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Comparative Approach to Flexural Behavior of Reinforced Beams with GFRP, CFRP, and Steel Bars

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Abstract

The replacement of conventional steel bars with GFRP or CFRP is one of the main topics discussed in this paper, including the main parameters and properties of the materials. The design procedures should account for the properties and will focus on the tensile strength and modulus of elasticity. It will also consider corrosion under environmentally aggressive conditions. This paper presents an experiment on the flexural behavior of concrete beams reinforced with GFRP and CFRP bars and compares these results with theoretical analysis based on different standards such as ACI, Eurocode, and CSA. Twelve reinforced concrete beams will be tested using four-point loading. The geometrical parameters of the tested beams are 130×220×2200 mm, reinforced with different diameters for GFRP and CFRP. The reinforcement ratio and strength of concrete influence the behavior of GFRP, CFRP, and RC beams and contribute to reduce the deflection and crack width. Based on this research, the closest approximation of the experimental results is observed with ACI standards. At this stage, these bars can be used in structures without strict requirements for exceeding the Serviceability Limit State. The non-integration of tension stiffening and regression performance of cracking moment in prediction expressions imposed the differences from experimental results.

Keywords: RC Beams; GFRP; CFRP; Deflection; Cracks.

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1. Introduction

For a long time, researchers and civil engineers have been searching for alternatives to steels and alloys to reduce the high costs of repair and maintenance of structures damaged by corrosion. Development of polymer materials and technology was also an indicator of the research on civil engineering structures. The most important impact is on applications in structures under severe environmental conditions. The use of polymer materials instead of steel bars in concrete led to the application of Fiber Reinforced Polymers (FRP) in the field of engineering in structure elements. The behavior of FRP bars under environmentally aggressive conditions, their light weight, non-magnetic characteristics, and mechanical properties such are tensile strength, are beneficial parameters for the replacement of conventional steel in elements of structures. However, use of these materials is limited because the modulus of elasticity, ductility, large creeps, bond between the FRP bars and concrete and high cost can disorient other parameters [1].

Theoretically, there are no conceptual differences between the classical theories of steel-reinforced concrete elements. According to CNR-DT [2], it is the different mechanical behaviors of FRP material that need to be considered, whose constitutive law is fundamentally linearly elastic up to failure.

Many researchers working in this field, reinforced members with FRP analyze the linear relations between stress and

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strain in FRP bars, positions and geometrical parameters of cracks, deflections of members, and general behavior of members. FRP bars include different type of fibers such as glass, carbon, or aramid combined with resin—epoxy, polyester or vinyl ester—and are known as GFRP, CFRP, and AFRP bars [3]. According to ACI 440.1R-06, the design of FRP-reinforced concrete members is governed by serviceability limit state (SLS) requirements. This is because the modulus of elasticity of FRP bars is much lower than steel bars and, therefore, affects the deformation response of FRP-reinforced beams.

This study considers different analyses from different researchers using different codes, to develop a comparative approach to critical points in the behavioral context of determined concrete members.

In this paper, the effect of GFRP and CFRP versus conventional steel is experimentally investigated, first regarding their mechanical properties (Table 1) and then as reinforcement in concrete members. The design of GFRP and CFRP beams is typically governed by serviceability requirements [4, 5]. The focus of this investigation is deflection and crack width as yielding effects of serviceability. This investigation also includes beams reinforced with conventional steel as the referent model for better comparative effects.

2. Research Methodology

First, several analytical calculations and FEM analysis were conducted with various approaches, to define the influential areas where referent linear variable differential transformer (LVDT) must be placed respectively where the main referent crack appears or maximum deflection. The process of investigation continued in the laboratory, where the concrete beams were cast under laboratory conditions, left for 30 days and then put into the testing machine one after another for examination. The beams were simply supported and subjected to a four-point bending load as shown in Figure 5. During the examination on the MCC8 Controls, characteristics records were kept of each beam, such as displacement versus time chart, crack width, deflections, and maximum load. After obtaining data from multifunctional console control MCC8, experimental and analytical data were compared and analyzed with different codes, such as ACI, Eurocode, and CSA. Besides the main prescribed codes, other approaches were also conducted for deflection evaluation [6] and crack evaluation [7], but not included in this research because of differences in approximation.

