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COMPARATIVE STUDY OF STEEL AND GLASS FIBRE REINFORCED POLYMER (GFRP) BARS IN RC MEMBERS

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ABSTRACT

Corrosion of steel reinforcements is one of the main problems in reinforced concrete structures that shortens their serviceability and reduces their strength. The RC structures in coastal environments are usually exposed to early deterioration and damage due to the existing extreme conditions that initiates corrosion. Glass fibre reinforced polymer (GFRP) bar is regarded as a next generation substitute material to conventional steel bars. This paper focuses on the behavior of GFRP reinforcement bars and steel reinforcement bars in RC members in terms of bonding and flexure. The flexural behaviours were compared experimentally and using finite element analysis (ABAQUS). 6 beam specimens were used having dimensions 750x150x150mm and were subjected to four-point bending test until failure. Reinforcement ratios of 1%, 1.4% and 2.1% were adopted using each type of the reinforcement bar. The ultimate load capacity and flexural strength of the steel RC beams was higher than GFRP RC beams, the failure modes experienced in both beam are generally shear failures but the GFRP RC beams exhibited more flexural cracks. The crack width of GFRP RC beams was higher than steel RC beams but is independent of the reinforcement ratios. The finite element analysis results closely agree with the experimental results.

Keywords: corrosion, glass fibre reinforced polymer (GFRP) bar, finite element analysis, reinforced concrete.

INTRODUCTION

Reinforced concrete is the most prevalent composite material used in construction in the world and particularly in North Cyprus [1]. Over the years the number of reinforced concrete structures has significantly rise from the year 1993 to 2016 according to data provided by State Planning Organization, (2016). This implies there is need for reconsideration in materials and methods of construction such as use of sustainable materials. Certain problems are associated with reinforced concrete structures in North Cyprus, some are; lack of quality control, inexperienced labourers, inadequate foundation isolation, inadequate vibration of concrete and lack of soil investigation [3]. These aforementioned problems in one way or the other results to cracks, honeycombing, segregation and subsequently corrosion of reinforcing bars. Corrosion is the most alarming factor that causes deterioration of RC structures over time preventing them from meeting their designed service life [4]. There has always been an interest for a material having both extreme strength and ductility. Strength gives a member the ability to carry load safely while ductility avoids sudden failure. Although steel has been the best option for years but corrosion exists as a setback to its overall performance [5]. Use of fibre reinforced polymer (FRP) bar as internal reinforcement in concrete elements is one of the preferred solution adopted around the world due to its positive results over the years [6].

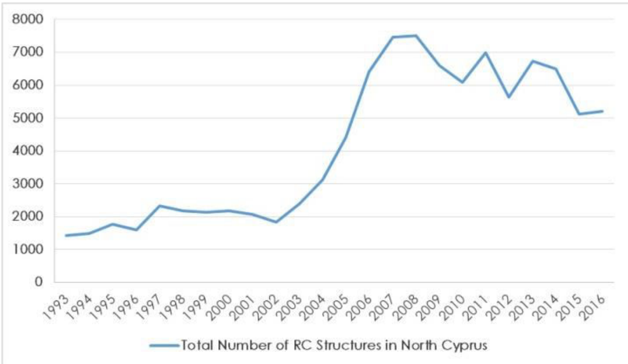

Figure-1. Number of RC structures in North Cyprus [2].

Cyprus is an island in the Eastern Mediterranean and has a lot of structures along the coastline and corrosion rate is high in coastal areas because of the elevated level of chlorides in the air, high relative humidity and high temperature [4], this calls for finding an alternative reinforcing material that can eradicate this menace.

Fibre reinforced polymer (FRP) bar particularly glass fibre reinforced polymer (GFRP) bar is regarded as an alternative material to steel as it is insusceptible to corrosion and chloride attack because it is a non-metallic material. It is manufactured from fibres and resin (Sonnenschein *et al*., 2016). It also has better tensile strength to weight ratio when compared to typical steel bar which is almost 3 times higher. Several researches have been carried out regarding the performance and durability of GFRP bar in RC members and they concluded that using GFRP bar is an excellent alternative to steel and most importantly if cost factor is involved [8].

