**The effect of shear strength on load capacity of FRP strengthened
beams with recycled concrete aggregate**

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Abstract. Shear failure of reinforced concrete members is brittle and catastrophic, and should be avoided. This work presents a comparative study of concrete beams with natural crushed stone and those incorporating recycled concrete aggregate (RCA). The study indicates that the equation presented in ACI 318-14 Code Requirements for predicting the shear strength of concrete is less conservative for RCA concrete beams. The presented experimental investigations show that strengthening with fiber reinforced polymer (FRP) fabrics can be designed so that the shear capacity of the beams with weaker (RCA) concrete is higher than that of the control beams with natural aggregate concrete. The design of the FRP strengthening system and the reasons for the effectiveness of the design are explained.

Keywords: Shear capacity, reinforced concrete beams, recycled concrete aggregate, FRP strengthening, failure mode.

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1. Introduction

The use of construction and demolition waste (CDW) in concrete has been studied extensively for the past four decades [1]. CDW consisting primarily of concrete rubble can be crushed and graded into coarse aggregate referred to as Recycled Concrete Aggregate (RCA). The use of RCAs in structural concrete is permitted by the construction codes of a number of countries, including Switzerland and Germany. These codes specify durability and strength requirements (based on standard tests), and maximum permissible amounts of non-concrete CDW materials (such as brick, wood and asphalt) as aggregates in structural concrete. Today, in several countries including Germany, Switzerland, Australia, and Spain concrete containing RCA is being marketed and used, although still in limited applications, in structural members [2]. Even though replacing natural aggregates with RCA may have little impact on reducing greenhouse gas emissions since most emissions occur due to cement production, recycling concrete decreases natural resource exploitation and reduces landfilling which both have a beneficial effect on the environment. In certain parts of the world there is a scarcity of natural aggregates close to major urban areas and the use of recycled aggregates may become more attractive and have lower overall environmental impacts when full life-cycle analysis is conducted [3]. The partial or full replacement of coarse natural aggregate (NA) with RCA is a growing trend in the concrete industry. This is mainly due to (1) increased awareness of the environmental impacts of landfilling, (2) tightening of landfilling regulations and enforcing minimum limits for the portion of CDW that must be recycled (particularly in Europe [4]), and (3) potential economic benefits due to the typically lower distances of demolition sites to recycling plants than to landfills [3].

When replacing NA with the same volume of RCA in concrete while keeping the mix volumetric proportions the same there is generally a 15%-25% reduction in 28-day compressive strength [1]. This reduction can be compensated for by using more cement in the mix. However, if the load capacity of a reinforced concrete member does not greatly depend on the strength of concrete, such a change in mix proportioning will not be necessary. Among different possible applications of RCA in structural concrete, flexural concrete members such as beams and slabs seem to be the most viable. It is well known that the load-carrying capacity of a reinforced concrete flexural members does not have a linear relationship with the compressive strength of the concrete [5]. In fact, the concrete compressive strength mostly affects the ability of flexural member to carry load after the maximum load is reached [5]. While large ductility after failure is an important and often desired characteristic of concrete structural members, in flexural members, post-failure ductility can be achieved not only by increasing the strength of concrete, but also by optimizing the beam design through selecting lower values of the reinforcement ratio (ρ_s).

Flexural and shear strengthening of in-service reinforced concrete members by attaching FRP fabrics or strips is today a common practice in structural engineering [6]. Standardized design procedures, such as those provided in ACI 440.2R [7], exist for the design and construction of externally bonded FRP systems for strengthening concrete structures. FRP strengthening is a practical option when the load carrying capacity of a structure needs to be increased due to design or construction errors, damage or usage changes requiring service loads higher than what the structure was originally designed for. If, as expected, the use of RCA in flexural members become commonplace, eventually retrofitting and strengthening methods used for conventional concrete members will also be used for their RCA counterparts. A study was recently performed

on the compressive strength of FRP-wrapped RCA concrete cylinders [8]. To the best knowledge of the authors no prior research has been performed on the topic of FRP strengthening of reinforced RCA concrete flexural members. A study of the previous work on RCA flexural members presented in the next section, shows that code predictions [9] can be less conservative for shear load carrying capacity than for flexural load carrying capacity of RCA concrete beams. Therefore, a strengthening system that is in particular effective in increasing the shear capacity of beams made with RCA concretes is attractive and may be found useful in the future.

