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Evaluation of Flexural and Shear Performance of Beams Made with Recycled Concrete Aggregates

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ABSTRACT

A mixture proportioning method proposed earlier by the writers, called the Equivalent Mortar Volume Method (EMV), is used to investigate under flexure and shear the serviceability and strength of reinforced concrete beams made with coarse recycled concrete aggregate (RCA). In this method, RCA is treated as a composite of residual mortar and natural aggregate, and the volumetric fraction and properties of each component are considered in proportioning the RCA- concrete. It is demonstrated, using the data from tests on a large number of beams, that the flexural and shear performances of RCA-concrete beams proportioned by EMV method are comparable to those of beams made of conventional concrete; therefore, existing flexural and shear design methods can be used, without modification, to design RCA-concrete beams, provided the RCA-concrete mix is proportioned by the EMV method.

INTODUCTION

Although well-maintained concrete structures have theoretically a long service life, practically they are often demolished before the end of their service life due to premature deterioration, obsolescence, and damage caused by natural or man-made disasters. Therefore, a large amount of concrete, resulting from the demolition of buildings and structures, is available for either disposal or recycling as aggregate. The disposal of large quantities of concrete requires extensive new landfill facilities and transportation away from cities and farm land. On the other hand, to replace the demolished structures, vast quantities of fresh aggregates will be required, which will result in depletion of resources and the degradation of the environment. The unbridled use of new resources is inconsistent with sustainable practices in land use, materials and energy consumption. The increased use of recycled aggregates in the construction industry will reduce the demand for fresh natural resources and the associated energy required to produce fresh aggregate. However, today only a small portion of demolition concrete is used in the construction of new structures.

Despite the obvious economic and environmental benefits of recycled concrete aggregate (Abbas et al., 2006), it is not widely used in structural applications. The reason is partly the belief among designers and owners of structures that concrete made with RCA, termed RCA-concrete, is inherently inferior to conventional concrete (C-concrete) made with virgin aggregate. This belief is partly due to the qualifier “recycled”, which often connotes used and depreciated and thus lower quality. This belief is bolstered by the findings of some existing studies (Topcu and Sengel, 2004), which have reported that RCA-concrete has some inherently inferior properties, such as higher creep and shrinkage and lower elastic modulus (Hansen, 1992). However, the results of previous studies are obtained from tests on RCA-concrete mixes that were proportioned by the mix proportioning methods applied to C-concrete and treating RCA as a homogeneous material (Dhir at al., 1999). Similarly, it has been reported that structural members made from RCA-concrete experience larger deflection and have relatively lower flexural and shear strength compared to the companion C-concrete members (Maruyama et al., 2004; Han et al., 2001). Consequently, the applicability of the existing empirical relations for calculating the flexural and shear resistances of reinforced C-concrete members to reinforced RCA-concrete members has been questioned (Etxeberria et al., 2007; Han et al., 2001). These issues may discourage the use of RCA-concrete as a viable structural material.

To address these concerns, in another investigation, described in detail elsewhere, the writers developed a new mix proportioning procedure for RCA-concrete, termed Equivalent Mortar Volume (EMV) method (Fathifazl et al., 2009a). In this method RCA is treated as a composite of residual mortar and original natural aggregate; hence, in proportioning RCA-concrete mixtures, the volume fraction and relevant property of each component is considered using the rule of mixtures. The main feature of the proposed EMV method is the recognition that the residual mortar and natural aggregate contribute to the RCA-concrete total mortar and total natural aggregate contents, respectively. This is in contrast to existing methods of RCA-concrete mixtures proportioning where RCA is treated as a homogeneous aggregate that merely replaces virgin aggregate and makes no contribution to the total mortar content of RCA-concrete. Concrete proportioned based on the EMV method has been found to have the same or superior fresh and hardened properties compared to an equivalent C-concrete with the same volume of fresh mortar as the total volume of mortar in the companion RCA-concrete (Fathifazl et al., 2008).

To verify this method, an extensive experimental study was performed, involving testing under flexure and shear, a number of reinforced RCA-concrete beams made with coarse RCA and companion control beams made of concrete containing only coarse virgin aggregate with similar properties as the coarse natural aggregate in RCA. The detailed experimental program and its results are reported elsewhere (Fathifazl, 2008; Fathifazl, 2009a, 2009b, and 2009c), and will not be repeated here. The purpose of this paper is to investigate whether some existing codes’ strength and serviceability provisions are applicable, without modification, to reinforced RCA-concrete beams.

