PRECAST ULTRA HIGH PERFORMANCE CONCRETE SLABS AND HYBRID FRP COMPOSITE GIRDERS FOR ACCELERATED BRIDGE CONSTRUCTION

Hai Nguyen, PhD, Research Associate, Nick J. Rahall II Appalachian Transportation Institute, P.O. Box 5425, Huntington, West Virginia 25703-0425, USA
Hiroshi Mutsuyoshi, PhD, PE, Professor, Department of Civil and Environmental Engineering, Saitama University, Shimo-Okubo 255, Sakura-ku, Saitama-shi 338-8570, Japan
Wael Zatar, PhD, Dean and Professor, College of Information Technology and Engineering; Director for Innovations and Asset Management of Sustainable Transportation Infrastructure Systems Program, Marshall University, 112 Gullickson Hall, One John Marshall Drive, Huntington, West Virginia 25755-2586, USA

ABSTRACT

This paper presents the development of composite girders consisting of hybrid fiber reinforced polymers (HFRP) I-girders and precast Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) slabs. HFRP I-girders consisting of multiple layers of carbon fiber reinforced polymers (CFRP) and glass fiber reinforced polymers (GFRP) were employed in the experimental investigation. Four full-scale composite girders were tested under four-point flexural loading. One girder was made of a full length precast UHPFRC slab connected to the HFRP I-girder. Twelve precast UHPFRC segments constituted slabs of the three other girders. Two connection types were used to connect the segments of the three precast UHPFRC segmental girders. The applicability of using epoxy and mortar in the connections was investigated. The test results indicated that the girder with epoxy connection between the precast segments exhibited more shear transfer across the segmental connections than the girder with mortar connection. The results showed that the flexural stiffness of the full length precast girder was almost similar to that of the girder with the segmental precast slabs connected by epoxy bonding. The ultimate strength of the full length precast girder was approximately 12% higher than that of the segmental girders. The study revealed that HFRP-UHPFRC composite girders provide a simple and sustainable solution for accelerated bridge construction.

Keywords: Hybrid Carbon/Glass Fiber Reinforced Polymer Girder, Ultra-High Performance Fiber-Reinforced Concrete Slab, Full Length Precast Slab, Segmental Precast Slabs, Flexural stiffness, Ultimate strength.

INTRODUCTION

According to the U.S. Department of Transportation (USDOT), 26.7% of the total bridges in the nation are either structurally deficient or functionally obsolete (USDOT 2007). The average annual cost to maintain highways and bridges for the 20-year period 2005–2024 is estimated to be $78.8 billion (USDOT 2007). Therefore innovations in materials, methods, and technologies are essential to find cost-effective solutions to address the deteriorating structures. FRP composites are receiving attention as a potential replacement for conventional materials in bridge construction since they provide the following advantages: high specific strength, high durability, excellent fatigue resistance, corrosion free, lightweight, reduced CO_2 emissions, rapid construction, and competitive life cycle cost.

Recently, a hybrid FRP (HFRP) I-girder has been developed for bridge applications. This girder optimized the combined use of carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) in a girder section with a specific ratio of flange to web width (Nguyen et al. 2010). While CFRP has higher tensile strength and stiffness, it is relatively expensive. On the other hand, GFRP is comparatively less expensive but its mechanical properties are lower than those of CFRP. In the HFRP I-girder, the top and bottom flanges were fabricated using a combination of CFRP and GFRP layers since they are subjected to high stresses under bending. The web was composed entirely of GFRP because it is usually subjected to lower stresses than the flanges. This way, the authors were able to utilize the advantages of both CFRP and GFRP in the HFRP girder for strength, stiffness and economy. Such girders can be used in severe corrosive environments and/or wherever accelerated bridge construction is required.

In a previous study by the authors (Nguyen et al. 2010), it was reported that the design of HFRP I-girders was governed by deformation rather than strength limitations. This was due to the low elastic modulus of FRP materials compared with equivalent steel or reinforced/prestressed concrete girders. Although the HFRP girders may have lower stiffness and strength comparable to conventional bridge girders, they can be combined with lightweight FRP grids or FRP decks to form pedestrian and/or bicycle bridges, where the live load is relatively small. The HFRP I-girders and GFRP grids were successfully applied to construct a pedestrian bridge in fishing port in Kure city, Hiroshima prefecture, Japan in 2011 (Fig. 1(a)). This bridge has a total length of 12 m and spans over the sea between a pier and a pontoon to replace for old steel bridges, which are susceptible to severe environmental attacks (Fig. 1(b)).

For road bridges and/or high volume traffic bridges, where the live load is significant, it is necessary to provide a concrete slab on top of the HFRP I-girder to carry the compression force. This helps to reduce stresses and deformation of the HFRP girder and to prevent delamination failure of the HFRP compression flange.

