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Full Length Research Paper

Ultimate capacity and reinforcement area requirement for bridge girder using various FRP re-bars

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An analytical study on the use of various types of fibre reinforced plastic rebars as reinforcement is presented in this paper. The structural element examined was a girder, which formed a bridge deck and was designed in accordance to the ACI code. Three types of fibres were considered in the reinforced plastic rebar, namely glass, aramid and carbon. A finite element model of the bridge girder was created using solid brick and embedded bar elements. A non-linear, three-dimensional analysis was performed to determine the deflections and stresses under an applied concentrated point load. The results revealed that the load capacity of the girder increased when fibre reinforced polymer (FRP) with same area as steel bars was used and the increase was related to the type of fibres used. Consequently, the reinforcement area required to attain a specified load capacity decreased when FRP was substituted for normal steel rebar.

Keywords: Bridge girder, FRP rebars, glass fibre, aramid fibre, carbon fibre.

INTRODUCTION

Reinforced concrete bridges are important structural elements. A large number of these elements exist in any country around the world. For example, in the United States, there are somewhere between 560,000 and 600,000 highway bridges (Boyd, 1997). The main problem with bridges is the rehabilitation cost, and the maximum part of this cost is due to the corrosion of the carbon steel bars. The corrosion of the carbon steel bars has been a serious issue for highway agencies around the world. In the United States, this problem appeared in the southern coastal states as far back as 75 years ago. It also appeared in the northern states after the use of salts to milt ice become common about 50 years ago. The corrosion of the steel reinforcement in concrete structures results in significant repair and rehabilitation costs. The annual cost of repair and maintenance in the UK and the Europe Union is around £20 billion, and in the United States, it's around $50 billion (Boyd, 1997). All highway agencies around the world are looking for an efficient solution to overcome the corrosion problem. The Federal Highway Administration (FHWA) in USA began experimenting with methods to extend the life of concrete with carbon steel reinforced bars around 1970 because of these corrosion issues. Many methods and materials were suggested for reducing the corrosion problem. Epoxy coating was suggested to protect the carbon steel from moisture and salts and to electrically isolate a rebar mat from other nearby mats that may be at different potentials (Kamaitis, 2008; 2009). High performance concrete (HPC) was another method suggested to overcome the corrosion problem. Galvanized rebar was also a method suggested to overcome the corrosion problem.

In the past several years, new FRP reinforcing bars have been introduced as an alternative to traditional structural materials, such as steel reinforcing bars. The main reason for the replacement of steel bars with FRP is the corrosion of steel reinforcement in concrete structures. When compared to conventional steel, FRP has greater tensile strength than steel rods; it is free from corrosion, which leads to reduce the strength; it is free from magnetization; and it is light (Taniguchi et al., 1993). It is generally expected that the use of FRP rebar will reduce the maintenance cost by over 80% in the long

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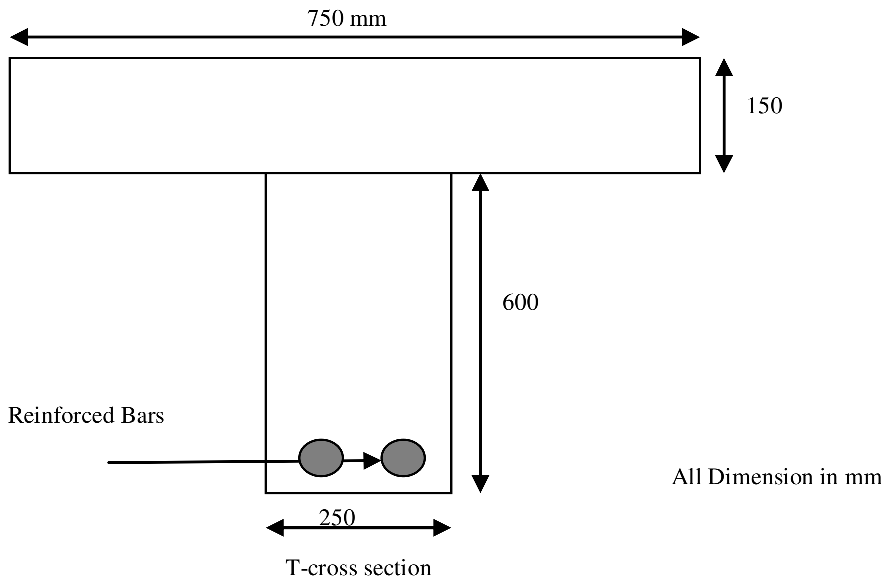