BRIEF DESCRIPTION OF EXPERIMENTAL PROGRAM

Materials Characterization

RCA from two different sources are used in the study, and are designated as RCA-M and RCA-V, which were obtained from demolition concrete recycling plants in Montreal (M) and Vancouver (V), respectively. The original natural aggregate in RCA-M is limestone and in RCA-V river-bed gravel.

Table 1 shows the weighted average absorption capacity (AC); bulk, saturated surface dry (SSD), and apparent specific gravities of RCA-M, RCA-V, natural limestone, river-bed gravel, and river sand. The average mass fraction, expressed in %, of the residual mortar for each RCA type, called the residual mortar content (RMC), is also shown.

Table 1. Average physical properties for coarse and fine aggregates

Aggregate	Moisture Content (%)	Absorption Capacity (%)	Specific Gravity			RMC (%)
			Bulk	SSD	Apparent	
RCA-M	1.1	5.4	2.31	2.42	2.64	41
RCA-V	1.3	3.3	2.42	2.50	2.64	23
Limestone	0.2	0.34	2.7032	2.71	2.73	-
River Gravel	0.2	0.89	2.7211	2.74	2.79	-
River Sand*	4	0.54	2.70	2.72	2.76	-

*Fineness modulus (F.M) of 2.60.

Mix Proportions

The full details of the mix proportions are presented elsewhere (Fathifazl, 2009a). However, for the sake of completeness, it is briefly recapped here. Two mix types were prepared for each RCA source using Type 1 Portland cement. Type 1 mixes were made with 100% virgin aggregates and proportioned by the conventional method of ACI for normal concrete (ACI Committee 211, 1997). Type 2 mixes involved the same type of cement and natural fine aggregate as Type 1 mixes, but their coarse aggregates were a blend of virgin aggregate and RCA and they were proportioned by the EMV method. The virgin coarse aggregate in mix types 1 and 2 had the same properties as the natural aggregate contained in the associated RCA. To compensate for the deficiency in the total natural aggregate volume of RCA-concrete compared to the companion C-concrete, the virgin aggregate content of each RCA-concrete mix was made equal to the total residual mortar volume in the same mix. This resulted in the total mortar and total coarse natural aggregate volumes of RCA-concrete being equal to those of the companion C-concrete. Table 2 presents the mixtures proportions, with the mix designation described in the last row of the table.

Table 2. Mix proportions of reinforced recycled concrete and control beams

Beam ID	RCA Content (%)	Mix Proportions (kg/m3)						
		Water	Cement	Sand	Coarse Aggregate		WRA (ml)	AE (ml)
					RCA	Natural aggregate		
EM	63.5	151	335	630	720	414	1055	35
CL	0	193	430	808	0	835	None	92
EV	74.3	161	358	645	813	281	1132	38
CG	0	191	424	763	0	900	None	91
Mix Designation Nomenclature: E or C: mix proportioned based on EMV (E) or conventional method (C); and 2) M, V, L or G: mix made with RCA-M (M), RCA-V (V), natural limestone (L) or natural gravel (G).								

Observe that the use of the EMV method lowers the cement and water requirements of the RCA-concrete mixes because the residual mortar reduces the fresh mortar requirement. Table 3 presents a summary of the hardened properties of the RCA-concrete mixes.

Table 3. Hardened properties of investigated concrete mixes

Mix ID	Mechanical Properties						Hardened Density kg/m3
	f'_c (MPa)		E_c (GPa)		f_t (MPa)		
	28 day	At time of beam test*	28 day	At time of beam test	28 day	At time of beam test	
EM	41.6	36.9	29.8	24.6	3.4	2.8	2333
CL	37.1	38	30.3	24.5	3.2	3.0	2308
EV	49.1	43.5	31.8	27.1	3.7	3.4	2364
CG	33.8	35.9	30.5	27.9	3.3	3.2	2308

* The strength or modulus of concrete on the day of testing the beams made of the pertinent concrete type.