(a) Newly constructed FRP bridge using HFRP I-girders (a) Old steel bridge

Fig. 1 First HFRP pedestrian bridge in fishing port in Kure city, Hiroshima prefecture, Japan

STUDY OBJECTIVES

Extensive studies have been carried out on behavior of the FRP-concrete bridge structure and the study results showed highly feasible for infrastructure applications from a structural engineering point of view (Bakeri and Sunder 1990, Fam and Rizkalla 2002, Correia et al. 2007, Deskovic et al. 1995a, Keller et al. 2007, and Van Erp 2002a). However, most of these studies were concerned with normal strength concrete (NSC). As a matter of fact, the use of NSC will require a larger cross-sectional area for the slab to attain a tensile failure for FRP girder, thus, resulted to a heavy composite structural system. This study focuses on the flexural behavior of the HFRP I-girder and precast Ultra High Performance Fiber Reinforced Concrete (UHPFRC) topping slab. UHPFRC has high ductility in both tension and compression due to crack-bridging effect of the high strength steel fibers included in UHPFRC. Therefore steel bars are not necessary to reinforce UHPFRC slab for shrinkage and temperature effects, thereby reducing the slab thickness and overall self-weight of the HFRP-UHPFRC composite girder system.

Flexural behavior of HFRP-UHPFRC composite girders using full length precast UHPFRC slab was investigated by Nguyen et al. 2013. The study showed that the use of HFRP-UHPFRC composite girders could result in significant improvements of the flexural strength and stiffness, as compared to those of single HFRP girders without the UHPFRC slab. The full length slab was manufactured in the factory and cured in a special condition to attain the highest quality of concrete. However, sizes of slabs in actual full-scale bridge applications are usually more than 3 times larger than the slabs used in this study. It may not thus be feasible to precast the slabs in the factory and carry them to construction site for assembling. This is because slabs are long and thin and they may be deformed and cracked during the transportation. Additionally, if the slabs are too long, they cannot be transported by trucks. Therefore, it is more reasonable to use cast-in-place method for full length and large size slabs. However, the curing condition for the UHPFRC slabs in the construction site may not be the same as that in the factory and it may result in lower quality of concrete.

To overcome the disadvantages of using the full length precast slabs, this study deals with segmental precast slabs. The use of precast segments has the following benefits: 1) They can be produced in the factory with highest quality of concrete; 2) They can be easily transported to the construction sites without any deformations, cracks, or damages. This preliminary research focuses on investigations of weakest types of connections between precast segments including epoxy and mortar connection. The main advantage of these types of connections is simple and it may reduce the construction cost because high-skilled workers are not required. More advanced connection method using prestressed tendon anchored at two ends of the slab to join precast segments is under investigation and results will be reported soon. The major aims of this study is to apply the HFRP-UHPFRC composite girder system to full-scale foot/road bridges in Tohoku area in Japan, where many bridges were destroyed by the earthquake and tsunami in 2011 and in need of rapid replacement.

FLEXURAL TESTING OF PRECAST HFRP-UHPFRC COMPOSITE GIRDERS

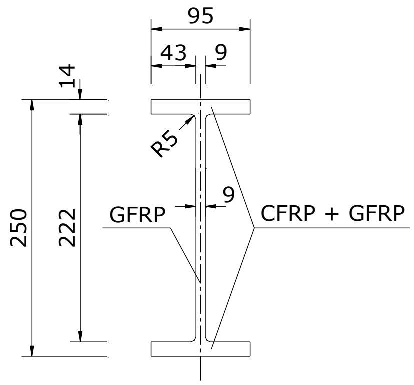
HFRP I-GIRDER

The HFRP I-girders were manufactured by pultrusion process using FRP layer composition as shown in Table 1. The top and bottom flanges of the I-girders are composed of CFRP and GFRP in order to increase strength and stiffness of the girder. Unidirectional carbon fibers (all carbon fibers are orientated in zero degree to the longitudinal direction) were used in the flanges of the girders. Bi-directional glass fabric (glass fibers are orientated in 0°/90° or ±45° directions) and glass continuous strand mat (glass fibers are orientated in random directions) were used in the flanges and the web of the girders to avoid the inherent anisotropic behavior of FRP materials. The web is composed entirely of GFRP because of the lower stresses, and to reduce the cost. The overall height of the HFRP girder is 250 mm and the flange width is 95 mm. The flange thickness is 14 mm and the web thickness is 9 mm. The cross-section of the HFRP I-girder is illustrated in Fig. 2(a). The mechanical properties of CFRP and GFRP are listed in Table 1.

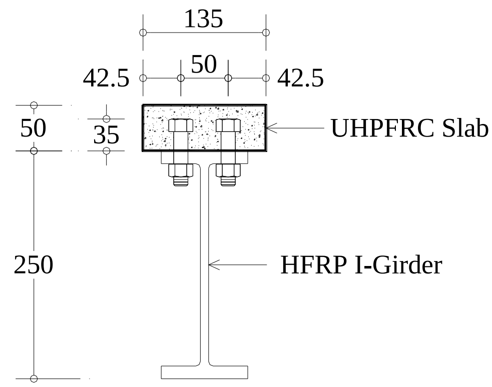
Table 1 Mechanical Properties of FRP Lamina

Parameters	Notation	CFRP 0°	GFRP 0/90°	GFRP ±45°	GFRP CSM[1]
Volume Fraction	V_f , %	55	53	53	25
Volume Content	Flange, %	33	17	41	9
	Web, %	0	43	43	14
Young's Modulus	E_{11}, GPa	128.1	25.9	11.1	11.1
	E_{22} , GPa	14.9	25.9	11.1	11.1
Shear Modulus	G_{12}, GPa	5.5	4.4	10.9	4.2
Poisson's Ratio	ν_{12}, —	0.32	0.12	0.58	0.31

