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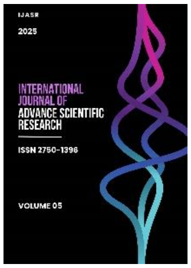

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Research Article

Flexural Behaviour Of Hybrid Steel-Frp Reinforced Concrete Beam

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Abstract

This paper presents the experimental test results of hybrid steel–glass fiber reinforced polymer (GFRP) reinforced concrete beams under four-point bending, and compares them with conventional steel reinforced concrete beams.

Keywords

Concrete, reinforced concrete beam, GFRP bar, steel bar, crack resistance, deflection, ultimate load, ultimate moment, cracking moment.

Introduction

In the construction industry, the long-term durability of reinforced concrete structures is considered one of the pressing issues. In reinforced concrete structures, the deterioration of concrete can lead to the exposure of steel reinforcement, and the slow penetration of oxygen from the air through cracks results in the formation of iron oxide in the steel reinforcement, which is primarily composed of iron [1–3].

The use of FRP reinforcement as an alternative to steel reinforcement in reinforced concrete structures has emerged as an innovative advancement in construction. Its advantages such as resistance to corrosion, absence of magnetic field generation, and non-conductivity of electricity are expanding its field of application. In addition, the high tensile strength and lightweight nature of composite polymer reinforcements make them increasingly attractive [4–6].

However, their tensile behavior follows a linear pattern and remains linear up to failure. As a result, concrete structures reinforced with composite polymer reinforcement are prone to brittle failure without any prior warning. Due to this factor, design codes require an excessive amount of reinforcement in concrete elements reinforced with composite materials, in order to reduce the probability of failure and minimize deformation. However, this approach is not economically justified [7–8].

The low modulus of elasticity of composite polymer reinforcements causes greater deflection and wider crack openings in flexural concrete elements compared to those reinforced with steel reinforcement of the same cross-section and quantity. Consequently, some challenges arise when calculating the serviceability limit states (SLS) of concrete beams reinforced with composite polymer reinforcement [9, 10, 11].

To address these issues, the idea of hybrid reinforcement of flexural elements using both steel and composite polymer reinforcements has emerged. The most effective solution involves placing the composite polymer reinforcement at the bottommost part of the tensile zone with the smallest protective concrete cover. In this arrangement, the steel reinforcement is embedded deeper inside the concrete, beneath the composite polymer reinforcement, thereby reducing its susceptibility to corrosion by increasing the protective concrete layer [1, 4, 6, 10].

As a result, the steel reinforcement contributes less to the load-bearing capacity of the element but plays a crucial role in providing ductility and flexibility [3, 8, 12, 13, 14]. Moreover, the presence of steel reinforcement helps reduce the number and width of cracks. Therefore, the combined use of steel and composite reinforcement enhances the durability and service life of reinforced concrete beams compared to conventional reinforced concrete [10, 15, 16].

Concrete cube specimens and their strength

Portland cement, sand, crushed stone, and water were used to prepare the specimens for the experimental study.

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Fig. 1. Sand used for the concrete mix

Fig. 2. Crushed stone used for the concrete mix

To determine the compressive strength of the concrete, cube specimens with nominal dimensions of 100×100×100 mm were prepared in four different series in accordance with the international standard requirements of GOST 10180-2012. The concrete mixes were placed into specially prepared molds (fig. 3), paying close attention to the specified requirements and the compaction of the concrete mix. The prepared cube specimens were labeled and cured under normal conditions for 28 days.

