one of the most investigated parameters of all the structural aspects of reinforced concrete due to the fact that shear failure usually occurs without giving any warning and may lead to casualties. Most of the research studies on shear strength were confined to the beams produced using conventional concrete (CC). Today, use of self-compacting concrete (SCC) and recycled aggregate concrete (RAC) is increasing enormously in the construction industry due to their numerous advantages. Use of SCC reduces the energy required for the transportation and compaction of concrete especially in high-rise buildings and structural elements with dense reinforcement. It also reduces the noise pollution due to the elimination of the use of concrete vibrators (Sonebi and Bartos 2002; Okamura and Ouchi 2003; Khatib 2008). RAC is a need of present time due to increasing rate of demolition of existing structures. Waste generated in the process is utilized for the production of concrete in the construction industry at a large scale (Xiao et al. 2005; Kapoor et al. 2016a, b; Xiao 2018).

Shear forces imposed on a beam are resisted by its internal shear forces generated through aggregate interlock, uncracked concrete in compression zone, dowel force of the longitudinal reinforcement and shear resisted by shear reinforcement (if present). Aggregate interlock plays a significant role in the transfer of shear stresses through a crack in concrete (Zsutty 1971; Okamura and Higai 1980; Mphonde and Frantz 1984; Ashour et al. 1992; Ahmad et al. 2018). SCC has lower amount of coarse aggregate content than

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CC (Sonebi and Bartos 2002; Ahmad and Umar 2018). It is established from the previous studies that reducing the content of coarse aggregate in concrete will reduce shear transfer through aggregate interlock action resulting in the reduction of its shear strength (Walraven 1981). As far as the shear strength of SCC beams is concerned, conclusions given by different authors are contradictory. Experimental results of Hassan et al. (2008) and Helincks et al. (2013) suggested that the difference in shear strength of SCC and CC beams with identical characteristics was found to be insignificant. Lin and Chen (2012) tested CC beams, SCC1 beams and SCC2 beams for the shear strength. Amount of coarse aggregate in CC and SCC1 beams was kept equal while SCC2 beams had lower coarse aggregate content than CC and SCC1 beams. Shear strengths of SCC1 and SCC2 beams were found to be greater and smaller than that of CC beams, respectively. Lima de Resende et al. (2016) concluded that ultimate shear stress of CC and SCC beams may or may not be equal depending upon the composition of concrete, beams size and the shear reinforcement ratio. Ultimate shear stress of the high-strength SCC beams with small transverse reinforcement ratio was found to be significantly lower than that of CC beams with identical characteristics.

Researchers also have different opinions that the shear strength of RAC beams will decrease or increase as compared to that of CC beams with the same strength class. Gonzalez-Fonteboa and Martinez-Abella (2007) tested beams having 50% RCA with a longitudinal reinforcement ratio of 3%. No significant differences in terms of deflection and shear strength were observed between CC beams and RAC beams. On the basis of experimental study on 20 beams having 0, 30, 50 and 100% RCA replacement ratio, Choi et al. (2010) concluded that shear strength of beam reduces for higher percentages of RCA. Fathifazl et al. (2010) reported that shear strength of RAC beams, both in terms of ultimate shear strength and deflection, are superior to that of beams made entirely with NCA.

In the literature, some papers discussed the accuracy of shear strength models for conventional concrete beams but none of them addressed the accuracy of shear strength models for beams produced with SCC and RAC (Hirata et al. 2013; Ahmad and Bhargava 2018). The inconsistencies among the results of shear strength of SCC and RAC beams raises a question that whether the provisions of shear in current design codes can be used to calculate the shear capacities of SCC and RAC beams? To answer this question, a database of SCC and RAC slender beams with and without shear reinforcement is generated. Experimental shear strengths are compared with those calculated by the ACI building code ACI-314-14, JSCE guideline no. 15 (2007), New Zealand concrete structures standard NZS 3101-2006 and Australian standard AS 3600-2009.

Shear design provisions

This section briefly discusses the shear design provisions of the above-mentioned design codes. This study is confined to the reinforced concrete beams without axial force; therefore, design provisions of non-prestressed members without axial forces are discussed. All the equations presented in this section are based on SI units.

ACI-318-14

Nominal shear strength of slender RC beams is calculated by adding the concrete contribution (V_c) and shear reinforcement contribution (V_s):

$$V_n = V_c + V_s,$$

$$V_c = \left(\sqrt{f_c'} + 120\rho\frac{V_u d}{M_u}\right)\frac{bd}{7} \le 0.3\sqrt{f_c'}bd,$$

where f_c' is the cylindrical compressive strength of concrete. ρ is the percentage of longitudinal reinforcement. V_u and M_u are the maximum factored shear force and bending moment, respectively, in the beam during application of external load. b and d are the width and effective depth of the beam, respectively.

$$V_s = \frac{A_v f_{vy} d}{s} \le 0.66\sqrt{f_c'}bd,$$

where A_v is the total area of shear reinforcement placed in s. s is the spacing of shear reinforcement. f_{vy} is the yield strength of stirrups ≤ 420 MPa.

Japanese code (JSCE guideline no. 15, 2007)

Design shear capacity of a member (V_{yd}) is calculated by the following equation:

$$V_{yd} = V_{cd} + V_{sd},$$

where V_{cd} is the design shear capacity of members without shear reinforcement, which is given by

$$V_{cd} = \beta_d \beta_p \beta_n f_{cvd} bd/\gamma_b,$$

where $f_{cvd} = 0.20\sqrt[3]{f_{cd}} \le 0.72\text{MPa}$. $\beta_d = \sqrt[4]{1000/d}$, when $\beta_d > 1.5$ take $\beta_d = 1.5$; d is in mm. $\beta_p = \sqrt[3]{100\rho}$, when $\beta_p > 1.5$ take $\beta_p = 1.5$. $\beta_n = 1$ for members without axial force. f_{cd} is the design compressive strength of concrete in MPa. b and d are the width and effective depth of the beam, respectively. ρ is the percentage of longitudinal reinforcement. V_{sd} is the shear capacity provided by shear reinforcement:

$$V_{sd} = \frac{A_w f_{wyd} d}{S_s} \times \frac{z}{\gamma_b},$$

where A_w is the total area of shear reinforcement placed in S_s. S_s is the spacing of shear reinforcement. f_{wyd} is the design yield strength of shear reinforcement. $f_{wyd} \leq 400$ MPa if $f'_c < 60$ MPa up to 800 MPa if $f'_c > 60$ MPa. z is the distance between resultant of compressive stress and centroid of tension reinforcement, generally taken as $0.87d$. γ_b is the member factor, generally taken as 1.10.

