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Flexural Behaviour of Concrete Beams Reinforced With GFRP Rebars

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Abstract:** This study reports test results of 12 concrete beams measuring 150mm wide × 180mm deep× 1200mm long reinforced with glass fiber-reinforced polymer (GFRP) bars subjected to a four-point loading system. The test specimens were classified into three groups according to the concrete compressive strength. The main variation done for each beam in all the three groups was a percentage of reinforcement (0.5%, 1%, 1.5% and 2%). Since all the beams were over reinforced failure occurred due to rupture of concrete at compression zone. The failure is initiated by a vertical crack at the midspan which extended up to compression zone of the beam and propagated horizontally which leads to bond failure between top concrete and compression reinforcement. The test results revealed that the crack widths and mid-span deflection significantly reduced by increasing the reinforcement ratio. The ultimate load increased by 7.5%, 16.8%, 27.7% as the reinforcement percentage increased from 0.5% to 1%, 1.55 and 2% respectively. The flexural provisions of structural design guidelines namely ACI 440.1R-06, ECP 208-2005, and CSA S806-12 were evaluated against the test data. ACI 440.1R-06 overestimates the moment resistance of GFRP bars as compared to other codes and experimental results. Whereas all the design guidelines predict nearly the same values for deflection. And for crack width approximation Toutanji's equation is more accurate compared to ACI equation.

***Keywords:** GFRP Rebar, Flexural Behaviour, Reinforcement Ratio*.

1. INTRODUCTION

1.1 General

One of the biggest challenges engineers today facing is the problem of ageing infrastructure, particularly with respect to reinforced cement concrete. The major cause of deterioration of reinforced concrete structures is corrosion of the reinforcing steel. Among others, one viable option is to reinforce concrete with rebars of glass fiber-reinforced polymer (GFRP), a noncorrosive material. GFRP reinforcing bars are made primarily of glass fibers. Being non-corrosive, GFRP bars can help extend the lifecycle of reinforced concrete structures substantially, as well as reduce their maintenance, repair, and replacement costs. While GFRP is becoming a viable reinforcement alternative, it presents design challenges which are different than those in the design of conventional steel reinforced concrete. One important challenge is consideration of a brittle failure mode in GFRP-reinforced members.

Other than the brittle failure mode, the major shortcoming of GFRP reinforcing bars is their relatively low stiffness, when compared to steel. This reduced stiffness, combined with other factors such as different bond behaviour and lower tension stiffening, results in deflections that are larger than conventional steel-reinforced beams, at any load stage. Because of these large deflections, designs may often be governed by deflection limitations. As such, it is critical that load-deflection behaviour can be accurately predicted.

1.2 Objectives

The fundamental objective of the present study is to investigate the flexural behaviour of GFRP reinforced concrete beams, with a focus on critically evaluating current design code provisions relating to design with GFRP. The design codes discussed are CSA S806-12 (Canadian standards for Design and Construction of Building Structures with Fiber-Reinforced Polymers), ECP 208-2005(Egyptian guidelines for the use of FRP in the civil construction**)**, and ACI 440.1R-06 (American standards for the Design and Construction of Structural Concrete Reinforced with FRP Bars).

2 EXPERIMENTAL WORK

The current research program was carried out to investigate the flexural behavior of concrete beams reinforced with longitudinal GFRP bars (top and bottom). The effects of both concrete grade and the increase in reinforcement ratio on the flexural behavior of the tested beams were studied.

2.1 Test specimen and setup

The test program consists of 12 beams split into three main series each of four beams cast with M50, M60, and M70 grades of concrete respectively. The main variation is a percentage of reinforcement ratio. Four different reinforcement ratios namely 0.5%, 1.0%, 1.5%, and 2.0% are used to access the flexure behaviour of test specimens strengthened with GFRP bars. The test specimens are named in a series A-B in which first letter A represents its grade of concrete and the second letter denotes its percentage of reinforcement. For example, 50-0.55 in which 50 represents the grade of concrete used in that beam is M50, and it is reinforced with GFRP bars equal to 0.5% of its cross section. The schematic representations of longitudinal as well as transverse reinforcement are shown in Figure 1 and Figure 2.