Fig. 3. Prepared cube specimens

Table 1. Results obtained from compressive strength testing of concrete cube specimens

No	Cube specimen series	Size *a*×*b*×*h*, mm	Ultimate load *P*, kN	Specimen strength, R_i MPaa	Average strength, R_m MPa	Normative strength R_n, MPa	Concrete grade	Modulus of elasticity *E*, MPa
1	Day 1, Series 1	102x100x101	377,89	35,9	34,4	31,2	B30	30449
2		100x101x99	349,47	33,2				
3		100x102x100	358,95	34,1				
4	Day 1, Series 2	100x99x101	383,16	36,4	34,9	31,7	B30	30568
5		102x101x101	346,32	32,9				
6		100x100x99	372,63	35,4				
7	Day 1, Series 3	100x101x101	386,32	36,7	33,3	30,2	B30	30097
8		99x100x101	344,21	32,7				
9		100x101x100	321,05	30,5				
10	Day 2	101x101x101	383,16	36,4	33,9	30,8	B30	30124
11		102x101x99	324,21	30,8				
12		99x100x100	363,16	34,5				

Geometric dimensions and reinforcement of the beams

For testing purposes, a total of 27 beams across 9 different series were prepared, each with a length of l=150 cm and an effective span of l_0=140 cm. The cross-sectional dimensions of the beams were b×h=15×20 cm. The concentrated load was applied at a distance of $l_0/3$ from the supports. The distance from the edge of the beam to the support was 5 cm. All beams were tested under four-point bending (fig. 4a).

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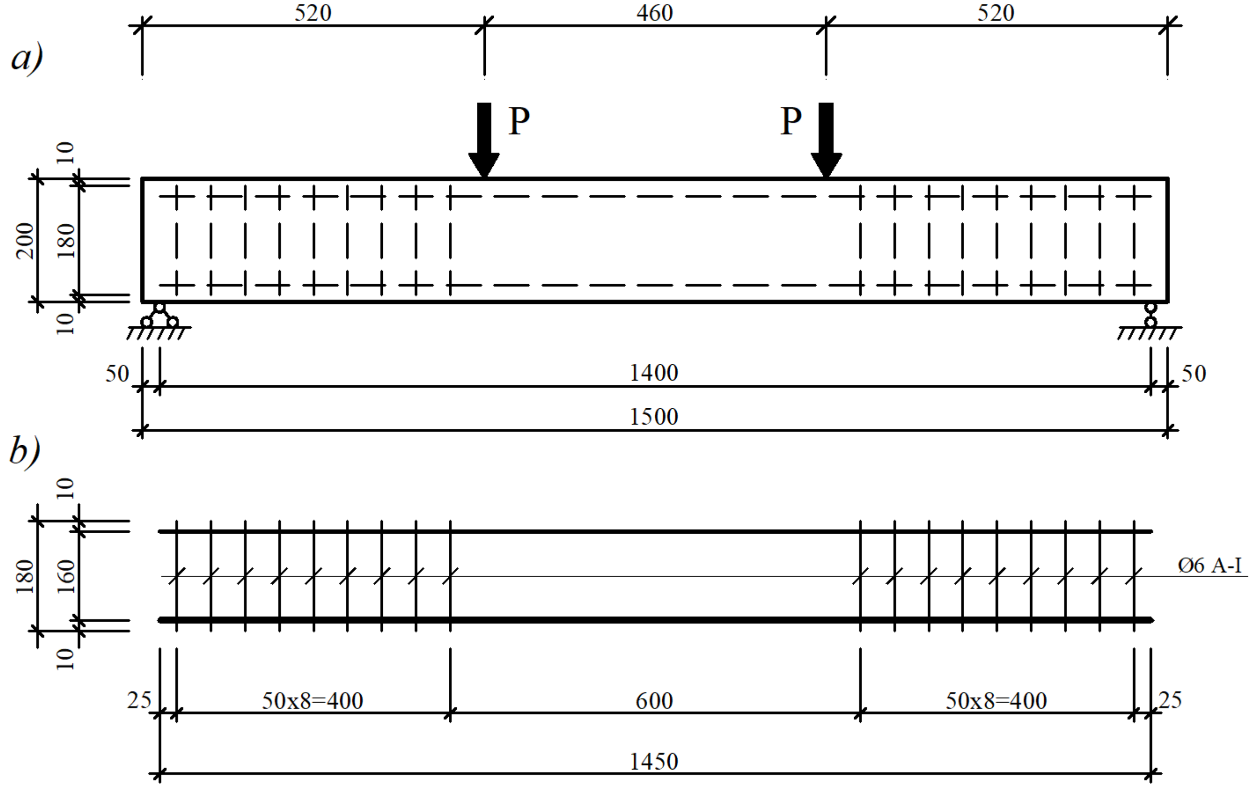
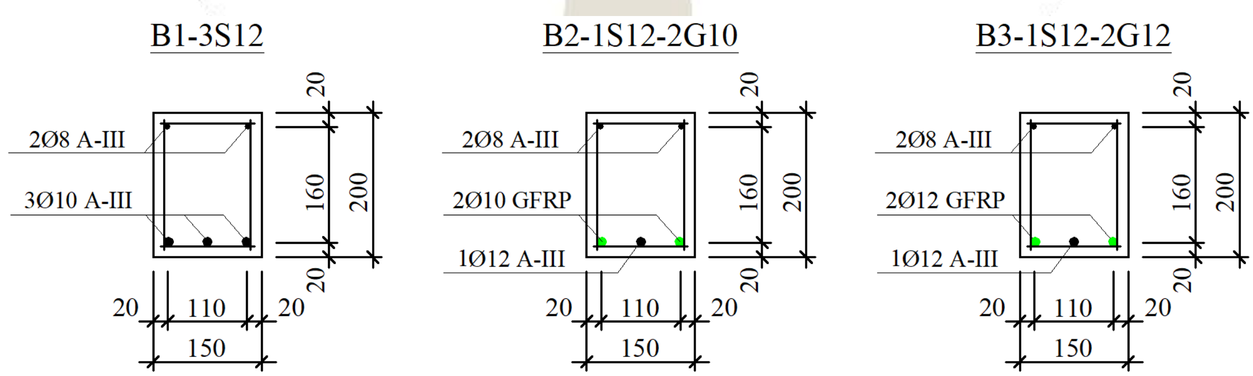


