Composite Behaviour of a Hybrid FRP Bridge Girder and Concrete Deck

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Abstract: This paper involves experimental investigation onto the composite behaviour of a hybrid FRP bridge girder with an overlying concrete deck. Two types of shear connections were investigated: epoxy resin adhesives alone and epoxy resin combined with steel u-bolts. The results showed that the steel u-bolts combined with epoxy resin provided a more effective connection; hence a full-size specimen was prepared based on this result. Four-point bending test was carried out to determine the behaviour of a full-scale composite hybrid FRP girder and concrete deck. The composite action resulted to a higher stiffness and strength with the hybrid FRP girder exhibiting higher tensile strain before final failure. There was a significant decrease in the compressive strain in the top flange of the FRP girder thereby preventing the sudden failure of the beam. The composite beam failed due to crushing of the concrete followed by shear failure in the top flange and web of the FRP girder.

Key words: hybrid FRP, fiber composites, concrete deck, composite action, shear connector.

1. INTRODUCTION

Fiber reinforced polymer (FRP) composite material which has been used extensively in the aircraft and automotive industries has provided a solution for civil engineering structures subjected to harsh environment. The high strength, light weight, non-corrosive properties, rapid and economical construction make the FRP composite an attractive material for bridge construction. However, due to their high cost, the advantages of FRP material can only be realized when new design concepts and structural systems with improved durability, reduced life-cycle costs and short construction period are developed.

A hybrid FRP composite for bridge girder application is now being developed in Japan. The innovative feature of this composite girder optimizes the combined use of carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) for maximum structural performance while minimizing the production cost (Mutsuyoshi *et al.* 2007). It is believed that this material can be competitive in the near future when life-cycle cost is taken into account. In view of this, several studies had been undertaken to advance the application of the hybrid FRP composite to bridge girder. The fundamental flexural behaviour of I-shaped FRP girders was studied as a first step of the project. However, the results of this investigation showed that the hybrid FRP girder exhibited large deflection and failed without utilizing the material's tensile strength fully. The separation of the laminates between the CFRP and the GFRP and the local buckling in the compressive flange caused the sudden failure of the girder. According to Bank (2006), the ultimate strength of the FRP material is not realized when local buckling or delamination occurs. These failure modes are more likely to control the design structures. Thus, further improvement and development are needed to increase the capacity of the hybrid FRP composite section.

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Many studies suggest that combining FRP composites with high stiffness and/or high compression strength of conventional materials has been proven effective (Keller *et al.* 2007). These have resulted in many research and developments focusing on hybrid systems that combine FRP composites with concrete (Hollaway and Head 2001). One of the earliest studies to produce FRP/concrete beam element was undertaken by Fardis and Khalili (1981). In their study, the FRP box beam provided partial confinement in the compressive zone, carry tensile force and shear force while the concrete provided compressive strength and rigidity preventing local buckling of the FRP casing. Zhao *et al.* (2000) investigated the flexural behavior of concrete filled carbon shell girders with reinforced concrete bridge deck and pultruded hybrid FRP rectangular girders with polypropylene fiber reinforced concrete deck. Their results showed that the concrete aided in carrying the compressive forces and stabilized the thin walled carbon shell against buckling. Using the concept of composite construction, the first fiber composite bridge in Australia was developed and installed in a quarry site in Queensland (Van Erp 2005). The high compression capacity of plain concrete was combined with high strength and low weight characteristics of fibre composites to control serviceability deflection. In addition, the use of FRP as stay-in-place bridge deck panel has been explored for a number of years (ACI 2006). In Japan, several bridges using FRP-concrete hybrid deck systems had already been developed (Yamada and Nishizaki 2007). These different studies show that FRP materials are often suited for applications with concrete for bridge infrastructures.

In future bridge projects, it is likely that the hybrid FRP composites will be used as a main girder to support concrete decks. Thus, an experimental study on a hybrid FRP I-shaped girder with a compositely acting concrete which can form a part of the bridge deck is conducted. It is conceived that placing a concrete deck on top of a hybrid FRP girder to carry the compressive forces will prevent the local failure at the thin compression flange and will lead to the efficient utilization of the superior characteristics of this material. An effective connection between the FRP girder and concrete deck is investigated since it is anticipated that shear failure in the interface between the concrete and the FRP can easily occur.

2. EXPERIMENTAL PROGRAM

2.1. Material Properties

The hybrid FRP composite girder used in this study has a flange width of 95 mm and a depth of 250 mm. The top and bottom flanges of the hybrid girder were comprised of CFRP and GFRP of 14.0 mm thickness and while the web has 9.0 mm thickness of GFRP alone. The details of the hybrid FRP profile are shown in Figure 1 and laminate composition is listed in Table 1. Test of coupon specimens were conducted to determine the elastic modulus and strength properties of the hybrid FRP composite I-girder. The tensile test was conducted following the ASTM 3916 and ACI 440 while the compressive test was conducted based on ASTM D3410-87. The effective mechanical properties of the flanges and web of the hybrid FRP composite girder are listed in Table 2.

2.2. Static Bending Test of Hybrid FRP Girder

The flexural behaviour of the hybrid FRP composite girder was presented as a background for this study. However, only the behaviour of the hybrid FRP girder with composition shown in Table 2 was discussed as this is the type of FRP girder used in the experimental investigation of the composite behaviour with a concrete deck. Table 3 summarises the description of full-size hybrid FRP girders with and without concrete

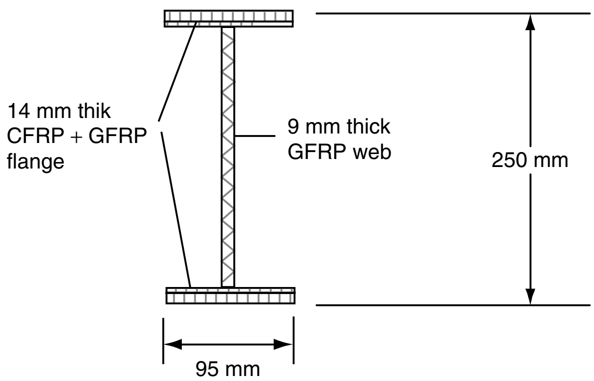


Figure 1. Details of hybrid FRP girder

Table 1. Composition of hybrid FRP girder

Types of lamina	Flange (%)	Web (%)
CFRP 0°	33	0
GFRP 0°/90°	29	31
GFRP ± 45°	13	42
GFRP Mat	25	27