2. Comparison of predicted vs experimental moment and shear capacities of NA and RCA concrete beams

As mentioned earlier, codes and guidelines that currently permit the use of RCA in structural concrete define criteria and allowable limits for the composition of RCA and for a variety of physical durability and performance requirements of the RCA. However, they do not offer new design equations for determining the load-carrying capacity of structural members made with RCA concrete. Therefore, it is important to find out how well the existing methods predict the load carrying capacities of RCA structural members. This can be done, in part, by analyzing the findings of numerous prior experimental investigations reported in the literature on the behavior of reinforced concrete structural members with NA and RCA. This study uses the provisions of ACI 318 [9], which is commonly used in the United States and several other countries, to predict the beam bending moment and shear capacities.

The specific goal of the study presented in this section was to determine whether, based on the test results from prior experimental investigations reported in the literature, the load-carrying

capacities of NA and RCA beams are predicted with the same margin of conservativeness according to the ACI Code. For the purpose of comparing the findings of different studies the metrics CF_M and CF_V are introduced for bending moment and shear load carrying capacities to normalize the results. CF_M incorporates the ratio of experimental to nominal moment capacities of both NA and RCA concrete beams. If the magnitude of CF_M is zero, it means that these two ratios are the same and that the code predicts the moment capacity of NA and RCA concrete beams with the same margin of conservativeness. If CF_M is negative, the code is less conservative in predicting the moment capacity of RCA beams:

$$CF_M = \left(\frac{(M_{\text{exp,RCA}} / M_{n,\text{RCA}})}{(M_{\text{exp,NA}} / M_{n,\text{NA}})} - 1 \right) \cdot 100 \quad (1)$$

A similar metric CF_V is introduced for the shear load carrying capacity:

$$CF_V = \left(\frac{(V_{\text{exp,RCA}} / V_{n,\text{RCA}})}{(V_{\text{exp,NA}} / V_{n,\text{NA}})} - 1 \right) \cdot 100 \quad (2)$$

In Equations (1) and (2) $M_{\text{exp,RCA}}$ and $M_{\text{exp,NA}}$ are the maximum experimental moment carrying capacity of RCA and NA concrete beams, respectively, which are produced and tested in the same study. $M_{n,\text{RCA}}$ and $M_{n,\text{NA}}$ are the nominal moment capacities of the RCA and NA concrete beams calculated using the provisions of ACI 318 Code, calculated from,

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \quad (3)$$

In this equation A_s is the area of longitudinal tensile reinforcement section, f_y is the yield strength of steel, d is the effective depth and a is the depth of Whitney Stress Block. Similarly,

the variables in Eq. (2) are the experimental and nominal shear capacities of the RCA and NA beams and calculated from,

$$V_n = 2\sqrt{f'_c}bd \quad (4)$$

2.1. Bending moment capacity

Sato et al. [10] produced RCA concrete with 100% NA by volume replacement batches using RCAs obtained from crushing concrete specimens with different w/c ratios incorporating the same type of natural aggregates used in control batches. They produced longitudinally reinforced concrete beams (with ρ_s ranging from $0.13\rho_b$ to $0.56\rho_b$) and tested them in a four-point bending test configuration. Analysis of the results of 18 of their control and 100% RCA beams reveals that for beams made of concrete with $w/c = 0.6$, the average value of CF_M is 2% when the NA is replaced with RCA produced by crushing concrete with $w/c = 0.6$. When both the source concrete and concrete used in the beams had the w/c ratio of 0.45, CF_M has an average value of 7%.

Fathifazl et al. [11, 12] produced control and RCA concrete batches with 64% and 74% by weight replacement of NA with commercially produced RCA. The RCA mix proportions were designed in order that the compressive strength values of RCA concretes ($f'_{c,avg} = 40$ MPa) were slightly higher than those of their control NA counterparts. They produced several doubly-reinforced concrete beams and tested them in a 3-point bending configuration. The results from

testing two control and two RCA concrete beams ($\rho_s = 0.5\rho_b$), each made from a different type of RCA or NA, show that CF_M has an average value of 5%.