Figure 1. Cross –sectional dimensions of reinforced concrete bridge girder.

term (Abbasi and Hogg, 2006). Furthermore, the use of FRP will reduce the area required to attain the specific load capacity and this will decrease the initial construction cost. During the last few years, many studies replaced steel with FRP to study the load capacity of structural elements, failure mode, and fire resistance for the structural elements reinforced with FRP. (Berg et al., 2006; Skuturna et al., 2008; He et al., 2007; Cosenza et al., 2002; Yonekura et al., 1993; Trejo et al., 2000; Gravina and Smith, 2008; Tannous and Saadatmanesh, 1999; Chen et al., 2007; Davalos et al., 2008; Baena et al., 2009; Barhim et al., 2004; Ali et al., 2005).

In this paper, the use of Glass Fibre Reinforce Polymer GFRP, Aramid Fibre Reinforce Polymer AFRP, and Carbon Fibre Reinforce Polymer CFRP bars instead of traditional steel bar is presented. The main objective of this study is to review the use of different FRP types for reinforcing bridge girders under ACI design code criteria. Different areas of the materials were used to give an indicator for the increase in load capacity, decrease of area required, failure stresses, and failure modes. The study investigated two ACI design code criteria or limitations. The minimum limitation is the minimum area required to prevent the temperature changes or shrinkage. The maximum limitation is the maximum area required to prevent the brittle failure by crushing of concrete before the yielding or rupture of the reinforcement.

Case study

A bridge RC girder was the case of study. The girder had a clear span of 10 m in length and a T-cross section as shown in Figure 1. The girder design is according to ACI code with a concentrated load at mid-span for the purpose of this study (not to model actual behaviour under vehicle loads). The girder was reinforced with two cases; one is a minimum area of 1960 mm^2 and the other is a maximum area of 4824 mm^2 in order to model the lower and upper limits according to the ACI-Code requirements. The properties of concrete, steel and different FRP types (Glass Fibre Reinforce Polymer GFRP, Aramid Fibre Reinforce Polymer AFRP, and Carbon Fibre Reinforce Polymer CFRP) are shown in Table 1.

Finite element modelling

Finite element modelling DIANA TNO Software was used to model the girder. Three-dimensional nonlinear analysis was used to model the bridge girder reinforced with different bars types. A solid brick element with 20-nodes was used to model concrete elements as an isotropic material. An embedded bar element was used to model the steel bars with von-misses plasticity criteria. An embedded bar composite element (special model by researcher) was used to model FRP bar as a linear material with no plasticity limit (it fails from rupture). Figure 2 shows the Finite Element (FE) modelling of the girder.

RESULTS AND DISCUSSION

FE analysis was adopted to study the effect of design criteria and different FRP types used to reinforce the girder. Girder load capacity, reinforced area required, failure deflection, failure stress and failure mode were investigated. The engineering design requirements attained were required capacity with lower cost, service-ability and safety requirements. According to ACI design code criteria, minimum reinforced area must be used to prevent thermal and shrinkage changes. The maximum area is limited by the code to prevent the brittle concrete before the yielding of the steel. From this point, the

Table 1. Material properties of concrete, and steel; GFRP; AFRP; CFRP bars.

Concrete		Reinforce material	Tensile stress(MPa)	Modulus of elasticity (GPa)
Compressive strengthen	30 MPa	Steel	414	200
Tensile stress	3.82 MPa	GFRP	600	42
Modulus of elasticity	24 GPa	AFRP	1200	83
Poison Ratio	0.2	CFRP	2070	152

