Visual Abstract

Effect of GFRP and steel reinforcement bars on the flexural behavior of RC beams containing recycled aggregate

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ABSTRACT. Concrete containing wastes from the demolition of old deteriorated buildings are produced enormously. Concrete is a brittle matrix that is usually reinforced by ductile reinforcement such as steel bars. However, due to the susceptibility of steel to corrosion, fiber-reinforced polymers (FRP) bars are used as an alternative reinforcement. The main drawback of FRP bars is their brittleness. These two types of reinforcements, i.e., steel and glass FRP (GFRP) bars, have been used in the present work. The flexural behavior of twelve RC beams reinforced with different ratios of GFRP or steel areas containing recycled aggregate has been experimentally studied and compared with beams without recycled aggregate. The present results show that beams reinforced with GFRP and containing recycled aggregate exhibit a lower load-carrying capacity, lower the first crack, and the highest deflection. All GFRP RC beams exhibited brittle failure, i.e., concrete crushing in the compression zone, except one beam, with 2□16 bars and concrete without recycled aggregate, which ruptured GFRP bars. However, ductile failure modes are observed for all beams reinforced with steel bars, i.e., yielding in steel bars followed by concrete failure. The novelty and advantage of the present results are that the large deflection of GFRP RC beams represents enough warning before failure, as found in ductile RC beams.

KEYWORDS. Recycled aggregates; GFRP bars; Steel bars; RC Beam.

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INTRODUCTION

Rao et al. [1] stated that most of the total solid waste produced in the world is construction and demolition waste. Recycling such solid waste is considered an environmental challenge [2-3]. Recycled aggregate is a sustainable solution for the environmental depletion of the construction sector [4]. Therefore, using this recycled solid waste to partially replace aggregates in concrete manufacturing has attracted considerable attention from many researchers [5-7]. The workability of the recycled concrete is reduced due to an increase in coarse aggregate porosity, and additional water may be required to get the same slump [8]. The microstructure of the Interfacial Transition Zone (ITZ) depends mainly on the w/c ratio of the new paste; moreover, the ITZ of the concrete made with recycled aggregate is influenced by the pre-saturation water, which can be released into the new paste, disrupting its microstructure [9]. When the recycled aggregate replacement level is increased by weight, the compressive strength decreases for normal, medium, or high-strength concrete [10]. Also, the concrete made with 10% recycled aggregate is weaker than concrete made with natural aggregate at the same water to cement ratio. This may be due to the influences of crushing types on recycled aggregates characteristics. The method of preparing recycled aggregate for mixing concrete influences the concrete workability when using different percentages of coarse recycled aggregate content (0%, 50%, and 100%). When using recycled aggregate that is water-saturated and a surface dry, similar workability is achieved. Besides, the same workability can be achieved after a prescribed time when using dried recycled aggregate and adding additional water during the concrete mix. The time taken to achieve the same workability is called the additional water time. The amount of recycled aggregate affects the amount of water absorption of concrete. This means that the increase of the recycled aggregate amount will proportionally increase the amounts of water absorption. Otherwise, increasing the recycled aggregate to 100% leads to an increase in the compressive strength up to 25% and increases the deflection to 4% [11]. This discrepancy may appear because the recycled aggregate was made by crushing waste concrete of laboratory test cubes and precast concrete columns.

The corrosion problem is initiated in hot countries such as the Middle East due to hot weather and high level of humidity [12]. FRP reinforcement emerged as a practical solution because of its non-corrosive properties, high strength-to-weight ratio, and good fatigue resistance [13]. Due to this brittle nature of FRP, it was recommended to direct the flexure members to fail in compression rather than fail in tension [14, 15]. In such brittle characteristics, this type of failure is less catastrophic and more progressive. To determine the deflection of FRP reinforced beams, most code guidelines adopted a simple elastic method, Branson's equation [16], and an effective moment of inertia equation to describe the reduced stiffness of cracked sections. The effective moment of inertia originally modified by ACI 316 [16] was adopted by ACI 440.1R-06 [17]. Most experimental results of GFRP reinforced beams in the literature [18-20] showed higher ultimate moment capacities than those predicted by most code guidelines. When using high-strength concrete in GFRP reinforced beams, a brittle behavior, sudden failure, and fewer width cracks were observed compared to beams made of normal concrete [21]. In Ref. [22], it was shown that CFRP-reinforced concrete beams' performance and behavior are comparable to the conventional steel-reinforced concrete beams. A new proposed tension stiffening model for concrete members reinforced with GFRP predicted significantly better experimental results than the existing models [23]. Considering the shear performance of steel-reinforced concrete beams cast with recycled aggregate [24], an equation for estimating the shear was proposed. It showed satisfactory results when verified against the experimental results. A study of the effect of recycled aggregate on the behavior of structural beams in flexure, shear, and bond showed a minimum difference in the peak load and load-deflection behavior attributed to the percentage replacement of natural concrete aggregate with recycled concrete aggregate [25]. It is worth noting that Hamad et al. [25] carried out a bond splitting (Bond beams) test to measure the bond strength.