New Zealand standard (NZS 3101-2006)

As in ACI 318-14, NZS 3101-2006, calculate the nominal shear strength of an RC beam (V_n) by combining the concrete contribution (V_c) and shear reinforcement contribution (V_s):

$$V_n = V_c + V_s,$$

$$V_c = k_a k_d v_b bd,$$

$$0.08\sqrt{f'_c} \leq v_b \leq (0.07 + 10\rho)\sqrt{f'_c} \leq 0.2\sqrt{f'_c},$$

where k_a is a factor that accounts for the maximum size of the aggregate. $k_a = 0.85$ if maximum aggregate size is 10 mm or less. $k_a = 1.0$ if maximum aggregate size is 20 mm or more. The value of k_a is linearly interpolated if the maximum aggregate size is between 10 and 20 mm. k_d is a factor that accounts for the size of the beam. $k_d = \left({}^{400}/_{d}\right)^{0.25}$, when $d > 400$ mm. d is in mm. k_d is taken as unity for the beams having shear reinforcement equal to or more than the minimum shear reinforcement. f'_c is the cylindrical compressive strength of the concrete. ρ is the percentage of longitudinal reinforcement. b and d are the width and effective depth of the beam, respectively. V_s is same as defined in "ACT-318-14". But $f_{vy} \leq 500$ MPa.

Australian standard (AS 3600-2009)

The nominal shear strength of a beam is calculated by adding the ultimate shear strength provided by concrete (V_{uc}) and shear reinforcement (V_{us}):

$$V_n = V_{uc} + V_{us},$$

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left(\frac{A_{st}}{b_v d_o}\right)^{1/3}.$$

For members with shear reinforcement equal to or greater than minimum shear reinforcement

$$\beta_1 = 1.1\left(1.6 - {}^{d_o}/_{1000}\right) \geq 1.1.$$

Otherwise,

$$\beta_1 = 1.1\left(1.6 - {}^{d_o}/_{1000}\right) \geq 0.8.$$

For members subjected to pure bending, i.e., without axial tension or compression

$\beta_2 = 1$, for members without axial loads. $\beta_3 = 1$, or $2d_o/a_v$ but not greater than 2, if diagonal compression is generated over the length a_v. a_v is the distance between support and the section where shear force is considered. $f_{cv} = f_c'^{1/3} \leq 4$ MPa. b_v and d_o are the width and effective depth of the beam, respectively. A_{st} is the area of steel provided in the tension zone.

$$V_{us} = \left(\frac{A_{sv} f_{sy\cdot f} d_0}{s}\right)\cot\theta_v,$$

A_{sv} is the cross-sectional area of shear reinforcement provided in s. s is the spacing of shear reinforcement. $f_{sy\cdot f}$ is the yield strength of stirrups or fitments ≤ 500 MPa. θ_v is the angle between compression strut and longitudinal reinforcement.

Comparison between experimental and calculated shear strengths

Experimental shear strengths (V_{exp}) are compared with the nominal shear capacities (V_n) calculated using the procedures of ACI 318-14, JSCE-2007, NZS 3101-2006 and AS 3600-2009 as discussed above. All the beams considered in this study were slender beams ($a/d > 2.5$) tested in pure flexure without any axial loads. Beams were having rectangular cross section, with or without shear reinforcement. Ratio of experimental to nominal shear strength (V_{exp}/V_n) of beams calculated as per the procedure of the above-mentioned building codes is termed as the strength ratio in this paper. Strength ratio greater than unity shows that the prediction is conservative while the shear strength ratio less than unity indicates unconservative predictions. Nominal shear strengths of the beams were calculated without considering the strength reduction factors.

Self-compacting concrete beams

Summary of the SCC beams considered in the study are given in Table 1 and their details are given in "Appendix". A total number of 103 beams were considered in the study out of which 75 were without stirrups and 28 were with stirrups. Range of concrete compressive strength (f'_c), shear-span-to-depth ratio (a/d), width (b), effective depth (d), percentage of longitudinal reinforcement (ρ), shear reinforcement index ($\rho_v f_y$) are given in Table 1.

Table 1 Summary of SCC beams considered in the study

Author (s)	No. of beams	f'_c (MPa)	a/d	b (mm)	d (mm)	ρ (%)	$\rho_v f_y$ (MPa)
Hassan et al. (2008)	10	45	2.5	400	100–667.5	1–2	0
Boel et al. (2010)	4	55.8–60.7	2.5–3	100	130	1.21	0
Safan (2012)	28	26–75	2.59–2.61	100	134–135	1.16–1.68	0
Lin and Chen (2012)	16	30.4–49.1	2.5–3.5	240	298	4.1	1.22–1.64
Arezoumandi and Volz (2013)	7	34.8–53.5	3–3.2	300	375–400	1.27–2.71	0–0.8
Helincks et al. (2013)	12	48-54.8	2.5-3	100	130	1–2	0
Biolzi et al. (2014)	8	42.64	2.5–4	170	260	0.909	0–1.31
Lima de Resende et al. (2016)	6	71.6	3	175	407–417	2–2.5	0.508–0.975
Alghazali and Myers (2017)	12	45.9–53.5	3	305	406.67	1.69–2.71	0–0.41
Total	103	26–71.6	2.5–4	100–400	100–667.5	0.909–4.1	0–1.64

Table 2 Statistical parameters for SCC beams

Procedure	Maximum	Minimum	Average	COV	Uncon-servative (%)
Beams without shear reinforcement ($\rho_v f_y = 0$)					
ACI 318-14	1.98	0.79	1.36	0.18	6.67
JSCE-2007	1.7	0.81	1.28	0.15	6.67
NZS 3101-2006	1.86	0.75	1.22	0.18	12
AS 3600-2009	1.43	0.72	1.11	0.14	18.67
Beams with shear reinforcement ($\rho_v f_y \geq \rho_v f_{y,min}$)					
ACI 318-14	1.97	1.04	1.57	0.18	0
JSCE-2007	1.93	0.93	1.55	0.17	3.57
NZS 3101-2006	1.90	1.01	1.51	0.19	0
AS 3600-2009	1.70	0.93	1.28	0.16	5.67

Overall evaluation

For SCC beams, maximum, minimum and average strength ratios, coefficient of variation and percentage of the unconservative results for different code provisions are given in Table 2. Graphical representations of strength ratio (V_{exp}/V_n) with experimental ultimate shear stress for SCC beams are shown in Fig. 1a, b. Statistical parameters given in Table 2 and graphical representation in Fig. 1a, b show that procedure of AS 3600-2009 produced least scatter for SCC beams with and without shear reinforcement since most of the points are concentrated near the line representing a strength ratio of unity. At the same time, this procedure produced maximum number of unconservative strength ratios.

Detailed evaluation

ACI 318-14 For beams without shear reinforcement, average strength ratio and coefficient of variation of strength ratio were found to be 1.36 and 18%, respectively. Only 6.67% of the results were found to be unconservative. Unconservative results include the beams tested by Hassan et al. (2008) having an effective depth of more than 450 mm and Biolzi et al. (2014) having longitudinal reinforcement smaller than 1%. For beams with shear reinforcement, maximum strength ratio, minimum strength ratio, average strength ratio and coefficient of variation of strength ratio were found to be 1.97. 1.04, 1.57 and 18%, respectively. None of the strength ratios was found to be unconservative.