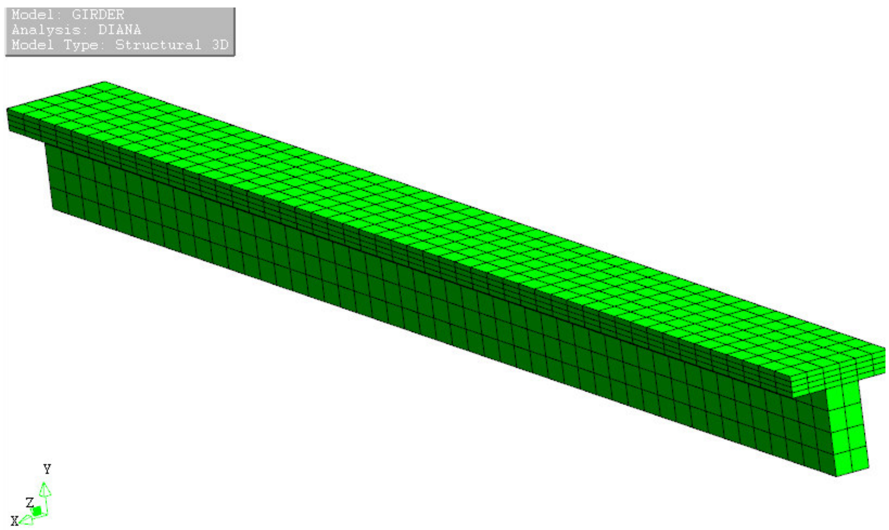


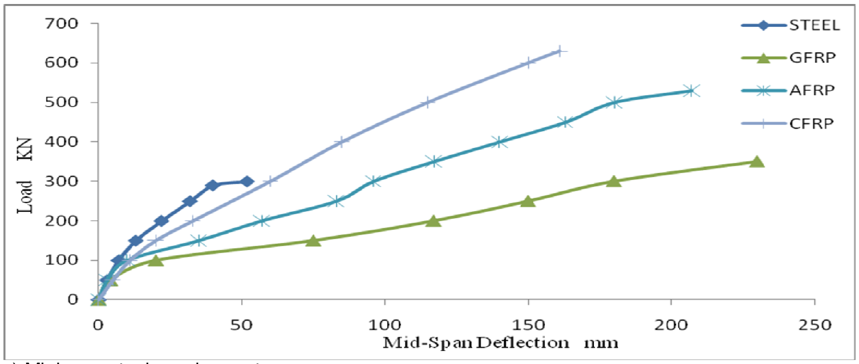
Figure 2. Three-dimensional modelling of the bridge girder.

the analysis with these two limits was done on the girder to understand the behaviour of the girder between these two limits. FRP is an important alternative material that is used in different engineering fields. Different types of FRP namely; glass, aramid and Carbon were used in this study.

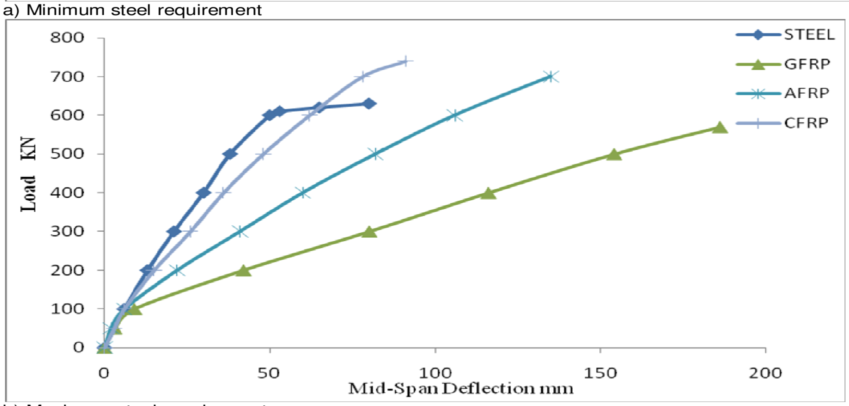
The main differences among the structural behaviours of these FRP types and steel are the ultimate tensile stress and the modulus of elasticity. The ultimate tensile stress directly affects the ultimate limit state of the girder and the failure mode while modulus of elasticity affect the deflection of the girder and failure ductility.

Load deflection curves are usually used as an indicator for the nonlinear behaviour of structural element under loads. Figure 3 shows the load deflection curve for the girder reinforced with the minimum and maximum reinforced limits, respectively. The figures show that even with the same area, the different reinforce materials give different load capacities and different mid-span deflections because these materials have different tensile stresses and different modulus of elasticity. The steel had the lowest capacity and the lowest deflection for the minimum code requirements while the GFRP had the lowest capacity and largest deflection for the maximum code requirements. GFRP had the largest deflection due to its minimum modulus of elasticity. CFRP had the largest capacity for both the minimum and maximum code requirements due to their maximum tensile stress. These figures show that the load capacity increases with the increase in tensile stress of the material used as reinforcement for the minimum code requirement while for the maximum requirement, the increase of load was limited by the crushing of concrete and not by the tensile stress of the materials. The load capacity increased based on steel as the baseline around 110% when CFRP was used and around 17% when GFRP was used for the minimum requirements. The load capacity increased around 16% when CFRP was used for the maximum requirement. Figure 4 show the load deflection curves for the girder reinforced with different materials and different areas to achieve the required load capacity for both code requirements.