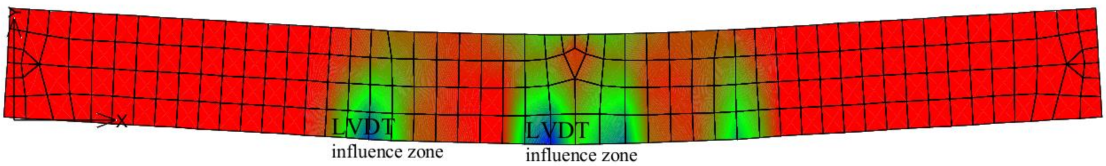

Figure 1. Influential zones as referent placements for transducers

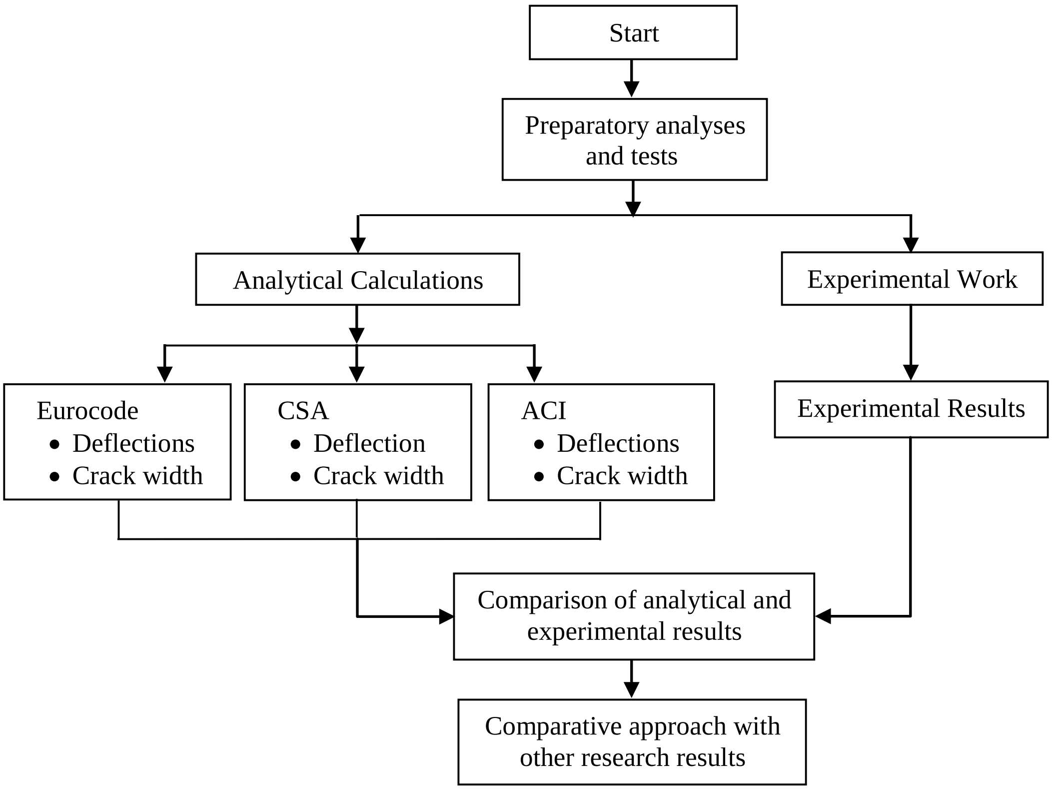

Figure 2. Flow chart of research module, analytical, and experimental approach

3. Materials and Methods

Due to the brittle nature of concrete under changing loading conditions and other factors not considered in the design (such as internal stresses resulting from casting), cracks in concrete infrastructures, cannot be completely avoided in practice. Design codes provide guidelines for checking the amount of reinforcement required in a structure to keep the crack width limited to a certain value at specified load-levels [2]. Checks in design codes are based mainly on the forces and bending moments in the cross-section of the structure and are unreliable for relatively thin plate-shaped structures, but conservative for non-standard structures with more complex loading and support conditions. Analysis of the cracks is based on the basic parameters used in EC 2, ACI 318 and CSA for cracking and deflections in concrete beams [1], [2] [8], [9]. The beams are divided into five groups due to differences in the type of reinforcement, as mentioned in Table 1. The expression and calculation procedure for different codes is presented in Figure 5.

ACI Committee 440 has modified the Gergely-Lutz equation for crack width prediction, where several experimental studies performed by different researchers [10-13] suggested incorporating the effects of the differing bond and mechanical properties of FRP reinforcement, compared to conventional steel reinforcement.

$$w = 2.2k_b \cdot \beta \cdot \frac{f_f}{E_{frp}} \sqrt[3]{d_c \cdot A} \quad (mm) \qquad \text{Cracks (ACI 440.1R} - \text{06 \& CAN/CSA)} \qquad (1)$$

The Eurocode 2 crack width equation is strain-based and can be adopted directly for the crack width of FRP-reinforced concrete members [6], [14-15]. Calibration is done through bond parameter β_1 and via parameter β_2 for long-term stress.