Fig. 4. Geometric dimensions of the beam (a) and reinforcement cage (b)

The beams were reinforced using prefabricated reinforcement cages. Since the beams were tested under normal cross-section bending, no transverse (shear) reinforcement was placed in the middle span of the beams. In the support zones, the spacing of the stirrups was set to 5 cm.

The length of the reinforcement cage was 145 cm, and its height was 18 cm. In all beams, two Ø8 A-III reinforcement bars were placed in the compression zone. Ø6 A-I steel bars were used as stirrups (fig. 4b).



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c)

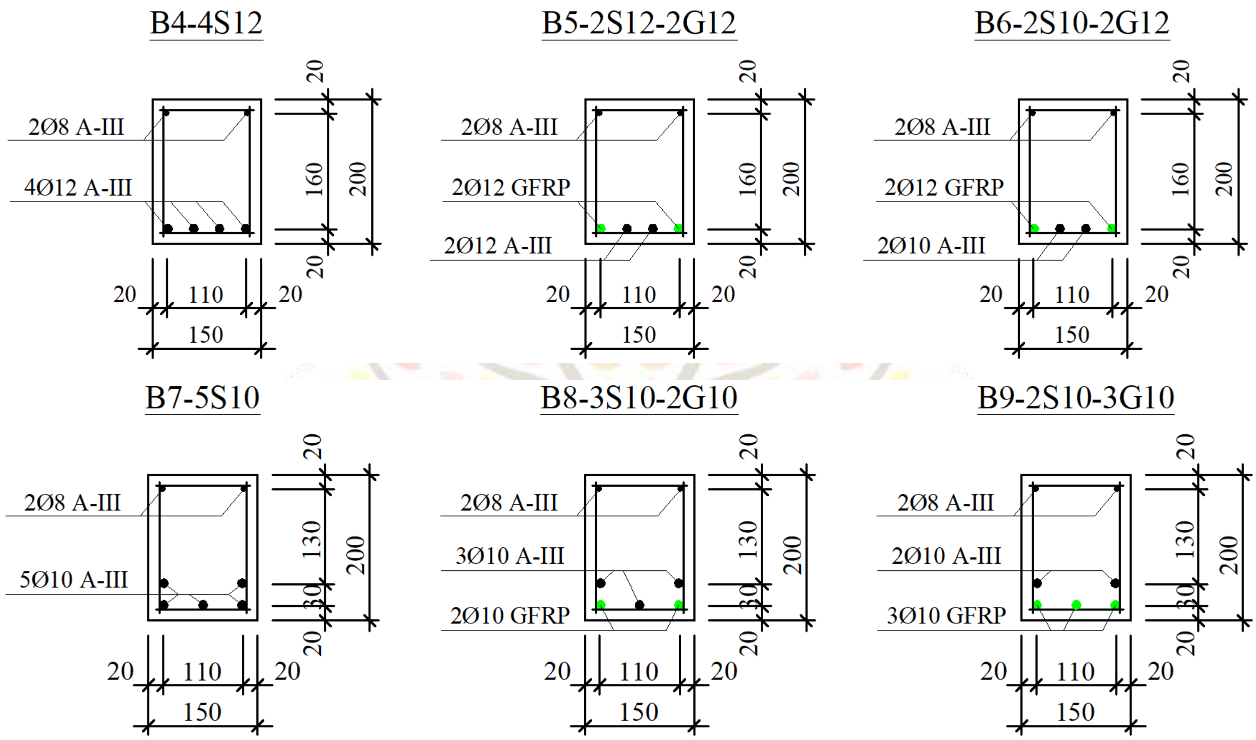

Fig. 5. Cross-sections of experimental beams and layout of reinforcement cages

B3-1S12-2G12

B6-2S10-2G12

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B8-3S10-2G10

B9-2S10-3G10

Fig. 6. Failure modes of beams under applied loading

As a result of the experimental tests, the failure of hybrid steel–composite reinforced concrete beams corresponded to Failure Mode 4. This mode is considered the closest to real-life structural behavior. In Modes 1, 2, and 3, the longitudinal reinforcement area relative to the beam's cross-section is insufficient, while in Modes 5 and 6, the reinforcement is considered excessive, which is not commonly used in construction practice.

In Mode 4, after the stress in the steel reinforcement reached its yield strength, crushing of the concrete in the compression zone was observed. However, the glass composite reinforcement did not rupture. The fact that the composite reinforcement remained intact allowed the beam to continue to behave elastically even after reaching its ultimate load-bearing capacity. In other words, upon unloading, a significant recovery of deflection in the beam was observed.

Strength and crack resistance of the beams

Table 2 compares the theoretical calculations, experimental test results, and ANSYS simulation results of the cracking moment for the tested beams. Additionally, the values of the applied load at which the first visible cracks appeared in the beams are presented.

Table 2. Crack resistance moment values in beam specimens

Beam notation	Ultimate load P_u, kN	Deflection *f*, mm	M_{crc}^{exp}, kN·m	M_u^{exp}, kN·m	$M_{crc}^{exp} / M_u^{exp}$
B1-3S12	109.7	29.9	4.61	25.60	0.18
B2-1S12-2G10	107.3	28.1	4.26	25.04	0.17
B3-1S12-2G12	119.6	30.2	4.47	27.91	0.16
B4-4S12	135.3	29.7	5.37	31.57	0.17
B5-2S12-2G12	138.1	23.4	4.83	32.22	0.15
B6-2S10-2G12	121.5	29.8	4.25	28.35	0.15
B7-5S10	116.2	25.0	5.15	27.11	0.19
B8-3S10-2G10	129.7	29.7	4.84	30.26	0.16
B9-2S10-3G10	118.1	28.1	4.41	27.56	0.16

According to the experimental test results of the beams, crack formation in beams reinforced with steel reinforcement developed later compared to hybrid-reinforced beams. That is, the first visible cracks were observed when the beam carried 17–19% of its ultimate load-bearing capacity. However, in the hybrid-reinforced beams, this value was 15–17% (table 2).

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