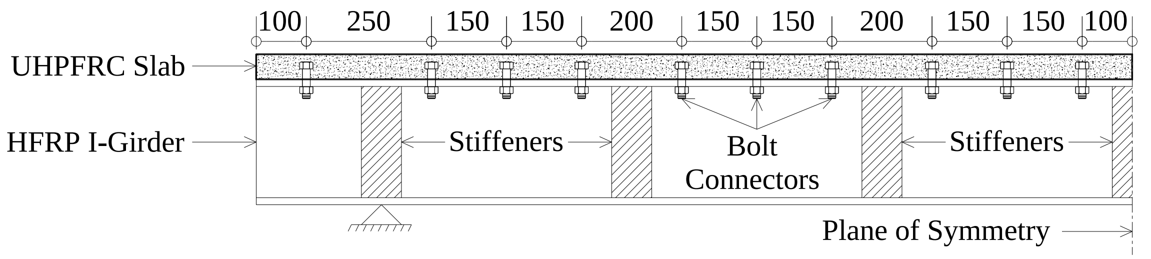
[1] Continuous Strand Mat



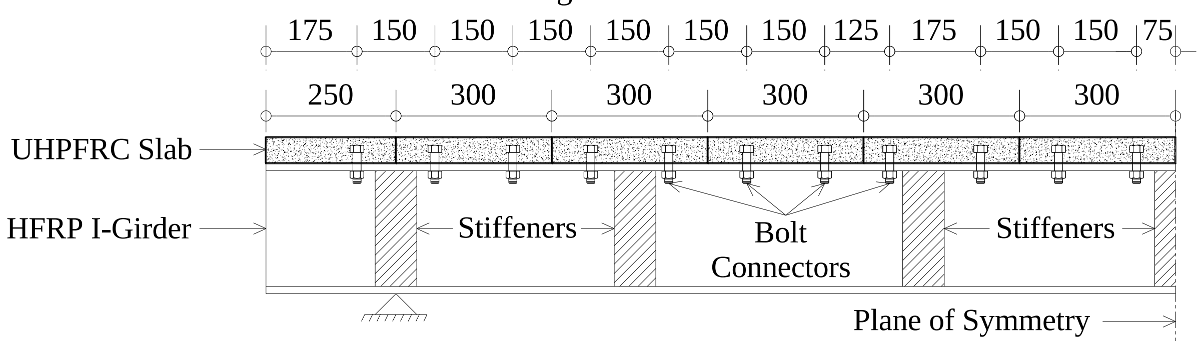
(a) Cross-section: control specimen



(b) Cross-section: HFRP-UHPFRC composite girders



(c) Locations of bolt shear connectors in full length precast UHPFRC-HFRP composite girders



(d) Locations of bolt shear connectors in segmental precast UHPFRC-HFRP composite girders

Fig. 2 Details of girders' cross-sections and locations of bolt shear connectors in all girders (units are in mm)

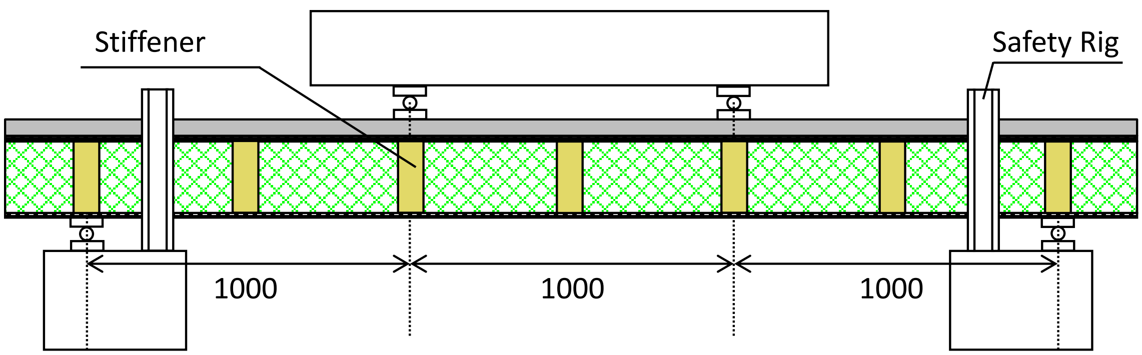
ULTRA HIGH PERFORMANCE FIBER REINFORCED CONCRETE (UHPFRC)

Mixture proportions of the UHPFRC are listed in Table 2. The UHPFRC is composed of water, premixed cementitious powder, sand, water reducing agent and steel fibers. The premixed cementitious powder includes ordinary Portland cement, silica fume and ettringite. The steel fibers have a diameter of 0.2 mm and a tensile strength of 2,000 MPa. They have lengths of 22 mm and 15 mm and two equal quantities of fiber lengths (50% for each fiber length) were used. The fibers were added at approximately 1.75% volume ratio. The UHPFRC slabs were precasted and cured at 85 degrees Celsius for 24 hours. Compression tests were performed on 100×200 mm cylinders of the UHPFRC to determine compressive strength and modulus of elasticity. Moduli of rupture tests were performed on 100×100×400 mm specimens to determine the tensile strength of the UHPFRC. Three specimens were tested for each material property and the test results showed that the UHPFRC had a compressive strength of 182 MPa and a tensile strength of 11.9 MPa. The tensile modulus of elasticity (Young's modulus) of the UHPFRC is almost the same as its compressive modulus of elasticity of approximately 46.1 GPa.

