Citation
Pournasiri, E. and Pham, T.M. and Hao, H. 2022. Behavior of Ultrahigh-Performance Concrete Bridge Decks with New Y-Shape FRP Stay-in-Place Formworks. Journal of Composites for Construction. 26 (3): ARTN 04022023
http://doi.org/10.1061/(ASCE)CC.1943-5614.0001214

Behaviour of Ultra-High Performance Concrete Bridge Decks with New Y-shape FRP Stay-In-Place Formwork

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Abstract

This study proposes using glass fibre-reinforced polymer (GFRP) as stay-in-place structural formwork for casting bridge decks with ultra-high performance concrete (UHPC). The GFRP stay-in-place formworks completely replace the bottom layer of rebars, and the top steel reinforcement is also replaced by a GFRP mesh to mitigate the corrosion damage. The formworks were either a flat GFRP plate with square hollow section (SHS) stiffeners or a flat GFRP plate with new Y-shape stiffeners. Concentric static tests on five 1:2.75 scale decks were performed to investigate the effect of stiffener's configuration and the influence of the concrete strength on the performance of bridge decks. Rotational fixity support was used to simulate a real bridge deck connection of supporting girders. All specimens with stay-in-place formwork exhibited punching shear failure. It was found that the use of Y-shape stiffeners significantly improved the load-carrying capacity of the proposed deck. Replacing normal concrete with UHPC further improved the loading capacity of the deck. The decks demonstrated excellent performance, with the load-carrying capacity 3.8 to 9.5 times higher than the established equivalent service load depending on the concrete strength and configuration of the GFRP stay-in-place formwork. Deflection at service load was less than span/1,600 for all the decks. Compared with normal strength concrete (34 MPa), UHPC improved the maximum load-carrying capacity of the deck from 91.4 kN to 149 kN.

Keywords: Stay-in-place formwork; Bridge deck; UHPC; GFRP; Punching shear

Introduction

Most of the conventional cast-in-place reinforced concrete (RC) structures need a temporary formwork. This type of formwork is often held in place by temporary scaffolding during concrete pouring and hardening. The cost of a temporary formwork system including the material and labour for conventional cast-in-place concrete structures usually exceeds 50% of the total cost of construction (Remy et al. 2011). The stay-in-place formwork is a well-known substitute for the temporary formwork. After Concrete pouring and hardening, this type of formwork stay in place as an integral part of the structure and acts as external structural reinforcements throughout the structure lifecycle (Wrigley et al. 2001). Steel decking (Rezaeian et al. 2020) is a classic stay-in-place formwork and it has been extensively used in bridge decks. Stay-in-place formwork is one of the innovative applications of fibre-reinforced polymer (FRP) materials primarily for their improved corrosion resistance compared to steel (Matta and Nanni 2009, Jayaprakash et al. 2015, Le et al. 2020, Tran et al. 2020a, b). Several FRP stay-in-place formwork configurations for decks have been proposed and investigated in previous studies. Flat plates with T-shape ribs (Oliva et al. 2008, Bank et al. 2010, Nelson et al. 2013, Nelson et al. 2014, Nelson and Fam 2014, Nicoletta et al. 2019), flat plates with perforated rib connectors (Zuo et al. 2018a, b), flat plates stiffened by bonded hollow square sections (Dieter et al. 2002, Alagusundaramoorthy et al. 2006, Hanus et al. 2009), corrugated plates (Honickman and Fam 2009, Liu et al. 2011, Fam and Nelson 2012, He et al. 2012, Richardson et al. 2014), a plate attached to the bottom of grid reinforcements (Ringelstetter et al. 2006, Fang et al. 2016), and pultruded hollow box sections with moulded grating (Gai et al. 2013) are examples of FRP stay-in-place formworks. Cost analysis of reinforced concrete bridge decks with FRP stay-in-place formworks showed a 24% ~ 57% project cost reduction when compared to conventional concrete bridge decks (Berg et al. 2006, Ringelstetter et al. 2006, Matta et al. 2007). Previous studies investigated different properties, such as failure modes, mechanical behaviour, the influence of formwork splicing, the effect of freeze-thaw cycles, and the response to fire of normal concrete decks using FRP stay-in-place formwork and it revealed that the shear failure is the dominant failure mode for this type of decks due to the absence of adequate shear reinforcement within concrete (Fam and Nelson 2012, He et al. 2012, Gai et al. 2013, Fang et al. 2016, Nicoletta et al. 2019). Therefore, improving the shear capacity would enhance the load-carrying capacity of decks with FRP stay-in-place formwork.