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Figure-2. Damage of an old building due to corrosion.

Figure-3. Honey combing.

Figure-4. Segregation.

It can be seen in Figure-2, an old building that was damaged as a result of corrosion; cracks and spalling of concrete can be observed, this was as a result of aggregates used that where fetched from the seaside [3]. Figures 3 and 4 shows a structure under construction in Nicosia, North Cyprus, honeycombing and segregation problem can be observed which is due to inadequate vibration of concrete, and this easily leads to ingress of corrosive agents. Before measures were taken, quarries existed in Kumköy and Gaziveren, in Güzelyurt district, North Cyprus, which were very close to the sea shore. Aggregates for construction works where fetched from these quarries which is cheaper than crushing part of the mountains, these aggregates requires washing because of the salt deposits existing on them but that wasn't considered. These aggregates were fetched and used for construction until 1993 when the government closed down the quarries and regarded the aggregates harmful for construction [9].

Cyprus exists within the seismic zone but in the less active part when compared to neighbouring Greece and Turkey [10]. Figure-5 shows the seismic activity in Cyprus over the years, this is why emphasis should be given regarding seismic performance of RC structures. Deterioration of RC structures is accompanied by corrosion which affects the bonding between the steel bars and concrete thereby increasing its seismic fragility (Pitilakis *et al*., 2014). Numerous studies have been carried out to determine the seismic performance of corroded RC members, they concluded that loss of strength and ductility was experienced and sometimes results to severe level of damage (Kivell *et al*., 2012). It is also known from the equation of the lateral earthquake force and base shear, the total earthquake force and total base shear is directly proportional to the weight of the structure. We can relate this to weight of GFRP bar which is ¼ weight of steel bar (ACI 440.1R-15, 2015), which implies using GFRP bars as reinforcement is associated with less total earthquake force and total base shear.

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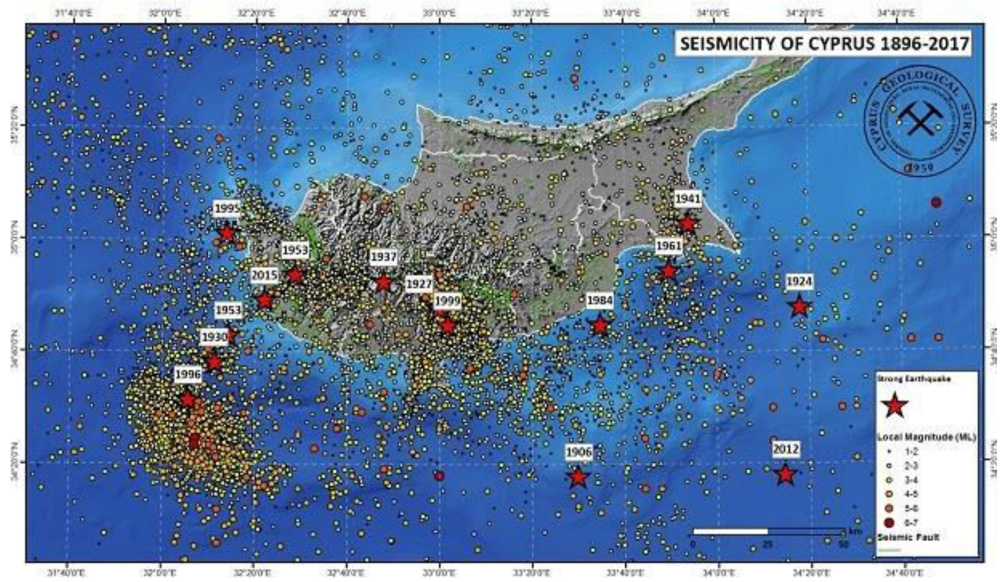

Figure-5. Seismicity of Cyprus 1896-2017 [14].