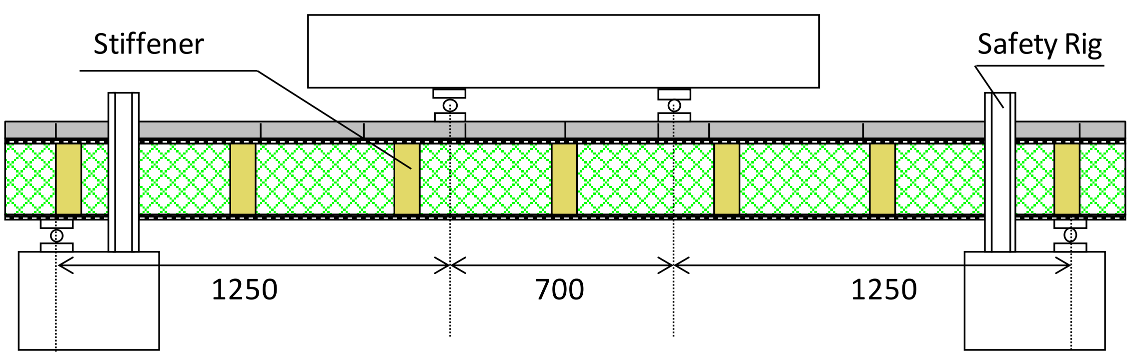
Table 2 Mix Proportions of UHPFRC

Air content, %	Unit quantity, kg/m^3				Steel fiber, kg/m^3
	Water	Premix cement	Sand	W.R. Admixture	
2.0	205	1,287	898	32.2	137.4

Four point bending tests were conducted on all girders. The experimental setup is shown in Fig. 3. Successive loads were applied through hydraulic jack until getting the girders to failure. The applied load, deflection at mid-span section and strains in the HFRP-UHPFRC composite section were recorded throughout the test.



(a) Full length precast UHPFRC-HFRP composite girder



(b) Segmental precast UHPFRC-HFRP composite girder
Fig. 3 Flexural girder test setup (units are in mm)

TEST VARIABLES

The test variables for the large-scale girder flexural tests are listed in Table 3. Four girders with a constant slab thickness of 50 mm and a slab width of 135 mm were tested (Fig. 2(b)). One girder was made of a full length precast UHPFRC slab connected to the HFRP I-girder (Fig. 2(c)). Twelve precast UHPFRC segments constituted slabs of the three other girders (Fig. 2(d)). Two connection types including epoxy bonding and mortar were used to connect the segments of the three precast UHPFRC segmental girders. Gap between two continuous segments was very close to zero for the girders with epoxy connection (the actual gap is equal to the thickness of the epoxy layer which is about 0.3-0.5 mm) and 10 mm for the girder with mortar connection as shown in Fig. 4. Mix proportions of mortar are presented in Table 4. Mechanical properties of mortar and epoxy bonding are listed in Table 5. Further details of mechanical properties of epoxy bonding can be found in Nguyen & Mutsuyoshi 2013.

Table 3 Flexural Girder Test Variables

Specimen name	Type of precast UHPFRC slab	Type of connection between UHPFRC segments	Gap between two continuous UHPFRC segments (mm)	N[1]	Length of each UHPFRC segment[2] (mm)	Total length of UHPFRC slab (mm)	Loading span (mm)
H-FL-LS10	Full length	n/a	n/a	n/a	n/a	3500	1000
H-S0-LS10	Segment	Epoxy bonding	0	12	300	3500	1000
H-S0-LS7	Segment	Epoxy bonding	0	12	300	3500	700
H-S10-LS7	Segment	Mortar	10	12	300	3500	700

[1] N = Number of UHPFRC segments
[2] Length of each UHPFRC segment at the end of the slab is 250 mm

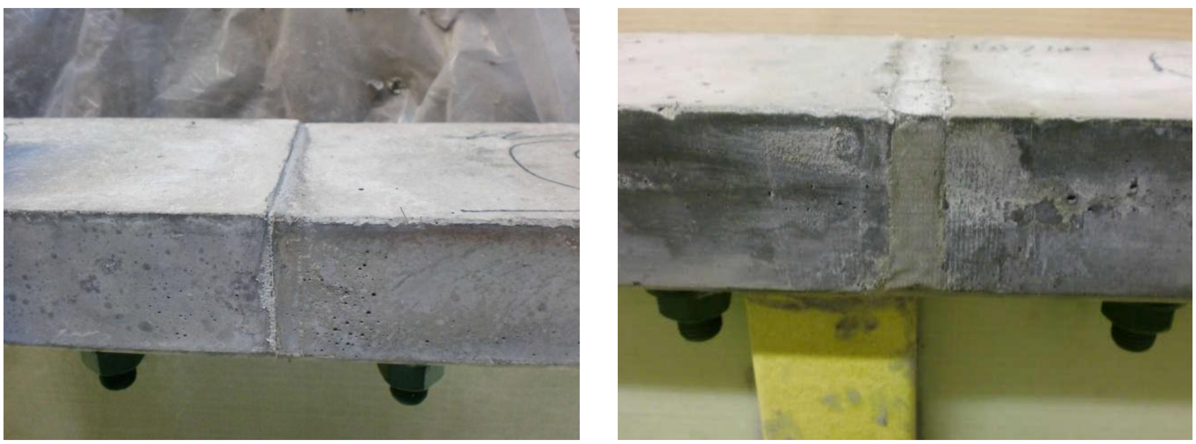
Table 4 Mix Proportions of Mortar

W/C (%)	Unit quantity (kg/m^3)				C+W : S
	Cement (C)	Water (W)	Fine aggregate (S)	Air entraining and high-performance water reducing agent	
30	970.7	291.2	1026.9	C*0.01	6 : 4