Bai and Sun [13] produced control and RCA concrete batches with RCA replacement of 50%, 70%, and 100% by weight. The RCA was produced in the laboratory by crushing waste test-specimen concrete (f'_c in the range of 30–40 MPa). Both the NA and RCA concrete batches were proportioned to have similar compressive strength values ($f'_{c,avg} = 46$ MPa). They produced longitudinally reinforced concrete beams and tested them in four-point bending configuration to achieve flexural-tension failure. The results from testing four of their beams with the same reinforcement ratio ($\rho_s = 0.17\rho_b$) shows that CF_M values were between 1.0% and 2.5% even when NA was replaced with 100% RCA.

Choi and Yun [14] produced two reinforced concrete beams ($\rho_s = 0.15\rho_b$), one with NA and the other with RCA obtained by crushing debris of demolished concrete structures, both having the same volumetric proportions. After the beams underwent a sustained bending load of half their theoretical capacity for one year, they were tested in a quasi-static three-point bending configuration and failed in flexural-tension mode. Using their test results, CF_M was found to be 5%.

Ignjatovic et al. [15] crushed and mixed concrete obtained from (1) a demolished concrete bridge, and (2) laboratory concrete waste with unknown strength to produce RCA. They designed and produced control and RCA concrete batches with 50% and 100% RCA by volume

replacement. The three types of concrete had very similar compressive strength values ($f'_{c,avg} = 46 \text{ MPa}$). They produced three sets of concrete beams with a wide range of reinforcement ratios, ρ_s ($0.1\rho_b$, $0.6\rho_b$, and $1.0\rho_b$). Every set consisted of three beams with different concrete types. The results of testing the 9 beams give values of CF_M (obtained by averaging those of 50% and 100% RCA concrete beams) of -5%, 0%, and 11% for beams with reinforcement ratios of $0.1\rho_b$, $0.6\rho_b$, and $1.0\rho_b$, respectively.

Kang et al. [16] produced normal-strength ($f'_c = 39 \text{ MPa}$) and high-strength ($f'_c = 65 \text{ MPa}$) NA concrete batches. They used the NA concrete mix proportions to produce normal- and high-strength RCA concrete batches with RCA replacement of 0%, 15%, 30% and 50% by weight. RCA was produced by crushing waste concrete. Each type of concrete was used to produce four reinforced concrete beams with different longitudinal reinforcement ratios, ρ_s . The beams were designed to fail in flexural-tension mode in a four-point bending test configuration. The results of testing 28 beams show that while the compressive strength of concrete was clearly affected by the RCA replacement (e.g. an average 21% reduction due to 30% RCA replacement), the magnitudes of CF_M were mostly smaller than 5%. CF_M values for the normal-strength concrete beams were all negative and their magnitudes grew with the increase in RCA replacement ratio and decrease in ρ_s . For example, for 50% RCA concrete beams, CF_M values were -11%, -10%, -9%, and -2% for ρ_s ratios of $0.15\rho_b$, $0.30\rho_b$, $0.45\rho_b$ and $0.55\rho_b$, respectively.

Ajdukiewicz and Kliszczewicz [17] produced three types of RCA by crushing three types of concrete incorporating gravel ($f'_c = 38$ MPa), crushed granite ($f'_c = 67$ MPa), and crushed Basalt ($f'_c = 91$ MPa). They used each aggregate type to produce RCA concrete batches with three different compressive strength levels (low; $f'_c = 36$ MPa, medium; $f'_c = 59$ MPa, and high; $f'_c = 95$ MPa). They also used gravel, crushed granite, and crushed Basalt to produce control NA concrete batches with similar strength levels. The RCA and control concrete batches were used to cast beams with two different ρ_s values which resulted in a wide range of ρ_s / ρ_b ratios, due to the very different values of f'_c . Some of the beams failed in the flexural-tension mode and the others failed in shear. Comparison of the test results of 20 NA and RCA beams made from concrete batches with the same class of compressive strength leads to the finding that magnitudes (absolute values) of CF_M and CF_V were less than 5% when NA concrete used for casting the beams and the NA concrete crushed for producing RCA had similar strength values and were made of the same type of NA. When different types of NA were used in the production of control concrete and in the production of concrete that was crushed into RCA (e.g., when NA concrete was made with granite and RCA was from crushing an NA concrete that was made with basalt), CF_M is in the range of -15% to 26%.