$$w = \beta \cdot s_{rf} \frac{M}{E_f \cdot A_f \cdot j \cdot d}\left[1 - \beta_1\beta_2\left(\frac{M_{cr} \cdot j \cdot d}{M \cdot 0.9 \cdot d}\right)^2\right] (mm) \qquad \text{Cracks (Eurocode 2)} \qquad (2)$$

The standard linear elastic approach using a constant effective moment of inertia yields very stiff behavior for FRP RC members [2], [14-15]. ACI 440.1R-06 modifies the model for evaluation of an effective moment of inertia, also including βb as the bond coefficient.

$$w\Delta = \frac{P \cdot a}{24EI_e}(3L^2 - 4a^2) \qquad \text{Deflections (ACI 440.1R} - \text{06)} \qquad (3)$$

CAN/CSA adopts the same modified Branson equation for the effective moment of inertia as in ACI 440.1R despite the correction factor β_b being based on several research results with limited test data and doubtful applicability in other loading and boundary conditions [3-5].

$$\Delta = \frac{P \cdot a}{24EI_{cr}} \cdot \left[3L^2 - 4a^2 - 8\left(1 - \frac{I_{cr}}{I_g}\right)\left(\frac{M_{cr}}{M_a}\right)^3 \cdot a^2\right] \qquad \text{Deflections (CSA A23.3} - \text{05)} \qquad (4)$$

Several researchers and model codes [6, 15] have pointed out that the model proposed for steel reinforcement by Eurocode 2 is reliable and adaptation is done through bond performance of FRP compared to steel.

$$\Delta = \left[1 - \left(\frac{M_{cr}}{M_a}\right)^2\right]\Delta_g + \left[1 - \left(1 - \left(\frac{M_{cr}}{M_a}\right)^2\right)\right]\Delta_{cr} \qquad \text{Deflections (Eurocode 2)} \qquad (5)$$

3.1. Material Properties

Different types of reinforced bars were used in our research. The first stage was oriented toward determination of real properties of reinforced FRP bars in the testing process. The results are presented in Table 2, based on the testing process according to the Standard ASTM D 7205. Metallic shells were set along the edges of the bars to avoid constriction of the FRP bars, as shown in Figure 2. The properties of conventional steel used from known parameters were based on previous research. Two types of FRP bars were used in our research GFRP (helically grooved) and CFRP (sand-coated), as shown in Figure 3.

Figure 3. Examination of mechanical properties of FRP bars

The mechanical properties of the tested GFRP and CFRP bars are presented in Table 1 while the properties of conventional steel bars were used from known parameters based on previous research [3].

Table 1. Mechanical properties of FRP bars

Mechanical properties	GFRP Ø6	GFRP Ø8	GFRP Ø10	CFRP Ø8	CFRP Ø10
Strain ε_{frp}^{*}	0.0204	0.0234	0.0256	0.0095	0.015
Tensile strength (MPa)	1022.1	1108.02	1194.3	1265.4	2000
Elasticity modulus (GPa)	55	55	55	155	155

Concrete mixes were prepared with the requested class of concrete, C 30/37. The compressive strength, modulus of elasticity, and other mechanical properties of concrete were determined by testing the standard cylinder, cubic specimens, prismatic specimens (150×150×600) mm.

Figure 4. Preparing of beams

4. Testing Set Up

Fifteen reinforced beams were prepared for testing – five sets with three samples – of which 12 were reinforced with GFRP and CFRP bars and three with conventional steel bars with a cross-section of 22 cm/13 cm and span 200 cm, as shown in Figure 5. The beams were reinforced with one layer of FRP in the tensile zone. The examined beams were reinforced in compression with two 6mm steel bars, and shear failure was avoided by providing closely spaced steel stirrups (6mm spacing in the shear span). In addition, stirrups spaced at 12mm were placed in the constant moment zone to ensure the positions of longitudinal bars and minimize the confinement provided by the stirrups.