RESEARCH SIGNIFICANCE

Optimal use of recycling construction and demolition waste in the concrete industry is one of the most important environmental challenges. This research work attempts to understand the behavior of reinforced concrete (RC) containing recycled coarse aggregate (RCA) from demolition waste either with ductile or brittle reinforcement. The characteristics of GFRP made by a manual method are tested. The characteristics of concrete made with the RCA from demolition were also studied and compared to the characteristics of new concrete, i.e., concrete with the natural coarse aggregate (NCA). Finally, a comparative study was made between the behavior of beams reinforced with steel reinforcements - new concrete and beams reinforced with the manually made GFRP bars and the recycled aggregate.

EXPERIMENTAL PROGRAM

The experimental program was divided into three stages. The first stage was the fabrication and testing of GFRP bars. A manual method was used in the fabrication of GFRP bars. Samples of the fabricated bars are tested in tensile and bond strength to get the GFRP mechanical characteristics. In the second stage, a comprehensive experimental test was carried out to measure the properties of both new concrete and concrete containing RCA. The flexural characteristics of the GFRP and steel-reinforced beams made with 11 recycled and new concrete were studied in the last stage. Four main aspects were studied, load-carrying capacity, deflection, load at first crack, and failure mode. The materials used in this research were natural sand, natural crushed stone (dolomite) for new concrete, dolomite-based crushed concrete as the source of recycled aggregate, ordinary Portland cement, steel bars, and GFRP bars.

Fabrication of GFRP bars

GFRP bars were fabricated in the experimental program using a hand lay-up technique [26]. This was done by stretching the thread of glass fiber between two hooks. The homopolymer polypropylene series resin used was fabricated in Vetrotex Company, USA. It consists of two main components, polyesters and hardeners. The physical and mechanical properties of the used glass fiber and resin are listed in table 1. The relation between the number of rolls and the final diameter of the bars was deduced, as listed in table 2.

Materials	Specific gravity	Tensile elongation %	Tensile strength (MPa)	Flexural strength (MPa)	Tensile modulus (GPa)	Flexural modulus (GPa)
Glass fiber	2.54	4.5%	3250	--	69	--
Resin	--	--	140	210	--	7.5

Table 1: The physical and mechanical properties of the glass fiber and resin as reported by the manufacturer.

Bar diameter	No. of rolls*
10 mm	35
12 mm	45
16 mm	65

Table 2: Relation between the number of rolls and diameter of GFRP bars (*The weight of one batch of rolls equal 20 kg).

For every 1 m of GFRP bars, the resin polyester was pushed with a 150 gm force using the movable hook. To press out the excessive quantity of the used resin, the movable arm will rotate and pull out the fibers to make it twisted fiber after being saturated with resin. When the clear distance between the two hooks reaches the required length, the process may be stopped [26], as shown in Fig.1. After sufficient curing of the GFRP bars, samples of bars were taken to examine their tensile and bond characteristics. Samples from GFRP bars having a length of 1.2 meters each were tested in tension to get tensile strength and Young's modulus to use these characteristics in the design equations of beams [27]. Using a standard procedure of pullout test according to ASTM D7913 [28], the bond strength specimens' tests were made to all GFRP bars diameters; 10, 12, 16 mm. Three samples representing each nominal diameter (10, 12, 16 mm) were tested to get the bond strength of the GFRP bars. During the fabrication of bars, no coating material was glued to the surface of bars to increase the bond strength between GFRP bars and the surrounding concrete. The bonding response was dependent completely on the normal surface of bars as resulted from initial fabrication.

a) The device for fabricating GFRP bars

b) Continuous fibers are tensioned between 2 hooks.

c) Saturating the fibers with the resin.

d) pulling out and twisting the fibers to squeeze out the excessive amount of the resin.

Figure 1: Fabrication processes of GFRP bars.

Properties of concrete with and without RCA

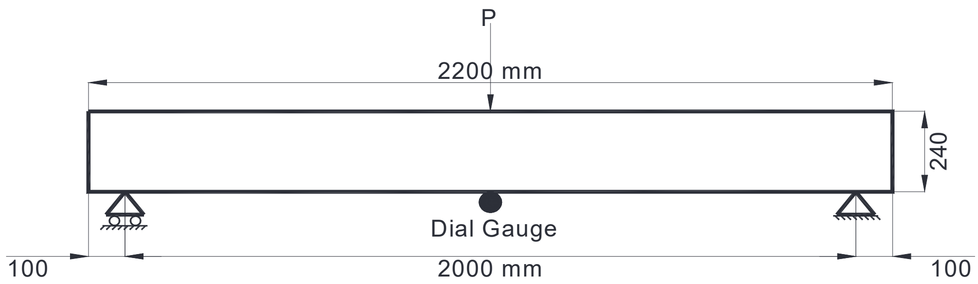
In this stage, a natural crushed stone (dolomite) was used as a coarse aggregate, which is common and available in Egypt. Dolomite had a specific weight of 2.61, a bulk density of 1.65 t/m^3, water absorption, and 2.05%. Fine aggregate (sand) had a specific weight of 2.36 and bulk density of 1.7 t/m^3. Recycled coarse aggregate had a specific weight of 2.41, bulk density of 1.74 t/m^3, and water absorption of 5.51%. The cement used was CEMI42.5N. For both recycled and new concretes, the compressive strength in 7 days and 28 days and indirect tensile strength in 28 days was determined. The tests were made according to ASTM C469-96 [29] and ASTM C469-94 [30], respectively. The mix design of new and recycled concrete was summarized in table 3.