Previous studies of the decks with FRP stay-in-place formworks used normal concrete. To the authors' best knowledge, there is no adequate investigation on the performance of decks with UHPC cast on GFRP stay-in-place formwork in literature yet. UHPC as a revolutionary cement-based material has been developed using various fibre types, binders, sand, and chemical admixes (Li et al. 2002, Herrera-Franco and Valadez-González 2005, Cho et al. 2008, Pournasiri et al. 2018). UHPC shows exceptional mechanical properties, including high compressive strength, high flexural and tensile strengths, high ductility and toughness, and high durability (Park et al. 2012, Shaikh et al. 2020, Tian et al. 2020). The excellent mechanical properties of UHPC can make it possible to overcome the shear weakness of conventional FRP stay-in-place formworks. Furthermore, UHPC has excellent durability that can reduce maintenance costs during the service life of structures (Graybeal and Tanesi 2007, Magureanu et al. 2012).

A previous study (Pournasiri et al. 2021) proposed a new Glass FRP (GFRP) stay-in-place formwork that features a base plate with Y-shape stiffeners and compared its structural performance to that of another GFRP stay-in-place formwork that consists of a base plate with square hollow stiffeners (SHS stiffeners for short hereafter). The results showed that utilizing Y-shape stiffeners can significantly improve the shear resistance and the maximum load-carrying capacity of the deck cast into FRP stay-in-place formworks. A comparison between decks stiffened by Y-shape stiffeners and T-ribs that have been used by a previous study (Nelson et al. 2013) showed that the load-carrying capacity of a similar deck with Y-shape stiffeners increased by 12.5%. The reason behind this can be attributed to the better shear performance of the Y-shape stiffener as a shear stud compared to the T-ribs and contributes to better structural integrity. Despite the deck with Y-shape stiffeners showing a higher shear resistance, it still failed in shear.

To improve the shear capacity of the deck with Y-shape stiffeners, this study proposes to replace normal concrete by UHPC. This paper presents an experimental study that focuses on the influence of UHPC on the structural behaviour of concrete decks cast on stay-in-place-formworks to improve the shear resistance. The use of UHPC together with GFRP stay-in-place formwork is proposed for the first time in this study. Five decks were fabricated and tested under concentric static loading. For the first time, this study introduced a fully non-corrosive bridge deck with stay-in-place formwork. The UHPC used in this study was reinforced by using non-corrosive polyvinyl-alcohol (PVA) fibres, instead of commonly used steel fibres. The structural behaviour of UHPC decks cast on different GFRP stay-in-place formworks is compared and analysed.

Experimental investigation

Test specimens and parameters

An experimental program was employed to quantify the structural capacity of UHPC cast on GFRP stay-in-place formworks under concentric loading. The 1,405 mm long and 604 mm wide deck specimens with a clear span of 665 mm in this study represent a 1:2.75 scaled concrete bridge deck with a 1,830 mm girder spacing commonly used in practice as also adopted in a study by Nelson and Fam (Nelson and Fam 2014). Table 1 provides a summary of the test matrix. The specimen name is explained in Fig. 1. In total, five decks were cast and tested in this study. Two control decks, namely SUCS and SUCF, were solid UHPC decks of 75 mm thick reinforced with top and bottom orthogonal layers of steel and GFRP bars with a total reinforcement ratio of 0.003, respectively. These decks will serve as a benchmark for comparing the decks with the GFRP stay-in-place formwork system and solid reinforced UHPC decks. The decks with GFRP stay-in-place formwork are referred to as SUSF and SUYF, which had seven SHS or Y-shape stiffeners with a total reinforcement ratio of 0.16 and 0.13, respectively, spanning the full length of the formworks in one direction, transverse to the direction of traffic. The stay-in-place formwork not only acted as structural formworks to support construction loads but also completely replaced the bottom layer of rebar reinforcements. Deck SNSF was constructed with the same design as Deck SUSF, but with normal concrete to make a comparison between UHPC and normal concrete. It is noted that the SHS stiffened formwork was used for the sake of environmental and economic advantages by the fact that it uses 22% less concrete than the conventional solid decks while the new Y-shape stiffeners were proposed to improve the shear resistance of concrete decks with GFRP stay-in-place formwork.

The total thickness of the decks with GFRP stay-in-place formworks was 65 mm, compared to the 75 mm thickness of the reinforced UHPC control decks to maintain the same effective depth of all these specimens (see Fig. 2). Rebar meshes were provided in the compressive zone for all the specimens. The GFRP stay-in-place formwork was extended 30 mm into the supports to simulate the 75 mm actual practice for full-scale structures. All the specimens were tested under the same loading and supporting conditions. Quasi-static monotonic loading was applied to the centre of the specimens.