JSCE-2007 Average strength ratio and COV were found to be 1.28 and 15%, respectively, for SCC beams without shear reinforcement. 6.67% of the results were found to be unconservative when JSCE method was found for calculating the nominal shear strength of SCC beams without stirrups. Unconservative strength ratios were found for the beams tested by Biolzi et al. (2014) with longitudinal reinforcement smaller than 1%. Strength ratios for the beams tested by Alghazali and Myers (2017) having high-volume fly ash and effective depth greater than 400 mm were also found to be unconservative. For beams with shear reinforcement, average strength ratio and COV were found to be 1.55 and 17%, respectively. Out of 28 beams with shear reinforcement, strength ratio of only one beam tested by Lima de Resende et al. (2016) having a concrete compressive strength of 71.6 MPa was found to be unconservative.

NZS 3101-2006 Average strength ratio and COV for SCC beams without shear reinforcement were found to be 1.22 and 18%, respectively. 14.67% predicted results were found to be unconservative. Unconservative results include the beams tested by Arezoumandi and Volz (2013) having beam depth greater than 450 mm and beams prepared with

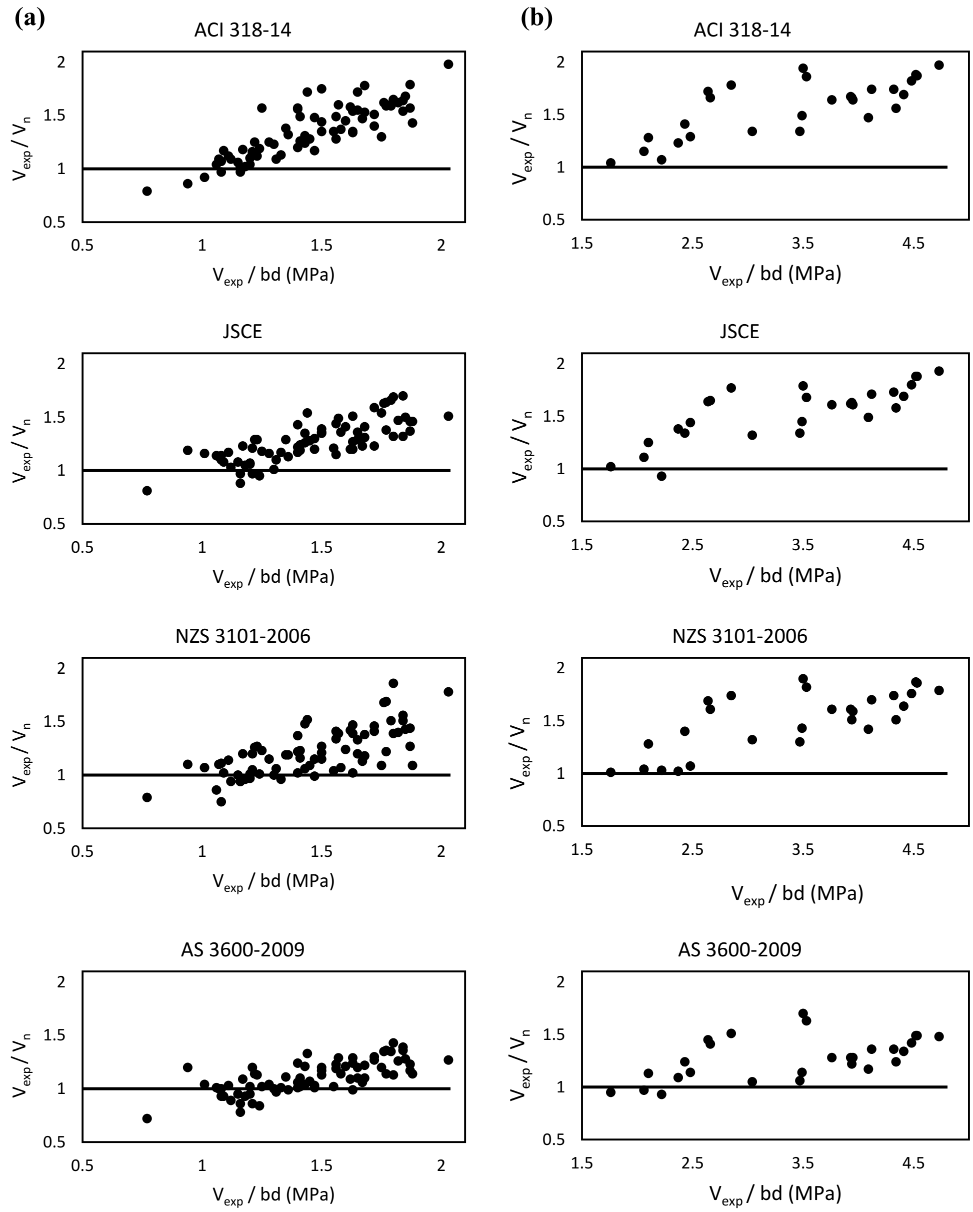

Fig. 1 **a** Strength ratios for SCC beams without shear reinforcement. **b** Strength ratios for SCC beams with shear reinforcement

high-volume fly ash tested by Alghazali and Myers (2017). For beams with shear reinforcement, average strength ratio and COV were found to be 1.51 and 19%, respectively. None of the shear strength ratios were found to be unconservative.

AS 3600-2009 For beams without shear reinforcement, average strength and COV were found to be 1.11 and 14%, respectively. 18.67% of the unconservative results were from the beams tested by Arezoumandi and Volz (2013) having a depth greater than 450 mm, Biolzi et al. (2014) having a longitudinal reinforcement smaller than 1% and beams tested by Alghazali and Myers (2017) prepared from SCC containing high-volume fly ash. For SCC beams with shear reinforcement, average strength ratio and COV were found to be 1.2 and 16%, respectively. Strength ratio for the beams with shear reinforcement tested by Lima de Resende et al. (2016) having concrete compressive strength 71.6 MPa were found to be unconservative.

Recycled aggregate concrete beams

Summary of the RAC beams considered in the study are given in Table 3 and their details are given in "Appendix". 109 RAC beams were considered in the study, 77 were without stirrups and 32 were with stirrups. Range of recycled aggregate replacement ratio (R_r), cylindrical compressive strength of concrete (f_c'), shear-span-to-depth ratio (a/d), width (b), effective depth (d), percentage of longitudinal reinforcement (ρ), shear reinforcement index ($\rho_v f_y$) are given in Table 3.