Table 2. Effective material properties of hybrid FRP composites

		Tension		Compression	
Parameters	**Units**	**Flange**	**Web**	**Flange**	**Web**
Elastic modulus	N/mm^2	50,670	15,300	48,350	19,780
Poisson's ratio	mm/mm	0.235	0.27	0.269	0.297
Maximum stress	N/mm^2	884	185	347	203
Failure strain	%	1.4	1.4	0.4	1.0

Table 3. Description of specimens for hybrid FRP composite beams

		Size of concrete deck (mm)		
Specimen name	**Test span (mm)**	**Width**	**Thickness**	**Type of shear connector**
FRP-14 mm	3000	none	none	none
FRP-conc 1	1000	275	75	Epoxy adhesives
FRP-conc 2	1000	275	75	Steel u-bolts + epoxy adhesives
FRP-conc 3	3000	400	100	Steel u-bolts + epoxy adhesives

deck. In this table, the specimen FRP-14 mm corresponds to the hybrid FRP girder with 14 mm thick flanges tested without concrete deck.

Four-point static bending test was conducted to examine the flexural behaviour of specimen FRP-14 mm. The schematic illustration of the experimental set-up is shown in Figure 2. Third point loading was applied making both the loading and the shear spans as 1000 mm. A hydraulic jack with a 3000 kN capacity was used to apply the load through a spreader beam. Safety rigs were provided near the supports to prevent lateral collapse of the beam. Teflon sheet was inserted between the safety rig and the FRP girder to minimize the frictional effects in case of any contact during the test. The specimen was instrumented with strain gauges to measure the strains on the FRP girder. Displacement transducers were installed to measure the displacement at midspan of the composite girder and data logger was used to record the data with the load–midspan deflection curve displayed in real time during the test to monitor the beam behaviour. The deflection, strains and failure mode were recorded during loading and until failure of the specimen.

2.3. Shear Connection for Hybrid FRP Girder and Concrete Deck

The connection between the deck and the girder was cited as the main problem in composite structures (Hollaway and Head 2001). Thus, an appropriate

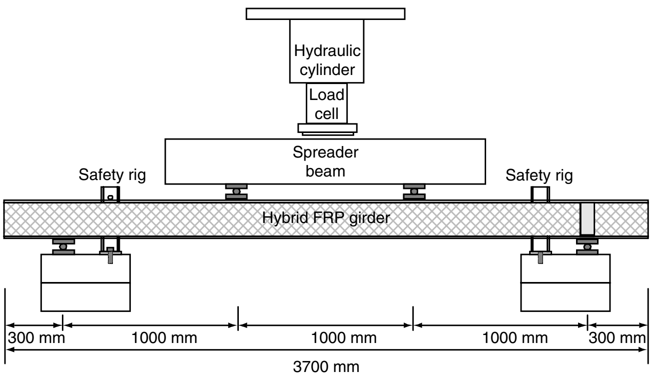


Figure 2. Schematic illustration of experimental set-up for FRP- 14 mm

shear connection between the interface of FRP girder and concrete deck is important to develop a composite structure. Similarly, the failure mechanism of a suitable shear connection should not be catastrophic. In literature, only a few examples can be found on this topic. Most studies on deck-to-girder connections involved FRP deck and girder made from conventional materials like steel, concrete and timber (NCHRP 2003; Cousins and Lesko 2004; Dagher 2005; Gurtler 2007). Hence, initial investigation on the behaviour of hybrid FRP girder with two differently bonded and compositely acting concrete decks was conducted.

Adhesive bonding, an adopted connection method for girder and deck system, and a combination of mechanical fasteners and adhesives to prevent debonding failure between the hybrid FRP girder and concrete deck were used in this study. Two 1.5 m long hybrid FRP girders with 1.0 m loading span were prepared as test specimens. FRP girder with an overlying precast concrete deck bonded with epoxy only (FRP-conc 1) and FRP girder with cast-in-place concrete deck connected with epoxy and steel u-bolts (FRP-conc 2) were subjected to three-point loading test. The concrete deck was 75 mm thick and 275 mm wide with a small amount of steel reinforcements (3 pieces 6 mm diameter) to facilitate casting. The concrete was cast simultaneously from a single batch to minimize the differences on the concrete strength between the two specimens. The concrete deck has a mean cylinder strength of 36 MPa obtained from compression test at 14 days (at the same age of testing the specimens). The dimensions and details of the specimens FRP-conc 1 and FRP-conc 2 are shown in Figures 3 and 4, respectively.

Epoxy resin adhesive with the help of 8 pieces 10 mm diameter bolts were used to connect the precast concrete deck while 10 mm diameter Grade 316 stainless steel u-bolts were used as shear studs as well as epoxy resins with gravel chips to develop full composite interaction between the hybrid FRP girder and cast in place concrete deck. Holes were drilled on the top flange of FRP girder to accommodate the steel u-bolts. Washers