In general, due to the corrosion attack to steel reinforcement it was estimated that up to 15% of all bridges are deficient structurally [15]. In United Sates, it was estimated that an approximate amount of $8.3 billion is associated to annual direct cost of repair and maintenance of these structures [15]. In Canada, the average cost of repair and maintenance of reinforced concrete structures in a year amount to almost $74 billion and in Europe, this amount is estimated to be around $3 billion per annum (Balendran *et al*., 2002). This maintenance cost can be curbed by adopting the use of FRP bars since it is economically efficient [17].

MATERIALS AND METHODS

Concrete

Ready mixed concrete of C30 grade is used for conducting this research work. The compressive strength test was performed according to British Standards EN 12390-3 and the results are shown in Table-1.

Table-1. Compressive strength of concrete cube specimens.

Concrete Specimen	Mass (kg)	Compressive Strength (MPa)
C1	12810	37.21
C2	12730	32.55
C3	12790	33.44
Average	12776.7	34.4

Glass Fibre Reinforced Polymer (GFRP) Bar

The GFRP reinforcing bar used for this experimental work is part of the Liana composite products produced by Ural Reinforcing Company which is based in Russia. GFRP bars of diameters 8mm, 10mm and 12mm were used for the various parts other of the experimental study. Table-2 shows the tensile properties of the GFRP bars when compared with the properties of those from previous studies.

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Table-2. Mechanical properties of GFRP bars.

Reference	Diameter (mm)	Ultimate stress (MPa)	Modulus of elasticity (GPa)	% Elongation
Liana composite*	8	1250	55	2.27
	10			
	12			
[19]	15.9	744	40.6	1.77
(Özkal *et al.*, 2018)	9	918	49.9	1.84
[21]	8	1374	59.99	2.06
	12	1160	60.19	1.77
(Shin *et al.*, 2009)	13	690	41	1.68
[23]	12	924	42.57	2.17
(Balendran *et al.*, 2004)	8	1150	47	2.45

*As provided by the manufacturer

Pull-out Test Specimen

An apparatus was made with 10mm thick steel and in such a way it can fit into a universal testing machine. Reinforcement bars of 300mm long were used and the schematic diagram of the pull out specimens is shown in Figure-7.

Figure-6. Pull-out apparatus.

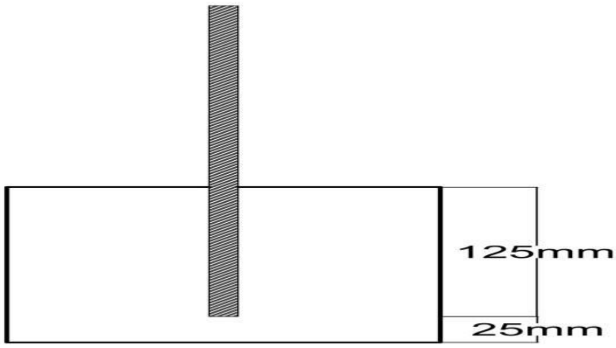

Figure-7. Diagram of pull-out specimen.

Figure-8. Pull-out test setup.

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Description and Geometry of Beam Specimens

The beams used have dimensions of 750x150x150mm. The specimens are prepared in two groups; group 1 beams reinforced with steel bars and group 2 beams reinforced with GFRP bars. The detail of the beam specimens is presented in Table-3 and the geometry is shown in Figures 9-11.

Table-3. Beam Specimen Details.

Group	Beam Specimen	Bar type	ρ (%)	Bottom reinforcement	Top reinforcement	Stirrups (steel)
1	G1-BM1	Steel	1	2φ10	1φ8	φ8 @ 100
	G1-BM2	Steel	1.4	2φ12	1φ8	φ8 @ 100
	G1-BM3	Steel	2.1	3φ12	1φ8	φ8 @ 100
2	G2-BM1	GFRP	1	2φ10	1φ8	φ8 @ 100
	G2-BM2	GFRP	1.4	2φ12	1φ8	φ8 @ 100
	G2-BM3	GFRP	2.1	3φ12	1φ8	φ8 @ 100