Knaack and Kurama [18] produced NA concrete as well as two types of RCA concrete with RCA replacement of 50% and 100% by volume. RCA was produced by crushing the demolished foundation of a 1920s building. NA and RCA concrete had similar compressive strength values ($f'_{c,avg} = 40$ MPa). They produced flexural-critical beams (with stirrups) and shear-critical beams

(without stirrups) with $\rho_s = 0.5\rho_b$, so that under a four-point loading configuration, the beams would fail in flexural-tension and shear modes, respectively. The results of testing 12 flexural-critical beams show that CF_M values were very small; 0% and -3% for the 50% and 100% RCA beams, respectively. The results of testing 12 shear-critical beams show that the average CF_V values for 50% and 100% RCA beams were 10% and -1%, respectively.

2.2. Shear capacity

The studies of Ajdukiewicz and Kliszczewicz [17], and Knaack and Kurama [18], who investigated both the flexural and shear capacities of reinforced RCA concrete were discussed in previous section. Four additional studies that exclusively investigated the shear properties of reinforced RCA beams are discussed here.

Etxeberria et al. [19] used commercially produced RCA to mix concrete batches with RCA replacement ratios of 25%, 50%, and 100% by volume. The mix proportions of concrete batches were adjusted in a way that they had very similar compressive strength values. For each type of concrete, a set of three longitudinally reinforced beams with $\rho_s = 0.84\rho_b$ and three different transverse steel shear reinforcement (stirrup) ratios, one of which was a beam with no stirrups (0%), were produced. The beams were tested in a four-point bending configuration and were designed to fail in diagonal shear. The results from testing all the sets showed that the shear load carrying capacity of beams with higher replacement ratios of NA with RCA were lower, although the concrete compressive strength values for all the beams were similar. In addition, the

results show that for the beams without stirrups CF_V is -14%. Using stirrups changed CF_V to -10% and -7% for the lower and higher shear reinforcement ratios, respectively.

Arezoumandi et al. [20] produced NA and 100% replacement RCA concrete batches with $f'_{c,avg}$ values of 36 MPa and 32 MPa, respectively. The source of the RCA was not reported. For each concrete type they produced three sets of beams with different longitudinal reinforcement ratios ρ_s ($0.38\rho_b$, $0.61\rho_b$, and $0.65\rho_b$), and designed them to have shear failures in a four-point bending setup caused by diagonal cracking. The results of testing 12 beams shows that (1) the shear capacity of RCA concrete beams were lower than those of control beams, and (2) for beams with reinforcement ratios of $0.38\rho_b$, $0.61\rho_b$, and $0.65\rho_b$, the average values of CF_V were -3%, -7% and -16%, respectively.

Kim et al. [21] produced control concrete and RCA concrete mixes with RCA mass replacement ratios of 0%, 50% and 100%. The source of the RCA is not reported. The mixes were proportioned so that the three types of concrete had very similar compressive strength values ($f'_{c,avg} = 33$ MPa). Each batch was used to produce 5 doubly-reinforced concrete beams with different cross-sectional dimensions (effective depth ranging from 300 mm to 600 mm), and average longitudinal reinforcement ratios of $0.8\rho_b$. The beams had no stirrups on one side of the midspan and were designed to undergo shear failure in a 4-point bending setup. CF_V values were very small for the three beams from each RCA batch with the largest dimensions (-1% and -3%

for 50% and 100% RCA concretes). However CF_V was significant for beams with smaller sections; -10% for both 50% and 100% RCA concrete beams.

Gonzalez-Fonteboa and Martinez-Abella [22] produced control and 50% RCA by volume concrete batches. The RCA was obtained by crushing demolition debris. The NA and RCA concretes had similar compressive strength values ($f'_{c,avg} = 40$ MPa). They used each type of concrete to produce longitudinally reinforced concrete beams with four different shear reinforcement ratios. The beams were tested in a four-point bending configuration and all failed in shear. No correlation between the shear reinforcement ratio and CF_V could be observed. CF_V was significant for most of the beams and had an average value of 16%. They performed a similar study [23] in which both the NA and 50% RCA concrete batches incorporated silica fume (silica fume to cement weight ratio of 8%). The results show that for all shear reinforcements CF_V was negative and had an average value of -6%.