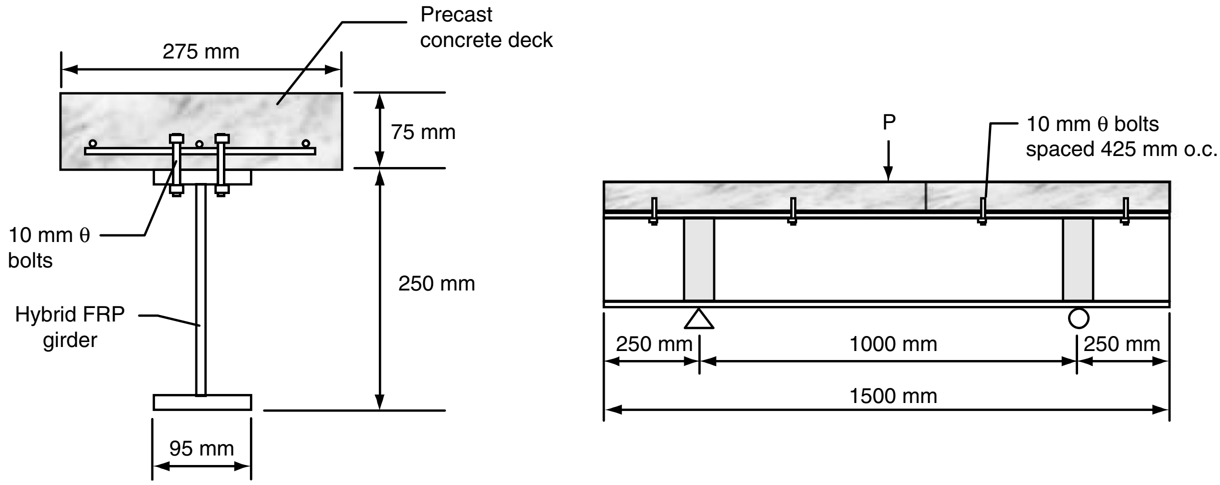


Figure 3. Specimen FRP-conc 1 dimensions and loading systems

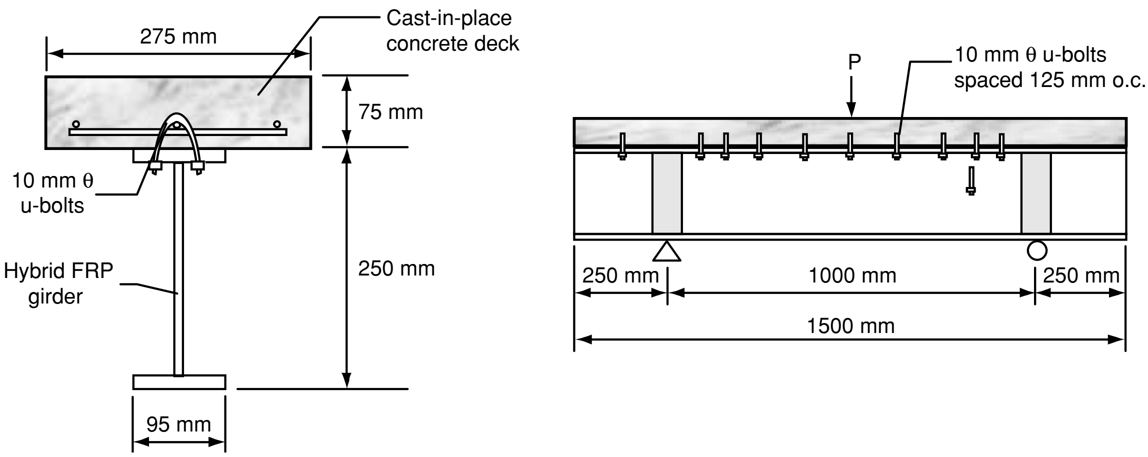
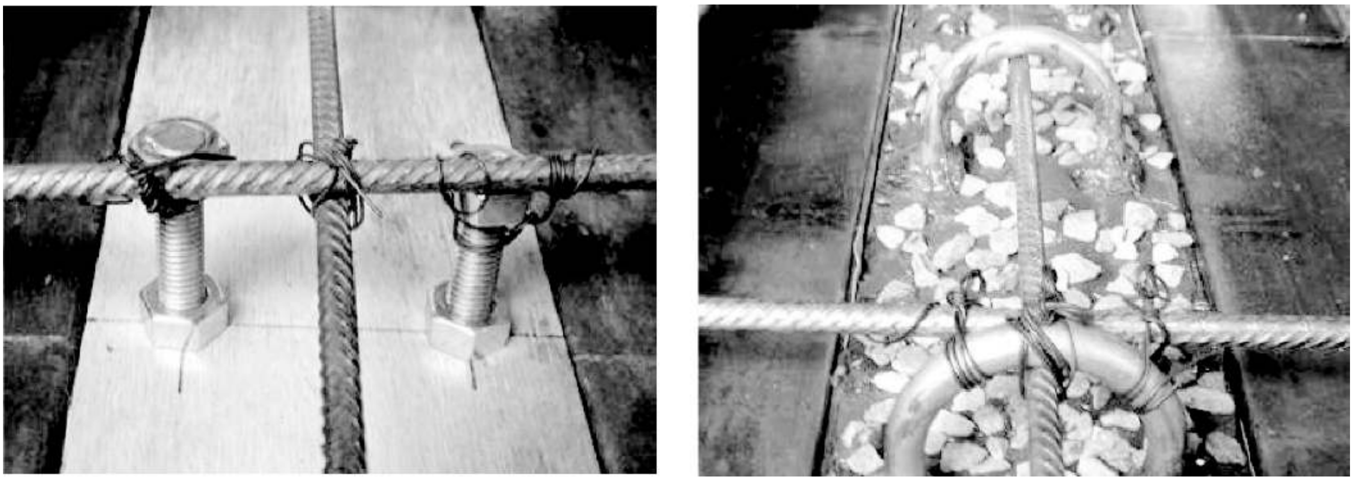


Figure 4. Specimen FRP-conc 2 dimensions and loading systems

were provided to the steel u-bolts to allow local stresses to spread more evenly. A two-part adhesive was mixed at 3:1 proportion and applied on the top flange of the hybrid FRP girder. Gravel chips were then distributed throughout the epoxy adhesives and cured for 24 hours. Figure 5 shows the actual shear connection for both specimens.

2.4. Full-Scale Test of Hybrid FRP Girder with Concrete Deck

Specimen FRP-conc 3 represents the FRP composite girder with cast-in-place concrete deck bonded both with epoxy and steel u-bolts. The length of the hybrid FRP girder is 3600 mm with a clear span of 3000 mm. Metal box stiffeners spaced at 333.33 mm on center were installed in the web to prevent premature failure and to ensure that only tensile failure will occur in the FRP girder. Grade 316 stainless steel u-bolts with a diameter of 10 mm and epoxy resin with gravel chips were used as shear connectors. The steel u-bolts were spaced at 150 mm on center between metal stiffeners. The concrete deck was 100 mm thick and 400 mm wide with steel reinforcement (5 pieces 16 mm diameter bars) to provide additional compressive force on the concrete section. Bottom reinforcements of 5 pieces of 16 mm diameter bars were used to delay the formation of tension cracks and to limit the crack width on the concrete deck. The width of the slab was kept at 400 mm based on the maximum length of the available steel loading plate. The concrete has a mean cylinder strength of 32 MPa obtained from compression test at 14 days (at the same age of testing the specimen). The steel reinforcement bars has an elastic modulus of 200 GPa and yield strength of 300 MPa. The dimensions and cross-sectional details of the specimen FRP-conc 3 are shown in Figure 6.