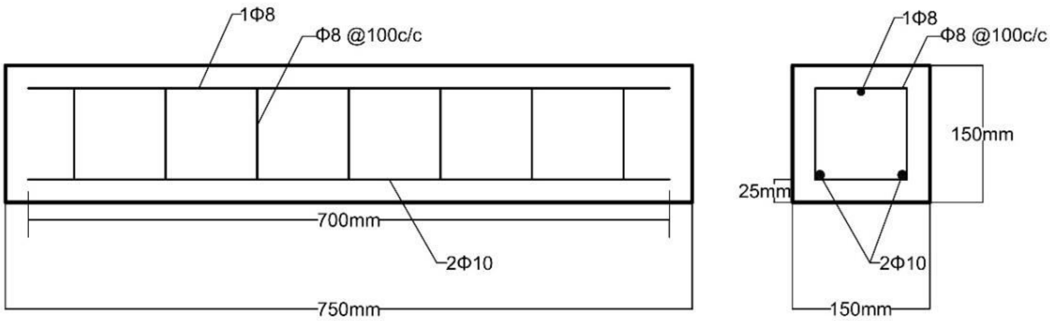

Figure-9. Details and dimension of Beam 1 for steel and GFRP.

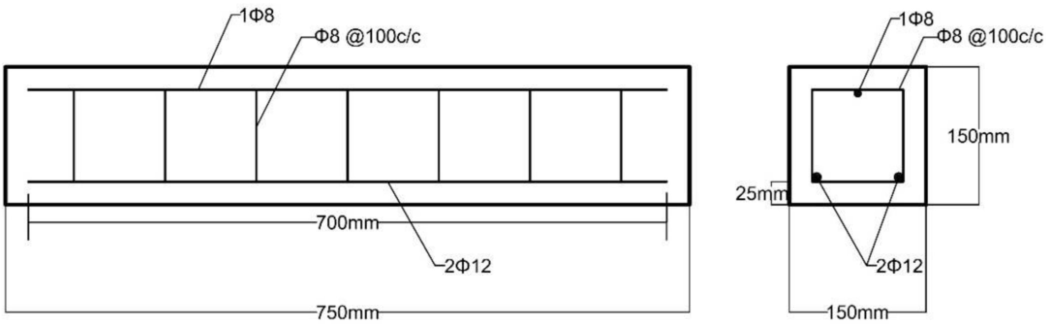

Figure-10. Details and dimensions of Beam 2 for steel and GFRP.

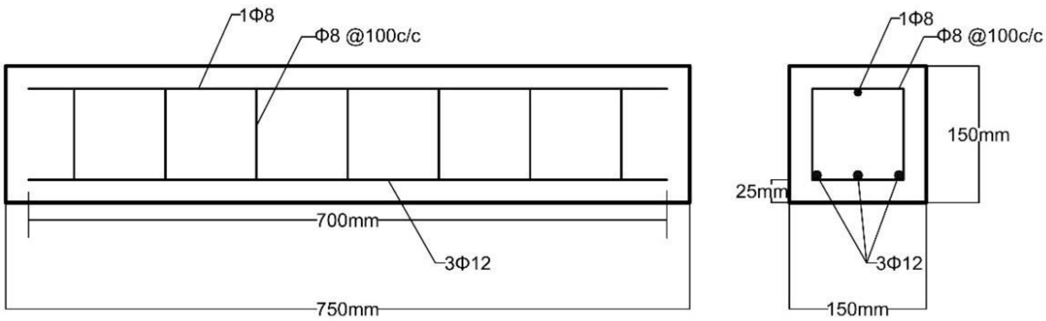

Figure-11. Details and dimensions of Beam 3 for steel and GFRP.

2.5 Four Point Bending Test

The four point bending test was performed in Chamber of Civil Engineers laboratory according to BS EN 12390-5, using automatic flexural testing machine (UTC-4550) with the supports positioned at 150mm from the both ends of the beams. The effective span of the beams was 450mm. the loading rollers were positioned at the top at 300mm from both ends of the beams. The loading setup and beam dimension is shown in Figure-12.