Test beams

Beams were tested to study their flexural or shear behavior and strength. All beams were rectangular, simple supported and tested under four point bending. The load was applied by a 1000 kN servo-controlled hydraulic actuator, attached to a rigid frame. The actuator applied the load by stroke control to a steel spreader beam supported by two heavy duty rocker-and-roller assemblies symmetrically located 300 mm from midspan of the beam and resting on its top surface. Table 4 gives the designation, basic dimensions, concrete and steel properties, reinforcement details, and shear span-to-depth ratio of each beam. The shear span, *a*, refers to the distance between the point load and the support on either side of the beam while *d* denotes the effective depth of the beam. All of the steel reinforcement used was Grade 400 bars in accordance with the specifications of CAN/CSA G30.18 (CAN/CSA, 1998).

Table 4. Structural details of RRC beams tested in flexure

Beam ID	a/d	h (mm)	d (mm)	As (ρs %)	A’s	L(mm)
EM-Min	2.63	350	304	3No.10 (0.493)	2No.10	2600
EM-Av	2.66	375	301	2No.20+3No.15 (1.991)	2No.10	2600
EM-Max	2.61	390	307	2No.25+2No.25 (3.256)	2No.10	2760
EM-CMP	2.65	385	302	2No.25+2No.25 (3.310)	2No.25	2700
CL-Av	2.66	375	301	2No.20+3No.15 (1.991)	2No.10	2600
CL-CMP	2.65	385	302	2No.25+2No.25 (3.310)	2No.25	2700
EV-Min	2.63	350	304	3No.10 (0.493)	2No.10	2600
EV-Av	2.66	375	301	2No.20+3No.15 (1.991)	2No.10	2600
EV-Max	2.61	390	307	2No.25+2No.25 (3.256)	2No.10	2760
EV-CMP	2.65	385	302	2No.25+2No.25 (3.310)	2No.25	2700
CG-Av	2.66	375	301	2No.20+3No.15 (1.991)	2No.10	2600
CG-CMP	2.65	385	302	2No.25+2No.25 (3.310)	2No.25	2700

Note: a= shear span, d= beam effective depth, h= beam height, A_s= total area of tension steel, A'_s= total area of compression steel, L: beam length

Beams tested under flexure

As described by Fathifazl et al. (2009b), twelve beams were designed according to the requirements of the CSA Standard A23.3-04 (CSA, 2004). Three singly reinforced beams with longitudinal tensile reinforcement ratio: minimum, "Min", average "Av", and maximum "Max", and one doubly reinforced beam were tested for each RCA source (see Table 4). The reinforcement ratio, ρ_s, varied from 0.493% to 3.310%. The beams with compression steel are identified by the letters CMP in their designation. For each RCA type, two control beams were also tested, these beams being made with 100% virgin coarse aggregates of the same type as the natural aggregate in the corresponding RCA. One of the control beams was singly reinforced with average reinforcement ratio while the other was doubly reinforced.

Beams tested under shear.

The test variables associated with a given test beam can be identified by considering its designation in Table 5.

RCA-concrete beams made of RCA-V or RCA-M are designated as EV or EM, respectively, while their respective C-concrete beams are designated as CG or CL. Symbols 1.5N, 2N, 2.7N, or 4N refer to the nominal a/d ratio of 1.5, 2, 2.7, or 4 and N signifies no shear reinforcement; L, M, H, or HH characterizes the beam depth as 250 mm (low), 375 mm (medium), 450 mm (high), and 550 mm (very high), respectively. These beams were similarly reinforced as the ones previously described. The beam without shear reinforcement is designated as NS, those with three or six times the minimum shear reinforcement according to CSA A23.3-04 (2004) and with stirrups made of smooth φ8 mm bar are denoted as 3SR and 6SR and those with stirrups made of deformed φ10 mm bar are designated as 6SD, respectively. For example, the beam designated as EV-6SR is made of RCA-V, has smooth bars as shear reinforcement with shear reinforcement ratio equal to six times the minimum shear reinforcement ratio. For each RCA type a 200 mm wide by 375 mm deep control beam was made of virgin aggregate concrete to compare its shear strength with that of an otherwise identical beam made of RCA-concrete.