(a) FRP-conc 1 (b) FRP-conc 2

Figure 5. Shear connections between hybrid FRP girder and concrete deck

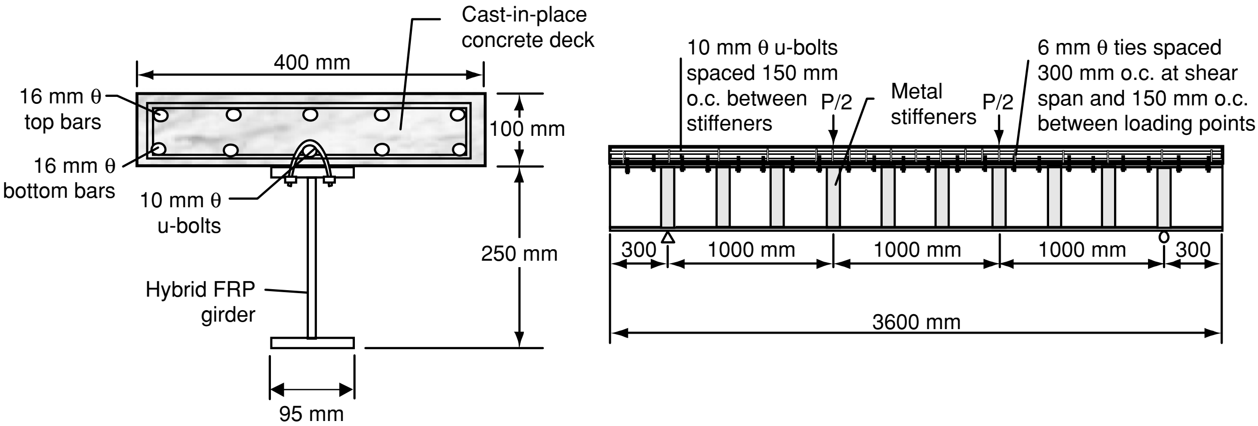


Figure 6. Dimensions and loading systems of specimen FRP-conc 3

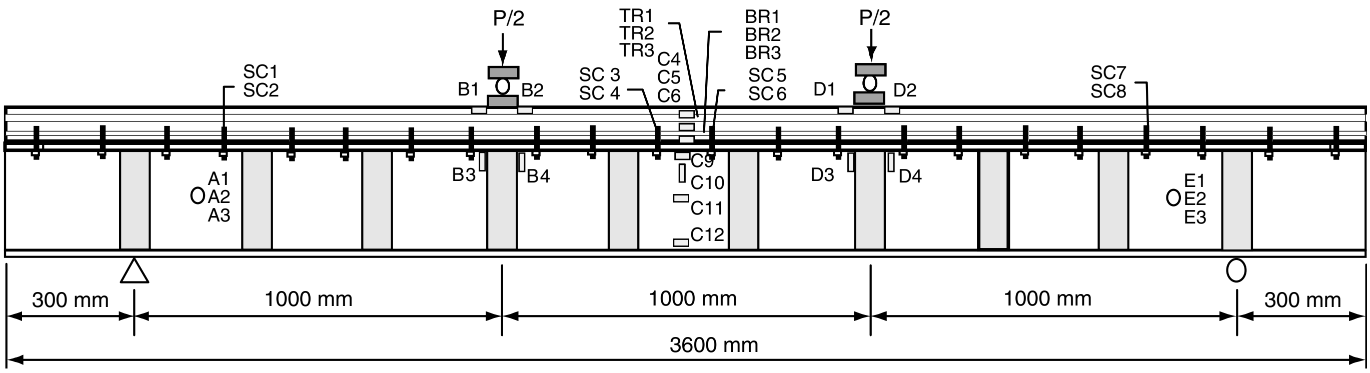


Figure 7. Location of strain gauges at specimen FRP-conc 3

A four point loading test was conducted on the hybrid FRP girder with compositely acting concrete deck following the set-up in Figure 2. Adhesive bonded single element metallic foil-type strain gauges were bonded on the FRP and concrete surfaces to measure the strain during the entire loading. Figure 7 shows the location of the strain gauges on the FRP-conc 3 specimen.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Flexural Behaviour of Hybrid FRP Girder

3.1.1. Load-deflection relationship

The load and midspan deflection of the hybrid FRP composite girder tested under four point loading is shown in Figure 8. The results showed that the hybrid FRP composite girder behaved linearly elastic with no signs of damage before final failure. The specimen FRP-14 mm failed at an applied load of 194.86 kN at a midspan deflection of 56.62 mm.

3.1.2. Load-strain relationship

Figure 9 show the relationship between the load and the strain of the top and the bottom flanges of specimen FRP-14 mm at the midspan. In this figure TF represents the average reading in the strain gauges attached to the top flange while BF represents the average strain gauge reading at the bottom flange of the FRP girder. The result showed that the measured strains in both tension and compression increased linearly with load. The specimen FRP-14 mm exhibited maximum tensile and compressive strains of 6,240 and 6,130 microns, respectively. These results showed that the measured strains at the top and the bottom flanges were almost equal throughout the test which indicates that the neutral axis of the section is located at the mid-height of the girder. It was expected therefore that failure will occur at the compressive flange since the hybrid FRP composites is stronger in tension than in compression.

The hybrid FRP specimen reached the recommended critical strain of 4,000 microstrains, the maximum strain type failure criterion in the compressive flange established from the test of coupons. This value represents the strain at which the delamination occur between the CFRP and the GFRP laminates under compression. However, the maximum tensile strains measured in specimen FRP-14 mm is only 45% of the recommended failure strains, respectively. This showed that the high tensile strength of the hybrid FRP laminates was not effectively utilized. This result further justified the addition of a concrete deck to provide stability on the thin compression flange. A section with a neutral axis located above the mid-height of the hybrid FRP girder would result to a higher tensile strain in the hybrid FRP girder and subjects the compressive flanges only to minor stresses.