2.3. Discussion of results of prior studies

From what has been presented above it can be concluded that in a majority of the investigations performed in the past CF_M is insignificant ($CF_{M,avg} = 0.1\%$) while CF_V tends to be negative with higher magnitudes ($CF_{V,avg} = -4.2\%$) as shown graphically in Figure 1. This implies that while replacing NA with RCA does not tend to negatively affect the accuracy of the predicted bending moment capacity by the current ACI Code, the prediction of shear capacity by the code is typically less conservative for RCA concrete. That may be because shear capacity depends greatly on the bond strength in the interface between the cementitious mortar and coarse

aggregate in the Interfacial Transition Zone (ITZ). Since there are two ITZs in RCA concrete; the zone between the old mortar and virgin aggregate in the RCA and the zone between RCA and the mortar of new concrete the shear capacity in RCA concrete beams is likely to be even more of a complex phenomenon than in NA concrete beams. In addition, the ITZs in the RCA could have previously been damaged in the crushing process. Because of these conditions the current (i.e., conventional) ACI equations for predicting shear capacity in concrete beams may not be appropriate to use for RCA concrete beams.

Based on this analysis it can also be concluded that when strengthening RCA beams with FRP it is important to pay special attention to the potential lower than predicted shear capacity of the RCA beams, even though the FRP strengthened bending moment capacity may be well predicted. It appears that in the future new design models will need to be developed to predict the shear capacities of RCA beams and RCA beams strengthened with FRP to avoid premature diagonal shear failures.

The experimental program presented in the next section presents results of a preliminary study of small RCA beams strengthened with FRP that was designed to further explore this issue. A new mechanism that may provide a new design method for FRP strengthened RCA beams in shear has been identified, as will be discussed.

Figure 1. Values of CF_M and CF_V based on prior studies from the literature.

3. Experimental program

3.1. Materials

In the experimental study conducted both NA and RCA coarse aggregates with size No.7 as specified in ASTM C33/C33M-13, with the particle size range of 4.75 mm to 12.5 mm (No.4 (3/16 in) to 1/2 in) were used. The aggregate maximum particle size was selected to comply with the regulations of the ACI 318 Code [24] for beam formwork dimensions and the spacing between the reinforcing bars and beam surfaces. The particle size gradations of the NA and RCA are presented in Figure 2. Crushed granite was used as the NA. The RCA was acquired from a local CDW recycling plant on Long Island, NY. The RCA contained minimal amounts of brick and asphalt and was obtained by crushing mostly non-structural concrete members, such as “slab-on-grade” or “curb-and-gutter” low-strength (20 MPa) concrete. ASTM graded washed sea sand and Type I portland cement was used in both the NA and RCA concrete mixes. Carbon fiber reinforced polymer (CFRP) fabrics [BASF 2015(a) [25], BASF 2015(b) [26]] were used for strengthening the beams and an epoxy resin [BASF 2015(c) [27]] was used to saturate the fabric

and attach the fabrics to the beams. Moisture properties and the specific gravity of the aggregates used in this study are presented in Table 1.

Figure 2. Particle size gradation of aggregates used.

Table 1. Moisture properties and specific gravity of aggregates.

Aggregate	Absorption, %	Total moisture, %	SG
NA	0.77	0.15	2.78
RCA	7.33	3.43	2.33
Sand	0.64	0.44	2.40

3.2. Beam design

The beams had a rectangular cross-section of 100 mm by 150 mm (4 in by 6 in) and a length of 914 mm (36 in) and were reinforced with one embedded Grade 60 No.10 steel bar (US No. 3).

Beam sections were designed according to ACI 318-14 [9] so that, in the absence of FRP

strengthening, they would achieve a very ductile flexural-tension failure ($\rho_s = 0.15\rho_b$) under a monotonically increasing load in a three-point bending setup on a span of $L = 813$ mm (32 in).

