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Experimental study on bridge decks reinforced with GFRP rebars

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ABSTRACT: Currently, FRP composites have been widely used as internal reinforcement for concrete bridge decks. However, a few experimental researches on the behavior of FRP-reinforced members are available. This paper addresses the features and results of an experimental study for FRP-reinforced bridge deck conducted to examine the applicability of a newly developed GFRP rebar on concrete structures. For the test, a steel-reinforced bridge deck with length of 4,000 mm, width of 3,000 mm and thickness of 240 mm was fabricated with respect to the Korean Bridge Design Code and two decks with GFRP reinforcements instead of steel bars were also fabricated. A point load was applied to the deck specimens through a rectangular steel plate with dimensions of 231 mm × 577 mm to simulate the tire contacting area of a wheel of a design truck. The decks were supported by two steel I-beams spaced at 2,200 mm and tests were performed under static loading until failure. Comparison of the load-displacement relationships and crack behaviors observed in this test revealed that, even if the deck which has reinforcement replaced by GFRP rebars exhibited 89% of load carrying capacity of the steel-reinforced one with same reinforcement ratio and satisfied the service limit state, the GFRP-reinforced deck might be used as highway bridge decks.

1 INTRODUCTION

The deterioration of concrete bridges exposed to aggressive environment develops primarily through the corrosion of steel bars. Among the solutions proposed to solve such problem, the use of non-corrosive material for the reinforcing bar (rebar) instead of steel bar has been suggested and researches aiming the development of rebar or tendon using fiber reinforced polymer (FRP) have been conducted by numerous researches (Noritake et al. 1993, Faza et al. 1997, Djamaluddin et al. 2004). Currently, several commercial products have been developed and are actually adopted in field applications (El-Salakawy et al. 2003a, b, Benmokrane et al. 2004).

Design codes are indispensable for FRP to be used as reinforcing material in concrete structures and relevant researches are under course to provide such codes. Several design codes and guidelines have recently been established, which allow the use of FRP bars as main reinforcement for concrete structures (CSA 2002, CHBDC 2006, ACI 2006). However, there are many subjects under corrections and discussions (Ospina & Nanni 2007).

Bridge decks constitute one of the most promising structural components for which FRP can be applied extensively. Accordingly, this paper addresses the features and results of an experimental study conducted to verify the applicability of a newly developed glass fiber reinforced polymer (GFRP) rebar on bridge deck. A total of three full-scale bridge decks were fabricated and subjected to one point static load until failure. Main variables are the type of rebar and the ratio of transverse bottom reinforcement. Test results were compared in terms of deflection, strain, and crack width at service and ultimate load levels.

2 EXPERIMENTAL INVESTIGATION

2.1 *Test specimens*

Figure 1 illustrates the dimensions of the deck specimens that are width of 3,000 mm, length of 4,000 mm and thickness of 240 mm. The width and thickness of the deck correspond to values usually adopted in Korea, and the length was chosen so as to include sufficiently the portion influenced by the wheel load. The deck was supported by two steel I-beams (600×300 mm) and steel studs were used as shear connector. The I-beams were spaced at 2,200 mm and braced by steel channels so as to introduce edge restraint.

Reinforcement ratio was calculated accordance with the Korea Highway Bridge Design Code (KHBDC 2005) and resulted in using 15.9 mm of diameter rebars spaced at 200 mm in the bottom transverse direction with a reinforcement ratio of 0.546%. For the bottom longitudinal and the top both directions, the spacing of 250 mm (reinforcement ratio 0.436%) was used. The reference specimen (Specimen D0) was fabricated as a deck with conventional deformed steel rebars. Two GFRP-reinforced decks were fabricated with dimensions identical to the steel-reinforced deck of which one deck (Specimen D1) has the reinforcing bars replaced with GFRP rebars, and the other one (Specimen D2) has double reinforcement ratio.

2.2 *Material properties*

The specimens were fabricated using normal-weight concrete with an average compressive strength of 30 MPa. This value was obtained at the same date of the test through compressive strength test with cylinder specimens (100 × 200 mm) which were exposed to the same environmental conditions as their reference decks.

Newly developed GFRP rebar shown in Figure 2 was used in this study (KICT 2006). The GFRP rebar has spiral deformation provided by the wrapping of a braided strand composed by fibers in order to improve the bonding performance between concrete and rebar. The bar was fabricated by using continuous longitudinal E-glass fibers impregnated in a thermosetting vinylester resin by the pultrusion method. The dimensions of the adopted GFRP rebar are a diameter of 15.9 mm (D16), a pitching of 14 mm and deformation height of 1.28 mm.