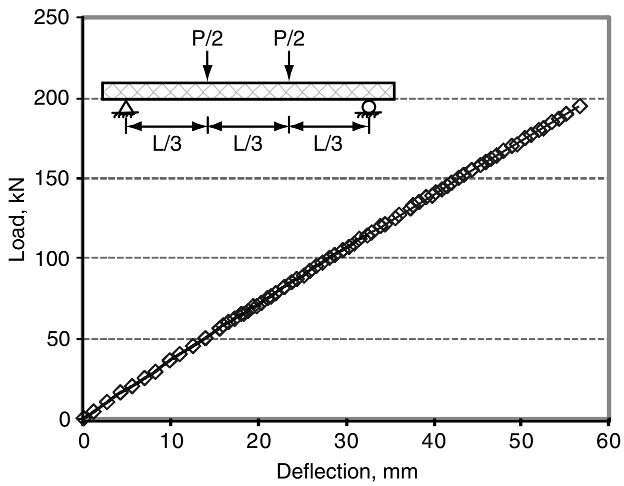


Figure 8. Load and midspan deflection relationship of specimen FRP-14 mm

3.1.3. Failure mode

Figure 10 shows the failure mode of specimen FRP-14 mm. The final failure mode was the delamination between the combined CFRP and GFRP and the GFRP laminates in the compression flange at the midspan

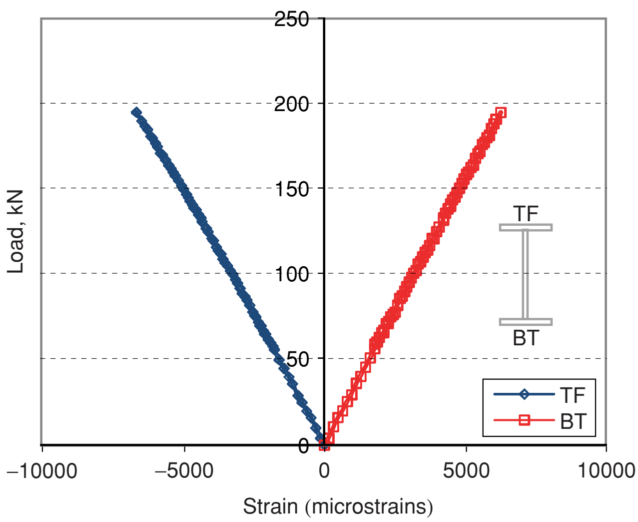


Figure 9. Load-strain relationship of specimen FRP-14 mm

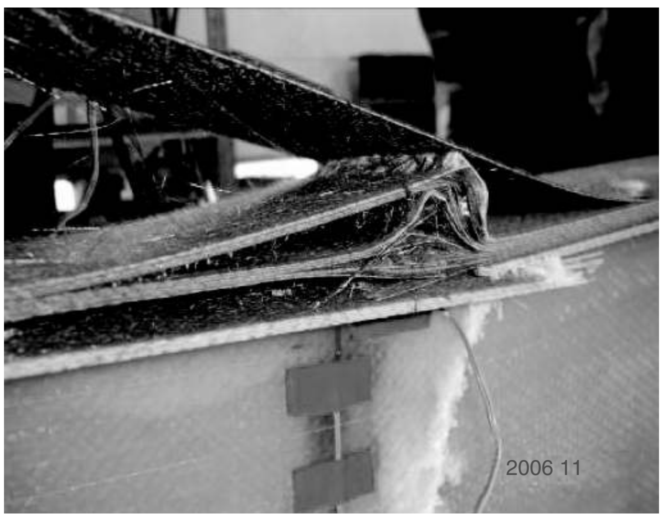


Figure 10. Delamination failure of specimen FRP-14 mm

followed by shear failure of the web. The hybrid FRP girder failed suddenly with no signs of damage before final failure. According to Bank and Yin (1999), the delamination between the stiff composites of the combined CFRP and GFRP and the comparatively soft GFRP is likely to occur due to their difference in stiffness and the high interlaminar tensile stress in the free edges of the compressive flange to separate the interface. It is essential therefore that this type of failure be eliminated to be able to efficiently use the hybrid FRP girder in bridge engineering applications.

3.2. Shear Connection

3.2.1. Load-deflection behaviour

The comparison of the load and midspan deflection of specimens FRP-conc 1 and FRP-conc 2 is shown in Figure 11. The result showed that the applied load in both specimens increased linearly with deflection until the cracking of the concrete at 180 kN (point A). However, a gradual decrease in the load was observed for both specimens at initiation of every debonding of the epoxy. The reduction started at a load of 200 kN in FRP-conc 1 (point B) and at 230 kN in FRP-conc 2 (point C). The debonding initiated the widening of the cracks in the concrete deck which resulted to a reduced effective section as shown by the load drop and the decrease in stiffness in the load-deflection curve. Noticeably, the load drop and the decrease in stiffness in both specimens occurred at almost the same amount of displacement.

Experimental results showed that FRP-conc 2 behaved slightly stiffer than FRP-conc 1 after concrete cracking until final failure. The adhesive bond provided by the epoxy enabled a smooth load transfer from the hybrid FRP girder to the concrete deck showing only a small drop in the load at every debonding initiation. At a load of 420 kN (point D), FRP-conc 1 failed due to the total debonding of the epoxy leading to the separation of the hybrid FRP girder and the concrete deck. In FRP conc-2, debonding of the epoxy occurred at an applied load of 400 kN at a mid-span deflection of 7.42 mm (point E). The load suddenly dropped to 350 kN but increased again. This result indicates that the u-bolts started contributing to the shear anchorage between the hybrid FRP girder and the concrete deck. The concrete flows to the opened holes created by the u-bolts forming dowels which may have provided the resistance. Similarly, reinforcement bars placed in the open holes with the concrete surrounding it improved transfer of horizontal shear forces between the hybrid FRP girder and the concrete deck. The section then failed at a load of 448 kN (point F) due to compressive crushing of the concrete under the loading point.

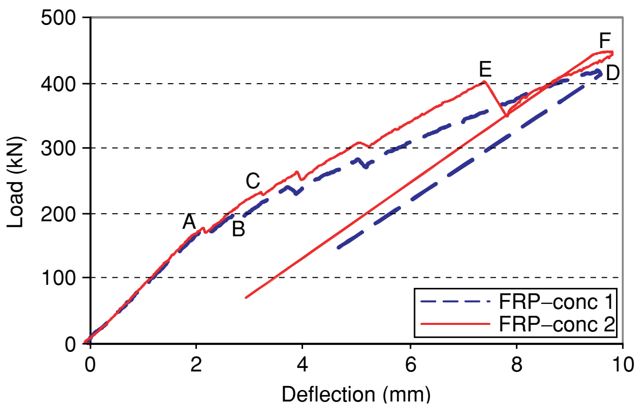
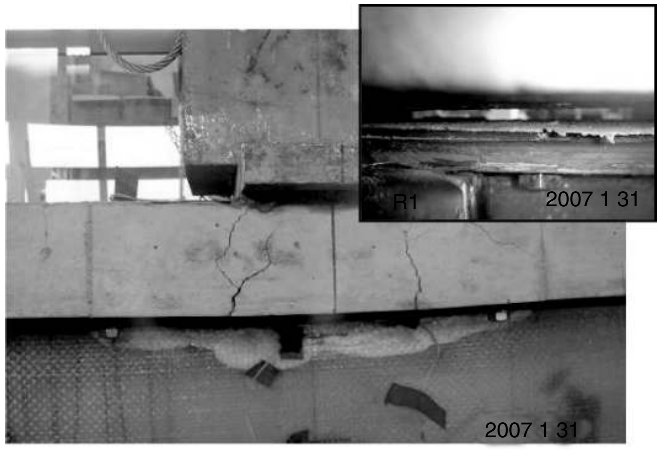