Table 5. Test beams details

Beam ID	a/d	Dimensions		Longitudinal Bottom Bars (ρ, %)
		d	L	
Effect of a/d ratio				
EM-1.5N	1.5	300	1900	2 No. 20 (1.00)
EM-2N	2	300	2200	3No.20 (1.5)
EM-2.7N	2.59	309	2600	3No.15+2No.15 (1.62)
CL-2.7N	2.59	309	2600	3No.15+2No.15 (1.62)
EM-4N	3.93	305	3400	3No.20+2No.20 (2.46)
EV-1.5N	1.5	300	1900	2 No. 20 (1.00)
EV-2N	2	300	2200	3No.20 (1.5)
EV-2.7N	2.59	309	2600	3No.15+2No.15 (1.62)
CG-2.7N	2.59	309	2600	3No.15+2No.15 (1.62)
EV-4N	3.93	305	3400	3No.20+2No.20 (2.46)

Table 5 (Cont.). Test beams details

Beam ID	a/d	Dimensions		Longitudinal Bottom Bars (ρ, %)
		d	L	
Size Effect				
EM-L	2.69	201	2080	2No.20+1No.15 (1.99)
EM-M	2.59	309	2600	3No.15+2No.15 (1.62)
CL-M	2.59	309	2600	3No.15+2No.15 (1.62)
EM-H	2.73	381	3180	2No.25+2No.15 (1.83)
EM-HH	2.73	476	3700	2No.25+2No.20 (1.68)
EV-L	2.69	201	2080	2No.20+1No.15 (1.99)
EV-M	2.59	309	2600	3No.15+2No.15 (1.62)
CG-M	2.59	309	2600	3No.15+2No.15 (1.62)
EV-H	2.73	381	3180	2No.25+2No.15 (1.83)
EV-HH	2.73	476	3700	2No.25+2No.20 (1.68)
Effect of Shear Reinforcement Ratio				Sirrups (ρ_w, %)
EM-NS	2.59	309	2600	No stirrups (0.0)
EM-3S-R	2.61	306	2600	No.8 @ 200mm (0.25)
EV-3S-R	2.61	306	2600	No.10@ 200mm (0.5)
EM-6S-R	2.65	302	2700	No.8@100mm (0.5)
EV-6S-R	2.65	302	2700	No.10@ 100mm (1.0)
CL-6S-R	2.65	302	2700	No.8@ 100mm (0.5)
CG-6S-R	2.65	302	2700	No.10@ 100mm (1.0)
EM-6S-D	2.66	301	2700	No.10@ 200mm (0.5)
EV-6S-D	2.66	301	2700	No.10@ 100mm (1.0)
CL-NS	2.59	309	2600	No stirrups (0.0)
CG-NS	2.59	309	2600	No.10@ 200mm (0.5)

EXPERIMENTAL RESULTS AND ANALYSIS

Beams tested under flexure

Table 6 summarizes the observed cracking moment and ultimate moment capacity of each beam. Assuming 40% of the ultimate load as the usual level service load, the service deflection, δ_s, of each beam was obtained from its load-deflection curve, and is shown in Table 5. The table also shows the predicted values of the preceding moments and the mid-span deflection (δ_s) at 40% of the failure load based on the ACI 318 code's provisions (ACI Committee 318, 2005).

Cracking Moment

According to Table 6, the $M_{cr}^{Exp.}/M_{cr}^{pred.}$ ratio based on Clause 9.5.2.3 of ACI 318 ranged from 0.61-1.37 with an average of 0.96 and standard deviation of 0.24. According to this range, it is observed that the calculated cracking moments using ACI 318 method overestimate the cracking moment of beams EM-Min, EM-Av and CL-Av. On the other hand, the ACI-predicted cracking moments overestimate the actual cracking moment of all the EV beams, except beams EV-CMP and CL-CMP. Note that the overestimation of ACI 318 is not solely due to the presence of RCA since the $M_{cr}^{pred.}$ value of beams made entirely of normal concrete (CL-Av and CG-Av), are also higher than their corresponding M_{cr}^{Obsvd} values. This might be partially attributed to the higher early shrinkage of RCA-concrete. It might be also due to the effect of aggregate angularity on the tensile strength which is not considered by ACI 318. However, according to Smith and Young (1955), the ACI 318 method for predicting cracking moment of a conventional reinforced concrete beam is known to be accurate within ±20% for normal concrete. Since the $M_{cr}^{obsvd.} / M_{cr}^{pred.}$ ratio for some of the EM and EV beams does not fall within this range, some modifications might be necessary to the ACI 318 equation for predicting the cracking moment of reinforced RCA-concrete beams, which requires further investigation.