No steel shear reinforcements (stirrups) were used in the beams. The results of the test of the beams without the FRP strengthening are reported elsewhere [28]. Due to the low reinforcement

ratios and the absence of any shear reinforcement none of the beams failed in shear (by design) and the prediction ratios CF_M and CF_V cannot be compared for these beams. The maximum bending load capacity of the non-strengthened NA and RCA beams were found to be very similar (approximately 25 kN).

The FRP strengthening system consisted of one 100 mm (4 in) wide fabric bonded to the soffit (bottom) of the beam and extending to just short of the supports. The shear strengthening consisted of closed hoop wraps of fabric bonded in 50 mm (2 in) wide strips and spaced at 125 mm (5 in) on-center as shown in Figure 3. The manufacturer reported properties (tensile strength, f_{tu} , Young's Modulus E_f , and strain at rupture ϵ_{fu}) of the FRP fabrics used for the flexural and shear strengthening are presented in Table 2. The FRP flexural strengthening system was designed according to ACI 440.02-08 [7] so that the longitudinal steel reinforcement would yield before the beam reached its ultimate (strengthened) load carrying capacity. For both NA and RCA beams, FRP shear strengthening and bending test configurations were chosen so that the beam failure would be caused by the formation and propagation of diagonal shear cracks after yielding of tension steel.

Figure 3. Configuration of the FRP strengthening system. (a) Side view, and (2) bottom view.

Table 2. Manufacturer supplied properties of the FRP fabrics used for flexural and shear strengthening [25, 26].

Strengthening	Width, mm (in)	Thickness, mm (in)	f_{fu} , MPa (ksi)	E_f , GPa (ksi)	ϵ_{fu}
Flexural	100 (4.0)	0.33 (0.0130)	3,792 (550)	228 (33,000)	0.0167
Shear	50 (2.0)	0.17 (0.0065)	3,792 (550)	228 (33,000)	0.0167

3.3. Concrete production and sample preparation

Most previous studies on the use of RCA in concrete have been conducted using normal-strength concrete. In this study, the volumetric method of ACI 211.1-91 was used to design and proportion high strength NA and RCA concrete mixtures. Both types of concrete had the same volumetric mix proportions as shown in Table 3, with a slight difference in the amount of superplasticizer to maintain the same workability.

In order to determine the mixing time the absorption rate of the RCAs were investigated. The absorption value of the RCA is significantly larger than that of NA (7.6% vs. 0.6% for the aggregates used in this study). Therefore, the amount of water that must be added to RCA concrete to compensate for the absorption of moisture by air-dried aggregate is significantly higher than that of NA concrete. In addition, if the absorption rate of RCA is slow, it may not become fully saturated during mixing the concrete or even before the concrete hardens. Therefore, the final w/c ratio of the RCA concrete and the cementitious binder strength may be more than intended. Vieira et al. [29] observed that the percent water absorption of oven-dry RCA is approximately 80% of its fully saturated condition after 5 minutes of immersion in water. After this period, the increase in water absorption was much slower tending to a value of 84% after 30 minutes. According to a study which led to recommendations in the German code (DIN 4226-100), RCA can be assumed to reach values of around 90% of the 24-hour water absorption value after 10 minutes [30].

Table 3. Mix proportion of the concrete batches. The weights of aggregates are for the saturated surface dry (SSD) condition.

Concrete	Weight, kg/m ³					
	water	cement	NA	RCA	Sand	SP
NA	244	554	1000	-	529	0.67
RCA	244	554	-	838	529	0.50

To obtain the absorption rate of the RCAs used in this study oven-dry RCAs were immersed in water and towel dried after different durations of immersion. Total moisture content in percent was obtained for each immersion duration as the percentage difference between the weight of the

towel-dried RCA and the weight of the RCA after drying in an oven at the temperature of 105 °C for 24 hours. The absorption rate is presented in Figure 4. The results show that approximately 82% of the maximum moisture content is achieved by the RCA after 30 minutes. Based on laboratory experience the average time from the beginning of mixing to the end of casting the specimens was approximately 30 minutes. Therefore, the absorption capacity value used for adjusting the mixing water was 82% of the total absorption capacity. It should be mentioned that more accurate mixing methods, such as a presoaking method (Ferreira et al. [31]) or a two-stage mixing method (Tam et al. [32]), have been proposed to tackle the issue associated with the moisture absorption rate of RCAs. However, the methods known to be most effective are usually more elaborate and time consuming than the conventional methods used with NAs.