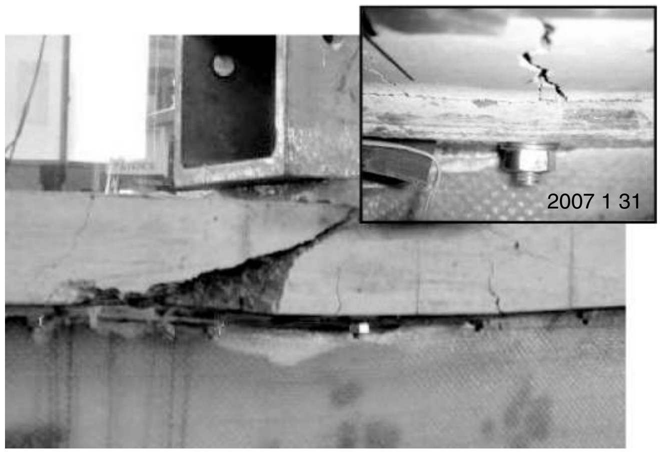
Figure 11. Load and midspan deflection of FRP-conc 1 and 2

3.2.2. Failure behaviour

Figure 12 shows the failure mode of the hybrid FRP girder with concrete deck. The shear crack in the concrete deck started at a load of 180 kN for both specimens due to lack of reinforcement. A sound was



(a) FRP–conc 1



(b) FRP–conc 2

Figure 12. Failure mode of hybrid FRP girder with concrete deck

heard at every debonding initiation between the hybrid FRP girder and concrete deck. The final failure of FRP-conc 1 was initiated by the total debonding of the epoxy between the hybrid FRP and concrete deck followed by buckling and compression failure in the web of the hybrid FRP girder. All the bolts embedded in the precast concrete deck to position the hybrid FRP girder were broken at failure. Similarly, failure of FRP-conc 2 occurred due to compressive crushing of the concrete followed by buckling and compression failure of the web. However, the concrete deck is still attached to the FRP girder even at final failure because of the presence of steel u-bolts.

It was observed that the failure mode of both composite beams was typically crushing of the concrete. This type of brittle failure mode is inherent to concrete due to the high strength of the FRP as tension reinforcing materials. The results also test showed that the use of steel u-bolts combined with epoxy resin and gravel chips provided a more effective shear connection than that of epoxy resin adhesive alone. This type of shear connection provided an almost full composite action and lead to a non-catastrophic type of failure. After the debonding of epoxy, an increase in the load carrying capacity was still observed, demonstrating the effectiveness of the steel u-bolts. The failure of the composite beam is typically due to the crushing of the concrete with no sign of local deformation observed in the hybrid FRP girder before final failure. In addition, failure in FRP-conc 2 with the concrete deck still attached to the FRP girder indicates the possibility of further capacity increase when a wider concrete deck and additional reinforcements are provided. It is based on this result that the full size specimen was prepared to determine the composite behaviour of a hybrid FRP girder and concrete deck.

3.2.3. FRP and concrete strain

The strains on the top and bottom flanges of the hybrid FRP composite girder without the concrete deck were approximately equal indicating that the neutral axis is in the middle of the girder section. In the composite hybrid FRP girder and concrete deck, the composite action increased the effective depth of the specimen and shifted the neutral axis towards the bottom of the concrete deck. Figure 13 shows the cross-sectional strain profile at the midspan of specimens FRP-conc 1 and 2 at different load levels. An approximate linear behavior was observed on both specimens at a lower load with the neutral axis remains roughly at the same location near the concrete slab bottom surface. Non-linearity in strain profile at higher loads was observed and a significant increase in strains on the top flange of the hybrid FRP after debonding of the epoxy, shifting the neutral axis away from the concrete deck.

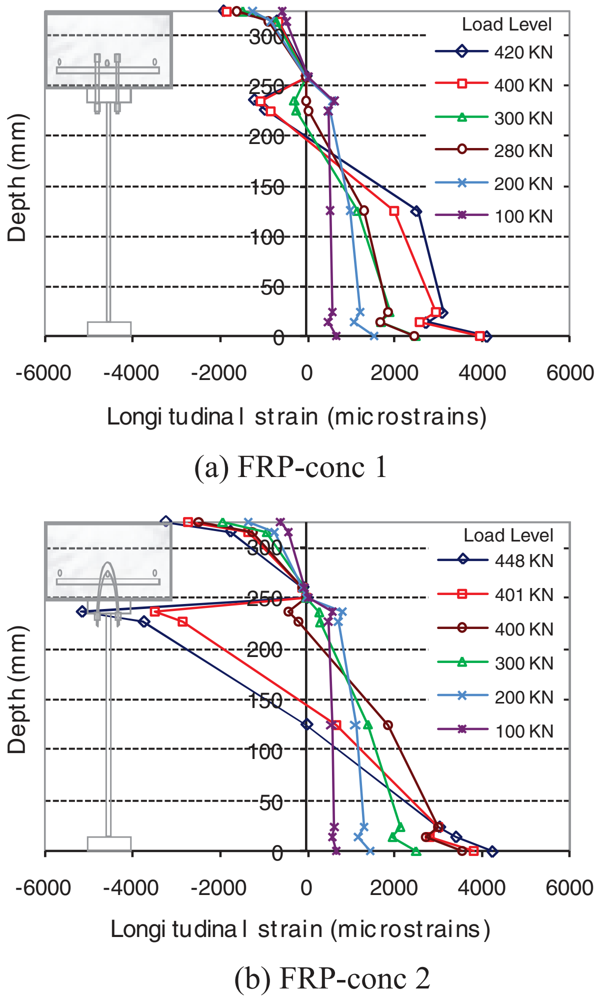


Figure 13. Longitudinal strain profile along the sections of FRP-conc 1 and FRP-conc 2

In the case of FRP-conc 1, the expected maximum strain in concrete of 3000 microstrains was not attained in the precast concrete deck due to debonding failure. The measured maximum strain prior to debonding was only 2400 microstrains in concrete and 4400 microstrains on hybrid FRP girder. For FRP-conc 2, the steel u-bolts eliminated the possibility of total bond failure but instead lead to the compression failure of the concrete at a strain of 4100 microstrains with the maximum measured strain on the hybrid FRP girder was 4500 microstrains. Since total debonding at the interface of the hybrid FRP and the concrete deck was prevented, higher strain was measured on the FRP. This shows that by taking advantage of the high compressive strength of concrete, the high tensile strength of hybrid FRP girder can be better utilized.