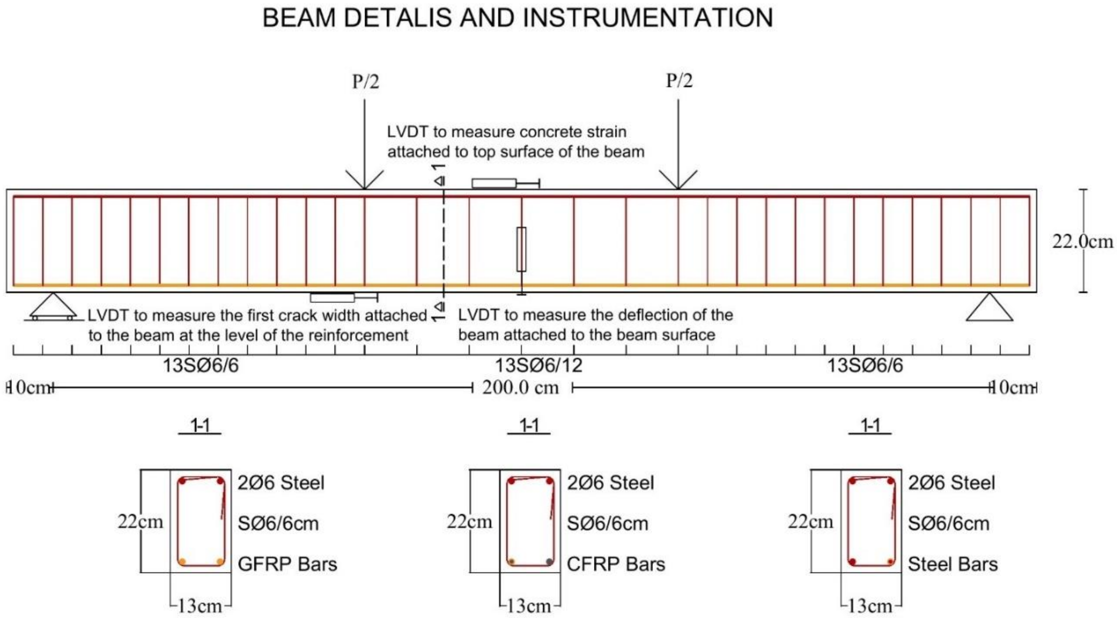

Figure 5. Beam details and instrumentation

Figure 5 shows the geometry and reinforcement details of the specimens. A linear variable differential transformer (LVDT) was used to measure the width of the first flexural crack right under the concentrated force. The beam was observed during the test until the first flexural crack appeared. As soon it appeared, the load was paused until the initial crack width was measured on the beam's side surface (at the reinforcement level). During the test, crack formation on the side of each beam was marked and the corresponding loads were recorded. Furthermore, compression concrete zones were instrumented with LVDT to measure the strain of concrete and another LVDT was inserted mid-span in the beam to measure the deflection. All beam specimens were tested under four-point bending over a clean span of 200cm (Figure 1). The load was monotonically applied using a 400 kN hydraulic actuator with a stroke-controlled rate of 300 N/s. The actuator, strain gauges, and LVDTs were connected to a data-acquisition unit to continuously record their readings.

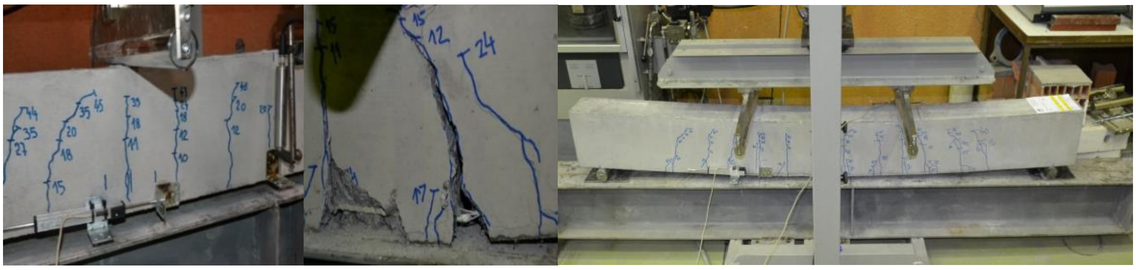

Figure 6. Beam during examination

During the examination, MCC8 equipment software was connected with LVDT for cracks and displacement and all necessary parameters, such as displacement versus time, level of load, level of cracks etc., were recorded. All collected data were exported to an Excel spreadsheet; cracks and displacements were measured in micrometers and graphical charts were collected directly from the equipment (Figure 7).

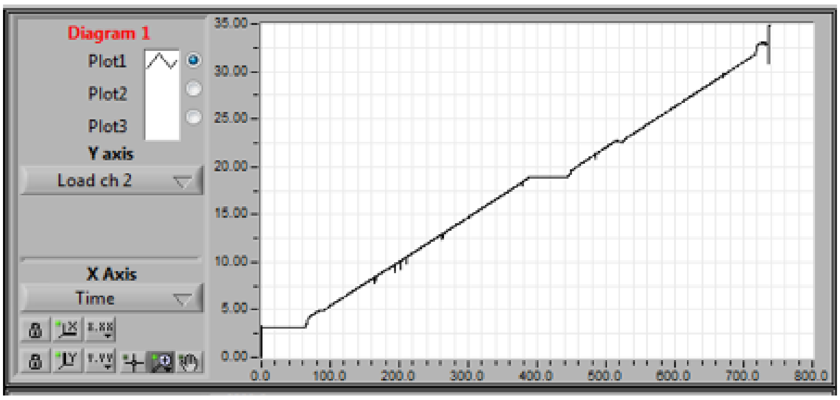

Figure 7. Displacement versus time chart and other parameters taken from MCC8 controls equipment