The flexural test setup of GFRP-RC beams under two point loads consisted of placing the specimens between two simple supports with an effective span of 1060mm. Two point loads were applied at a distance 367mm from both the supports. An upper spreader beam was placed above the point loads to equally distribute the concentrated load from the machine. During testing, cracks have been marked, deflections are measured, and the corresponding loads were noted to observe the behaviour of the beam at various load increments. Sequence, pattern, and width of the cracks until failure were also examined. The loading configuration for flexure test has been shown in Figure 3;

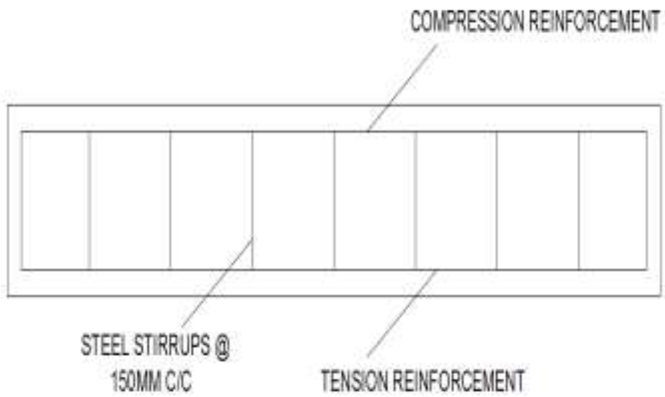


Figure 1: Detailing of longitudinal and transverse reinforcement

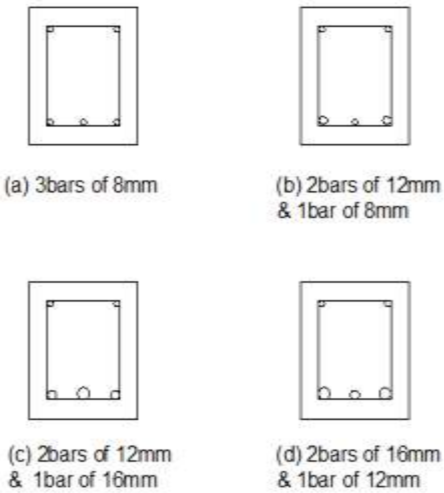


Figure 2: Cross sections with varying percentage of reinforcement

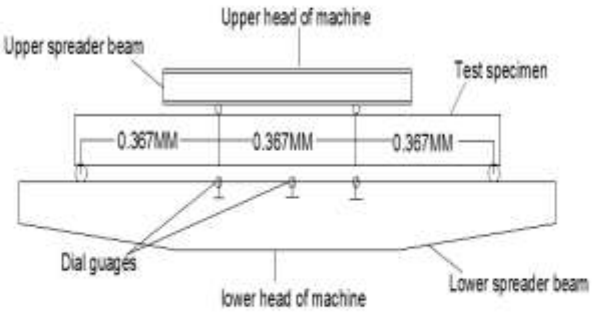


Figure 3: Loading configuration for Flexure test

2.2 Test results

This section presents the experimental results from the twelve specimens and is split into two main sections, based on the grade of concrete and reinforcement ratio. The section based on the grade of concrete discusses and compares the results between three different grades and the section based on reinforcement ratio deals with the results of beams with different reinforcement ratios.

Table 1: Results of test specimens reinforced with GFRP bars

Specimen	f_{ck}*(MPa)*	*ρ(%)*	M_{exp}	Δ_{exp}	W_{exp}
1	50	0.5	92	24.5	1.13
2	50	1.0	97	21.3	0.72
3	50	1.5	101	19.6	0.64
4	50	2.0	110	15.8	0.52
5	60	0.5	105	26.4	1.65
6	60	1.0	117	22.2	0.88
7	60	1.5	124	18.7	0.55
8	60	2.0	131	16.1	0.45
9	70	0.5	119	25.6	1.79
10	70	1.0	128	21.2	0.96
11	70	1.5	139	17.2	0.6
12	70	2.0	152	14.6	0.5

Ultimate load for beams of M50 grade concrete reinforced with 0.5%, 1.0%, 1.5%, 2% are 92kn, 97kN, 101kn and 110kN respectively. And deflections in beams were 24.5mm, 21.3mm, 19.6mm and 15.8mm for varying percentages of GFRP reinforcement. And also crack widths reduced by 36.28%, 43.36% and 54% reinforcement ratio increased from 0.5% to 1%, 1.5% and 25 respectively.