Type of Concrete	Cement (kg)	Natural Coarse Agg. (kg)	Recycled Coarse Agg. (kg)	Fine agg (kg)	W/C
New Concrete	416.7	992	0	724.5	0.48
Recycled Concrete	416.7	496	496	724.5	0.48

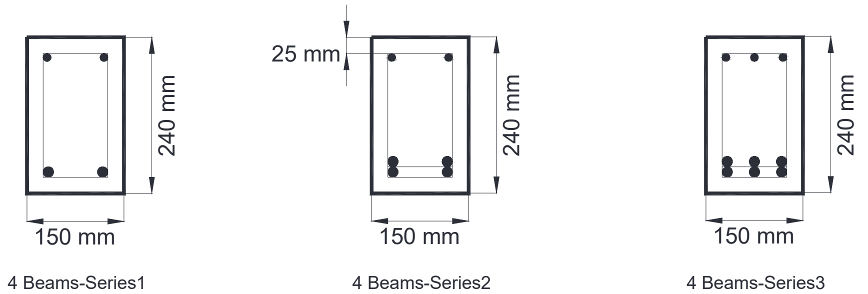
Table 3: Mix design of new and recycled concrete.

Preparation of Beams

In this stage, GFRP RC beams and steel RC beams with and without RCA were prepared. A three-point bend test was adopted to determine the first crack, deflection, load carrying capacity, and failure mode of RC beams as a function of reinforcement type (GFRP and steel rods) and matrix type (concrete with and without RCA). The twelve beams were categorized into three series; each series was composed of four beams with identical reinforcement areas but different types of reinforcement (GFRP and steel rods) and different types of concrete (with and without RCA). In the first series, tensile reinforcement was 2ϕ16 mm, while top reinforcement was 2ϕ10 mm. In the second series, tensile reinforcement was 4ϕ16 mm, while top reinforcement was 2ϕ12 mm. In the third series, tensile reinforcement was 6ϕ16 mm, while top reinforcement was 3ϕ12 mm. All beams had 2200 mm long provided with f8 mm steel stirrups at 167 mm center to center (6ϕ8/m), 2000 mm span, 240 mm depth, and 150 mm width. The experimental setup, instrumentation, and details of the test beams are shown in Fig. 2. Details of reinforcements of all beams are listed in table 4.



(a) The test set up and instrumentation of beams
(b)



(c) Details of beams

Figure 2: The test setup, instrumentation, and details of beams

Beams*	Bottom (Tensile) reinforcement	Top (Compression) reinforcement
BNG1 BRG1	2 GFRP rods ϕ 16	2 GFRP rods ϕ 10
BNS1 BRS1	2 steel rods ϕ 16	2 steel rods ϕ 10
BNG2 BRG2	4 GFRP rods ϕ 16	2 GFRP rods ϕ 12
BNS2 BRS2	4 steel rods ϕ 16	2 steel rods ϕ 12
BNG3 BRG3	6 GFRP rods ϕ 16	3 GFRP rods ϕ 12
BNS3 BRS3	6 steel rods ϕ 16	3 steel rods ϕ 12

Table 4: Details of reinforced concrete beams (*Identification Codes: B: Beam; N: New concrete; R: Recycled concrete; G: GFRP bars; S: Steel bars; 1, 2, and 3: Series number).

Mechanical properties of reinforcements and concretes

Three samples from each nominal diameter (10, 12, and 16 mm) were tested to get its tensile characteristics. The mechanical properties of steel bars are summarized in table 5. The results of tensile and bond strength of GFRP bars are presented in table 6. As expected, the GFRP bars had a typical linear behavior up to failure, i.e., exhibited brittle failure, as shown in Fig. 3. For all nominal diameters, the ultimate tensile strength (UTS) of GFRP bars is higher than those corresponding in steel bars, as shown in tables 5 and 6.

	Steel bars		
Nominal Diam. (mm)	10	12	16
Yield stress (MPa)	360	430	510
UTS (MPa)	415	520	600

Table 5: Tensile properties of steel bars.