Material properties

As shown in Fig. 3, two different GFRP panels were fabricated in this study and used as stay-in-place formworks. The SHS GFRP stay-in-place formwork was 600×604 mm^2 and it consisted of a 3.2 mm thick GFRP plate, stiffened by seven 38.1 mm pultruded SHSs. The Y-shape GFRP stay-in-place formwork of 600×604 mm^2 was similar to the former panel and it contained a 3.2 mm thick GFRP plate, which was stiffened by seven 36.5 mm pultruded Y-shape sections. The stiffeners were bonded to the plate using high strength and low viscosity epoxy resin at 80 mm centre to centre spacing after proper surface preparation including sanding and cleaning. Both stay-in-place formworks were designed so that the cross-section area of GFRP was the same for both decks with SHS stiffeners and Y-shape stiffeners. The ultimate tensile strength and modulus of elasticity of stiffeners in the longitudinal direction were 206.8 MPa and 20.7 GPa, respectively. The corresponding properties in the transverse direction were 48.2 MPa and 5.5 GPa, respectively. The ultimate tensile strength and modulus of elasticity of GFRP plates in longitudinal direction were 165.5 MPa and 13.8 GPa, respectively. The corresponding properties in the transverse direction were 51.7 MPa and 6.9 GPa, respectively (Treadwellgroup 2020). As shown in Fig. 4, all GFRP reinforcing bars were sand-coated surface size #2 bars (32-mm^2 nominal cross-section area) with a nominal tensile strength of 1,100 MPa and modulus of elasticity of 60 GPa (Pultrall Inc. 2021). 6-mm diameter deformed steel bars were provided in the control deck SUCS as both tensile and compressive reinforcements. Five coupons were tested in tension according to ASTM A615 (2020). The tensile tests showed a yield strength and modulus of elasticity of 550 MPa and 200 GPa, respectively. A two-part epoxy was used as a high modulus, high strength, and low viscosity epoxy adhesive with a manufacture-reported bond strength of 50.5 MPa (WEST-SYSTEM 2020).

Deck SNSF was cast from a normal concrete batch with the 10 mm maximum aggregate size. The normal concrete had a compressive strength of 34 MPa on the day of testing according to ASTM C39 (2021), and tensile strength of 3 MPa as per ASTM C496 (2017). Decks SUSF and SUYF were cast with UHPC. The mix proportion of normal concrete and UHPC are given in Tables 2 and 3. 2% PVA fibres with 12 mm long and 0.2 mm diameter were used as fibre reinforcements. Manufacture data reports a tensile strength of 1,000 MPa and an elastic modulus of 29 GPa (Kurary 2021). PVA fibres were adopted in this study to construct completely metal-free bridge decks with excellent loading capacities and corrosion-free feature. The decks were transferred to the steam curing room after 24 hours and stored for 3 days at 85°C. According to the manufacturer, the glass transition temperature is 100°C (Treadwellgroup 2020, Pultrall Inc. 2021). The UHPC had a compressive strength of 140 MPa and splitting tensile strength of 12 MPa on the day of testing.

Fabrication of deck specimens with stay-in-place formwork

The GFRP stay-in-place formwork was placed in the middle of a special wooden formwork. As shown in Fig. 5, SHS stiffeners were obstructed by polystyrene foam to avoid penetration of fresh concrete in the hollow sections. As seen in Fig. 5, the top rebar mesh of 6.35 mm GFRP bars was then placed in position for all the specimens. Bonding between concrete and formwork is a vital factor in the stay-in-place formwork system. Adhesive bonding (wet bonding) mechanism was used for the GFRP–concrete interface to promote composite action. A thin layer of wet epoxy adhesive applied to the surface of GFRP stay-in-place formwork approximately 20 minutes before pouring wet concrete on GFRP stay-in-place formwork.

Test Setup and Instrumentation

Similar to the previous studies (Bouguerra et al. 2011, Boles et al. 2015, Pournasiri et al. 2021), the deck specimens were bolted to the supports to simulate similar restraints as in reality due to monolithically connection between deck and girders. Tied bolts restrained the deck against rotation and lateral sliding. The complete assembly of the test setup is illustrated in Fig. 6.