Table 5 Mechanical Properties of Mortar and Epoxy Bonding

Material	Compressive strength (N/mm^2)	Modulus of elasticity (kN/mm^2)
Mortar	88.3	29.9
Epoxy bonding	70-76	4-6.4

All girders in this study were designed with full shear interaction between the slab and the girder (i.e. there is no slip between the slab and the girder). They were designed to fail in both tension and compression. Stainless steel headed bolt shear connectors were used. Headed bolt diameter of 16 mm and the bolt-hole diameter of 17 mm were used. The surface of the top flange of the HFRP I-girder was roughened with sandpaper to have better bonding between the girder and the slab. Details of the girders' cross-section are shown in Fig. 2(a)-(b). The total length of each girder is 3,500 mm with the loading span is either 700 mm or 1000 mm as shown in Fig. 3. The wooden/GFRP stiffeners were installed at a spacing of 500 mm on both sides of the web to prevent web buckling. It is necessary to emphasize that due to the unavailability of the GFRP box stiffeners at the time of preparing for girder with full length precast UHPFRC slab (girder H-FL-LS10), a decision was made to use the wooden stiffeners as a temporary solution for this girder. That decision did not affect to the behavior of girder H-FL-LS10 since the test results of the same girder using GFRP stiffeners are almost the same with those using wooden stiffeners. However, for real infrastructure applications, it is recommended to use FRP stiffeners in HFRP-UHPFRC composite girders to improve durability of structures. The stiffeners were bonded to the HFRP girders using epoxy bonding. The spacing of headed bolts was determined from the shear connection tests to prevent premature bolt shear failure as shown in Fig. 2(c)-(d). A torque wrench was used to apply 20 N-m torque to the bolts in all girders.



(a) Connected by epoxy bonding (Gap = 0)   (a) Connected by mortar (Gap = 10 mm)

Fig. 4 Connection method between two continuous UHPFRC segments

RESULTS AND DISCUSSION

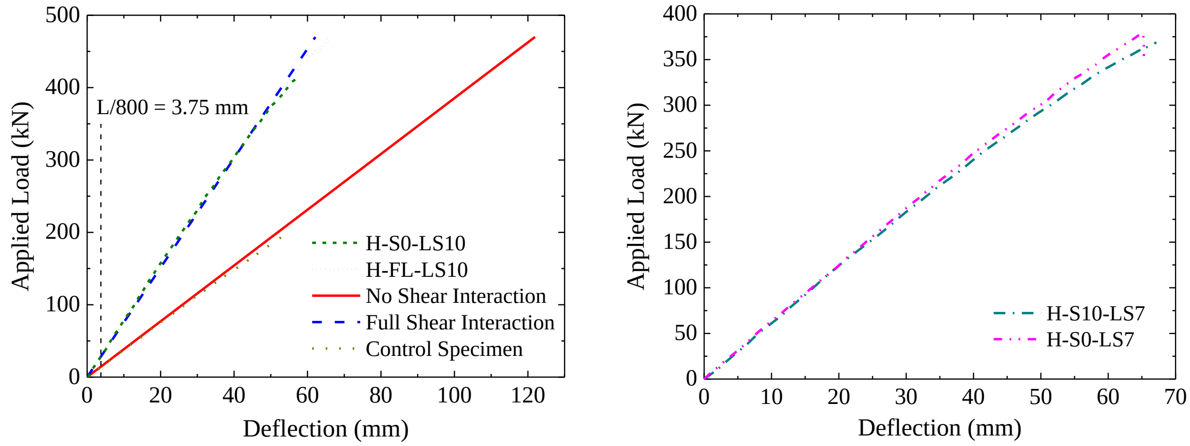
Comparisons of the load versus mid-span deflection curves of all tested girders are shown in Fig. 5. Figure 5(a) shows comparisons of two girders with a loading span of 1000 mm. A full length precast slab was used for girder H-FL-LS10 and segmental precast slabs were used for girder H-S0-LS10. The gap between two continuous segments in girder H-S0-LS10 was very close to zero. For comparison purposes, the load-deflection curves of a single HFRP girder without UHPFRC slab (control specimen), composite girders with no shear interaction (i.e. there is no connection between the slab and the girder) and full shear interaction are also included in Fig. 5(a). The control specimen was tested under four point bending. The girder span was 3000 mm and the loading span was 1000 mm. The control specimen behaved almost linearly up to the failure load which was 195 kN. The failure mode of the control specimen was crushing of fibers near the loading point followed by delamination of the compressive flange.

Figure 5(a) shows that both girders H-S0-LS10 and H-FL-LS10 behave linearly up to the load at which the major debonding of the epoxy layer happened. Before the major debonding, stiffness of girder H-S0-LS10 (with segmental slabs) is almost the same with that of girder H-FL-LS10 (with full length slab). Stiffness of these girders is almost similar to the computed stiffness of the composite girder with full shear interaction. This result indicates that these girders have almost full shear interaction before the major debonding happened. Stiffness reductions in girders H-S0-LS10 and H-FL-LS10 were observed at the loads of 380 kN and 430 kN, respectively due to the major debonding. The difference in the major debonding loads of these two girders attributing to their difference in locations and spacing of bolt shear connectors as shown in Fig. 2(c)-(d). The load-carrying capacity of girder H-S0-LS10 is approximately 12% lower than that of girder H-FL-LS10. Finally, these girders failed in similar failure modes which were spalling of the concrete near the loading point followed by delamination of the HFRP compression flange and web crushing as shown in Fig. 6(a)-(b). Tensile failure of the HFRP bottom flange at mid-span section was observed in girder H-FL-LS10 indicating that the high tensile strength of the CFRP in the bottom flange was utilized effectively.