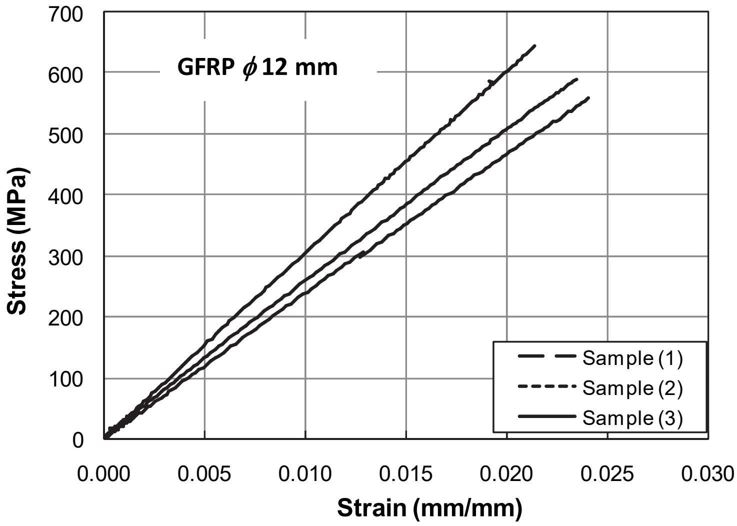


Figure 3: Stress-strain curve of ϕ12 GFRP.

Nominal/Actual Ave. Diam. (mm)	Average Tensile strength (MPa)/ (Coefficient of variation (%)	Average Bond strength (MPa)/ (Coefficient of variation (%)
10/10.23	435.6 / (5.1%)	7.0 / (4.3%)
12/12.32	597.3 / (7.2%)	5.0 / (9.2%)
16/16.5	620.4 / (7.6%)	3.1 / (8.0%)

Table 6: Tensile and bond strength of GFRP bars.

Fig. 4 shows new and recycled concrete's compressive and tensile strengths at 7 and 28 days. It is shown that the compressive strength of concrete with RCA was less than those of concrete without RCA by 28.8 % and 13.3 % for 7 and 28 days, respectively. Furthermore, it was found that the 28 days tensile strength of concrete with RCA was less than that of concrete without RCA by 30 %. This reduction may be attributed to the presence of the cement paste or a certain amount of mortar surrounding the particles of RCA. The percentage of water absorption of concrete with RCA was 60% higher than that of concrete without RCA. This may be due to the attached mortar around the RCA particles. These observations are in agreement with the results obtained in the literature [31].

Structural behavior of RC beams

The experimental results of all tested beams (first crack, deflection, load-carrying capacity, and mode of failure) are summarized in table 7. In general, the first crack load and ultimate load of all RC beams containing RCA are lower than those of corresponding RC beams without RCA. The opposite trend was observed for maximum deflection.

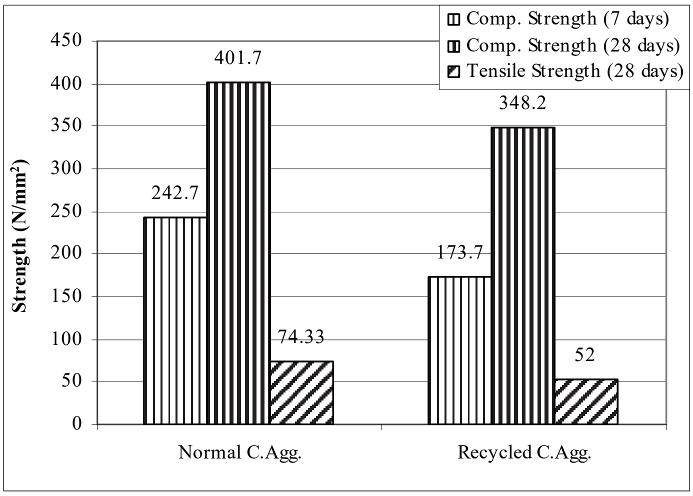


Figure 4: Compressive and tensile strength of concrete with and without RCA.

Beam	First crack Load (kN)	Ultimate Load, P_{ULT}, (kN)	Maximum Deflection (mm)	$P_{ULT\text{-}RCA}/P_{ULT\text{-}NCA}$ For beams failed due to concrete crushing	Failure mode
BNG1	15	57	27		Rupture in FRP
BRG1	12	54	35	N/A	Concrete crushing
BNS1	18.5	70	22		Yielding in steel followed by Concrete Crushing
BRS1	15	63	25	0.90	Yielding in steel followed by Concrete crushing
BNG2	21	70	25		Concrete crushing
BRG2	17	62	32	0.89	Concrete crushing
BNS2	28	79	18		Yielding in steel followed by Concrete crushing
BRS2	24	78	22	0.99	Yielding in steel followed by Concrete crushing
BNG3	31	79	23		Concrete crushing
BRG3	25	72	31	0.91	Concrete crushing
BNS3	39	89	17		Yielding in steel followed by Concrete crushing
BRS3	28	85	19	0.96	Yielding in steel followed by Concrete crushing

Table 7: Flexural test results of all RC beams

In the beginning, a comparison between the measured values of ultimate loads of GFRP RC beams and those predicted by the ACI 440.1R-06 provision [17] was made. The approach of the ACI code was based on the forces equilibrium, strain compatibility of sections and considers the equivalent stress block. The equations of moment capacity depend on whether the reinforcement ratio is lesser or higher than the balanced ratio:

$$\rho_f = \frac{A_f}{bd} \tag{1}$$

$$\rho_{fb} = 0.85\beta_1 \frac{f_c^{'}}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \tag{2}$$

