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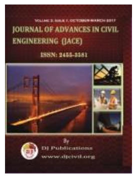

RESEARCH ARTICLE

Empirical Formulation for Prediction of Flexural Strength of Reinforced Concrete Composite Beams

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ABSTRACT

The guidelines of ACI 440.2R – 08, are based on the knowledge gained from experimental research, analytical work and field applications of Fiber Reinforced Polymer (FRP) systems used to strengthen concrete structures. ACI 440.2R – 08 declare that the design procedures have not, in many cases, been thoroughly developed and still require research and this research remains on going. This paper investigates the behavior of Reinforced Cement Concrete (RCC) beams strengthened externally by Glass Fiber Reinforced Polymer (GFRP) strips under flexure. The experimental investigation suggests that cross-sectional area of GFRP directly affects the moment capacity of RCC-GFRP composite beams. Based on the results, a new empirical model is proposed to predict the moment capacity of composite beams especially with high GFRP cross-sectional area.

Keywords: Composite beams, Flexure tests, GFRP, Reinforced Concrete (RCC), Strengthening.

1. INTRODUCTION

The GFRP strengthening or retrofitting of existing RCC beams show that retrofitted beams support higher loads. Externally bonded steel plates, steel or concrete jackets and external post-tensioning are just some of the many traditional techniques available [1] considerably for higher GFRP area initially but doesn't increase. It was found that the actual moment capacity found out experimentally for higher GFRP area was lesser as compared to the moment capacity calculated as per ACI 440.2R-08.code.The present paper focuses on this issue and proposes a new empirical model to predict the moment capacity of RCC beams strengthened with GFRP strips. I0n Europe, developed FRP systems as alternative to steel plate bonding. Bonding steel plates to the tension zones of concrete members with adhesive resins proved to be a good alternative technique for increasing flexural strength. This technique has been used to strengthen many bridges and buildings around the world. [2] investigated behaviour of RCC beams strengthened by FRP. Experimental and analytical investigation of the behavior of RCC beams strengthened in flexure by means of different combinations of externally bonded hybrid GFRP/CFRP sheets, the predicted results by ACI-2R-08, and experimental results from the tests were compared and showed good agreement [3]. [4] investigated many parameters like compressive strength of concrete, modulus of elasticity of FRP, tensile strength of FRP, effective depth of concrete and width of FRP that affect the capacity of reinforced concrete beams strengthened with FRP strips. Due to the use of FRP, the moment capacity of reinforced concrete beams increased. [5] investigated the flexural strength of reinforced concrete beams in case of low strength and strengthened by CFRP plate. The parameters taking into account was main reinforcement ratio and pre- load level. All tested beams were under four points loading. The experimental and analytical results showed that the flexural capacity and stiffness of strengthened and repaired beams using CFRP laminate were increased compared to

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those of control beams and the ductility of strengthened beams was greatly reduced compared to the control. [6] looked out at the flexural behavior of RC beams strengthened with GFRP and analyzed the different kinds of failure, ultimate moment capacity, deflection, load of first crack and tensile strength of composite beams. Result from tests compared with ACI 440 showed that deflections, width of cracks and the cracks' extent are further used toward the usual RC beams. [7, 11] explored the flexural behavior of RC beams strengthened by GFRP. Different sizes of GFRP laminates were used to achieve flexural strength and load carrying capacity of RC beams by applying two point bending flexural tests conducted up to failure. In the present paper flexural strength of RCC beams strengthened by GFRP is investigated experimentally. Two types of cement with two different characteristic compressive strength and varying width and number of layers of GFRP strips were used for this study. The results of flexural strength from analytical calculations using ACI 440.2R – 08 are compared with experimental results and further an empirical model is developed to predict the moment capacity of RCC beams strengthened with GFRP strips.

2. EXPERIMENTAL WORK

A series of RCC beams strengthened with GFRP strips (RCC-GFRP composite beams) along with additional control RCC beams (without GFRP), were tested in this experimental study to find out flexural capacity.