3.3. Full-Size Hybrid FRP Girder with Concrete Deck

3.3.1. Load-deflection behaviour

The load and midspan deflection behaviour of a full-size hybrid FRP girder with an overlying concrete deck is shown in Figure 14. Based on the figure, the load capacity of FRP-conc 3 increased linearly with deflection until an applied load of 196 kN and a reduction in stiffness was observed until final failure. The reduced stiffness can be attributed to the development of diagonal cracks within the shear span which contributed to the downward deflection of the beam. The decrease in stiffness could also be due to some slips which took place in the experimental beam. Before the ultimate failure, widening of the cracks in the concrete deck and an increased amount of deflection were observed even without an increase in the applied load. Failure of the specimen was reached at this point hence the test was stopped. When the applied load was released, the beam did not return to its initial height indicating that the steel reinforcements had buckled.

FRP-conc 3 failed due to the crushing of the concrete at the shear span followed by shear failure of the top flange and web of the FRP girder at an applied load of 427 kN with a mid-span deflection of 73.9 mm. Consequently, the result of the experimental investigation showed that the composite action with a concrete deck could overcome deflection limitations inherent in hybrid FRP girder and the higher load carrying capacity at final failure could be attained compared to hybrid FRP girder alone.

The measured deflection was compared with the code-stipulated deflection to determine its applicability to bridge structures. Based on the AASHTO guidelines, the maximum deflection design criteria is length/800 for highway bridges and length/500 for pedestrian bridges (Bank 2006). This means that for a 3000 mm length, the hybrid FRP girder with concrete deck is allowed to deflect only up to 3.75 mm and up to 6 mm for highway and pedestrian bridges. This corresponds to an applied load of 77 kN and 46 kN, respectively. These allowable deflection values correspond to an applied load of only 15 kN and 22 kN for hybrid FRP composite girder without concrete deck. This result showed that the composite action with a concrete deck could overcome deflection limitations inherent to the hybrid FRP girder and the structure can be designed with a higher load carrying capacity to conform to the serviceability limit for bridge girder applications.

3.3.2. Failure behaviour

Figure 15 shows the failure mode of the hybrid FRP girder with concrete deck. The development of diagonal cracks within the shear span of the concrete deck started at a load of 246 kN. The crack width started to increase with the continuous load application which leads to the compression failure of the concrete deck near the loading point followed by shear failure on the top flange and web

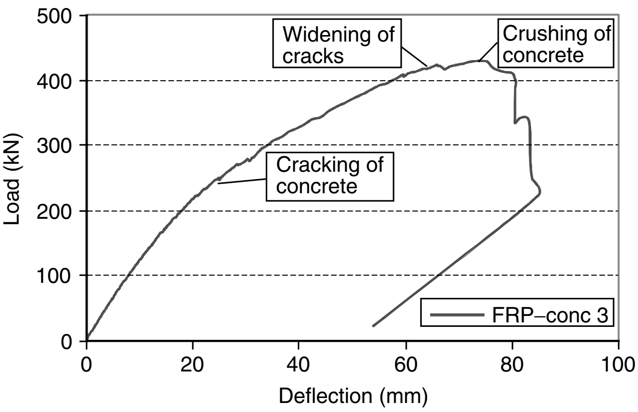


Figure 14. Load and mid-span deflection relationship of FRP-conc 3

(a) Compression failure of concrete deck

(b) Shear failure of FRP girder

Figure 15. Failure mode of specimen FRP-conc 3

of the hybrid FRP girder. The shear failure in the top flange of hybrid FRP girder may be due to stress concentration in the holes provided for the u-bolts. This is not the expected mode of failure as the composite beam was designed to fail by rupture of the FRP in order to determine the strain level where hybrid FRP girder will fail in tension. However in actual design, failure of the concrete may be a preferred failure mode because cracks in the concrete deck give an adequate warning of impending failure of the structure. This mode of failure was not catastrophic which is advantageous from the civil engineering point of view.

3.3.3. Load-strain relationship of hybrid FRP girder

The load strain relationship on the top and bottom flanges of the hybrid FRP girder tested with a concrete deck is shown in Figure 16. In this figure TF1-2-3 represents the average reading in the strain gauges attached to the top flange while BF1-2-3 represents the average strain gauge reading at the bottom flange of the FRP girder. An almost linear load strain relationship was observed in both the top and bottom flanges at lower load levels. The strain plots then became non-linear when the concrete cracked at a load of around 246 kN. The strains in the top flange of hybrid FRP girder shifted from positive to negative direction which indicated that the FRP girder started to carry compressive forces.

The composite action of the hybrid FRP girder and concrete deck resulted to a higher tensile strain in the FRP girder. The maximum tensile strain measured for FRP-conc 3 was 9050 microstrains compared to only 6245 microstrains for specimen FRP-14 mm. This level of strain is almost 65% of the ultimate tensile strain of hybrid FRP girder while only 45% of the ultimate tensile strain was achieved when tested without the concrete deck. Moreover, there is a significant decrease in the compressive strain on the top flange of the FRP girder thereby preventing the separation of the laminates and the sudden failure of the beam. This showed that the superior characteristics of the FRP material was efficiently utilized with the addition of the concrete deck in the thin compression flange. The result showed that in composite construction, the need for expensive CFRP in the upper flange of the hybrid FRP girder may be eliminated as it is subjected only to minor stresses.