When $\rho_f < \rho_{fb}$

$$M_n = 0.8 A_f f_{fu} \left(d - \frac{\beta_1 c_b}{2} \right) \tag{3}$$

$$c_b = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) d \tag{4}$$

When $\rho_f > 1.4\rho_{fb}$

$$f_f = \left(\sqrt{\frac{\left(E_f \varepsilon_{cu}\right)^2}{4} + \frac{0.85\beta_1 f_c^{'}}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} \right) \le f_{fu} \tag{5}$$

$$M_n = A_f f_f \left(d - \frac{a}{2} \right) \tag{6}$$

$$a = \frac{A_f f_f}{0.85 f_c^{'} b} \tag{7}$$

Also M_n can be calculated from

$$M_n = \rho_f f_f \left(1 - 0.59 \frac{\rho_f f_f}{f_c^{'}} \right) bd^2 \tag{8}$$

Theoretical and experimental ultimate loads are compared in Table 8. There is a good agreement between them.

Crack pattern and Mode of Failure

Crack pattern and mode of failure of steel RC beams are typical of under-reinforced RC beam behavior, as shown in Fig. 5. a and b. The first crack loads ranged from 15 to 39 kN based on the tensile reinforcement ratio and concrete type, see table 7. In general, steel RC beams with RCA had lower first crack loads than steel RC beams without RCA, as listed in table 7. Flexural cracks propagate upwards as loading progress but remain very narrow throughout the loading history. With increasing load, flexural-shear cracks initiated propagated towards the point of load. With a further increase in the applied load, crushing of concrete at the top compression side around the point of load application occurred at the ultimate load after yielding steel reinforcement. On the other hand, all GFRP RC beams failed due to concrete crushing in the compression zone, i.e., brittle failure, except beam BNG1, which failed due to the rupture of tensile GFRP bars, i.e., catastrophic failure, Fig. 5. c-e.

Beam Code	Pu ACI 440 (kN)	Pu exp. (kN)	Pu exp. /Pu ACI 440
BNG1	56	57	1.02
BNG 2	66	70	1.06
BNG 3	77	79	1.03
BRG 1	56	54	0.96
BRG 2	61	62	1.02
BRG 3	70	72	1.03

Table 8: Ultimate theoretical and experimental loads.

(a)
(b)
(c)
(d)
(e)

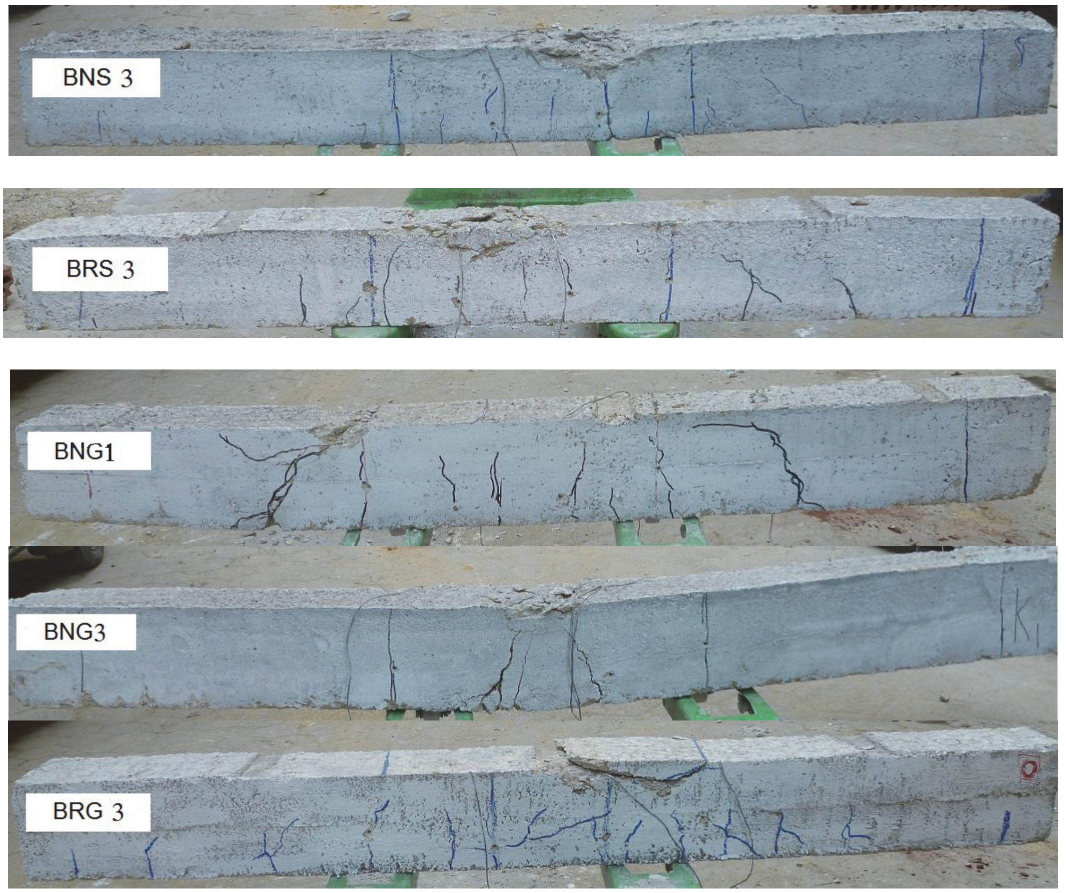


Figure 5: Crack pattern and mode of failure of RC beams.