3. VARIABLES IN EXPERIMENTAL STUDY

In the present study three widths of GFRP strips with single and double layer and two grades of concrete were used as variables. Two types of cements were also used to make the empirical model generic. The effect of these variables on capacity and flexural behaviour was studied. GFRP strips were bonded to RCC beams using epoxy resin. GFRP strip widths used were of 25, 50 and 100mm having thickness of 0.34mm.

4. MATERIAL PROPERTIES

4.1. Steel reinforcement

HYSD bars conforming to IS 1786 - 2008 (Fe500), were used as main enforcement and mild steel bars conforming to IS 432-1966 were used as stirrups (Fe 250).

4.2. Cement

Two types of cement namely 43 Grade Ordinary Portland Cement (OPC) conforming to IS 8112-1998 and Portland Slag Cement (PSC) conforming to IS 455-1989, was used.

4.3. Concrete

Concrete mix was designed according to BS5282 and IS10262:2009. The mix proportions with (W/C) ratios of 0.405 and 0.37 were used for two grades of concrete M35 and M45 respectively.

4.4. GFRP strips

GFRP strip conforming to ACI-440.2R-2008 [1] were used having modulus of elasticity 75900MPa, tensile strength 875MPa.

4.5. Epoxy

Epoxy resin of make Gold-bond® 1893 was used as bonding between concrete and GFRP having compressive strength of 550 kg/cm^2.

4.6. Details of specimens

RCC beams of cross section 150mm x 150mm x length 700mm were used for the experimental work. RCC beams were cast using two bars of 8mm diameter as tension reinforcement and 6 mm diameter stirrups at 80mm spacing. Schematic details of RCC-GFRP composite beam specimens are given in figure 1.

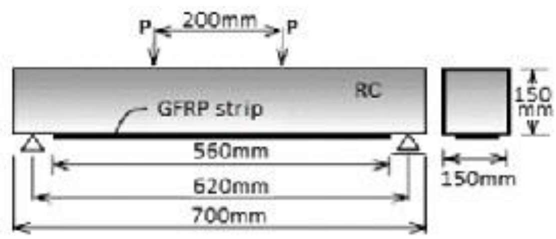

Figure 1.Details of the RCC-GFRP composite beam test specimens.

GFRP strips were bonded to the reinforced concrete beams in the middle at 580 mm length using epoxy resin, centrally placed along the span. GFRP strips were bonded to the beam-soffit along the length (i.e. the longitudinal- axial direction) and centrally positioned across the width of the beam.

5. TESTING OF BEAMS

Four points loading was applied for all beams as shown in figure 1. All the beam specimens were tested in accordance with section-8 of IS516-1959 [8]. All beams specimens were tested under a load control regime with a load increase of 2kN initially and 1kN after initiation of crack. Loading was stopped at the ultimate beam specimen and this load was recorded as collapse load. Bending moment at ultimate was then calculated as shown in equation (5.1)

$$M_{exp} = P * a \quad (5.1)$$

where P: load (kN)

a: distance between support and first load (m)

Analytical moment capacity M_{aci} is calculated as per ACI 440-2R-08. Comparison of experimental and analytical moment capacity as per ACI-440-2R-08 code is given in table A1.

5.1. Analysis of RCC-GFRP Composite Beams – Proposed Equations

It is clear that from the results listed in table A1 that cross sectional area of GFRP and grade of concrete affect the moment capacity of RCC-GFRP composite beams.

From the comparison of experimental and analytical moment capacity values presented in table A1, it was observed that, for RCC-GFRP composite beams having higher GFRP area, the ACI code over estimates the moment capacity, for all the series of test specimens. It is therefore necessary to develop an empirical model to predict moment capacity by taking into consideration the experimental results. Under the action of loads, concrete below the natural axis of the beam experiences tensile stresses. During tensile cracking, concrete typically shows a quasi-brittle plastic behaviour along with compressive crushing ultimate mode. The ultimate stress corresponds to the onset of micro-cracking in the concrete material. The tensile strength, f_{cr} as per IS 456-2000[8] is given as shown in equation (5.2)

$$f_{cr} = k\sqrt{f_{ck}} \quad (5.2)$$

where, f_{cr} = Modulus of rupture of concrete (flexural tensile strength) in MPa.

k= Flexural Stress Assessment Factor (FSAF)

f_{ck}=Characteristic compressive strength of concrete cube in MPa.