The results show that girders H-S0-LS10 and H-FL-LS10 exhibit significant higher stiffness and load-carrying capacity than those of the single HFRP I-girder without a slab (control specimen). The stiffness of these two girders was about 2 times higher than that of the control specimen. The load carrying capacities of girders H-S0-LS10 and H-FL-LS10 were 2.1 and 2.4 times, respectively higher than that of the control specimen.

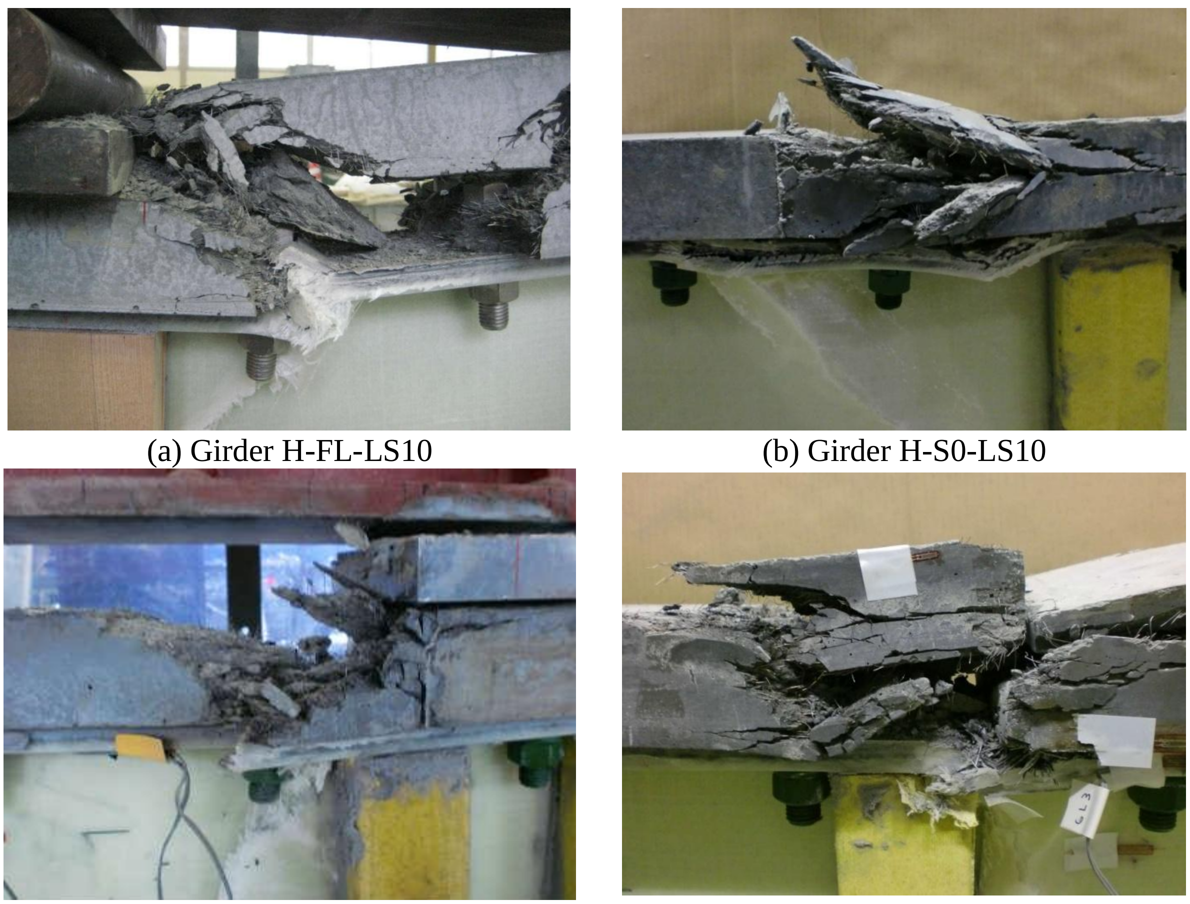
It can be seen from Fig. 5(a) that the stiffness of the composite girder with no shear interaction is only 5% higher than that of the control specimen. The result indicates that the total moment applied to the composite girder with no shear interaction is approximately equal to moment acting in the HFRP I-girder alone.

Figure 5(b) shows the comparisons of the load versus mid-span deflection curves between two girders using segmental slabs. Gaps between two continuous segments in girders H-S0-LS7 and H-S10-LS7 were approximately zero and 10 mm, respectively. Stiffness of girder H-S0-LS7 was almost the same with that of girder H-S10-LS7 with the load range from zero to approximately 150 kN. When the load increased from 150 kN to the failure, stiffness of girder H-S0-LS7 was slightly higher than that of girder H-S10-LS7 attributing to the local failure of the mortar connections in this girder. These girders failed in the similar failure modes, which were the concrete spalling near the loading point followed by delamination of the compression flange and web crushing as shown in Fig. 6(c)-(d).