These curves show that different fibre areas and properties under the same load capacity have similar nonlinear behaviour. According to FE analysis, the microstructure of the fibre cannot be simulated while considering the tensile stress and modulus of elasticity. The steel had the lowest deflection according to the maximum modulus of elasticity. The required area to attain 300 or 630 kN with different fibre types becomes

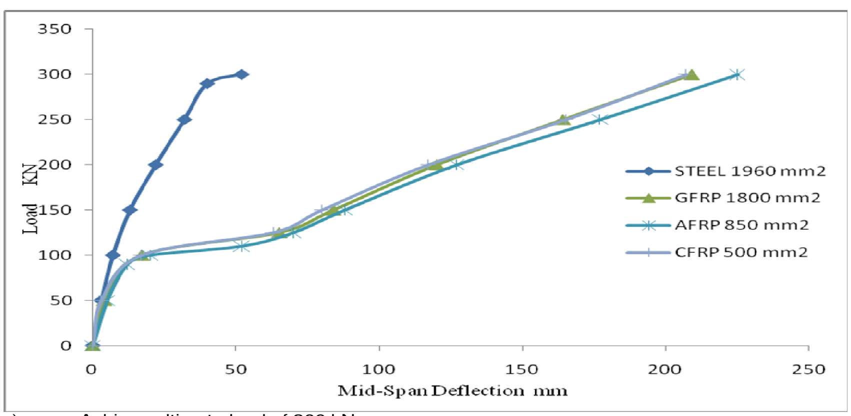


a) Minimum steel requirement

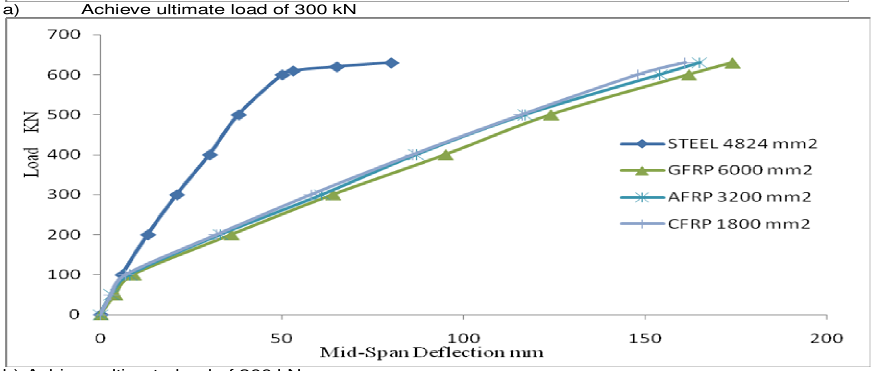


b) Maximum steel requirement

Figure 3. Load vs. deflection curves for the bridge girder.