Modulus of rupture, is a measure of a specimen's flexural tensile strength before rupture. Thus, “k” indicates flexural tensile strength on the onset of cracking. The factor for assessment of flexural tensile stress (FSAF or k = 0.7 as per IS 456: 2000 [9] or 0.62 as per ACI 318 [10], provides the extent of flexural stiffness, indirect tensile strength and cracking resistance of concrete composite. Hence, larger value of “k” is desirable.

Hence, for shear, shear de-bonding and flexural de-bonding modes of ultimate of RCC-GFRP composite beam, an empirical model is developed to predict its moment capacity. The moment capacity of RCC-GFRP composite beam and corresponding values of FSAF depend on the cross-sectional area of GFRP laminates. It is therefore necessary to establish a relation between revised FSAF and percentage area ratio of GFRP which is defined as the ratio of cross sectional area of GFRP strip to the concrete cross section and is expressed in percentage as A_r.

From the experimental moment at failure, the flexural tensile strength of the RCC-GFRP composite beam was calculated using equation (5.3).

$$\sigma = \frac{M}{Z} \quad (5.3)$$

where σ = actual flexural tensile strength corresponding to ultimate moment MPa,

M = moment at ultimate in N.mm,

Z = Section modulus of the beam on overall cross section in mm^3.

Revised FSAF k_2 for RCC-GFRP composite beam was calculated using equation (5.4).

$$k_2 = \frac{\sigma}{\sqrt{f_{ck}}} \quad (5.4)$$

where f_{ck}: Characteristic compressive strength of concrete cube MPa or design compressive strength whichever is less.

Values of revised FSAF k_2 thus found out are plotted against A_r for all beams tested in figure B1.

From figure B1 it is observed that the value of k_2 increases with A_r and variation of k_2 with A_r follows a linear relationship up to $A_r = 0.191$ whereas, for $0.191 \le A_r \le 0.38$ the value of k_2 is stabilized at 2.2. It is necessary to note that the value of k_2 stabilizes at a certain value (2.2 in present work) beyond the area ratio of 0.191. This is not reflected in ACI440-2R-08 code [1]. Thus, the relation between k_2 and A_r for all values can be expressed as given in equation (5.5) and (5.6).

From the graph, the relation between k_2 and A_r for all experimental values is expressed as below:

$$k_2 = 5A_r + 1.2; 0 \le A_r \le 0.191 \qquad (5.5)$$

$$k_2 = 2.2; 0.191 \le A_r \le 0.38 \qquad (5.6)$$

It is observed that value of k_2 due to strengthening of GFRP increased from 1.2 for RCC beams up to 2.2 for RCC-GFRP composite beams with Ar = 0.38. This proves the effectiveness of the composite action imparted by the GFRP laminates in enhancing bending tensile strength of RCC-GFRP composite beams. However, for design consideration, it is prudent to take the lower bound of k_2 value. Hence from figure B1 the lower bound value equation is shown in (5.7) and (5.8):

$$k_2 = 5.44A_r + 1.05; 0 \le A_r \le 0.191 \qquad (5.7)$$

$$k_2 = 2.09; 0.191 \le A_r \le 0.38 \qquad (5.8)$$

Based on these relations, the moment capacity of RCC-GFRP composite beams can be calculated as per following steps shown in equation (5.9), (5.10) and (5.11)

- Calculation of flexural tensile stress based on revised FSAF factor k_2

$$\sigma_{ct} = k_2\sqrt{f_{ck}} \qquad (5.9)$$

- Calculation of section modulus of the RCC-GFRP composite beam:

$$Z = \frac{bh^2}{6} \qquad (5.10)$$

where Z = Section modulus of beam on overall cross section of the beam only

b = Width of the beam

h = Depth of the beam

- Finally, calculation of ultimate moment capacity is carried out using the proposed equation

$$M_{Pred} = \sigma_{ct}Z \qquad (5.11)$$

Based on above mentioned steps, the ultimate moment capacity for all the RCC-GFRP composite beams tested were calculated and compared with the corresponding analytical values using ACI-440-2R-08 code as well as experimental values as shown in table A2.