Figure 4. Absorption rate of the RCA used in this work.

For each batch of concrete six cylinders (100 mm by 200 mm) and four identical reinforced beams were cast for compression and three-point bending tests, respectively. Concrete specimens were produced, cured, and tested at room temperature. Each concrete batch was mixed for 10 minutes and cast into the beam and cylinder specimens. The forms were covered with plastic

384 sheets for 24 hours. Then the specimens were demolded and transferred to an environmental
385 chamber with the relative humidity of 98% and kept for 27 days. The beams were removed from
386 the environmental chamber and kept in the laboratory environment for one week so that the
387 surfaces of the beams become fully dry. The sharp edges were rounded as prescribed by ACI
388 440.2R and the FRP fabric and binder were applied according to procedures of the FRP
389 manufacturer.

391 *3.4. Instrumentation and testing*

392 To measure the strain in shear strengthening strips (especially, when diagonal cracks propagate
393 to the zone the strips confine), two external strain gages (strain gages 1 and 2) were attached
394 vertically to the two strips closest to the mid-span of the beam (Figure 3.a). To measure the
395 tensile strain in flexural FRP strengthening strip, two strain gages (strain gages 3 and 4) were
396 attached to the strip in the longitudinal direction (Figure 3.b).

397
398 An Instron servo-hydraulic test machine was used to test the beams. The test setup consisted of a
399 simply supported loading configuration with semi-roller supports to prevent restraint to axial
400 elongation (Figure 5). The beams were loaded at midspan (center-point) and tested in a
401 displacement control static mode using a hydraulic actuator with a capacity of 250 kN (56 kip).
402 Displacement rate of 0.012 mm/sec (0.0005 in/sec) was applied. The parameters measured and
403 recorded during the monotonic testing were beam deflections and the applied load. The
404 compressive strength of 100 mm by 200 mm (4 in by 8 in) cylindrical concrete specimens were
405 measured according to ASTM C39/C39M – 12. All the beams and cylinders were tested 6

months after FRP strengthening fabrics were applied on the beams. The 6 month delay was due to the wait for strengthening supplies and other research priorities.

Figure 5. Three-point bending test configuration in Instron test machine

4. Results and discussions

Load vs. mid-span displacement results for RCA and NA concrete beams are presented in Figure 6. The average values of yield loads, first drop loads, and maximum loads for both RCA and NA concrete beams, as well as concrete compressive strength f'_c values, are presented in Table 4.

Figure 6. Load vs. midspan displacement for (a) NA and (b) RCA concrete beams.

Table 4. Average values of yield load, first drop load, and maximum load for NA and RCA concrete beams.

Concrete beam	f'_c , MPa (ksi)	Yield load kN (kip)	Load at first drop kN (kip)	Maximum load kN (kip)
NA	77.8 (11.28)	39.8 (8.90)	52.1 (11.70)	55.2 (12.4)
RCA	51.8 (7.51)	39.7 (8.92)	62.4 (14.04)	68.4 (15.37)

The average values of f'_c for NA and RCA concretes were 77.8 MPa and 51.8 MPa, respectively. The results show that the yield load is almost the same for both RCA and NA concrete beams (39.8 kN for NA vs. 39.7 kN for RCA concrete beams). This is expected as the yield load of reinforced concrete beams is minimally affected by the compressive strength of concrete [5]. However, the first drop loads and maximum loads of RCA beams were 20% and 24% higher than those of their NA counterparts (Table 4). More details on the design criteria and theoretical predictions for the unstrengthened and the FRP strengthened NA and RCA beams and more experimental data can be found in [33]. To understand the reason for this counterintuitive finding the behavior of the FRP strengthened beams after yielding of the tension steel is discussed in greater detail in what follows.