Table 3 Summary of RAC beams considered in the study

Author (s)	No. of beams	R_r (%)	f'_c (MPa)	a/d	B (mm)	D (mm)	ρ (%)	$\rho_v f_y$ (MPa)
Han et al. (2001)	2	100	31.23–31.89	3–4	170	270	1.1	0
Gonzalez-Fonteboa and Martinez-Abella (2007)	4	50	39.3–41.5	3.3	200	303	2.98	0–1.1
Etxeberria et al. (2007)	9	25–100	39.75–42.38	3.3	200	304	2.97	0.653–1.197
Ajdukiewicz and Kliszczewicz (2007)	6	100	39.3–107.8	3.2	200	250	1.55	0.66
González-Fonteboa et al. (2009)	3	50	41.45–44.49	3.3	200	303	2.98	1.05–1.95
Fathifazl et al. (2010)	3	63.5–74.3	36.9–43.5	2.61–2.66	200	301–306	3–4	1.32–2.38
Choi et al. (2010)	15	30–100	18.05–19.65	2.5–3.25	200	360	0.53–1.61	0
Fathifazl et al. (2011)	3	63.5–74.3	41.6–49.1	2.7–4	200	305–309	1.62–2.46	0
Al-Zahra et al. (2011)	2	25–50	29.58–30.42	2.5	100	180	1.9	0.915
Knaack and Kurama (2014)	8	50-100	35.7-43.6	3.875	150	200	1.34	0
Arezoumandi et al. (2015)	12	50–100	30–35.52	3–3.2	300	380–406	1.27–2.71	0
Katkhuda and Shatarat (2016)	8	50–100	23.2–35.55	2.5–3	150–206	267	1–2	0
Choi and Yun (2017)	11	30–100	23.2–27.2	3–5	400	525	1.88	0
Ignjatovic et al. 2017	6	50–100	42.4–46.3	4.2	200	238	2.5	0.42
Rahal and Alrefaei (2017)	12	10–100	27.2–30.48	2.99	150	388	0.79	0
Pradhan et al. (2018)	5	100	42.82	2.6	200	265	0.75–1.31	0–1.056
Total	109	10–100	18.05–107.8	2.5–4.2	100–400	180–525	0.53–4	0–2.38

Table 4 Statistical parameters for RAC beams

Procedure	Maximum	Minimum	Average	COV	Unconservative (%)
Beams without shear reinforcement ($\rho_v f_y = 0$)					
ACI 318-14	2.29	0.87	1.32	0.18	7.8
JSCE-2007	1.73	0.85	1.25	0.15	7.8
NZS 3101-2006	1.69	0.79	1.15	0.17	18.2
AS 3600-2009	1.53	0.75	1.11	0.15	24.67
Beams with shear reinforcement ($\rho_v f_y \geq \rho_v f_{y,min}$)					
ACI 318-14	2.59	1.04	1.68	0.24	0
JSCE-2007	2.54	1.0	1.59	0.20	0
NZS 3101-2006	2.42	0.95	1.44	0.24	3.12
AS 3600-2009	1.96	0.86	1.32	0.19	3.12

Overall evaluation

For RAC beams, statistical parameters maximum, minimum and average (V_{exp}/V_n), coefficient of variation and percentage of the unconservative results found for different code provisions are given in Table 4. Graphical representations of strength ratio (V_{exp}/V_n) with experimental ultimate shear stress for SCC beams are shown in Fig. 2a, b. Statistical parameters given in Table 4 and graphical representation in Fig. 2a, b show that procedure of AS 3600-2009 and JSCE-2007 produced least scatter for RAC beams with and without shear reinforcement. But, the procedure of JSCE produced only 7.8% unconservative results for RAC beams without shear reinforcement, contrary to AS 3600-2009 which produced 23.37% unconservative results.

Detailed evaluation

ACI 314-14 Average strength ratio and COV for RAC beams without shear reinforcement were found to be 1.32 and 18%, respectively. Unconservative strength ratios were found for the beams tested by Arezoumandi et al. (2015) with $R_r = 100\%$ and Rahal and Alrefaei (2017) having a longitudinal reinforcement of 0.79%. For RAC beams with shear reinforcement average values of strength ratio and COV were 1.68 and 24%, respectively. None of the strength ratios were found to be unconservative for RAC beams with shear reinforcement.

JSCE-2007 For RAC beams without shear reinforcement, average strength ratio and COV were 1.25 and 15%, respectively. Unconservative strength ratios were found for the beams tested by Gonzalez-Fonteboa and Martinez-Abella (2007), Katkhuda and Shatarat (2016) and Rahal and Alrefaei (2017). These beams were either having smaller amount of longitudinal reinforcement or 100% recycled aggregate replacement ratio. Average of the strength ratio and COV for RAC beams with shear reinforcement were 1.59 and 20%, respectively. JSCE method does not yield unconservative estimate of the shear strength of RAC beams with shear reinforcement.