6. DISCUSSION

It was observed that when highest area of GFRP was used, the experimental moment capacity was less than the moment capacity calculated based on ACI-440-2R-08[1] code. This is indicated in table A2 by the ratio (M_{aci}/M_{exp}) greater than 1.00. This was due to separation of GFRP strips before failure of concrete. Such type of failure occurred because of development of greater pull out force between GFRP strips and concrete than that of the load capacity of RCC-GFRP composite beam.

The moment capacity in the present study is less than that in ACI code. Because of this, the partial interaction was assumed and this is the real behaviour in site so that the deflection becomes more and the moment capacity becomes less in presences of slip between RC beams and GFRP layer.

The suggested empirical formula can be used in design of RC beams strengthened by GFRP because of the given moment capacity for composite beams being less than the actual moment strength capacity so that the designed one is not over and safe.

Table A2 also gives ratio of predicted moment capacity to that of experimental moment capacity (M_{pred}/M_{exp}).This ratio varies from 0.82 to 1. This shows that the moment capacity predicted by the empirical model is conservative and does not overestimate the capacity of RCC-GFRP composite beam. The ratio of predicted moment capacity to that of analytical moment capacity (M_{pred}/M_{aci}) for all the beam is also given in table A2.

It is evident from the above discussion that the developed empirical method can be used successfully in design and analysis of RCC-GFRP composite beams.

7. CONCLUSION

The analytical method to predict the moment capacity and ultimate modes are still not fully developed. It was observed that the analytical moment capacity of RCC-GFRP composite beams, calculated as per ACI-440-2R-08 code was higher than the actual experimental moment capacity for RCC beams strengthened with larger GFRP area. From the comparison between experimental, analytical (using ACI method) and predicted moments worked out using developed empirical model it can be concluded that the developed model can predict the moment capacity of RCC-GFRP composite beams conservatively and more accurately, especially for higher GFRP areas.

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APPENDIX A

Table A1.Comparison of experimental and ACI moment capacity for RCC-GFRP composite beams

Specimen ID	Cross-sectional Area of GFRP (mm^2)	Grade of concrete	M_{aci} (kN.m)	M_{exp} (kN.m)	
				OPC	PSC
Control	0	M35	4.12	4.41	4.41
G25-1L	10.75	M35	4.88	4.83	4.83
G25-2L	21.5	M35	5.62	6.14	6.09
G50-1L	21.5	M35	5.62	5.67	5.67
G50-2L	43	M35	7.08	7.50	8.09
G100-1L	43	M35	7.08	7.56	7.67
G100-2L	86	M35	9.86	7.72	7.98
Control	0	M45	4.12	4.83	4.73
G25-1L	10.75	M45	4.94	4.94	5.04
G25-2L	21.5	M45	5.70	6.51	6.30
G50-1L	21.5	M45	5.70	6.09	6.30
G50-2L	43	M45	7.18	8.61	8.30
G100-1L	43	M45	7.18	7.98	7.88
G100-2L	86	M45	10.06	8.93	8.40

Table A2. Analytical, experimental and predicted moment capacities of RCC-GFRP composite beams