For M60 grade concrete as the reinforcement ratio increased from 0.5% to 1.0%, 1.5% and 2.0% the ultimate load increased by 11.4%, 18.1%, 24.8% and the deflections reduced by 15.91%, 29.17%, 39% respectively. And there was a tremendous reduction in crack widths i.e crack widths reduced by 46.67%, 66.67% and 72.72% as the percentage of GFRP reinforcement is increased. Compared to M50 and M60 grade concretes M70 concrete exhibited greater load carrying capacity as well as lower deformations. Crack widths reduced by 46.37%, 66.5% and 72.06% for an increment in reinforcement ratio from 0.5% to 1.0%, 1.5%, 2.0% respectively.

3. RESULTS AND DISCUSSION

The following discussion focuses on comparing the experimental results presented in Chapter 5 with the behaviour predicted by using current design codes and guidelines. The documents discussed are design codes CSA S806-12 and ECP 208-2005, and the report ACI 440.1R-06 with the focus being placed on the moment resistance, deflection, and cracking behaviour.

3.1 Moment resistance

ACI 440.1R-06 Provisions: As per ACI 440.1R-06 design guidelines the moment resistance capacities of the beams are calculated using the following equation and the results are tabulated in table 6.1.

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

ECP 208-2005 Provisions: According to design guidelines of ECP 208-2005 the moment resistance of the beams reinforced with varying percentages of GFRP rebars are computed using the equation given below. The results of moment resistance are presented in table 6.1.

$$M_u = \left(\frac{A_f\, f_{fe}^{\,*}}{\gamma_f}\right)\left(d - \frac{a}{2}\right)$$

CSA S806-12 Provisions: According to CSA S806-12 the ultimate limit moment for an over reinforced beam can be calculated using the following equation with top concrete strain constant which is equal to ($\varepsilon_c = -3.5\times10^{-3}$). And the results are tabulated in table 6.1.

$$M_r = T\left(d - \frac{\beta_1 c}{2}\right) = A_f\, f_f\left(d - \frac{\beta_1 c}{2}\right)$$

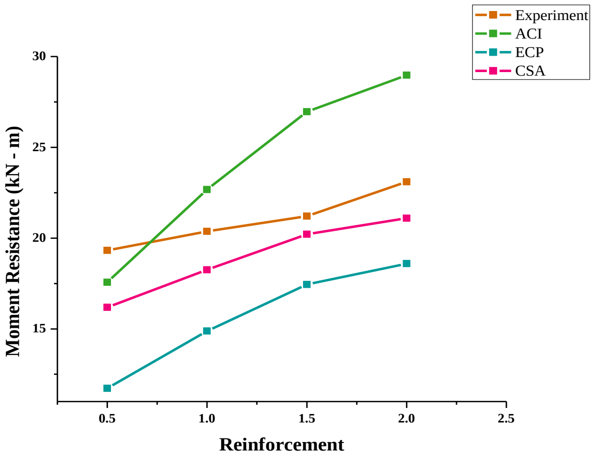


Table 2 compares the experimentally obtained results with predicted results by considering various design guidelines. The above figures indicate that For the higher volume of reinforcement the ECP 208-2005 equation and CSA S806-12 equation both perform well for moment resistance capacity of the beams strengthened with GFRP bars.

In all the three cases ACI 440.1R-06 provisions have overestimated the moment resistance capacity of GFRP reinforced beams as compared to other two design guidelines. For the same elastic properties, cross sections, and percentage of reinforcement ACI guidelines predicts higher results than experimentally obtained results. Whereas CSA and ECP guidelines predicts lower values of moment resistance capacity than the experimentally obtained results.