In the case of GFRP RC beams, the failure mode of beams may be predicted by computing the GFRP reinforcement ratio and comparing it to the balanced GFRP reinforcement ratio from equations (1) and (2). The ratio of the balanced FRP reinforcement was a ratio when the crushing in the concrete happened at the same time as the rupture in FRP when the ratio of FRP reinforcement is less than the balanced ratio, ($\varrho_f < \varrho_{fb}$), the failure mode is expected to be a rupture in FRP.

Oppositely, when the balance ratio of the FRP reinforcement is less than the ratio of the FRP reinforcement ($\varrho_{fb} < \varrho_f$), the mode of failure is expected to be crushing of concrete at the top middle part of the beam. It may be noted that the balanced ratio for FRP reinforcement ϱ_{fb} is lower than the balanced ratio for steel reinforcement ϱ_b [32-34]. Beam BNG1 (2 #16) had a GFRP reinforced ratio of ϱ_f = 0.0112, which is less than the balanced ratio (ϱ_{fb}) =0.0116. The first crack at the tension face of the beam appeared at 12 kN. When increasing the load, another crack in the tension face appeared. After that, the shear cracks almost 45 degrees. At a load of 57 kN, the beam failed due to GFRP rupture, see Fig. 5.c. Beams BNG2 and BNG3 were made of new concrete and had a reinforced ratio of FRP bars ϱ_f = 0.0223 and 0.033, respectively. Both ratios were bigger than the balanced ratio, and the first cracks appeared at mid-span at the loads 17 kN and 25 kN. At loads 70 kN and 79 kN, the beams failed in the compressive region due to crushing in the middle of the beam. The mode of failure of beams BNG3 was illustrated in Fig. 5.d. The ϱ_f of beams with RCA, BRG1, BRG2, and BRG3; were similar to beams BNG1, BNG2, and BNG3, but the ϱ_{fb} ratio was lower due to using of a lower strength concrete. The balanced ratio ϱ_{fb} = 0.001. The first crack of BRG1, BRG2, and BRG3 appeared at 15, 21, and 27 kN. The beams failed in the compression zone at loads of 54, 62, and 72 kN, respectively, as shown in Fig. 5. It is worth noting that failures in BNS3 and BNG3 are quite similar, and it complies with what was stated in the literature, i.e., steel and GFRP have a similar effect on the flexural behavior of the beams. However, RC beams with RCA, i.e., lower concrete's compressive strength, showed a significant difference in the crack patterns, see Fig. 5.b. (BRS3) and Fig. 5.e. (BRG3). Where BRG3 showed diffuse cracks near the supports may be due to the lower concrete's strength and lower modulus of elasticity of the reinforcements (GFRP bars). It is clear from table 7 that the ratios of ultimate load of RC beams with RCA to that of RC beams with NCA ($P_{ULT-RCA}/P_{ULT-NCA}$) are lower in the case of GFRP RC beams than steel RC beams. Therefore, the ultimate loads of GFRP RC beams are more affected by the compressive strength of concrete than steel RC beams due to the lower modulus of elasticity of GFRP. This is a limitation on the use of either GFRP bars or RCA.

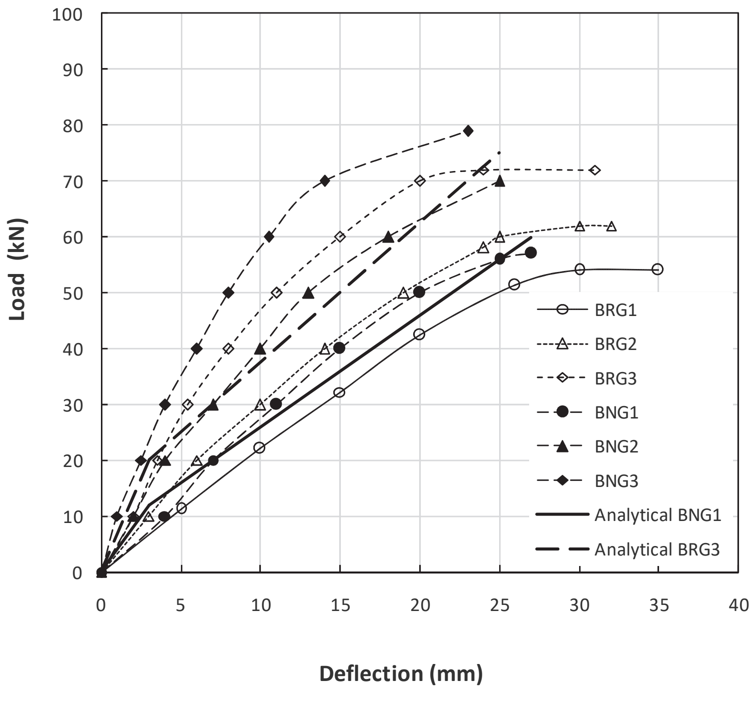


Figure 6: Load-central deflection curves of GFRP RC beams.