Specimen ID	**Moment Capacity (kNm)**			**(M_{aci}/ M_{exp})**	**(M_{pred}/ M_{exp})**	**(M_{pred}/ M_{aci})**
	(M_{exp})	**(M_{aci})**	**(M_{pred})**			
OPC-35control	4.41	4.12	3.49	0.93	0.79	0.85
OPC-35-G25-1	4.83	4.88	4.36	1.01	0.90	0.89
OPC-35-G25-2	6.14	5.62	5.22	0.92	0.85	0.93
OPC-35-G50-1	5.67	5.62	5.22	0.99	0.92	0.93
OPC-35-G50-2	7.50	7.08	6.96	0.94	0.93	0.98
OPC-35-G100-1	7.56	7.08	6.96	0.94	0.92	0.98
OPC-35-G100-2	7.72	9.88	6.96	1.28	0.90	0.70
OPC-45 control	4.83	4.18	3.96	0.87	0.82	0.95
OPC-45-G25-1	4.94	4.94	4.94	1.00	1.00	1.00
OPC-45-G25-2	6.51	5.70	5.92	0.88	0.91	1.04
OPC-45-G50-1	6.09	5.70	5.92	0.94	0.97	1.04
OPC-45-G50-2	8.61	7.19	7.89	0.84	0.92	1.10
OPC-45-G100-1	7.98	7.19	7.89	0.90	0.99	1.10
OPC-45-G100-2	8.93	10.08	7.89	1.13	0.88	0.78
PSC-35 control	4.41	4.12	3.49	0.93	0.79	0.85
PSC-35-G25-1	4.83	4.88	4.36	1.01	0.90	0.89
PSC-35-G 25-2	6.09	5.62	5.22	0.92	0.86	0.93
PSC-35-G50-1	5.67	5.62	5.22	0.99	0.92	0.93
PSC-35-G50-2	8.09	7.08	6.96	0.88	0.86	0.98
PSC-35-G100-1	7.67	7.08	6.96	0.92	0.91	0.98
PSC-35-G100-2	7.98	9.88	6.96	1.24	0.87	0.70
PSC-45 control	4.73	4.18	3.96	0.88	0.84	0.95
PSC-45-G25-1	5.04	4.94	4.94	0.98	0.98	1.00
PSC-45-G25-2	6.30	5.70	5.92	0.90	0.94	1.04
PSC-45-G50-1	6.30	5.70	5.92	0.90	0.94	1.04
PSC-45-G50-2	8.30	7.19	7.89	0.87	0.95	1.10
PSC-45-G100-1	7.88	7.19	7.89	0.91	1.00	1.10
PSC-45-G100-2	8.40	10.08	7.89	1.20	0.94	0.78

APPENDIX B

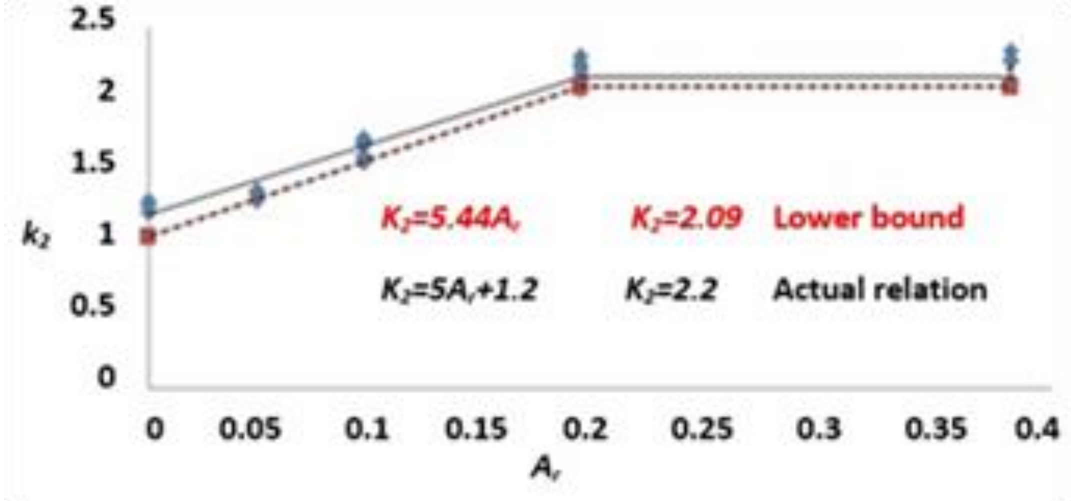

Figure B1.Variation of FSAF (k_2) with area ratio (Ar) at collapse for all RCC-GFRP composite beams