The tensile and bond properties of the GFRP rebar were determined by performing tensile and tests on representative specimens in accordance with CSA S806-02 (2002) and ACI (2004), respectively. The mean tensile strength and elastic modulus of this rebar are 1,066 MPa and 47.8 GPa, respectively. The nominal tensile strength of rebar obtained by subtracting three times of standard deviation to the mean tensile strength was 988 MPa. The mean bond strength in a concrete block with compressive strength of 30 MPa and dimensions of 200×200×200 mm is 10.8 MPa for the embedded length of five times of diameter.

The deformed steel rebar used for the steel-reinforced concrete deck has diameter of 15.9 mm with yielding strength and elastic modulus of 400 MPa and 200 GPa, respectively.

2.3 *Test set-up*

The deck specimens were tested under a point load applied at the mid-span as illustrated in Figure 3. The load was applied through a steel plate (231 × 577 mm), which is equivalent to the tire contacting area of a wheel specified in KHBDC (2005).

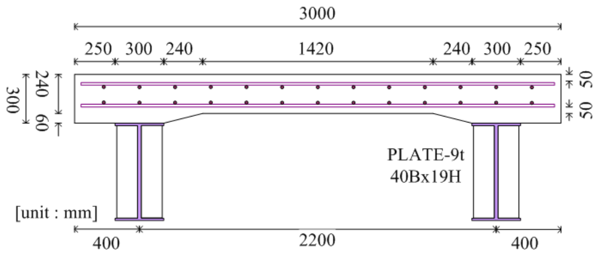

Figure 1. Dimensions of cross-section of deck

Figure 2. GFRP rebar

Prior to the failure test, the load was applied to the specimen with small increment until visible concrete crack has occurred. Omega-shaped extensometers were installed at the crack locations to measure the width of crack. The load was reduced to zero and then applied monotonically with displacement control until failure of the specimen. During the test, loading was stopped several times to mark the development of cracks at the bottom of the deck. The load was applied using an actuator with capacity of 3,000 kN at a loading rate of 0.5 mm/min.

Electrical resistance strain gages were mounted at the surface of the rebars and concrete to measure the strains. In addition, the deflection at the bottom of the deck was measured using six linear variable displacement transducers (LVDTs).

3 TEST RESULTS AND DISCUSSION

3.1 *Deflection characteristics*

The results of tests are summarized in Table 1. Figure 4 shows the load-deflection behavior measured at the center of the specimens. The service load of 124.8 kN was calculated using the maximum wheel load of 96 kN with impact factor of 0.3 according to KHBDC (2005).

All deck specimens showed punching shear failure after occurrence of flexural cracking. The load levels measured at punching failure were 845 kN, 755 kN, and 870 kN for Specimens D0, D1, and D2, respectively. The punching load level of the GFRP-reinforced deck is approximately 89% of the steel-reinforced deck with the same reinforcement ratio. If the reinforcement ratio for the GFRP-reinforced deck was doubled, it became 103%.

It was reported by the researches of El-Gamal et al. (2004, 2005) that the type of rebar nor the reinforcement ratio (that is, the flexural stiffness of reinforcement) do not significantly affect the deflection behavior of the restrained decks. However, the deflection observed in Specimen D1 was larger than that of Specimen D0 although both specimens had same reinforcement ratio, and that difference increased with larger load level. Even if slight differences exist for the properties of the specimens and their restraint methods, the difference observed in the test results seems attributable to the flexural stiffness (reinforcement ratio times elastic modulus) of Specimen D1, which was smaller by about 24% than that of the steel-reinforced deck, while El-Gamal adopted FRP-reinforced decks presenting flexural stiffness more than 70% of steel-reinforced deck.

Table 1. Summary of test results

Deck	Ultimate load (kN)	Max. deflection (mm)		Max. strain $(\times10^{-6})$		Max. crack width at service (mm)	Failure mode
		Service load	Failure	Service load	Failure		
D0	845	0.66	14.8	136	7,058	0.06	Punching
D1	755	0.99	19.6	176	5,278	0.18	Punching
D2	870	0.74	15.4	42	2,219	0.07	Punching