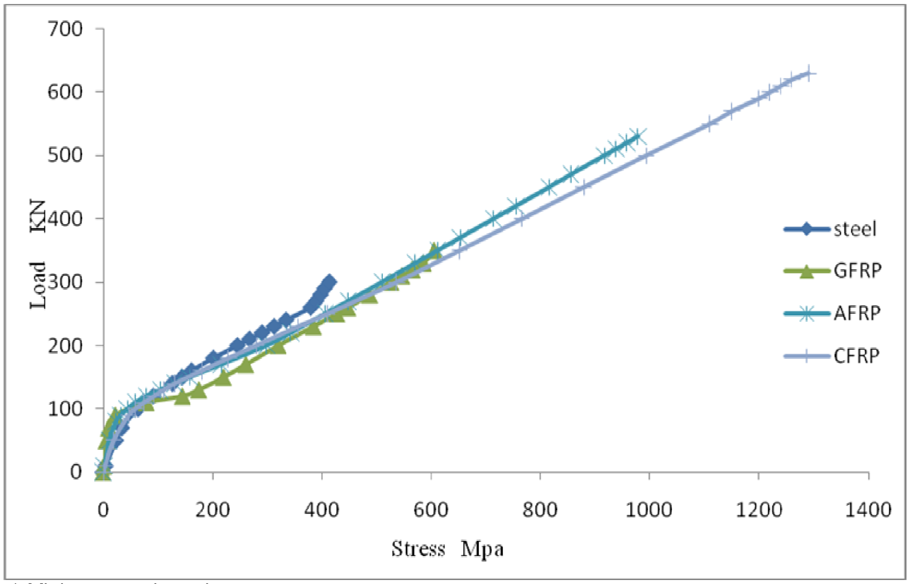


a) Achieve ultimate load of 300 kN

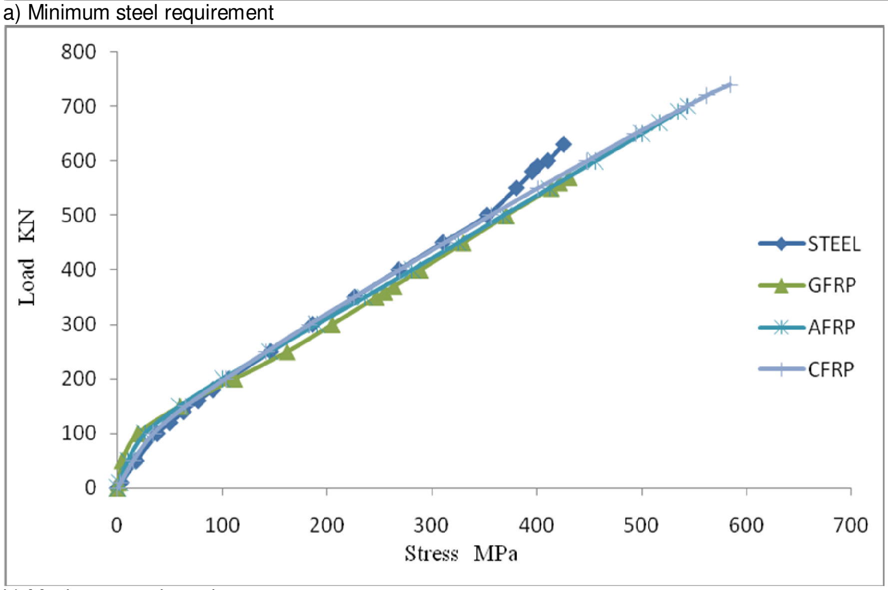


b) Achieve ultimate load of 300 kN

Figure 4. Load vs. deflection curves for the bridge girder with different reinforce cross-section area.



a) Minimum steel requirement

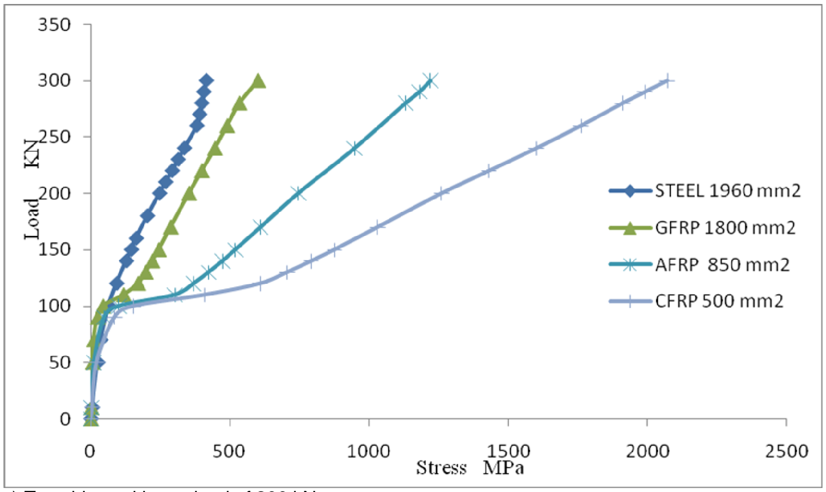


b) Maximum steel requirement

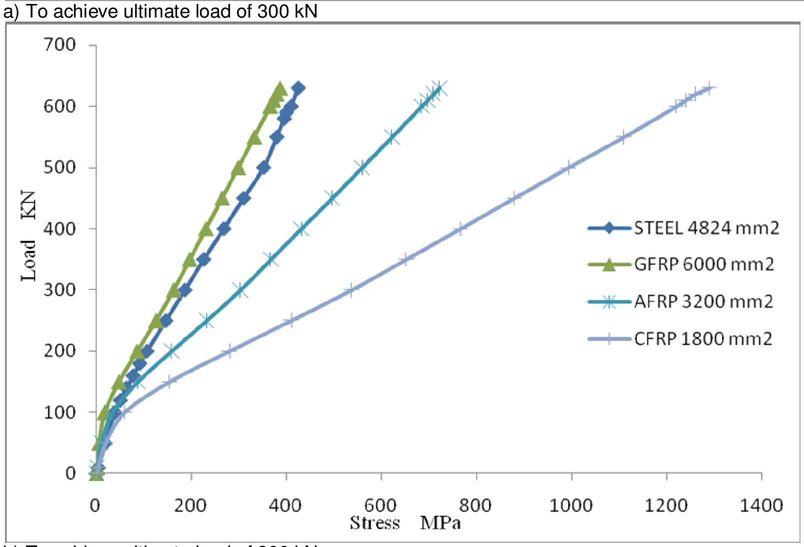
Figure 5. Load vs. stress curves: girder with different reinforce materials.