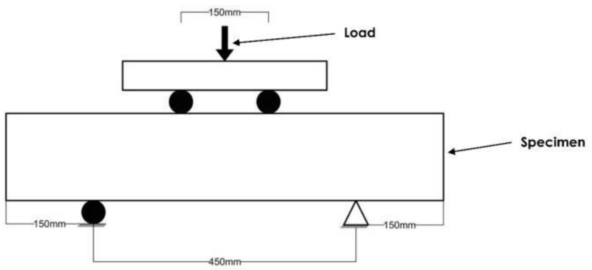

Figure-12. Diagram of Loading Arrangement of Beams in Flexural Testing Machine.

Figure-13. Beam flexural test setup.

The flexural testing machine has a loading capacity of 200kN. The load was applied until failure at a constant pace rate of 0.05MPa/s. Flexural strength of the beams can be computed using the equation below;

$$f_{cf} = \frac{P_u \times I}{d_1 \times d_2^2} \quad \text{(MPa)} \tag{1}$$

Testing Bonding Behaviour

The test was performed according to ASTM C234-91a and the ultimate bond strength was computed using equation below;

$$\tau_u = \frac{P_u}{\pi DL} \quad \text{(MPa)} \tag{2}$$

FINITE ELEMENT MODELLING

ABAQUS was used for the finite element analysis. All element to be used in the analysis are defined. The definition of the materials contains all the necessary material behaviours such as elastic material behaviour in a linear static stress analysis.

The beam was modelled as a 3D model of dimension 750x150x150mm with span length of 450mm between the supports. The reinforcement bars are assumed to be perfectly bonded into the concrete mass. Therefore, in fixing the reinforcement bars into the host element (concrete mass), the embedded option is selected. Sample of the 3D models can be seen in Figure-14.

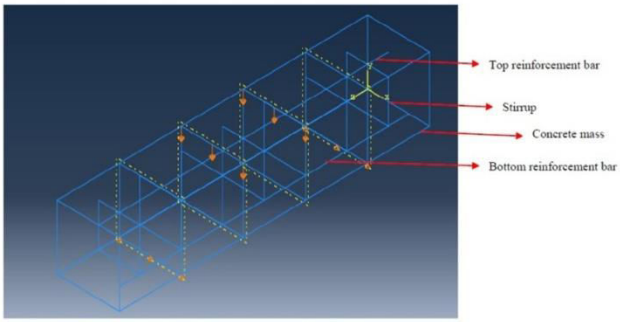

Figure-14. Geometry of Beam Model.

Before the analysis begins, the meshing of the model must be done which involves dividing the whole model into smaller parts prior to load application. Selection of appropriate mesh density is an important factor in determining the convergence of the results, the mesh density used can be seen in Figure-15.

Four point bending test was performed on the beams and load was applied on the model. The support was modelled as roller and pinned support just as in the experimental work and was defined in the boundary condition option.

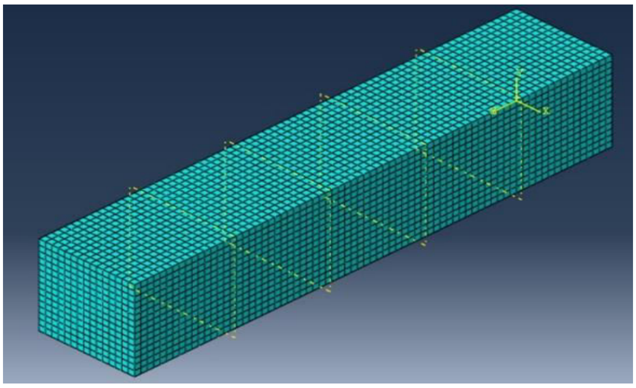

Figure-15. Meshing of the Model.