Flexural behavior of RC beams

The experimental results of load-central deflection for GFRP RC beams and steel RC beams are shown in Figs. 6 and 7, respectively. Relative linear elastic behavior was observed in all beams until reaching the cracking limit of the beam at the tension face. It can be seen that the deflections of GFRP RC beams with RCA are higher than those of GFRP RC beams without RCA by about 1.3 to 1.55 times, see Fig. 6 and table 7. The same trend was observed in the case of steel RC beams, see Fig. 7. The deflection increase is attributed to the recycled concrete's modulus of elasticity and strength

reduction. This means the material is easier to be deformed and cracked. A greater deflection was obtained at a lower 1oad in beams made of recycled aggregate than beams made with new concrete [8].
Furthermore, the deflection of RC beams reinforced with GFRP was higher than that of corresponding beams reinforced with steel by about 1.7 times. This may be due to the lower GFRP bar's modulus of elasticity than that of the steel. These observations agree with the results found in the literature; it was found that beams reinforced with GFRP had a higher deflection by about 2.5 to 3.0 times than that of the beam reinforced with steel [33].

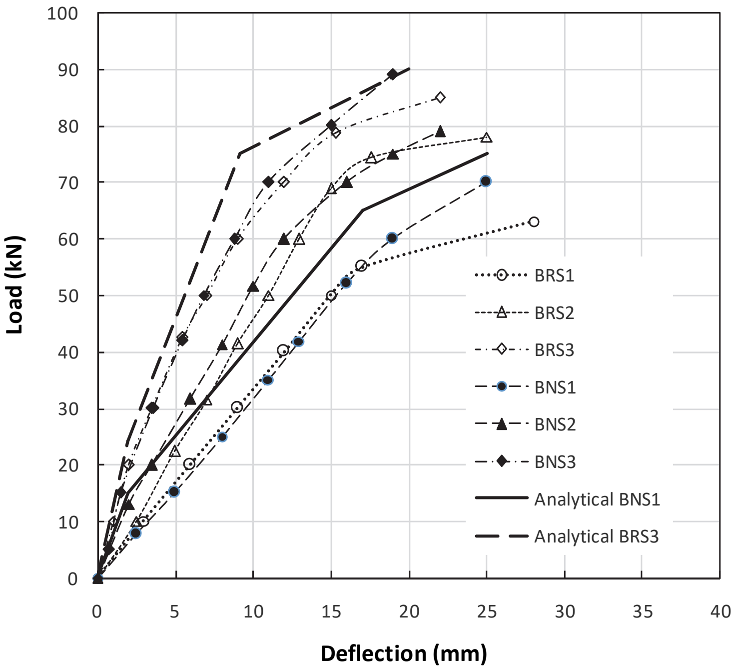


Figure 7. Load-central deflection curves of steel RC beams.

The present experimental results have been compared with the analytical model proposed previously by Sallam and colleagues [35-36], see Figs 6 and 7. Assumptions and details of the analytical model can be found in Refs. [35-36]. In both cases (steel and GFRP RC beams), the final failure of the beams is due to concrete crushing in the compression zone, while in steel RC beams, the tensile reinforcement reached its yield stress before concrete crushing. However, in GFRP RC beams, concrete crushing occurred before the tensile reinforcement reached its ultimate tensile strain. In the case of steel RC beams, BNS1 and BRS3 (Fig. 7), There is good agreement between the analytical data and the experimental results, and this may be due to the occurrence of steel yielding before concrete crushing. However, there is a fair agreement between the analytical data and the experimental results of BGN1 and BGR3 (Fig. 6). It may be because concrete crushing occurred after the first crack in the tensile zone without any events between them. Therefore, the analytical model cannot describe the nonlinearity of the P-d curve of GFRP RC beams.
Fig. 8 shows the effect of the presence of RCA and reinforcement type and area on the maximum deflection. By comparing the reduction in maximum deflection due to the increase of reinforcement area in different cases, it can be stated that the increase of steel area is more pronounced than that of GFRP in decreasing the maximum deflection in the case of concrete without RCA. In the case of lower strength concrete (concrete with RCA), the increase of reinforcement area for both types of reinforcement has the same effect. The effect of RCA in concrete is more pronounced in the case of GFRP RC beams.
Fig. 9 shows the effect of the presence of RCA and reinforcement type and area on the ultimate load of RC beams. It can be seen that the ultimate loads of GFRP RC beams with/without RCA were slightly lower than those of steel RC beams. This may be attributed to the steel interlocking effect, which improves the bond between steel bars and concrete, increasing load-carrying capacity. When the reinforcement bars cannot transfer the bond force, cracks parallel to the rebar are developed [34]. Furthermore, it can be observed that the RC beams made of recycled aggregate and reinforced with either GFRP or steel reinforcement had a load-carrying capacity lower than that beams made of natural coarse aggregate by about 5%, 11 %, and 9% for beams BRG1, BRG2, and BRG3, respectively, and about 10%, 1.3%, and 4% for beams

BRS1, BRS2, and BRS3, respectively. This reduction is attributed to the lower compressive strength of concrete made with recycled aggregate than concrete made with natural aggregate.