(a) Girders with a loading span of 1000 mm (b) Girders with a loading span of 700 mm

Fig. 5 Comparisons of load versus deflection curves at mid-span section



(a) Girder H-FL-LS10 (b) Girder H-S0-LS10

(c) Girder H-S0-LS7 (d) Girder H-S10-LS7

Fig. 6 Failure modes of HFRP-UHPFRC girders in flexure

Figure 7 shows the distribution of longitudinal strains along section depth of tested girders. Almost linear strain distributions through the cross-sections were observed for girders H-FL-LS10, H-S0-LS10, and H-S0-LS7 as shown in Fig. 7(a)-(c). The result indicates that these girders show almost full composite action until the final failure. On the other hand, nonlinear strain distributions were observed through the cross-section of girder H-S10-LS7 as shown in Fig. 7(d). This may be attributed to the local failure of the mortar connections used to connect the UHPFRC segments in girder H-S10-LS7. This girder finally failed due to the totally mortar splitting between two segments and concrete spalling near the loading point followed by delamination of the compression flange and web crushing (Fig. 6(d)).

Maximum tensile strains recorded at the HFRP tension flange of girders H-FL-LS10, H-S0-LS10, H-S0-LS7, and H-S10-LS7 at failure are approximately 10,970 μ, 9,400 μ, 10,960 μ, and 10,860 μ, respectively. These strain levels are significantly higher than the 6,000 μ recorded at failure in the tension flange of the single HFRP girder tested without slab. This result shows that the addition of the UHPFRC slab on the HFRP girder resulted to the effective utilization of the high tensile strength of the CFRP. Summary of results for all tested girders is presented in Table 6.

Table 6 Summary of Results for All Tested Girders

Girder	P_u [1] (kN)	δ_u [2] (mm)	ε_{cu} [3] (μ)	ε_{tu} [4] (μ)	Failure mode
H-FL-LS10	469.7	66.3	4,290	10,970	Severe spalling – UHPFRC slab (SS-LP); Delamination – HFRP top flange (SS-LP); Web crushing (SS-LP); Delamination – HFRP bottom flange (FS)
H-S0-LS10	411.7	57.0	3,140	9,400	Severe spalling – UHPFRC slab (FS-LP); Delamination – HFRP top flange (FS-LP); Web crushing (FS-LP)
H-S0-LS7	379.7	65.2	3,440	10,960	Severe spalling – UHPFRC slab (SS-LP); Delamination – HFRP top flange (SS-LP); Web crushing (SS-LP); Delamination – HFRP bottom flange (FS)
H-S10-LS7	368.7	67.2	2,210	10,860	Severe spalling – UHPFRC slab (SS-LP); Delamination – HFRP top flange (SS-LP); Web crushing (SS-LP); Delamination – HFRP bottom flange (FS)
Control specimen	194.9	53.3	n/a	6,000	Delamination – HFRP top flange (SS-LP); Web crushing (SS-LP)

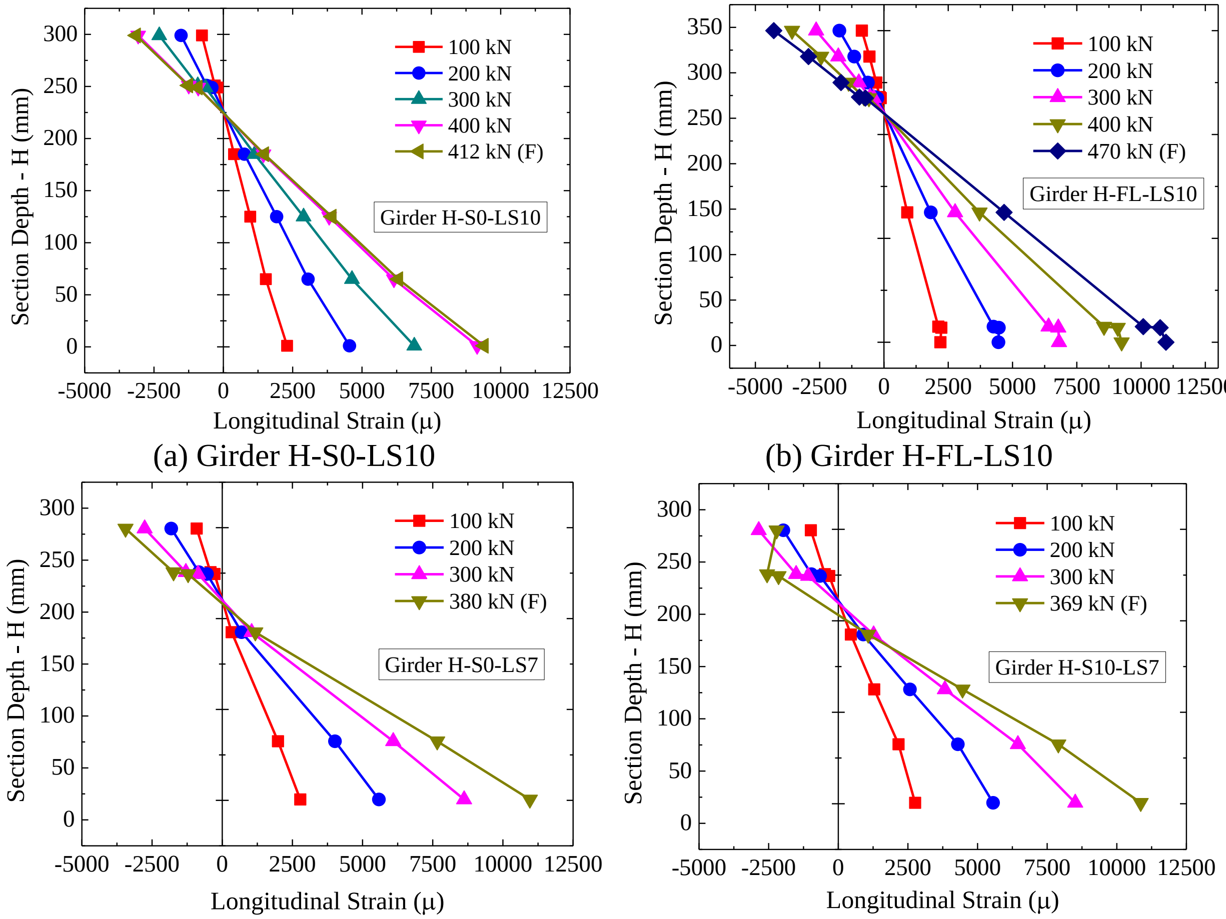
Note: FS = flexural span; SS-LP = shear span – near the loading point; FS-LP = flexural span – near the loading point