Ultimate Moment

Generally, the $M_{cr}^{Exp.} / M_{cr}^{pred.}$ ratio based on ACI 318 ranged from 1.04-1.15 with an average of 1.09 and standard deviation of 0.04. According to this range, it can be observed that the ACI flexural design provisions are still applicable to all reinforced concrete beams made of RCA-concrete proportioned by the EMV method.

Midspan Deflection

According to Table 6, the $\delta_s^{Exp.} / \delta_s^{pred.}$ ratio for the different reinforced RCA-concrete beams varies over the range 0.87-2.09 using ACI 318 method.

According to the range of $\delta_s^{obsvd.} / \delta_s^{pred.}$ values, the ACI 318 method predicts the deflection of RCA-concrete beams with average and maximum longitudinal reinforcement ratios reasonably well (2-15% higher than the observed values), but for the beams with minimum reinforcement ratio, the ACI predicted values are 46-52% lower than the observed values. This due to the inapplicability of the ACI tension-stiffening model to members with very low reinforcement ratio.

Table 6. Predicted and observed flexural performance of RCA-concrete beams

Beam ID	$M_{cr.}^{Exp.}$ (kN-m)	$\frac{M_{Cr.}^{Exp.}}{M_{Cr.}^{Pred.}}$	$M_{Ult.}^{Exp.}$ (kN-m)	$\frac{M_{Ult.}^{Exp.}}{M_{Ult.}^{Pred.}}$	$\delta_s^{Obsvd.}$ (mm)	$\frac{\delta_s^{Exp.}}{\delta_s^{Predct.}}$
EM-Min	13.0	0.75	46.0	1.08	1.54	1.86
EM-Av	13.8	0.72	149.2	1.15	2.72	0.89
CL-Av	19.2	0.94	142.7	1.10	2.67	0.94
EM-Max	21.1	1.04	221.9	1.06	3.04	0.97
EM-CMP	24.7	1.17	246.1	1.12	3.61	1.08
CL-CMP	29.5	1.37	229.1	1.04	3.24	1.08
EV-Min	16.2	0.86	46.7	1.09	1.04	2.09
EV-Av	15.2	0.70	150.2	1.14	2.62	0.90
CG-Av	19.2	0.97	139.1	1.08	2.67	1.01
EV-Max	13.4	0.61	225.2	1.04	2.63	0.87
EV-CMP	29.1	1.27	245.7	1.11	3.64	1.13
CG-CMP	23.0	1.16	226.5	1.04	3.87	1.30
Average		0.96		1.09		1.18
Standard Deviation		0.24		0.04		0.40

Beams tested under shear

The beams tested under shear involved beams with and without shear reinforcement. The experimental results for the beams without shear reinforcement are summarized in Table 7. The table shows the corresponding nominal shear stress, $v_c^{Exp.} = V_{max}/bd$ at the section of maximum shear. Note that *b* and *d* are the section width and effective depth, respectively. Since in these beams the shear resistance is derived from the so-called concrete contribution v_c , the predicted v_c values based on the Canadian Standard A23.3 (CSA, 2004) and the ACI-318 code are calculated and compared with the experimental nominal shear stress values. The table also shows the ratio of $v_c^{Exp.} / v_c^{pred.}$. The two codes differ significantly in their shear design provisions as described by Collins et al. (1996). Thus it is useful to check if the results of either deviate significantly from the experimental data.

Since the current experimental program involves most of the major parameters which affect the shear resistance of reinforced concrete beams without shear reinforcement, such as beams size, shear span/depth ratio and longitudinal reinforcement ratio, the results in Table 8 indicate that the provisions of current standards can be applied to the shear design of RCA-concrete beams without shear reinforcement because they all give a conservative estimate of the actual shear strength of these beams.

Table 7. Ratio of observed to predicted ultimate shear resistance of RCA-concrete beams without shear reinforcement