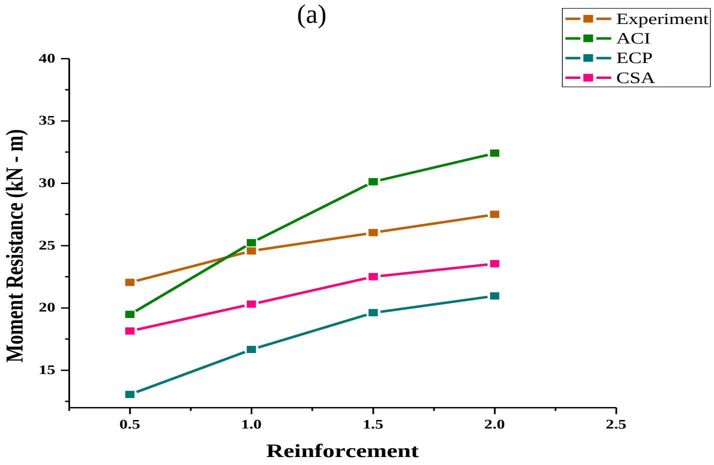
The experimentally obtained results for beams strengthened with GFRP rebars well agree with the results of other authors. This indicates that ACI guidelines have overestimated the strength of GFRP rebars and on the other hand ECP and CSA have underestimated the strength of GFRP rebars.

Table 2: Moment Resistance of Test Specimens.

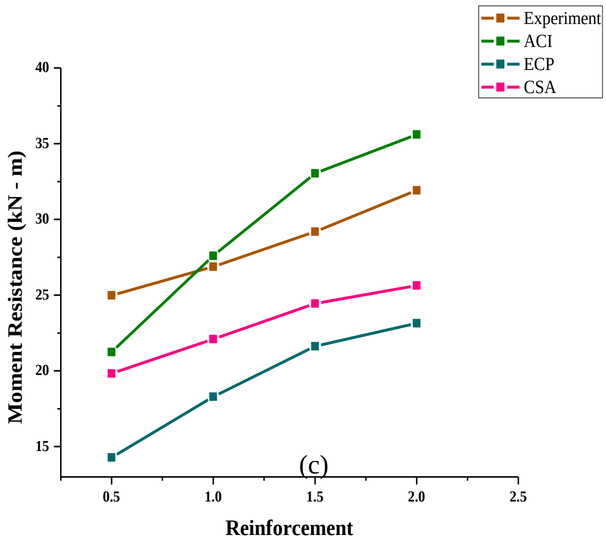
Specimen	M_{EXP}	M_{ACI}	M_{ECP}	M_{CSA}
M50-0.5	19.32	17.57	11.73	16.19
M50-1.0	20.37	22.67	14.88	18.25
M50-1.5	21.21	26.96	17.44	20.21
M50-2.0	23.10	28.97	18.60	21.10
M60-0.5	22.05	19.48	13.06	18.14
M60-1.0	24.57	25.23	16.66	20.31
M60-1.5	26.04	30.12	19.61	22.50
M60-2.0	27.51	32.42	20.96	23.55
M70-0.5	24.99	21.24	14.29	19.83

M70-1.0	26.88	27.60	18.30	22.10
M70-1.5	29.19	33.04	21.62	24.45
M70-2.0	31.92	35.61	23.15	25.63

(a)



(b)

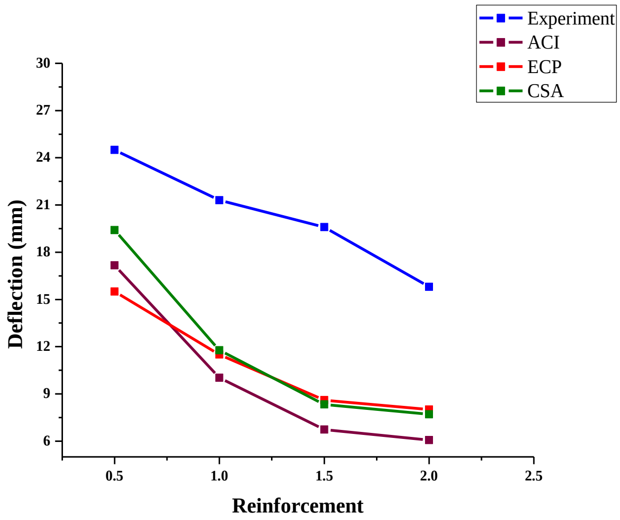


(c)

Figure 3: Comparison between experimental and predicted results of moment resistance for a) M50, b) M60 and c) M70 grade concrete reinforced with GFRP bars

3.2 Deflection

Beams with more reinforcement than balanced ($\rho_f > \rho_{fb}$) are defined as over reinforced and fail by crushing of concrete at the top surface. Because the majority of beams designed in practice are over-reinforced, all the twelve beams presented here were designed in such a manner. The design process for an over-reinforced beam involves supplying more reinforcement than that of calculated.