RESULTS AND DISCUSSIONS

Bond Behaviour

A total of 4 specimens were used for testing the bonding behaviour. Specimen S1 and S2 containing steel bars failed due to bar pull-out because of slipping of bar against the concrete, this is as a result of weak bonding. Specimen G1 and G2 containing GFRP bars failed to concrete splitting, this is because the bonding of GFRP and concrete is excellent. The average ultimate bonding strength of the G1 and G2 is 30% higher than that of S1

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and S2. Figure-16 shows a curve comparing the bond strength of the various specimens.

Table-4. Summary of Pull-out Test Results.

Specimen	Bar Diameter (mm)	Bar Type	Embedded Length (mm)	P_u (kN)	τ_u (MPa)	Failure Mode*
S1	10	Steel	125	22.1	5.6	BP
S2	10	Steel	125	22.3	5.7	BP
G1	10	GFRP	125	25.9	6.6	CS
G2	10	GFRP	125	31.7	8.1	CS

*BP - Bar pullout, CS - Concrete splitting

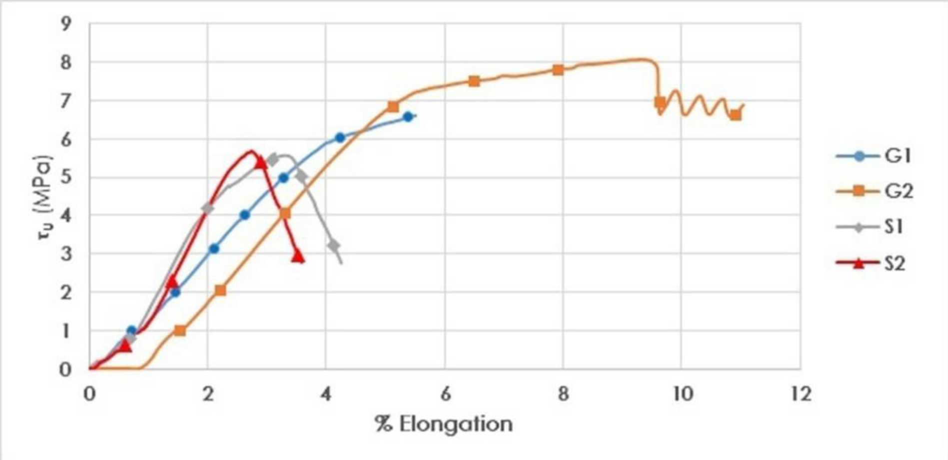

Figure-16. Bond strength of specimens.

Figure-17. S1 & S2 Specimen Failure.

Figure-18. G1 & G2 Specimen Failure.

Flexural Behaviour

The summary of the flexural behavior of the experimental and analytical results is presented in Table-4.

Ultimate Load Capacity and Flexural Strength

It can be seen that the ultimate load capacity and flexural strength of the group 1 beams is higher than that of group 2 beams in the experimental and FEA results. The experimental ultimate load capacity of G1-BM1 is

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17% more than G2-BM1 and 40.7% for the FEA result having reinforcement ratio of 1%, G1-BM2 is 5% more than G2-BM2 and 39.7% for the FEA result having reinforcement ratio of 1.4% and G1-BM3 is 33.6% more than G2-BM3 and 31.5% for the FEA result having reinforcement ratio of 2.1%.

Failure Modes

The failure mode of group 1 and group 2 beams are generally shear failure but group beams exhibited more flexural cracks, this is also similar to the failure modes experienced in the FEA results.

Crack Pattern and Crack Width

The group 1 beams exhibits cracks which initiates from the supporting points and propagates to the extreme compression zone but less flexural cracks are observed which is not the case in G1-BM1 that exhibit some wider cracks. The group 2 beams also exhibits cracks at supporting points which propagates to the compression zone but with more flexural cracks, this is due to brittleness nature of the GFRP bar. The maximum crack width was determined according to ACI 440.1R-15, it can be seen that maximum crack width for the group 2 beams is significantly higher than group 1 beams. It can be observed that the crack width of group 2 beams is independent on the reinforcement ratio of the beams but depends on type of reinforcing bar, the GFRP reinforced beams has higher crack width than steel reinforced beams.