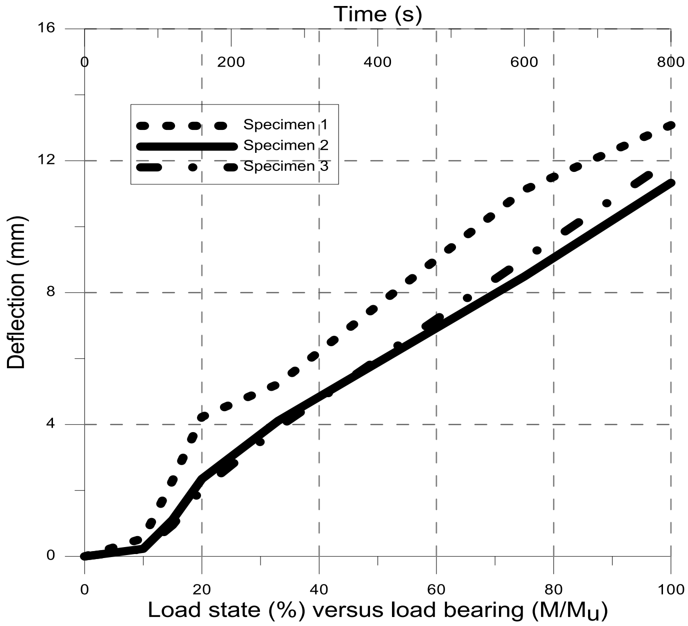

Figure 8. Deflections versus load bearing (M/Mu) versus time in beams reinforced with Ø10 CFRP

5. Analysis of Results

As shown in the results in Table 2, the maximum strength of GFRP and CFRP-reinforced bars is very high, but their SLS stage ends very quickly. GFRP-reinforced bars are characterized by deep cracks that occur rapidly in the direction of the force lines, a phenomenon not emphasized at this level in reinforced concrete slabs with conventional steel bars. This condition is due to the low modulus of elasticity of the GFRP bars. CFRP bars are seen to be more utilized since they have a module of elasticity about three times greater than GFRP bars, however, they are limited due to their poor bond with the concrete, as a result of the smooth surface.

Table 2. Serviceability Limit State (SLS) of tested beams

Specimen	Limit of SLS (kN)	Maximum strength (kN)	SLS percentage (%)
Steel Ø6	13.42	13.75	97.6
Steel Ø8	27.40	31.70	86.4
Steel Ø10	31.00	38.90	79.6
GFRP Ø6	8.21	29.24	28.0
GFRP Ø6	9.59	35.00	27.4
GFRP Ø8	9.54	37.00	25.7
GFRP Ø8	10.57	43.00	24.5
GFRP Ø10	15.43	70.00	22.0
GFRP Ø10	15.69	72.11	21.7
CFRP Ø8	23.74	59.00	40.2
CFRP Ø8	20.98	72.00	29.1
CFRP Ø8	23.39	72.90	32.0
CFRP Ø10	29.18	80.00	36.4
CFRP Ø10	27.81	85.00	32.7
CFRP Ø10	28.30	84.00	33.7

5.1. Cracks and Deflection Parameters in Testing Beams

The flexural capacity of an FRP-reinforced flexural member is dependent on whether the failure is governed by concrete crushing or FRP rupture. The failure mode can be determined by comparing the FRP reinforcement ratio with the balanced reinforcement ratio (that is, a ratio where concrete crushing and FRP rupture occur simultaneously). Because FRP does not yield, the balanced ratio of FRP reinforcement is computed using its design tensile strength. However, once the beam cracks, the stiffness of the GFRP-reinforced concrete beam decreases faster than the control beam, resulting in a larger deflection of the GFRP-reinforced beam. Crack propagations were observed during the tests. The SLS for all testing beams is presented in Table 5. The balanced reinforcement ratio and nominal flexural strength defined in this paper can be obtained by conducting a sectional analysis in different stages of SLS theory, including the percent of ratio "Moment-M/M_u".