ACI 440.1R-06 Provisions: As per ACI 440.1R-06 design guidelines the midspan deflection of the beams are calculated using the following equation and the results for all the beams are tabulated in table 6.2.

$$\Delta = \frac{Pa}{48E_c I_e}\left(3L^2 - 4a^2\right)$$

ECP 208-2005 Provisions: According to design guidelines of ECP 208-2005 the midspan deflections of the beams reinforced with varying percentages of GFRP rebars are computed using the equation given below. The results of deflection are presented in table 6.2.

$$\Delta = \frac{PL^3}{48E_c I_e}$$

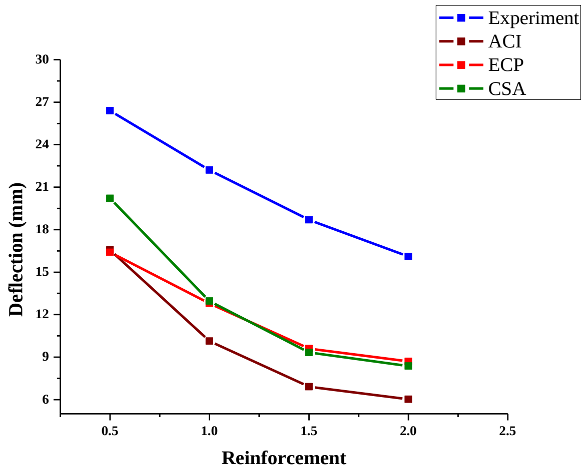
CSA S806-12 Provisions: Deflection procedure according to CSA S806-12 states that the deflection shall be calculated by formulae based on the integration of curvature at sections along the span. The equation applicable to the test setup adopted in this experimental investigation is given below. And the results for the same are presented Table 6.2.

$$\Delta = \frac{PL^3}{48E_c I_{cr}}\left[3\left(\frac{a}{L}\right) - 4\left(\frac{a}{L}\right)^3 - 8\eta\left(\frac{L_g}{L}\right)^3\right]$$

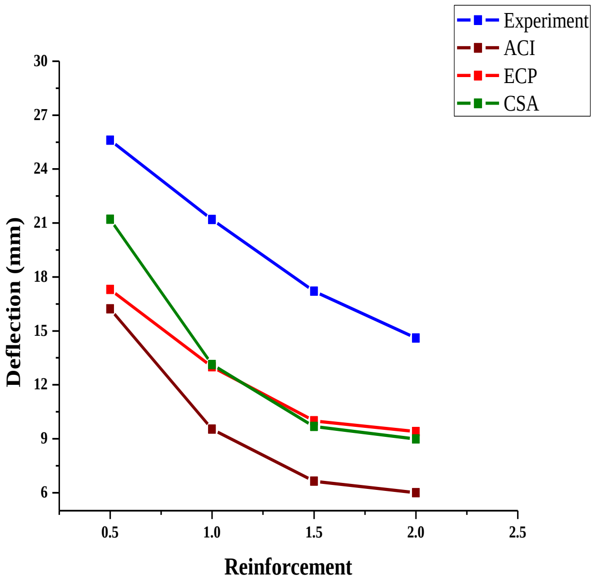
Table 3: Mid-span deflections of test specimens.

Specimen	Δ_{EXP}	Δ_{ACI}	Δ_{ECP}	Δ_{CSA}
M50-0.5	24.50	17.16	15.50	19.40
M50-1.0	21.30	10.02	11.50	11.77
M50-1.5	19.60	6.74	8.60	8.33
M50-2.0	15.80	6.07	8.00	7.70
M60-0.5	26.40	16.56	16.40	20.21
M60-1.0	22.20	10.13	12.80	12.96
M60-1.5	18.70	6.91	9.60	9.33
M60-2.0	16.10	6.03	8.70	8.37
M70-0.5	25.60	16.23	17.30	21.21
M70-1.0	21.20	9.53	13.00	13.13
M70-1.5	17.20	6.64	10.00	9.69
M70-2.0	14.60	6.00	9.40	8.99

(a)



(b)



(c)

Figure 4: Comparison between experimental and predicted results of midspan deflection for a) M50, b) M60 and c) M70 grade concrete reinforced with GFRP bars