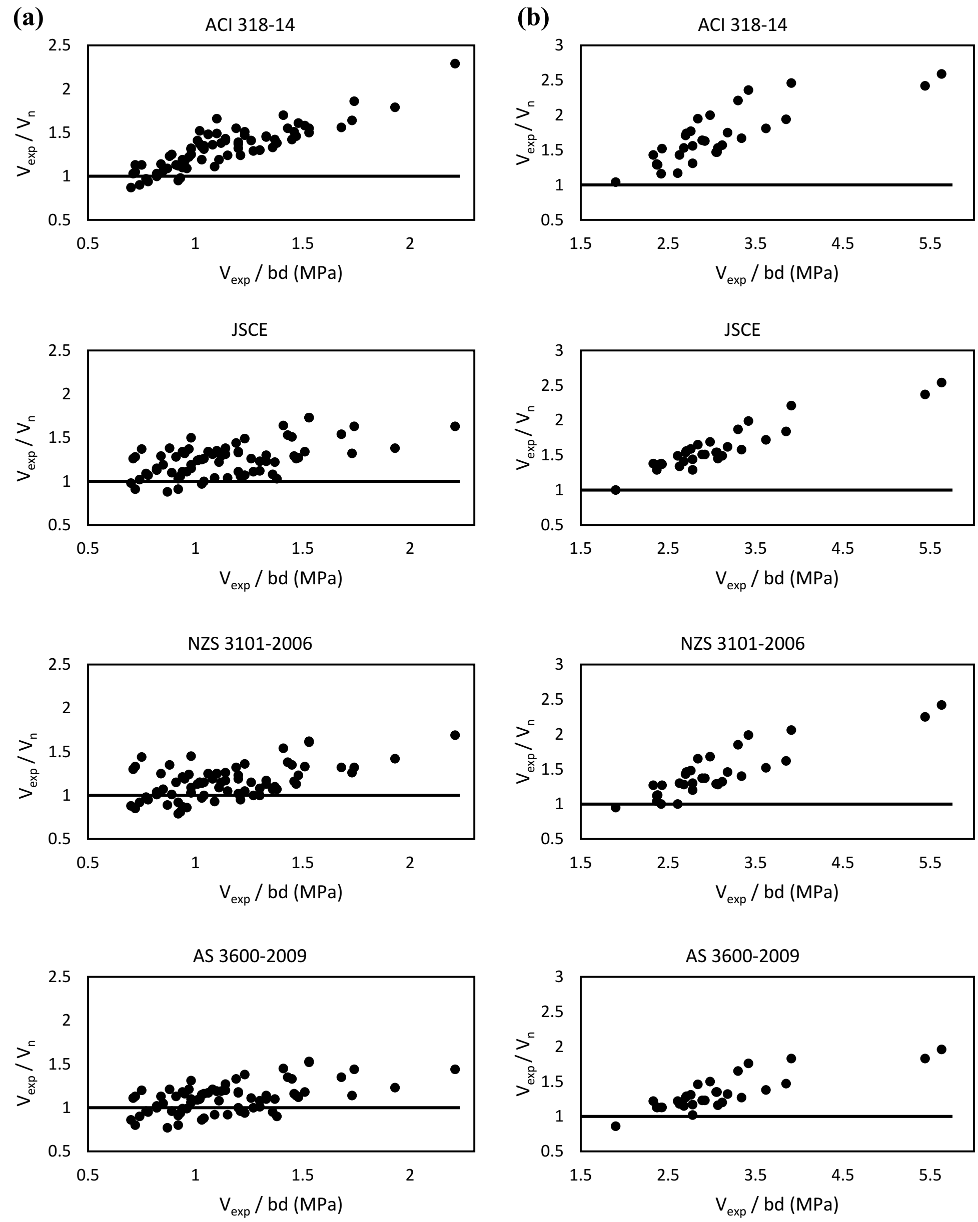

Fig. 2 **a** Strength ratios for RAC beams without shear reinforcement. **b** Strength ratios for RAC beams with shear reinforcement

NZS 3101-2006 This method yielded an average strength ratio and COV of 1.15 and COV of 17%, respectively, for the RAC beams without shear reinforcement. This method gives unconservative strength ratios for the beams tested by Choi et al. (2010) and Knaack and Kurama (2014) having $R_r = 100\%$, Arezoumandi et al. (2015) having beam depth greater than 400 mm and $R_r = 100\%$, and Rahal and Alrefaei (2017) having a longitudinal reinforcement ratio of 0.79%. Average strength ratio and COV were found to be 1.44 and 24%, respectively, for RAC beam with shear reinforcement. Unconservative estimate was found for only one beam tested by Al-Zahra et al. (2011).

AS 3600-2009 Australian standard gives an average strength ratio of 1.11 and a COV of 15% for RAC beams without shear reinforcement. Strength ratios for 16 out of 77 beams were found to be unconservative. Unconservative estimates of strength ratios were found for the beams tested by Choi et al. (2010), Knaack and Kurama (2014), Arezoumandi et al. (2015), and Rahal and Alrefaei (2017). These beams were having either 100% recycled aggregate replacement ratio, depth greater than 400 mm or longitudinal reinforcement percentage smaller than 1%. Average strength ratio and COV were 1.32 and 19%, respectively, for the RAC beams with shear reinforcement. Strength ratio of one beam tested by Al-Zahra et al. (2011) was found to be unconservative.

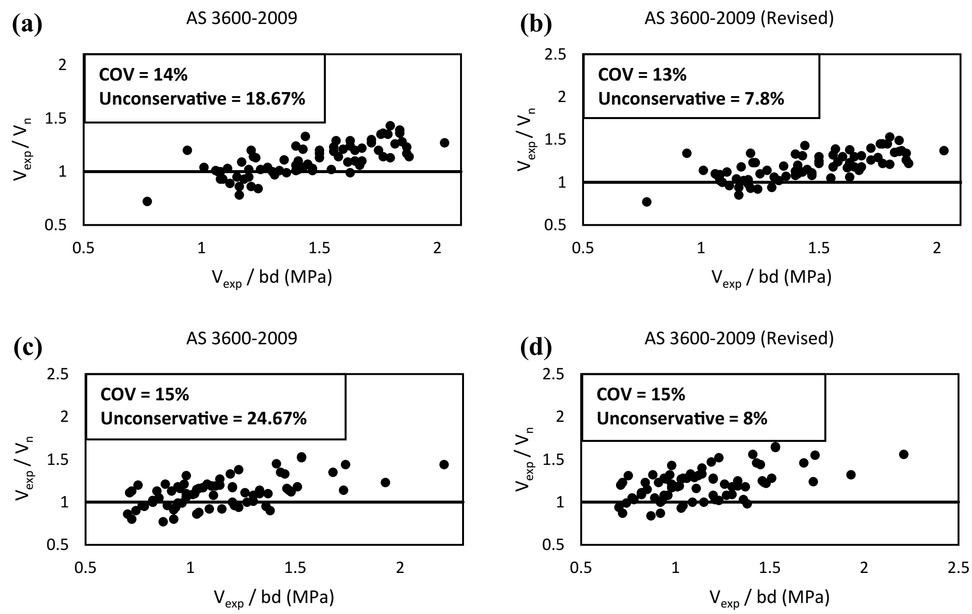

Fig. 3 **a** Strength ratios for SCC beams without shear reinforcement as per AS 3600-2009. **b** Strength ratios for SCC beams without shear reinforcement after reducing size factor in AS 3600-2009. **c** Strength ratios for RAC beams without shear reinforcement as per AS 3600-2009. **d** Strength ratios for RAC beams without shear reinforcement after reducing size factor in AS 3600-2009

Revision proposed in AS 3600-2009

After detailed statistical analysis, it was found that AS 3600-2009 has the minimum strength ratio and scatter for the collected database. A modification in the size effect factor of AS 3600-2009 is recommended and the results are plotted in Fig. 3. The use of modified size effect factor reduced the unconservative results from 18.67 to 7.8% for SCC beams and 24.67% to 8% for RAC beams.

Conclusions and recommendation

SCC is different from CC in terms of quantity and size of coarse aggregate while RAC is different from CC in terms of type of aggregate. Provisions for shear strength given in international code of practices are for CC beams. This paper evaluated shear strength provisions of four design codes: ACI 318-14, JSCE-2007, NZS 3101-2006 and AS 3600-2009 for the beams produced with SCC and RAC. On the basis of evaluation, the following conclusions can be made:

- In general, all the four design codes produced more scatter for RAC beams as compared to SCC beams. Average strength ratios were also found to be more for RAC beams as compared to SCC beams.
- For SCC and RAC beams without shear reinforcement, all the four codes produced unconservative estimates of the shear capacities having effective depths larger than 450 mm or longitudinal reinforcement ratio less than 1%. Minimum average strength ratio with least scatter was found for AS 3660-2009 but at the same time it produced 18.67% and 24.67% unconservative results for SCC and RAC beams, respectively. Critical assessment of the strength ratios showed that most of the unconservative estimates were for the beams with larger depths. Therefore, it is recommended to modify the factor β_1 to $1.1\left(1.5 - {}^{d_o}/_{1000}\right) \geq 0.8$ to account for the size effect of the member. This modification reduced the percentage of unconservative results to 8% for both SCC and RAC beams with a COV of 13% for SCC beams and 15% for RAC beams.
- All the four codes produced conservative estimates of the shear strength for SCC and RAC beams with shear reinforcement. AS 3600-2009 produced conservative estimates with minimum average strength ratio and least scatter.