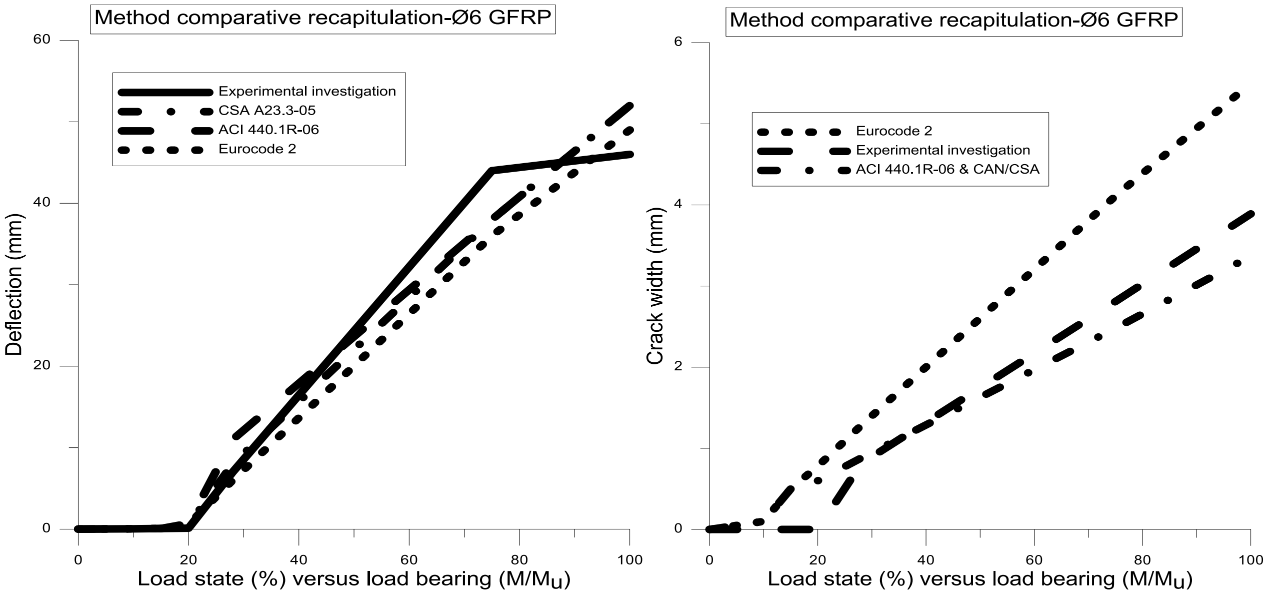

Figure 9. Comparative calculation methods for deflection and crack widths in beams reinforced with Ø6 GFRP

From the observed results, if extrapolated backward, the plot of moment versus deflection or crack width will pass through zero, as a crack will not form immediately after application of force. The differences are clearly emphasized in beams reinforced with Ø6 GFRP, known as under-reinforced beams; from experimental investigation, we observed a critical point in the sensing of behavior differences, which is related to the cracking moment.

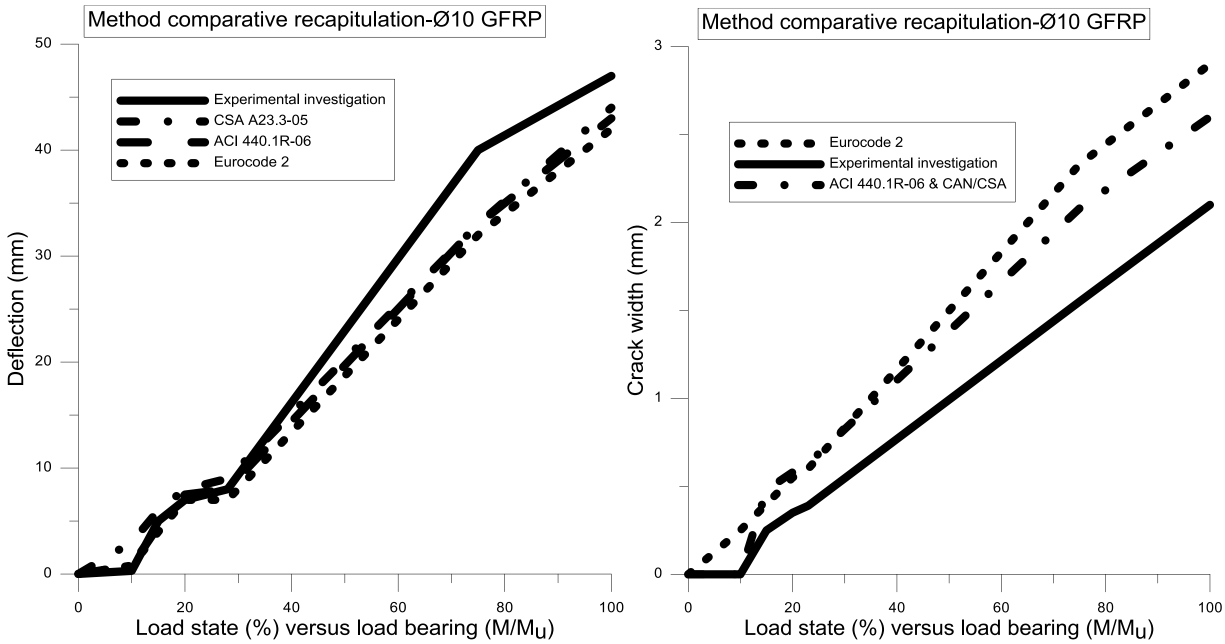

Figure 10. Comparative calculation methods for deflection and crack widths in beams reinforced with Ø10 GFRP