Table 3 shows that, for all twelve beams, the current code provisions under predict ultimate midspan deflection. Underprediction of deflection is undesirable, particularly at service levels. The cause of this discrepancy can be seen in Figure 4(a), Figure 4(b) and Figure 4(c), which compare the predicted midspan deflection behaviour to the experimental behaviour of beams of different grades of concrete strengthened with different percentages of GFRP bars. The above figures indicate that, beyond cracking, the three methods do not provide significantly different results. The figure also shows that the major cause of the discrepancy between the predicted and experimental results can be attributed to an underprediction of cracked stiffness (or the difference in slopes post-cracking). The results above indicate that the described deflection prediction methods do not accurately predict ultimate deflection for this test series. All the three methods predict an ultimate deflection of approximately 60% of the experimental results.

3.3 Crack widths

ACI Code Equations: As per ACI 440.1R-06 provisions, the maximum crack width at any given load can be computed using the following equation. And the results for all the beams are presented in table 6.3.

$$w = 2\frac{f_f}{E_f}\beta k_b\sqrt{d^{2} + \left(\frac{s}{2}\right)^{2}}$$

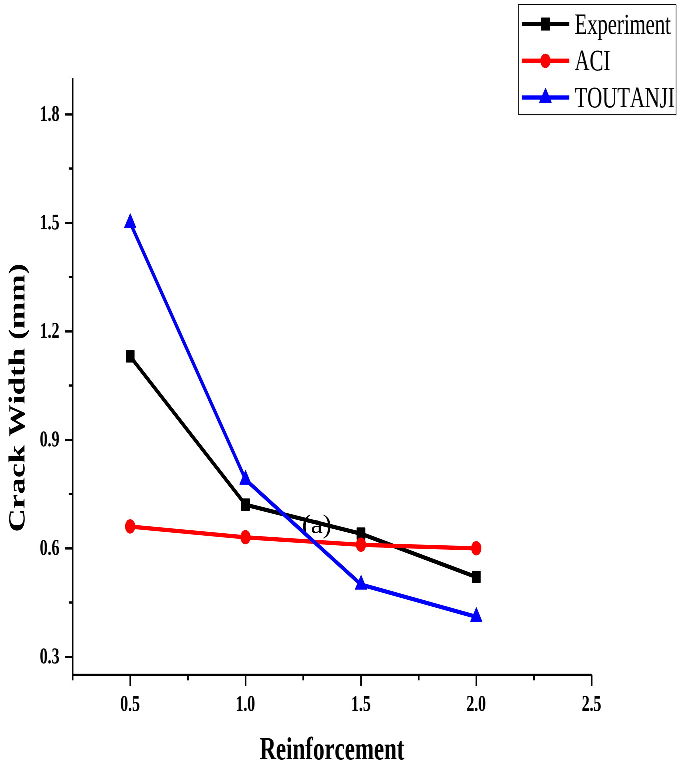
Toutanji's Equation: For predicting the maximum crack widths in GFRP-RC beams Tountanji recommended the following equation. And the crack widths for test beams are tabulated in table 4.

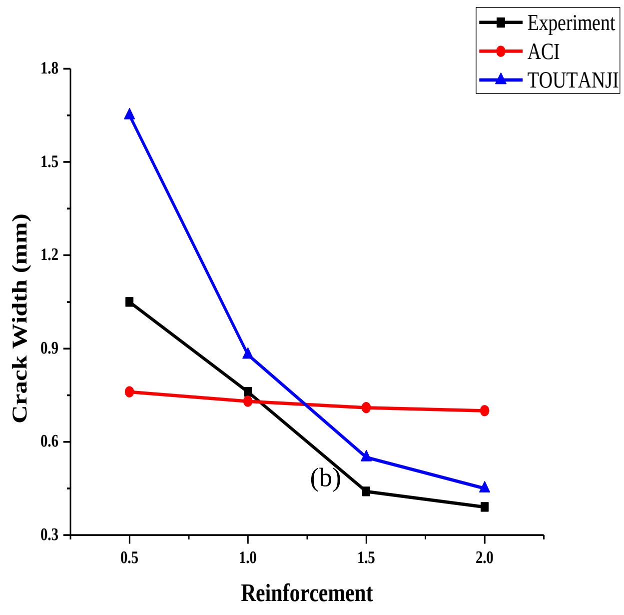
$$w = 0.2\frac{f_f}{E_f}\beta(\rho_f)^{-0.5}\sqrt[3]{d_c A_{ct}}$$

Figure 5(a) and Figure 5(b) indicates that for beams that are over-reinforced Toutanji's model predicts maximum crack width more accurately than the current ACI code provisions. The Tountanji's model is slightly more accurate than ACI code for over-reinforced beams in this test series. This analysis does not take into account long-term effects such as creep, which may be significant in GFRP reinforced beams. Further research is needed in the area of cracks in order to fully understand the crack development.