Table-5. Summary of flexural tests result.

Beam Specimen	Bar type	ρ (%)	Initial Cracking load (kN)	w_{max} (mm)	w (mm)	P_u (kN)	FEA P_u (kN)	f_{cf} (MPa)	FEA f_{cf} (MPa)	Failure Mode*	FEA failure mode*
G1-BM1	Steel	1	55	0.3	4.5	103.64	89.34	13.8	11.9	DC	DC
G1-BM2	Steel	1.4	94	0.35	2	107.17	97.49	14.3	13	DT	DC
G1-BM3	Steel	2.1	134	0.25	0.7	152.87	106.57	20.4	14.2	DT	DC
G2-BM1	GFRP	1	35	2.5	2	88.5	63.48	11.8	8.5	FF	DC
G2-BM2	GFRP	1.4	51	2.4	3	102.06	69.68	13.6	9.3	DT	DC
G2-BM3	GFRP	2.1	63	1.6	4	114.48	78.86	15.3	10.5	DC	DC

* DC- Shear compression, DT- Diagonal tension, FF- Flexural failure

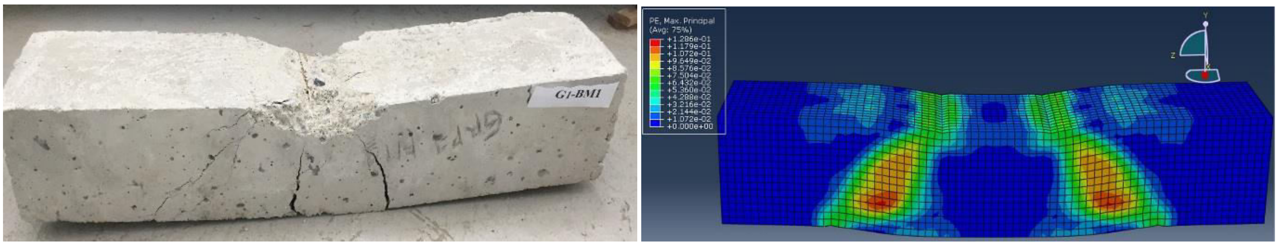

Figure-19. Failure mode in group 1 beams.

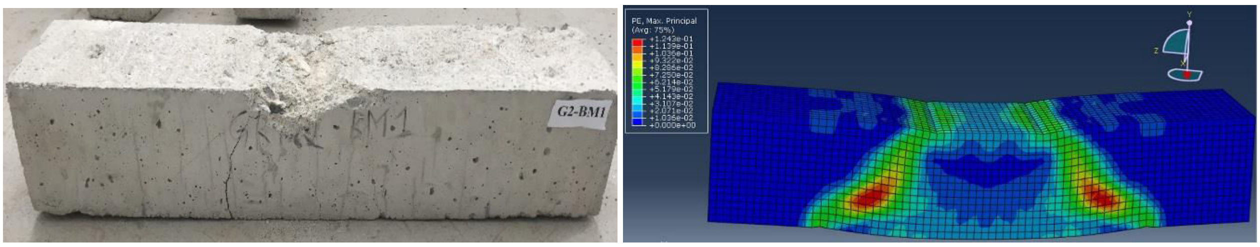

Figure-20. Failure mode in group 2 beams.

Cost comparison

The cost of steel reinforcement cages and GFRP reinforcement cages is presented in Table-6. It shows that the GFRP reinforcement cages cost more than the steel reinforcement cages but the steel reinforced beams where able to resist more loads than the GFRP reinforced beams. The reinforcement cage of G2-BM1 cost 11% more than G1-BM1, G2-BM2 costs 13% more than G1-BM2 and G2-BM3 costs 14% more than G1-BM3. But this difference in cost can be covered if the ease of transport and handling, its non-corrosive nature is considered. This means labour cost can be reduced and the high

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maintenance cost due to corrosion problem can be avoided.