Compliance with ethical standards

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

Appendix

SCC and RAC beams without shear reinforcement

Author	Specimen	R_r (%)	f'_c (MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	V_{exp} (kN)
Self-compacting concrete beams									
Hassan et al. (2008)	1SCC150	–	45	2.5	400	102.5	0.01	10	73.8
	2SCC150	–	45	2.5	400	100	0.02	10	81.2
	1SCC250	–	45	2.5	400	202.5	0.01	10	115.83
	2SCC250	–	45	2.5	400	197.5	0.02	10	127.98
	1SCC363	–	45	2.5	400	310.5	0.01	10	152.766
	2SCC363	–	45	2.5	400	305.5	0.02	10	166.192
	1SCC500	–	45	2.5	400	447.5	0.01	10	180.79
	2SCC500	–	45	2.5	400	442.5	0.02	10	226.56
	1SCC750	–	45	2.5	400	667.5	0.01	10	250.98
	2SCC750	–	45	2.5	400	650.5	0.02	10	314.84
Boel et al. (2010)	SCC1-2.5	–	60.7	2.5	100	130	0.0121	8	23.96
	SCC1-3	–	60.7	3	100	130	0.0121	8	22.35
	SCC2-2.5	–	55.8	2.5	100	130	0.0121	8	21.2
	SCC2-3	–	55.8	3	100	130	0.0121	8	20.31
Arezoumandi and Volz (2013)	NS-4-1	–	53.5	3	300	400	0.0127	19	129.6
	NS-4-2	–	39.6	3	300	400	0.0127	19	127.2
	NS-6-1	–	53.5	3.2	300	375	0.0203	19	177.75
	NS-6-2	–	39.6	3.2	300	375	0.0203	19	168.75
	NS-8-1	–	53.5	3.2	300	375	0.0271	19	210.37
	NS-8-2	–	39.6	3.2	300	375	0.0271	19	185.62
Safan (2012)	D1/10	–	75	2.59	100	135	0.0116	19	23.62
	D1/12	–	75	2.61	100	134	0.0168	19	25.19
	G1/10	–	56	2.59	100	135	0.0116	19	17.95
	G1/12	–	56	2.61	100	134	0.0168	19	20.77
	D2/10	–	64	2.59	100	135	0.0116	19	19.84
	D2/12	–	64	2.61	100	134	0.0168	19	21.84
	G2/10	–	47	2.59	100	135	0.0116	19	16.2
	G2/12	–	47	2.61	100	134	0.0168	19	17.42
	D3/10	–	53	2.59	100	135	0.0116	19	19.30
	D3/12	–	53	2.61	100	134	0.0168	19	23.71
	G3/10	–	37	2.59	100	135	0.0116	19	21.19
	G3/12	–	37	2.61	100	134	0.0168	19	22.51
	D4/10	–	55	2.59	100	135	0.0116	19	18.9
	D4/12	–	55	2.61	100	134	0.0168	19	22.37
	G4/10	–	37	2.59	100	135	0.0116	19	18.22
	G4/12	–	37	2.61	100	134	0.0168	19	18.89
	D5/10	–	51	2.59	100	135	0.0116	19	19.57
	D5/12	–	51	2.61	100	134	0.0168	19	22.51
	G5/10	–	33	2.59	100	135	0.0116	19	14.71
	G5/12	–	33	2.61	100	134	0.0168	19	18.76
	D6/10	–	48	2.59	100	135	0.0116	19	21.6
	D6/12	–	48	2.61	100	134	0.0168	19	22.11
	G6/10	–	30	2.59	100	135	0.0116	19	18.9
	G6/12	–	30	2.61	100	134	0.0168	19	20.1
	D7/10	–	41	2.59	100	135	0.0116	19	15.12
	D7/12	–	41	2.61	100	134	0.0168	19	19.69
	G7/10	–	26	2.59	100	135	0.0116	19	19.44
	G7/12	–	26	2.61	100	134	0.0168	19	16.75

Author	Specimen	R_r (%)	f'_c (MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	V_{exp} (kN)
Helincks et al. (2013)	SCC-1a	–	54.56	2.5	100	130	0.01	16	23.92
	SCC-1a	–	54.56	3	100	130	0.01	16	22.36
	SCC-1b	–	48	2.5	100	130	0.015	16	23.66
	SCC-1b	–	48	2.5	100	130	0.02	16	23.4
	SCC-1b	–	48	3	100	130	0.015	16	24.05
	SCC-1b	–	48	3	100	130	0.02	16	24.31
	SCC-2	–	48	2.5	100	130	0.01	16	21.19
	SCC-2	–	48	3	100	130	0.01	16	20.28
	SCC-3a	–	54.8	2.5	100	130	0.01	16	23.27
	SCC-3a	–	54.8	3	100	130	0.01	16	19.5
	SCC-4a	–	52.72	2.5	100	130	0.01	8	23.01
	SCC-4a	–	52.72	3	100	130	0.01	8	22.88
Biolzi egt al. (2014)	SCC40-M–N-1	–	42.64	2.5	170	260	0.00909	15	51.71
	SCC40-M–N-2	–	42.64	2.5	170	260	0.00909	15	49.06
	SCC40-L-N-1	–	42.64	3.5	170	260	0.00909	15	47.29
	SCC40-L-N-2	–	42.64	3.5	170	260	0.00909	15	53.92
	SCC40-XL-N-1	–	42.64	4	170	260	0.00909	15	34.03
	SCC40-XL-N-2	–	42.64	4	170	260	0.00909	15	47.73
Alghazali and Myers (2017)	50-5 N	–	53.5	3	305	406.67	0.0169	10	149.2
	50-6 N	–	53.5	3	305	406.67	0.0203	10	143.8
	50-8 N	–	53.5	3	305	406.67	0.0271	10	144
	60-5 N	–	45.9	3	305	406.67	0.0169	10	142.5
	60-6 N	–	45.9	3	305	406.67	0.0203	10	175
	60-8 N	–	45.9	3	305	406.67	0.0271	10	150
	70-5 N	–	52.9	3	305	406.67	0.0169	10	146
	70-6 N	–	52.9	3	305	406.67	0.0203	10	162
	70-8 N	–	52.9	3	305	406.67	0.0271	10	154
Recycled aggregate concrete beams									
Han et al. (2001)	R-3.0-N	100	31.23	3	170	270	0.011	25	55.08
	R-4.0-N	100	31.89	4	170	270	0.011	25	50.95
Gonzalez-Fonteboa and Martinez-Abella (2007)	V0RCS	50	41.45	3.3	200	303	0.0299	25	83.88
Choi et al. (2010)	RARAC30-H2.5	30	19.65	2.5	200	360	0.0161	25	73.44
	RARAC30-H3.25	30	19.65	3.25	200	360	0.0161	25	72.72
	RARAC50-H2.5	50	19.32	2.5	200	360	0.0161	25	79.2
	RARAC50-H3.25	50	19.32	3.25	200	360	0.0161	25	64.08
	RARAC100-H2.5	100	18.05	2.5	200	360	0.0161	25	76.32
	RARAC100-H3.25	100	18.05	3.25	200	360	0.0161	25	51.84
	RARAC30-L2.5	30	19.65	2.5	200	360	0.0053	25	51.12
	RARAC30-M2.5	30	19.65	2.5	200	360	0.0083	25	70.56
	RARAC30-H2.5	50	19.65	2.5	200	360	0.0161	25	73.44
	RARAC50-L2.5	50	19.32	2.5	200	360	0.0053	25	51.84
	RARAC50-M2.5	100	19.32	2.5	200	360	0.0083	25	60.48
	RARAC50-H2.5	100	19.32	2.5	200	360	0.0161	25	79.2
	RARAC100-L2.5	100	18.05	2.5	200	360	0.0053	25	54
	RARAC100-M2.5	100	18.05	2.5	200	360	0.0083	25	63.36
	RARAC100-H2.5	100	18.05	2.5	200	360	0.0161	25	76.32