Table 4: Crack widths of test specimens

Specimen	W_{EXP}	W_{ACI}	$W_{TOUTANJI}$
M50-0.5	1.13	0.66	1.50
M50-1.0	0.72	0.63	0.79
M50-1.5	0.64	0.61	0.50
M50-2.0	0.52	0.60	0.41
M60-0.5	1.05	0.76	1.65
M60-1.0	0.76	0.73	0.88
M60-1.5	0.44	0.71	0.55
M60-2.0	0.39	0.70	0.45
M70-0.5	1.08	0.86	1.79
M70-1.0	0.78	0.82	0.96
M70-1.5	0.47	0.80	0.60
M70-2.0	0.46	0.79	0.50





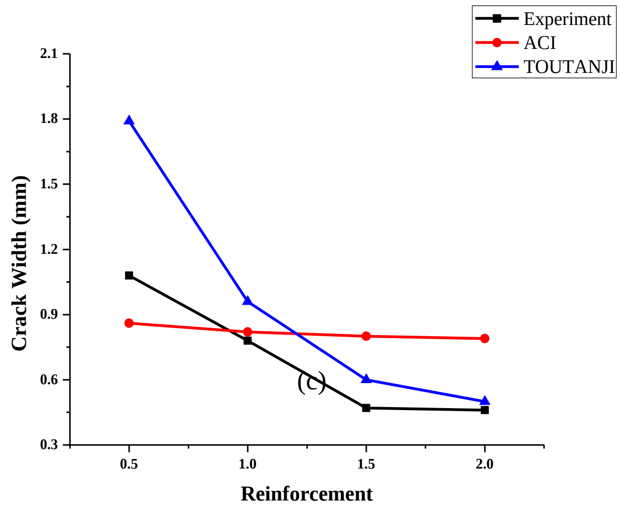


Figure 4: Comparison between experimental and predicted results of crack widths for a) M50, b) M60 and c) M70 grade concrete reinforced with GFRP bars

CONCLUSIONS

The following conclusions have been drawn from the current study.

At the earliest stages of research into glass-fiber reinforced polymer reinforcing bars, it was recognized that this material has a large potential for use in concrete structures, to be suitable as a reinforcing material, design equations must account for such properties as low stiffness and brittle failure mode.

As early as 1970's, it was understood that the flexural behaviour of GFRP reinforced concrete beams could be predicted using equations developed for steel reinforced beams, particularly with respect to moment resistance. Since that era, both the mechanical properties of GFRP bars and the understanding of their behaviour have significantly improved.

It is recognized, for instance, that the use of GFRP bars results in significantly less tension stiffening effects than steel bars. Due to this, and other phenomena, modifications to the steel-reinforced concrete equations must be made in order to accurately predict the unique behaviour of GFRP-reinforced concrete in flexure.

Increment in percentage of reinforcement ratio from 0.5% to 1%, 1.5%, 2% causes reduction if crack widths by 46.67%, 66.67% and 72.72% and deflection by 15.91%, 29.17%, 39% respectively.

Both increasing grades of concrete and percentage of reinforcement tend to increase the moment resistance of beans reinforced with GFRP bars.

From the current study, it can be concluded that the ACI 440.1R-06 provisions overestimate the moment resistance compared to ECP 208-2005, CSA S806-12 as well as experimentally obtained results.

ECP 208-2005 underssssestimates both moment resistance and deflection behaviour of GFRP reinforced beams compared experimentally obtained results and other provisions.

The Tountanji's model is slightly more accurate than ACI code for over-reinforced beams in this test series. This analysis does not take into account long-term effects such as creep, which may be significant in GFRP reinforced beams.

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