Table-6. Total cost of reinforcement cages of specimens.

Specimen		Bottom Reinforcement	Top Reinforcement	Stirrups (steel)	Total ($)
G1-BM1	Diameter	Φ10	Φ8	Φ8	1.45
	Length (m)	1.4	0.7	3.5	
	Price ($)	0.497	0.159	0.795	
G1-BM2	Diameter	Φ12	Φ8	Φ8	1.67
	Length (m)	1.4	0.7	3.5	
	Price ($)	0.714	0.159	0.795	
G1-BM3	Diameter	Φ12	Φ8	Φ8	2.03
	Length (m)	2.1	0.7	3.5	
	Price ($)	1.071	0.159	0.795	
G2-BM1	Diameter	Φ10	Φ8	Φ8	1.66
	Length (m)	1.4	0.7	3.5	
	Price ($)	0.644	0.224	0.795	
G2-BM2	Diameter	Φ12	Φ8	Φ8	1.86
	Length (m)	1.4	0.7	3.5	
	Price ($)	0.84	0.224	0.795	
G2-BM3	Diameter	Φ12	Φ8	Φ8	2.28
	Length (m)	2.1	0.7	3.5	
	Price ($)	1.26	0.224	0.795	

CONCLUSIONS

The following conclusions are retrieved from this research work;

a) **Tensile behaviour;** the tensile strength of glass fibre reinforced polymer (GFRP) bar is significantly higher than that of steel bar. The average tensile strength of GFRP bar is about 65% higher than that of steel bar.

b) **Bond behaviour;** the ultimate bond strength of specimen G1 and G2 containing GFRP bar is significantly higher than specimen S1 and S2 containing steel bars. The failure mode of specimen S1 and S2 is bar pull-out which is due to weak bonding because of slippage between the steel bars and the concrete. But specimen G1 and G2 failed due to concrete splitting which shows that GFRP bar bonds well with concrete. The average ultimate bond strength of the GFRP bar embedded in concrete is 30% higher than that of the steel bar.

c) **Flexural behaviour;** the beams were loaded until failure and it can be seen that the ultimate load and flexural strengths of group 1 beams is higher than group 2 beams. The ultimate load capacity of the G1-BM1 is 17% higher than G1-BM2, G1-BM2 is 5% higher than G2-BM2 and G1-BM3 is 33.6% higher than G2-BM3. Similarly, the flexural strength of group 1 beams is higher than group 2 beams. This clearly shows that the GFRP reinforced beams are able to withstand a great amount of loads so it is advised for them to be overly reinforced. Comparing the experimental and FEA result also shows that the group 1 beams have higher ultimate load capacity and flexural strength than group 2 beams. Generally, it can be seen that higher reinforcement ratio means higher stiffness of the beams. Also, the FEA results can be seen to closely agree with the experimental results

d) **Failure mode;** experimentally the group 1 beams generally exhibited shear failure while the group beams also exhibited shear failures but with a lot of flexural cracks, this mode of failures closely agrees with the FEA results which is also shear failures.

e) **Crack pattern and crack width;** due to the brittleness nature of GFRP bar, the group 2 beams exhibited more cracks and higher crack width than group 1 beam. The computed maximum crack width based on the equation provided by ACI 440.1R shows that crack width of group 2 beams is also higher than that of group1 beams. It can be observed that as the reinforcement ratio of group 1 beams increase the crack width decreases but the crack width of group 2 beams seems to be independent on the reinforcement ratios.

f) **Cost comparison;** the initial cost of GFRP bar is higher than steel bar, so does the cost of the reinforcement cages. The price increase is between 11 - 14% but this difference can be overlooked when you involve the ease of transport and handling due to its light weight and also, its non-corrosive nature that will help in curbing the high maintenance cost due to corrosion problem. This makes it great alternative material to conventional steel bar.

In general, it can be observed that GFRP can be adopted as reinforcement material because it could bonds well with concrete and resist a considerable about of load with the benefits of being lightweight and non-corrosive.

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