Beams with balanced reinforcement and over-reinforced GFRP beams behave more roughly in relation to beams with conventional reinforcement. Increasing the percentage of GFRP will influence in the stiffness of the beams and in yielding effects of this changing module [17]. The calculation methods especially approximate in deflection prediction up to an interval of 50% (M/M_u) of load-bearing capacity. The non-integration of the tension-stiffening principle leads to a difference of up to 18% in intervals of 75–80% M/M_u. Crack prediction methods are also more accurate with increased percentage of GFRP reinforcement, while the pre-cracking behavior and linear plot nature remain the critical points in terms of approximation.

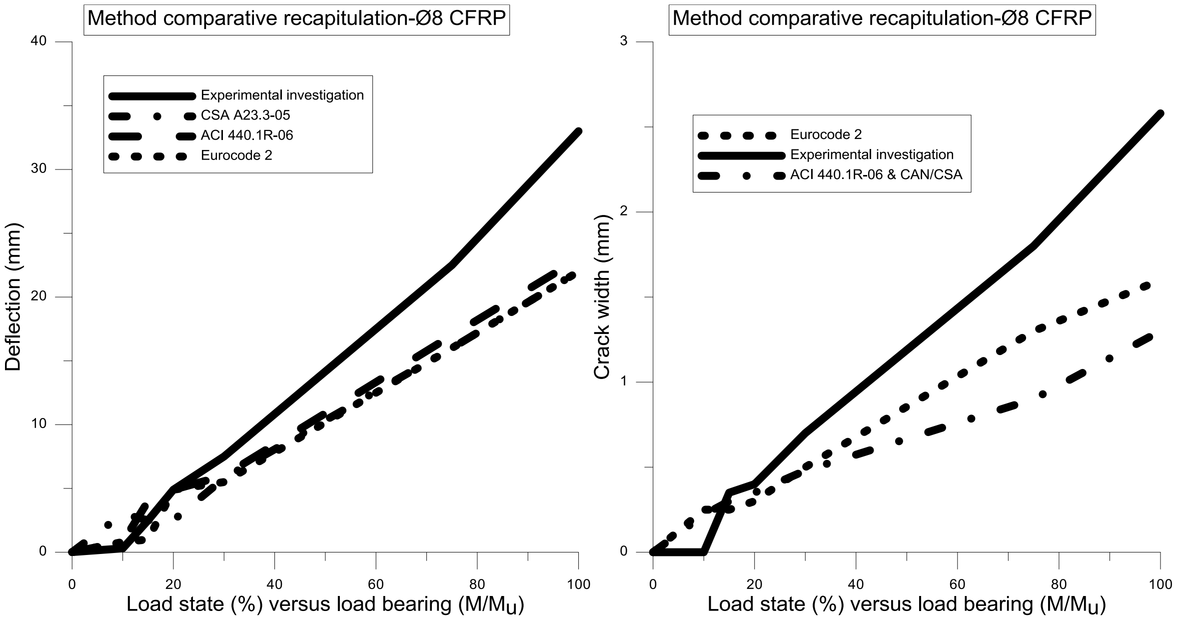

Figure 11. Comparative calculation methods for deflection and crack widths in beams reinforced with Ø10 GFRP

Beams reinforced with CFRP are characterized by lower deflections and stiffer cracking behavior, imposed by the material properties of CFRP bars [18-20]. In term of deflection, all the methods especially approximate in the first stages of the pre-cracking phase. For beams reinforced with CFRP, deviance of results is imposed by not considering the tension-stiffening principle. Compared to beams reinforced with GFRP, the cracking behavior differs in results but not in nature of linearity. The first cracks occur when concrete reaches its tensile strength and regression must be performed by implementing corrective bond coefficients.