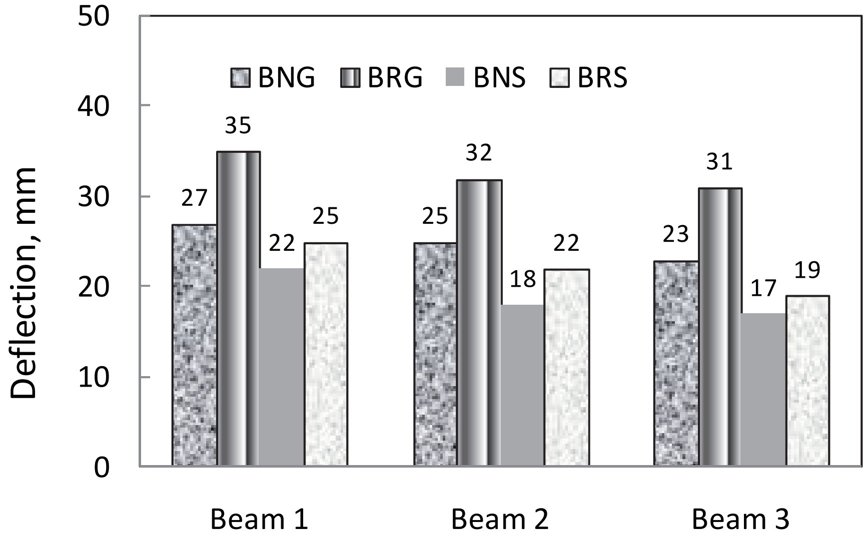


Figure 8: Maximum deflection for all RC beams.

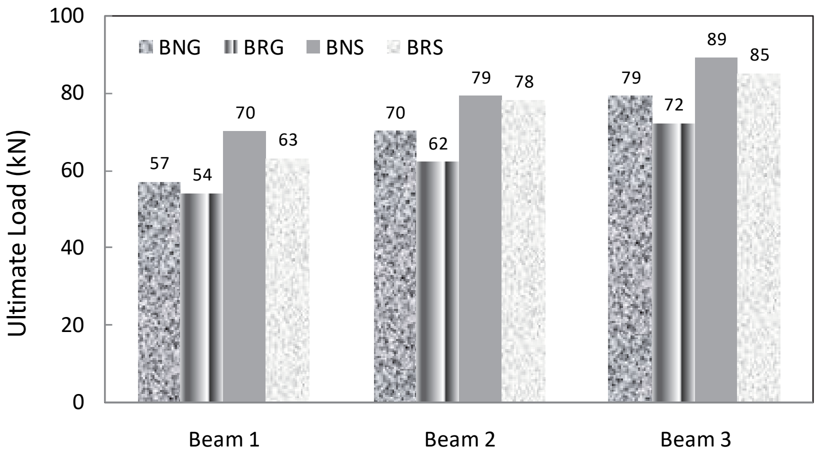


Figure 9: Ultimate Loads values for beams.

Conclusion

Based on this study, the following conclusions can be drawn:

- ✓ The characteristics of concrete made with RCA showed an acceptable lower compressive strength value than concrete without RCA (≈ 13% reduction) and a higher absorption ratio (≈ 60 % higher than natural aggregate concrete).
- ✓ The GFRP RC beams with RCA had sufficient load-carrying capacities compared to those beams without RCA (≈ reduction of 5%, 11%, and 9 % for reinforcement ratios 0.011, 0.022, and 0.033, respectively).
- ✓ The steel RC beams with RCA had a good load carrying capacity compared to those without RCA (≈ reduction of 10%, 1.3%, and 4 % for reinforcement ratios 0.011, 0.022, and 0.033, respectively).
- ✓ ll GFRP RC beams had lower ultimate loads than those of corresponding steel RC beams. These reductions are

about 19%, 11%, and 11% for beams without RCA and 14%, 21%, and 15% for beams with RCA, considering reinforcements ratios (0.011, 0.022, and 0.033), respectively.

- ✓ The use of RCA increases the deflection of RC beams compared to RC beams without RCA due to the lower modulus of elasticity of GFRP.
- ✓ The ultimate loads of GFRP RC beams are more affected by the compressive strength of concrete than steel RC beams due to the lower modulus of elasticity of GFRP. This is a limitation on the use of either GFRP bars or RCA.
- ✓ Based on the strength, durability, and sustainability points of view, the economic benefit is considered another limitation of using either GFRP or RCA. Therefore, additional research should be conducted to cover the durability and sustainability of such materials.

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