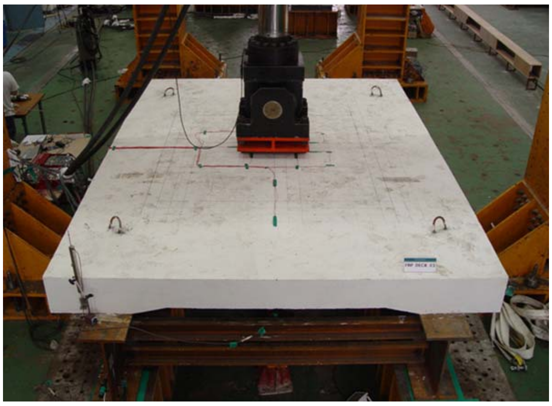

Figure 3. Test set-up

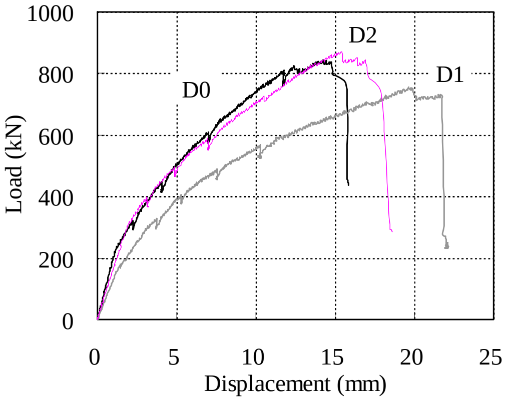

Figure 4. Load-deflection behavior

This presumption is confirmed by the results of Specimen D2 with increased flexural stiffness reaching about 48% of that of Specimen D0, where the load-displacement behavior was similar to the steel-reinforced deck as shown in the results of El-Gamal. Accordingly, additional experimental studies regarding to the restraint method, span length, and reinforcement ratio are necessary to examine the behavioral characteristics of GFRP-reinforced decks.

The measured deflections under the service load were 0.66, 0.99 and 0.74 mm respectively for the steel- and GFRP-reinforced decks, which are less than the allowable value (L/800 = 2.38 mm) recommended by KHBDC (2005).

3.2 *Cracking behavior and crack widths*

Figure 5 shows the cracking patterns of the specimens at the failure. Specimens D0 and D2 exhibited a large number of cracks propagating radially despite of relatively small crack widths, whereas Specimen D1 with reinforcement ratio equal to Specimen D0 showed a small number of cracks but relatively large crack widths. This can be explained by the poor distribution of the load provoking the local concentration of the load in Specimen D1 induced by the low modulus of elasticity of the GFRP rebar and the different bonding characteristics from the conventional steel one.

Figure 6 shows the load versus maximum crack width of the specimens. Specimen D1 developed larger crack width than Specimen D0 under similar load level. On the other hand, the width of crack developed in Specimen D2 was similar to that of Specimen D0 until approximately 300 kN but reduced maximum crack width beyond 300 kN. This behavior can be attributed to the denser arrangement of rebars in Specimen D2 compared to Specimens D0 and D1, which helps the redistribution of load.

FRP being a non-corrosive material, FRP rebar may constitute a fair alternative in the case where corrosion of steel becomes the major factor in the crack width limitation. Most design codes in the world specify the crack width limits for the reinforced concrete structures. However, allowable crack widths for FRP-reinforced concrete members are given in few number of design codes. Among them, Canadian Highway Bridge Design Code (CHBDC 2006) proposes a value of 0.5 mm for structures exposed to harsh environment and a value of 0.7 mm for other members. KHBDC (2005) is still not giving any crack width limitations for FRP-reinforced concrete members but only prescribes the use of values corresponding to 0.005 and 0.0035 times the cover thickness for steel-reinforced concrete members under normal and highly corrosive environments, respectively.

As shown in Table 1, Specimen D0 satisfied the allowable crack width with a value of about 0.068 mm under the service load level specified in design guidelines for steel-reinforced members. The crack widths of GFRP-reinforced decks being 0.139 mm and 0.074 mm also satisfied it proposed in CHBDC (2006). Moreover, these values also satisfied the allowable crack width specified in KHBDC (2005) for steel-reinforced concrete structures.