Author	Specimen	R_r (%)	f'_c (MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	V_{exp} (kN)
Fathifazl et al. (2011)	EM-2.7	63.5	41.6	2.7	200	309	0.0162	19	103.9
	EM-4	63.5	41.6	4	200	305	0.0246	19	83.2
	EV-4	74.3	49.1	4	200	305	0.0246	19	105.6
Knaack and Kurama (2014)	S50-1a	50	43.6	3.875	150	200	0.0134	19	44
	S50-1b	50	43.6	3.875	150	200	0.0134	19	39.1
	S50-2a	50	40.2	3.875	150	200	0.0134	19	43.7
	S50-2b	50	40.2	3.875	150	200	0.0134	19	41.2
	S100-1a	100	41.4	3.875	150	200	0.0134	19	36.4
	S100-1b	100	41.4	3.875	150	200	0.0134	19	38
	S100-2a	100	35.7	3.875	150	200	0.0134	19	39.9
	S100-2b	100	35.7	3.875	150	200	0.0134	19	36.1
Arezoumandi et al. (2015)	RC50NS-4	50	32.03	3	300	406	0.0127	25	116.93
	RC50NS-4	50	35.52	3	300	406	0.0127	25	112.06
	RC50NS-6	50	32.03	3.2	300	380	0.0203	25	151.62
	RC50NS-6	50	35.52	3.2	300	380	0.0203	25	148.2
	RC50NS-8	50	32.03	3.2	300	380	0.0271	25	172.14
	RC50NS-8	50	35.52	3.2	300	380	0.0271	25	168.72
	RC100NS-4	100	30	3	300	406	0.0127	25	114.49
	RC100NS-4	100	34.14	3	300	406	0.0127	25	113.27
	RC100NS-6	100	30	3.2	300	380	0.0203	25	143.64
	RC100NS-6	100	34.14	3.2	300	380	0.0203	25	124.26
	RC100NS-8	100	30	3.2	300	380	0.0271	25	131.1
	RC100NS-8	100	34.14	3.2	300	380	0.0271	25	140.22
Katkhuda and Shatarat (2016)	R50-3	50	25.2	3	206	260	0.019	20	49.5
	R100-3	100	23.2	3	206	260	0.019	20	46.45
	T50-3	50	28.05	3	206	260	0.019	20	55
	T100-3	100	26.6	3	206	260	0.019	20	55.61
	R50-L-2.5-LR	50	27.95	2.5	150	260	0.0103	20	54.87
	R50-M-2.5-LR	50	35.55	2.5	150	260	0.0103	20	55.67
	R100-L-2.5-LR	100	31.85	2.5	150	260	0.0103	20	46.86
	R100-M-2.5-LR	100	38.7	2.5	150	260	0.0103	20	56.47
Choi and Yun (2017)	S-2.5-A100	100	23.2	2.5	400	525	0.0188	20	259.34
	S-3-A100-1	100	23.2	3	400	525	0.0188	20	227.11
	S-3-A100-2	100	23.2	3	400	525	0.0188	20	239.01
	S-4-A100-1	100	23.2	4	400	525	0.0188	20	250.91
	S-4-A100-2	100	23.2	4	400	525	0.0188	20	216.98
	S-5-A30-1	30	27.2	5	400	525	0.0188	20	235.44
	S-5-A30-2	30	27.2	5	400	525	0.0188	20	239.21
	S-5-A60-1	60	25.6	5	400	525	0.0188	20	205.91
	S-5-A60-2	60	25.6	5	400	525	0.0188	20	206.08
	S-5-A100-1	100	23.2	5	400	525	0.0188	20	206.51
	S-5-A100-2	100	23.2	5	400	525	0.0188	20	219.30
Ignjatovic et al. (2017)	RAC-50-1	50	46.3	4.2	200	238	0.025	31.5	91.75
	RAC-100-1	100	42.4	4.2	200	238	0.025	31.5	105

Author	Specimen	R_r (%)	f'_c (MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	V_{exp} (kN)
Rahal and Alrefaei (2017)	35A-0-100	100	28.64	2.99	150	388	0.0079	25	55.06
	35A-0-10	10	29.28	2.99	150	388	0.0079	25	43.07
	35A-0-20	20	28	2.99	150	388	0.0079	25	45.05
	35A-0-20R	20	28.24	2.99	150	388	0.0079	25	40.62
	35A-0-35	35	28.24	2.99	150	388	0.0079	25	49.47
	35A-0-50	50	30.48	2.99	150	388	0.0079	25	45.63
	35A-0-75	75	29.28	2.99	150	388	0.0079	25	47.49
	35-S-0-5	5	29.92	2.99	150	388	0.0079	25	47.96
	35-S-0-10	10	27.84	2.99	150	388	0.0079	25	56.57
	35-S-0-35	35	28.32	2.99	150	388	0.0079	25	53.08
	35-S-0-50	50	27.2	2.99	150	388	0.0079	25	54.77
	35-S-0-75	75	28.08	2.99	150	388	0.0079	25	47.78
Pradhan et al. (2018)	RAC-B1	100	42.82	2.6	200	265	0.0075	20	81.1
	RAC-B2	100	42.82	2.6	200	265	0.0075	20	81.3
	RAC-B3	100	42.82	2.6	200	265	0.0131	20	92.3