less and less with an increase of the tensile stress of the materials. These figures show that the required area became smaller with the increase in the tensile stress. When CFRP is used, the reduction was around 75% while the reduction was around 56% when AFRP was used. Using GFRP to attain 630 kN increases the required area around 24% and because the difference between the tensile stress of GFRP and steel is less than the difference between the modulus of elasticity for these two materials. Figure 5 shows the load stress curves for the lower and upper design required areas of the different materials.

The curves show that the relation is the same for all the materials used. The load of 100 kN was a reversal point at which the micro-cracks grow to cross the reinforced position. Before the reversal point, the concrete is still resistant to cracks and this protects the reinforced material. That is why at this zone, the reinforced material still has small values of stress. After the cracks cross the reinforced position, stresses at the tension zone are taken by the reinforced material, which can be seen as the stress increases gradually after the reversal point. Figure 6 shows the load stress curves for the girder with different materials and different areas to achieve the load capacity of 300 and 630 kN. These curves show that the stress increased gradually after the reversal point as we mentioned previously. They also show that the CFRP has the maximum stress due to the minimum area needed to



a) To achieve ultimate load of 300 kN



b) To achieve ultimate load of 300 kN

Figure 6. Load vs. stress curves: girder with different reinforce materials and cross-section areas.

Table 2. Analysis results: Girder reinforced according to the minimum reinforce requirement.

Material	Area (mm^2)	Crack Load (KN)	Failure Load	Failure mode
Steel	1960	60	300	Yield of steel
GFRP	1960	50	350	Rupture of fibre
GFRP	1800	50	300	Rupture of fibre
AFRP	1960	50	530	Crushing of concrete
AFRP	850	50	300	Rupture of fibre
CFRP	1960	60	630	Crushing of concrete
CFRP	500	50	300	Rupture of fibre

attain the required capacity. This is related to the high tensile stress of CFRP.

Tables 2 and 3 show the materials and area used, concrete micro-crack load, and failure mode of the girder with both design code requirements. The results showed that the failure mode of the different materials for the minimum design requirement depends on the tensile stress of the material. Most of the girders failed by the steel yielding or the fibres rupturing when we used material with lower tensile stress or less cross sectional

Table 3. Analysis results: Girder reinforced according to the maximum reinforce requirement.

Material	Area (mm^2)	Crack Load (KN)	Failure Load	Failure mode
Steel	4824	80	630	Yield of steel
GFRP	4824	60	570	Crushing of concrete
GFRP	6000	60	630	Crushing of concrete
AFRP	4824	60	700	Crushing of concrete
AFRP	3200	60	630	Crushing of concrete
CFRP	4824	70	740	Crushing of concrete
CFRP	1800	60	630	Crushing of concrete

area. CFRP and AFRP with area according to the minimum code requirement failed by crushing of concrete due to large values of tensile stress. On the other hand, the results showed that the failure mode according to the maximum design requirement was by crushing of concrete, and this is related to the balance state of stresses along the cross section when the concrete has reached the ultimate strain state and the reinforced material still has enough stress to prevent the rupture.

Conclusions

Based on the present study, some conclusions can be drawn about replacing steel with FRP for reinforcing concrete as follows:

1. Replacement of steel with different FRP types depends on the tensile stress and modulus of elasticity of FRP.
2. Replacement with FRP gives a high increase in load capacity especially when high tensile stress fibre is used; however, it gives more deflection than with steel. This becomes a limit for the serviceability requirements.
3. Replacement with FRP gives a high reduction in the area required to attain design load especially when high tensile stress fibre is used. This is considered an economical factor for choosing the fibre.
4. For the upper design requirement, the crushing of concrete still controls the failure mode even when FRP is used with the equivalent area to attain the required design capacity. This gives necessity to avoid the design of the structural elements with upper design requirements.

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