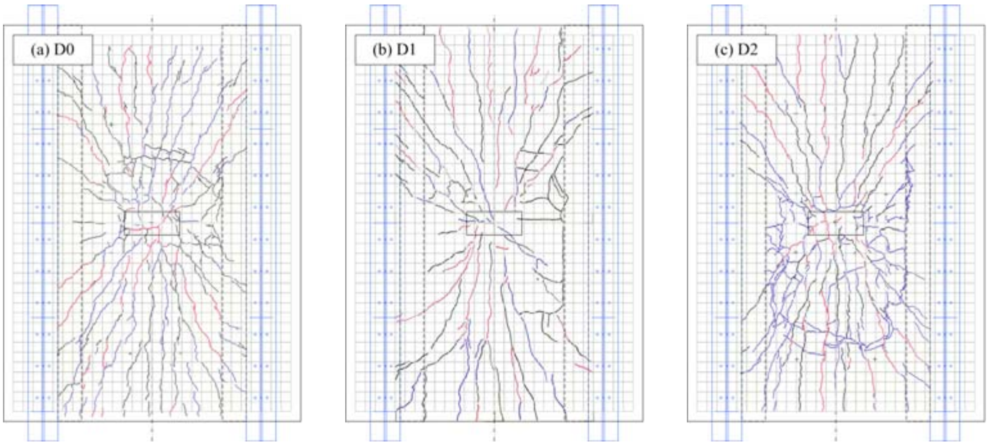

Figure 5. Crack pattern at the bottom surface of the specimens

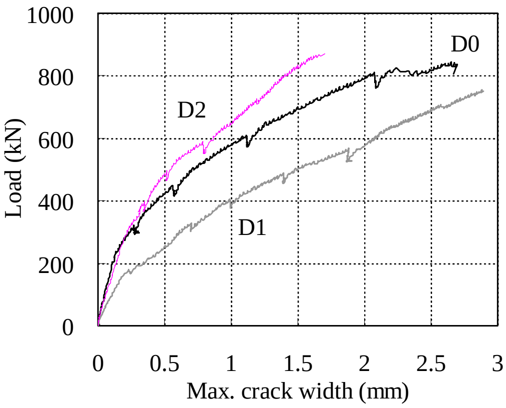

Figure 6. Load-maximum crack width behavior

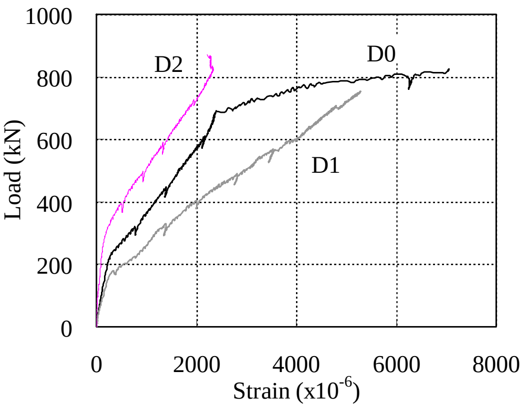

Figure 7. Load-strain behavior

3.3 *Strain characteristics*

Figure 7 shows the strains of the reinforcing bars measured in the bottom transverse reinforcement according to the applied load. After degradation of the stiffness of the decks due to cracking, the GFRP rebar behaved linear-elastically until punching failure, while steel rebar yielded at about 684 kN after exhibiting linear behavior.

ACI (2006) specifies the use of a value corresponding to the guaranteed tensile strength multiplied by the environmental reduction factor for the tensile strength of FRP rebar. Considering this environmental reduction factor of 0.7, the tensile strength adopted for the GFRP rebar is 691.3 MPa, and the ultimate strain is 14,450×10^{-6}. The strain of the GFRP rebar measured at punching failure of Specimen D1 was 5,278×10^{-6}, which reached merely 37% of the ultimate strain (about 15% for Specimen D2).

Moreover, ACI (2006) recommends the stress limit for the creep rupture of GFRP rebar corresponding to 20% of the tensile strength (strain of 2,890×10^{-6} for the adopted GFRP rebar). As shown in Table 1, this condition is satisfied since extremely small strains were measured in the rebars under the service load level (respectively 176×10^{-6} and 42×10^{-6} for Specimens D1 and D2).

4 CONCLUSIONS

One steel-reinforced deck and two GFRP-reinforced decks have been fabricated and tested to investigate the behavioral characteristics of decks reinforced with GFRP rebars. The results of this study can be summarized as follow.

1) The deck specimens reinforced with GFRP rebars showed the punching failure and overall behavior was similar to that of the steel-reinforced deck. However, the crack pattern observed in GFRP-reinforced deck with reinforcement ratio identical to the steel-reinforced deck exhibited smaller number of cracks with relatively larger crack width compared to the steel-reinforced deck in which a large number of cracks were propagating.
2) If the case where GFRP rebar simply replaced steel one in deck (Specimen D1) with reinforcement ratios of 0.546% and 0.436% for the bottom transverse reinforcing bar and other rebars respectively, the ultimate load carrying capacity of the GFRP-reinforced deck ran around 89% compared to the steel-reinforced deck. On the other hand, when the reinforcement ratio for the GFRP-reinforced deck was doubled, it became 103%.
3) For the reinforcement ratio equal to that of the steel-reinforced deck, the deflection of the GFRP-reinforced deck was larger than the steel-reinforced deck under the same load

level. However, if the reinforcement ratio was doubled for the GFRP one, similar values were measured and smaller maximum crack widths occurred with the steel one.

4) The serviceability limits (deflection, crack width) relevant to the steel-reinforced deck were also satisfied by the GFRP-reinforced deck with the same reinforcement ratio.

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