In both the NA and RCA concrete beams, after the yield load diagonal cracks start to propagate into the zones confined by FRP shear hoops (Figure 8). The beam failure starts and progresses with debonding of FRP flexural strip (Figure 8). As mentioned earlier, the shear strips were applied over the flexural strips. Therefore, the shear strips function as a confining system that can postpone debonding of the flexural strip. This confining system becomes more effective if the shear strips undergo tension and are strained.

Figure 7. Diagonal crack in two of the beams formed after yielding of tension steel and propagating in the zones confined by the shear straps.

Figure 8. A beam after testing.

Figure 9 shows the load versus strain in shear strips (measured by Gages 1 and 2) for NA and RCA concrete beams. These graphs show that there is almost no strain in the shear strips up to the loads 17 kN (3.8 kips) and 13 kN (3.0 kips) for NA and RCA concrete beams respectively. These loads are associated with the initiation of formation of tension cracks in concrete.

Figure 10 (load versus strain measured by Gages 3 and 4) shows that there is a sudden increase in tensile strain in the flexural strips at these loads. After the initiation of the flexural–tension cracks, small compressive strains are measured by Gages 1 and 2 (which may be due to an arching effect, or the depth compression of the beams as they settle into the supports) until the flexural–tension cracks change direction and form diagonal cracks that propagate towards the load point and intersect the shear hoops causing a sudden increase in the strain in the hoops. For each beam, the level of load at which a diagonal crack enters the zone confined by the strips are different since these cracks do not initiate in the same location nor propagate with the same pattern even in the beams with the same type of concrete.

Figure 9. Strain in the shear strengthening strips for (a) NA and (b) RCA concrete beams

Figure 10. Strain in the flexural strengthening strips for (a) NA and (b) RCA concrete beams

It is important to closely observe the measured strain in the shear strips after the yield load, where shear cracks has already developed and propagated into shear strip zone, and before the first drop load, which is associated with the initiation of debonding of flexural strip. Yield load for both NA and RCA concrete beams is 40 kN (9 kips) (Figure 6 and Figure 10). The average strain values in the shear strips at the load of 40 kN (9 kips) are 0.00024 and 0.00060 for NA and RCA concrete beams, respectively (disregarding one outlier) (Table 5). At the load of 45 kN (10 kips) the strain values are 0.00065 and 0.00115 for NA and RCA concrete beams, respectively. The higher values of strain in shear strips in RCA beams are expected as the compressive strength of RCA concrete is approximately 35% lower than that of NA concrete, reflecting the lower shear strength of the RCA concrete. Figure 11 shows the surfaces of shear cracks in NA and RCA beams after testing. The crack in the RCA beam has an observably smoother surface and has passed through the weak coarse aggregates (RCAs) suggesting the lower shear strength of RCA concrete. Due to the lower shear strength of RCA concrete, the diagonal shear cracks in RCA beams grow wider at lower loads, and therefore, for a given load the shear cracks in RCA

beams are more widely open causing more tensile strain in shear strips and stronger confinement of flexural strips.

Table 5. Average vertical strain in FRP shear strips (measured by Gages 1 and 2) for two different load levels

Load, kN	Average strain	
	NA Beams	RCA Beams
40 (Yield load)	0.00024	0.0006
45	0.00065	0.00115

Figure 11. Surface of shear cracks in an NA concrete beam (top beam) and a RCA concrete beams (bottom beam).

5. Conclusions

A study comparing the predictions of ACI 318 Code for load carrying capacity of reinforced concrete beams and the bending test results from experimental investigations were presented.

The study shows that while the provisions of the code predicts the flexural strength of reinforced

NA and RCA concrete beams with similar accuracy, the code predictions for the shear capacity of RCA concrete beams are less conservative than those for NA concrete beams.

The presented experimental study investigated a hybrid shear-flexural FRP strengthening system. The results show that when flexural FRP strips are effectively restrained and confined by FRP shear strips, if failure is caused by shear cracking, beams made with weaker concrete (here, RCA concrete), will have higher load carrying capacity. This is due to the fact that failure in strengthened beams occurs due to debonding of flexural strips. The flexural strips are confined by shear strips, which provide a higher confinement when they are stressed due to the propagation and widening of shear cracks. For weaker concrete shear cracks propagate and widen at lower loads. Therefore, the shear strips are more stretched and confine the flexural strips more effectively.

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