Beam Designation	$v_c^{Exp.} / v_c^{pred.}$				$v_c^{Exp.}$ (MPa)
	Method				
	CSA A23.3-04		ACI-318		
	Simplified	General	Eq.11.3*	Eq.11.5*	
EM-1.5N	3.14	3.08	3.07	3.58	3.45
EM-2N	2.85	2.60	2.79	3.25	3.14
EM-2.7N	1.71	1.58	1.66	1.94	1.87
CL-2.7N	1.51	1.36	1.46	1.71	1.67
EM-4N	1.38	1.26	1.35	1.57	1.51
EV-1.5N	3.03	3.06	2.96	3.45	>3.61
EV-2N	2.77	2.59	2.71	3.16	3.22
CG-2.7N	2.50	2.23	2.43	2.84	>2.70
EV-4N	1.61	1.51	1.57	1.83	1.92
EM-L	2.09	1.90	2.19	2.56	2.47
EM-M	1.71	1.58	1.66	1.94	1.87
CL-M	1.51	1.36	1.46	1.71	1.67
EM-H	1.39	1.22	1.29	1.50	1.45
EM-HH	1.25	1.11	1.09	1.27	1.22
EV-L	2.64	2.78	2.78	3.23	>3.39
CG-M	2.50	2.43	2.43	2.84	>2.70
EV-H	1.44	1.33	1.33	1.55	1.63
EV-HH	1.31	1.14	1.14	1.33	1.40

*Refers to the equation numbers in ACI 318 code

For the beams with shear reinforcement, Table 7 shows the observed and the predicted strength of these beams. In this case the codes expression involve the contribution of concrete v_c as mentioned earlier and the shear reinforcement contribution v_s. Note when calculating the predicted shear resistance of these beams using the code expression, i.e. $v_r^{pred} = v_c + v_s$, all material resistance factors are set equal to one.

According to Tables 7 and 8, it can be observed that all the tested beams had larger shear strength than their predicted values by both the simplified and general methods of the two codes, regardless of the a/d ratio, beam size, shear reinforcement ratio, or the RCA source. In many cases the predicted values are rather conservative, particularly for the beams with lower a/d ratios or lower overall effective depth. Consequently, for RCA-concrete designed by the EMV method, one can safely use the existing codes expressions for calculating v_c.

Table 8. Ratio of observed to predicted ultimate shear resistance of RCA-concrete beams with shear reinforcement

Beam Designation	$V_u^{Exp.}/V_u^{pred.}$				$V_u^{Exp.}$ (kN)
	Method				
	CSA A23.3-04		ACI-318		
	Simplified	General	Eq.11.3	Eq.11.5	
SEM-NS	1.71	1.58	1.66	1.94	103.9
SEM-3S	1.04	1.18	1.20	1.28	171.9
SEM-6SR	1.14	1.32	1.39	1.45	>308.3
SCL-6SR	1.06	1.22	1.29	1.35	>287.0
SEM-6SD	1.37	1.56	1.67	1.74	340.8
SEV-NS	-	-	-	-	-
SEV-3SR	1.38	1.57	1.59	1.70	>235.0
SEV-6SR	1.12	1.30	1.36	1.42	>307.8
SCG-6SR	1.06	1.23	1.30	1.35	283.8
SEV-6SD	1.29	1.47	1.56	1.64	327.4
CL-NS	1.51	1.36	1.46	1.71	92.82
CG-NS	2.50	2.43	2.43	2.84	149.47

Conclusions

In this paper, the results of an investigation into the flexural and shear behavior of reinforced recycled concrete beams were presented. The focus of the study was the effect of the proposed EMV concrete mix design method on the shear and flexural capacity of RCA-concrete beams, and the applicability of current flexural and shear design provisions of some major codes to reinforced RCA-concrete beams. Based on the results of the study, provided the RCA-concrete mix is designed by the EMV method, the following conclusions are reached:

- The general flexural theory and ACI flexural design provisions for conventional reinforced concrete members were found to be applicable to RCA-concrete beams at different longitudinal tension reinforcement ratios, with or without compression steel. Therefore, there is no need to develop a new flexural design method for reinforced recycled concrete beams if the RCA-concrete used in these members is designed by the EMV method.

- The ACI 318 method for predicting the immediate deflections of conventional reinforced concrete members were found to be applicable to reinforced RCA-concrete beams made with RCA-concrete proportioned by EMV method.
- The observed cracking moments of the RCA-concrete beams with or without compression steel were generally lower than those of the companion control beams. The predicted cracking moments of the beams based on the ACI 318 relationship were found to be higher than the corresponding observed values.
- Both simplified and general methods of Canadian Standard and ACI-318 for calculating shear capacity were found to be conservative when applied to predict the shear resistance of practically all the RCA-concrete beams tested in this study. Therefore, there is no need to develop a new shear design methods for reinforced recycled concrete beams.

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