[1] Ultimate load

[2] Deflection corresponding to the ultimate load

[3] Ultimate compressive strain of the UHPFRC slab at mid-span section

[4] Ultimate tensile strain of the HFRP girder at mid-span section



(a) Girder H-S0-LS10 (b) Girder H-FL-LS10

(c) Girder H-S0-LS7 (d) Girder H-S10-LS7

Fig. 7 Distribution of longitudinal strains along section depth of girders

Although the behavior of HFRP-UHPFRC composite girders is brittle, it can be used safely for bridge applications. According to AASHTO LRFD bridge design specifications (AASHTO 2007), live-load deflection for girder bridges should not exceed L/800, where L is span of the girder. The deflection limit for girders H-S0-LS10 and H-FL-LS10 is L/800 = 3.75 mm. This deflection level is associated with a design load of 28 kN for these girders. This design load limit is equal to only 6% and 6.8% of the ultimate load of girders H-S0-LS10 and H-FL-LS10, respectively.

Table 7 shows the comparisons for weight and ultimate load of the HFRP-UHPFRC composite girder (girder H-S0-LS10) and HFRP-Normal strength concrete (HFRP-NSC) composite girder (Mutsuyoshi et al. 2011). It can be seen that the ultimate load of girder H-S0-LS10 is only 4% lower than that of the HFRP-NSC girder while its weight is only one fourth of the HFRP-NSC girder. This indicates that the use of segmental precast UHPFRC slabs is more effective than the cast-in-place NSC slab in terms of strength, weight, and durability.

Table 7 Comparisons of Weight and Ultimate Load of Composite Girders

	HFRP-UHPFRC composite girder (girder H-S0-LS10)		HFRP-Normal strength concrete composite girder (Mutsuyoshi et al. 2011)	
	HFRP I-girder[1]	UHPFRC slab[1]	HFRP I-girder[1]	NSC slab[1]
Dimensions (mm)	$250\times95\times14\times9$ $H\times B\times t_f\times t_w$	135×50 $B\times t$	$250\times95\times14\times9$ $H\times B\times t_f\times t_w$	400×100 $B\times t$
Ultimate load (kN)	412		428	
Composite girder weight (kg)	28.7	60.5	28.7	336
	Total: 89.2 kg		Total: 364.7 kg	

[1] Length of the HFRP I-girders and the UHPFRC/NSC slabs is 3500 mm for all tested girders

CONCLUSIONS

This paper presents an experimental study of composite girders consisting of pultruded HFRP I-girders and full length/segmental precast UHPFRC topping slabs. The bolt shear connectors and epoxy were used to connect the HFRP I-girder and UHPFRC slab. The main conclusions from the study are as follows:

1. The use of segmental precast UHPFRC slabs may benefit for actual full-scale bridge applications using HFRP-UHPFRC composite girders with long spans and having a large size of slabs. This is attributed to the high quality of precast UHPFRC segments and the ease of transporting precast segments to the construction site without deforming and damaging them.

2. The use of epoxy bonding is simple and efficient to connect the precast segments. The test results showed that the flexural stiffness of the girder with the segmental precast slabs connected by epoxy bonding was almost similar to that of the girder with the full length precast slab. However, the ultimate strength of the segmental girder with epoxy connection was approximately 12% lower than that of the full length precast girder indicating a not fully shear transfer across the segmental connections. Further investigations of connection methods for the UHPFRC segments are required to attain fully shear transfer across the segmental connections.

3. The use of mortar to connect the precast segments is not recommended for real bridge applications. This is because of the differences in mechanical properties between the mortar and the UHPFRC leading to high stress concentrations at the segmental connections. Since the compressive strength and modulus of the mortar is smaller than those of the UHPFRC, stiffness reduction observed in girder with mortar connection is attributed to the compressive failure of the mortar.

4. The HFRP-UHPFRC girders can be potentially used for rehabilitating and strengthening of existing structures. They can also be used for new bridge construction with high strength, high durability, low weight and moderate stiffness.

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