Table 3. Cracks and deflection values of tested beams

Beams	Code	Cracks (SLS)	Deflections (SLS)	Cracks (75%)	Deflection (75%)	Cracks (100%)	Deflections (100%)
Steel Ø6	ACI	0.20	5.94	0.25	6.34	0.26	6.51
	CSA	0.20	3.41	0.25	4.80	0.26	5.08
	EC2	0.29	1.92	0.40	3.51	0.41	3.82
	EXP	0.01	2.15	0.47	7.30	0.52	9.70
Steel Ø8	ACI	0.26	6.04	0.32	7.16	0.35	7.91
	CSA	0.26	5.60	0.32	6.81	0.35	7.62
	EC2	0.27	5.05	0.32	6.35	0.36	7.19
	EXP	0.24	6.50	0.26	8.30(90%)	2.14	18.65
Steel Ø10	ACI	0.22	5.23	0.24	5.66(82%)	0.30	6.69
	CSA	0.22	4.99	0.24	5.45(82%)	0.30	6.74
	EC2	0.24	4.65	0.26	5.14(82%)	0.32	6.49
	EXP	0.21	7.30	0.25	8.30(82%)	2.28	31.4
GFRP Ø6	ACI	0.89	8.68	2.44	38.13	3.25	51.76
	CSA	0.89	11.16	2.44	38.17	3.25	51.58
	EC2	1.33	6.12	4.06	35.86	5.45	49.82
	EXP1	0.73	7.57	2.91	44.8	3.81	44.90
	EXP2	0.69	1.7	2.25	36.29	2.98	49.61
GFRP Ø8	ACI	0.89	5.28	2.44	27.25	3.25	36.87
	CSA	0.89	7.4	2.44	27.38	3.25	36.84
	EC2	0.69	4.5	2.33	26.22	3.12	35.96
	EXP1	0.71	8.29	2.91	43.25	3.81	48.31
	EXP2	0.68	4.10	2.80	23.11	3.90	31.53
GFRP Ø10	ACI	0.58	6.89	1.99	31.21	2.66	41.81
	CSA	0.58	8.31	1.99	31.22	2.66	41.77

	EC2	0.61	6.75	2.2	30.73	2.94	41.4
	EXP1	0.31	8.30	1.54	39.87	2.04	46.61
	EXP2	0.34	6.69	1.69	30.33	2.49	40.64
CFRP Ø8	ACI	0.38	6.64	0.92	15.78	1.23	21.04
	CSA	0.38	6.23	0.92	15.63	1.23	20.92
	EC2	0.49	5.52	1.20	15.31	1.61	20.68
	EXP1	0.70	8.06	1.86	22.50	2.59	32.22
	EXP2	0.38	8.30	0.60	18.50	0.79	28.43
	EXP3	0.34	8.29	0.87	22.80	1.27	33.42
CFRP Ø10	ACI	0.35	3.98	0.78	8.92	1.05	11.89
	CSA	0.35	3.90	0.78	8.87	1.05	11.86
	EC2	0.37	3.69	0.85	3.76	1.14	11.77
	EXP1	0.29	8.29	0.76	21.06	1.07	30.19
	EXP2	0.46	4.11	1.20	8.49	1.62	11.33
	EXP3	0.30	3.91	0.70	9.02	0.93	12.03

6. Conclusions

- In RC beams with GFRP and CFRP bars, it is observed that increasing the bar diameter increases their bearing capacity and decreases their SLS. The results are shown in Table 4.

Table 4. Maximum strength and percentage of SLS

Specimen	Maximum strength (kN)	SLS percentage (%)
GFRP Ø6	29.24	28.0
GFRP Ø6	35.00	27.4
GFRP Ø8	37.00	25.7
GFRP Ø8	43.00	24.5
GFRP Ø10	70.00	22.0
GFRP Ø10	72.11	21.7
CFRP Ø8	59.00	40.2
CFRP Ø8	72.00	29.1
CFRP Ø8	72.90	32.0
CFRP Ø10	80.00	36.4
CFRP Ø10	85.00	32.7
CFRP Ø10	84.00	33.7

- Currently, it is difficult to directly replace steel bars with GFRP bars in construction due to the low modulus of elasticity, big deformations, and low percentage of SLS, as seen in Table 3.
- Reinforced beams with GFRP and CFRP bars behaved linearly up to failure based on the linear characteristics of FRP bars and their lower modulus of elasticity—especially of GFRP bars—than conventional steel bars.
- As different manufacturers may improve some properties of these bars in the future, their use at this stage is limited. Currently, they can be used for constructions that do not have rigorous SLS condition criteria, especially in place of steel bars in skimmers subject to aggressive ambient conditions, such as salt water.
- Results of this study's experimental analysis for deflections and cracks in concrete beams show closer behavior to ACI 318 than other codes.
- Differences in deflection prediction are imposed by non-integration of principles of tension stiffening.
- Approximation of cracking behavior is done by performing a regression of cracking moment and implementing corrective bond coefficients.

7. Funding

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8. Conflicts of Interest

The authors declare no conflict of interest.

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