3.3.4. Load-strain relationship on steel u-bolts and concrete under the loading points

Figure 17(a) shows the load-strain relationship of the steel u-bolts at shear span and at the constant moment region. In this figure, SC1–2, SC3–4, SC5–6 and SC7–8 represent the load strain relation attached to the

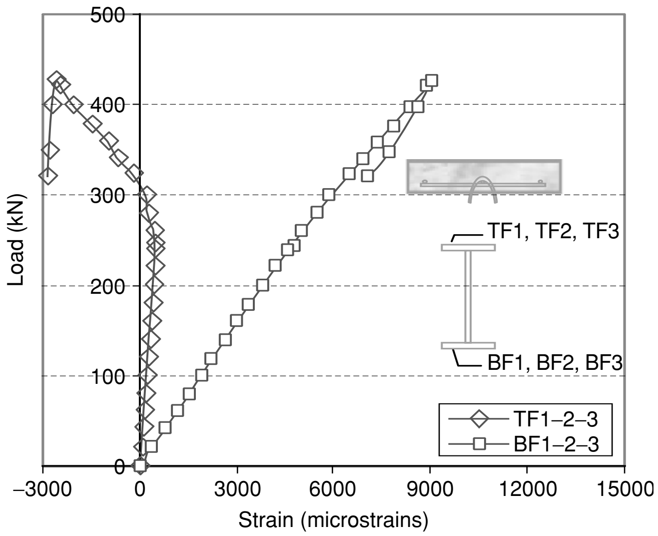


Figure 16. Load strain relationship of the top and bottom flanges of FRP-conc 3

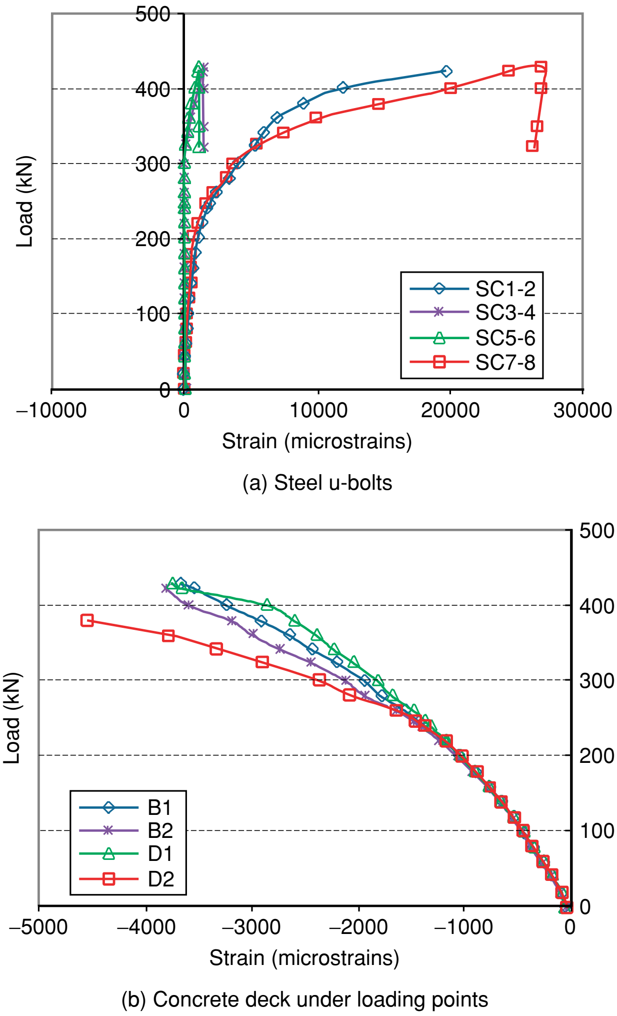


Figure 17. Load strain relationship of steel u-bolts and of concrete deck under loading points

steel u-bolts. Based on the figure, larger strain was developed in the steel u-bolts at shear span than at the constant moment region. This indicates that the steel u-bolts at shear span carried most of the horizontal force while the shear connector at midspan was subjected to only minimal force. This result is significant as it indicates that the number of u-bolts can be reduced by providing mechanical anchorages at the shear span only and eliminating the u-bolts at the constant moment region. This will reduce the cost of not only the materials but also in labor since there is no need to bore holes at the top flange of the girder at the constant moment region.

Figure 17(b) shows the load-strain relationship of the concrete deck under the loading points. Similar behavior was observed in all the strain gauges on the concrete deck under the loading points until the concrete cracked at an applied load of 246 kN. From this point on, the strain in all the gauges read differently until final failure. The top flange of the hybrid FRP girder started to carry compressive forces at a load of 246 kN based on Figure 16. At an applied load of 420 kN, the strain in the concrete deck under the loading point reached the maximum strain of 3000 microstrains and the top flange and the web of FRP girder failed in shear.

5. CONCLUSIONS

The investigation on the flexural behaviour of the hybrid FRP girder showed that failure of this girder was induced in the compressive flange with the tensile strength of the material far below its available tensile strength. Thus, it was conceived that the superior characteristics of FRP girder can be efficiently utilized if a stiffer material such as concrete is provided on the thin compressive flange which can form a part of the bridge deck.

A hybrid FRP girder with a compositely acting concrete deck was tested statically under four-point loading to examine the flexural behavior of this construction system. The result showed that the use of both steel u-bolts and epoxy resin with gravel chips provided a more effective shear connection than that of epoxy resin adhesive alone. This type of shear connection provided an almost full composite action and lead to a non-catastrophic type of failure. The composite action reduced the strain at the top flange due to the increase in the depth of the neutral axis. Thus, the combined use of steel u-bolts and epoxy adhesives can be a suitable shear connection between the hybrid FRP girder and the concrete deck to develop a composite bridge infrastructure.

The addition of the concrete deck on top of the hybrid FRP girder to resist compressive forces resulted not only to an increase in the stiffness and strength of the composite section but also to a reduced deflection at failure. There was a significant decrease in the compressive strain in the top flange of the FRP girder thereby preventing the separation of the laminates and the sudden failure of the beam. Consequently, the hybrid FRP girder exhibited high tensile strain showing that the FRP material was utilized more efficiently than using it alone. The composite beam failed due to crushing of the concrete in the shear span followed by shear failure in the top flange and web of FRP girder.

This experiment is an initial attempt to investigate the behavior of hybrid FRP girder with an overlying concrete deck. Nonetheless, the result of the experimental investigation has confirmed the importance of composite construction involving hybrid FRP girder and concrete deck. Consequently, a study to investigate the composite behaviour of hybrid FRP I-girder with Ultra high strength Fiber reinforced Concrete (UFC) deck is now being conducted for a more durable, lightweight and easy to install bridge structure. It is anticipated that this will eventually lead towards the development of an optimized hybrid FRP composite section with maximum structural performance for bridge girder application.

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