R_r recycled aggregate replacement ratio; f'_c cylindrical compressive strength of concrete; a/d shear-span-to-depth ratio; b width; d effective depth; ρ longitudinal reinforcement ratio; d_a maximum aggregate size; V_{exp} experimental shear strengthSCC and RAC beams with shear reinforcement

Author	Specimen	R_r (%)	f'_c(MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	$\rho_v f_y$ (MPa)	V_{exp} (kN)
Lin and Chen (2012)	–	S11	32.8	3	240	298	0.041	10	1.22	268.91
	–	S12	38.7	3	240	298	0.041	10	1.22	294.66
	–	S13	47.8	3	240	298	0.041	10	1.22	320.40
	–	S14	39	2.5	240	298	0.041	10	1.22	323.98
	–	S15	40.2	3.5	240	298	0.041	10	1.22	281.07
	–	S16	42.3	3	240	298	0.041	10	1.63	338.28
	–	S17	39	3	240	298	0.041	10	1.63	310.39
	–	S18	40.3	3	240	298	0.041	10	1.44	315.40
	–	S21	30.4	3	240	298	0.041	10	1.22	217.42
	–	S22	42.9	3	240	298	0.041	10	1.22	282.5
	–	S23	49.1	3	240	298	0.041	10	1.22	308.96
	–	S24	38.4	2.5	240	298	0.041	10	1.22	323.27
	–	S25	39.5	3.5	240	298	0.041	10	1.22	249.60
	–	S26	39.9	3	240	298	0.041	10	1.63	281.78
	–	S27	39.5	3	240	298	0.041	10	1.64	292.51
	–	S28	38.5	3	240	298	0.041	10	1.44	248.17
Arezoumandi and Volz (2013)	–	S-8-2	34.8	3.2	300	375	0.0271	19	0.8	231.75
Biolzi et al. (2014)	–	SCC40-M-S-1	42.64	2.5	170	260	0.00909	15	1.31	109.61
	–	SCC40-M-S-2	42.64	2.5	170	260	0.00909	15	1.31	104.75
Lima de Resende et al. (2016)	–		71.6	2.8	175	409	0.025	19	0.975	252.65
	–	V2	71.6	2.8	175	409	0.025	19	0.813	250.51
	–	V3	71.6	2.8	175	409	0.025	19	0.659	173.92
	–	V4	71.6	2.8	175	409	0.025	19	0.508	150.30
		V5	71.6	2.8	175	416	0.02	19	0.659	128.12
	–	V6	71.6	2.8	175	407	0.025	19	0.962	158.11
Alghazali and Myers (2017)	–	50-8S	53.5	3	305	406.67	0.0271	10	0.41	330.5
		60-8S	45.9	3	305	406.67	0.0271	10	0.41	327.3
	–	70-8S	52.9	3	305	406.67	0.0271	10	0.41	354.1

Author	Specimen	R_r (%)	f'_c(MPa)	a/d	b (mm)	d (mm)	ρ (%)	d_a (mm)	$\rho_v f_y$ (MPa)	V_{exp} (kN)
Recycled aggregate concrete beams										
Gonzalez-Fonteboa and Martinez-Abella (2007)	50	V24RC	39.3	3.3	200	303	0.0298	25	0.6	164.3
	50	V17RC	41.5	3.3	200	303	0.0298	25	0.85	177
	50	V13RC	40.5	3.3	200	303	0.0298	25	1.1	233.6
Etxeberria et al. (2007)	25	HR-25-1	42.38	3.3	200	304	0.0297	25	0.6528	238
	25	HR-25-2	42.38	3.3	200	304	0.0297	25	0.90304	169
	25	HR-25-3	42.38	3.3	200	304	0.0297	25	1.1968	186.5
	50	HR-50-1	41.34	3.3	200	304	0.0297	25	0.6528	164
	50	HR-50-2	41.34	3.3	200	304	0.0297	25	0.90304	176
	50	HR-50-3	41.34	3.3	200	304	0.0297	25	1.1968	220
	100	HR-100-1	39.75	3.3	200	304	0.0297	25	0.6528	168
	100	HR-100-2	39.75	3.3	200	304	0.0297	25	0.90304	163
	100	HR-100-3	39.75	3.3	200	304	0.0297	25	1.1968	189.5
Ajdukiewicz and Kliszczewicz (2007)	100	ORNm-b2	58.3	3.2	200	250	0.0155	16	0.65548	118.5
	100	GRNl-b2	39.3	3.2	200	250	0.0155	16	0.65548	116.5
	100	GRRl-b2	59.6	3.2	200	250	0.0155	16	0.65548	118.5
	100	GRNm-b2	89.1	3.2	200	250	0.0155	16	0.65548	121
	100	GRNh-b2	59.6	3.2	200	250	0.0155	16	0.65548	119
	100	GRRh-b2	107.8	3.2	200	250	0.0155	16	0.65548	130.5
Belen et al. (2009)	50	V24RCS	43.25	3.3	200	303	0.0299	25	1.05	147.33
	50	V17RCS	44.49	3.3	200	303	0.0299	25	1.5	192.92
	50	V13RCS	41.45	3.3	200	303	0.0299	25	1.95	202.36
Fathifazl et al. (2010)	63.5	EM-3S-R	36.9	2.61	200	306	0.03	12.5	1.32	170.13
	63.5	EM-6S-D	36.9	2.66	200	301	0.04	12.5	2.38	338.92
	74.3	EV-6S-D	43.5	2.66	200	301	0.04	12.5	2.38	327.48
Al-Zahra et al. (2011)	25	B11	30.42	2.5	100	180	0.019	20	0.915	47.25
	50	B12	29.58	2.5	100	180	0.019	20	0.915	34.25
Ignjatovic et al. (2017)	50	RAC-50-2	46.3	4.2	200	238	0.025	31.5	0.42	142
	100	RAC-100-2	42.4	4.2	200	238	0.025	31.5	0.42	135
	50	RAC-50-3	46.3	4.2	200	238	0.025	31.5	0.42	157
	100	RAC-100-3	42.4	4.2	200	238	0.025	31.5	0.42	163
Pradhan et al. (2018)	100	RAC-B6	42.82	2.6	200	265	0.0131	20	1.056	161.9
	100	RAC-B7	42.82	2.6	200	265	0.0131	20	1.056	162.1

R_r recycled aggregate replacement ratio; f'_c cylindrical compressive strength of concrete; a/d shear-span-to-depth ratio; b width; d effective depth; ρ longitudinal reinforcement ratio; d_a maximum aggregate size; $\rho_v f_y$ confinement index; V_{exp} experimental shear strength

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