Two stiff steel beams with I-section, 454 mm depth, 190 mm width, and 2,250 mm length were tied to the ground using 20 mm diameter high strength threaded rods. The thickness of the web and flanges was 12 mm, and beams were stiffened with three 12 mm plats at both sides along with the web. 16-mm steel plates were placed to the top flange of the beams to provide a 540 mm clear span due to the short length of flanges. Finally, steel square hollow sections were then used to clamp the deck from the top using eight 20 mm threaded rods. The load was applied to the specimens with a loading rate at 1 mm/min at the centre using two hydraulic jacks through a 91×182 mm^2 steel loading plate over a 12.7 mm neoprene pad in contact with the concrete surface (scaled tire pad) according to American Association of State Highway and Transportation Officials (AASHTO) (2007). The load was measured with two 200 kN load cells. Deflections were measured using linear variable differential transformers (LVDTs) at various points along the longitudinal and transverse centrelines. Fig. 7 shows the positions of the strain gauges attached to the GFRP stiffeners. The strain of the GFRP stiffeners was internally measured in the longitudinal and transverse directions at various points along the same axes using 5 mm strain gauges. The strain of the GFRP plate at the mid-span was also measured in the longitudinal direction using strain gauges.

Experimental Results

This section presents the test results of the performances of solid reinforced UHPC control decks and decks with stay-in-place formworks. Figs. 8 and 10 show the load-deflection and mid-span strain responses of all the deck specimens while Fig. 9 displays the failure modes and crack patterns of the decks.

Performance of decks under equivalent design service load

Nelson and Fam (2014) performed a full-scale test to establish an equivalent service load of 122.5 kN representing the half-axle load of the CL-625 design truck considering the dynamic loading effect. Based on their analysis, the equivalent service load for a 1:2.75 scaled bridge deck is 24.3 kN. The peak loads for decks with stay-in-place formwork in this study were ranged from 91.4 to 230.9 kN which were 3.8 to 9.5 times higher than the established design service load. The deflection of Decks SUSF, SUYF, and SUCF at service loads ranged from 0.20 to 0.41 mm, which was far smaller than the AASHTO limit for concrete vehicular bridges of *L*/800 (0.83 mm) and the most stringent US Department of Transportation (DOT) limit of *L*/1600 (0.42 mm), where the span *L* was taken as the centre-to-centre support spacing of 665 mm (AASHTO 2007, Fu et al. 2015, Grubb et al. 2015). The results demonstrated that the deflection of these decks at the service load was smaller than the most stringent allowable deflection. Therefore, all these decks were deemed to satisfy the requirements of ASHTTO and US DOT (AASHTO 2007, Grubb et al. 2015) at the serviceability and ultimate state.

Performance of reinforced UHPC decks

Two solid reinforced UHPC decks SUCS and SUCF were tested to provide a benchmark for the analyses. The general cracking pattern of the solid reinforced UHPC decks is shown in Fig. 9. The load increased linearly during the early stage of the test and no crack was observed up to approximately 30 kN. As illustrated in the figure, the formation of the primary cracks of the decks initiated at the tension face and midspan of the deck under the loading area when reaching the cracking bending moment. After the initial cracking, more new cracks developed between the existing primary cracks. This cracking behaviour was very different from the typical development of flexural cracks in a conventional RC deck which usually concentrates on a few cracks at the tension zone. This phenomenon indicated the ability of UHPC to undergo multiple cracking before tensile failure due to an excellent stress redistribution. Tensile failure of UHPC is the consequence of elongation and pulling out of fibres from the matrix. As a result, the fibres nearby must carry higher stress. When the applied load increased, additional cracks formed and the fibres bridging the existing cracks experienced higher stress.

Just before the peak load of Deck SUCS (113.5 kN), the fibres at primary cracks began to pull out. These cracks' width became significantly wider than other cracks in the deck, as shown in Fig. 9. Thereafter, the flexural failure of the deck was precipitated by the local bond failure between fibres and the UHPC matrix as well as rebar yielding. The yielding point is for general behaviour of the deck to represent its behaviour when changing from linear to nonlinear behaviour. The behaviour of Deck

SUCF was similar to Deck SUCS up to the peak load, while a web-shear collapse occurred suddenly for Deck SUCF at the peak load (194.3 kN). 70% increment of the applied load was observed in control UHPC decks by replacing steel bars with GFRP bars. This improved load-carrying capacity will be discussed subsequently. Unlike Deck SUCS which failed in flexure, the failure of Deck SUCF was governed by a critical diagonal crack farthest from the mid-span led to a sudden shear failure of the deck. This failure occurs when the principal tensile stress exceeds the tensile strength of concrete in absence of shear reinforcements (ACI 2019).

As can be seen from Fig. 8, the load-defection curves of Deck SUCS exhibited four distinct phases.

- Phase one, linear elastic behaviour, the mid-span deflection developed with the load linearly increased up to approximately 31 kN. The maximum deflection in this phase was approximately 0.33 mm. No noticeable crack was observed in UHPC up to this level. The appearance of the crack indicates the end of phase one.
- Phase two, deflection hardening and crack propagation, the mid-span deflection increased nonlinearly with the load up to approximately 90 kN. Minor cracks were initiated at the bottom of the specimen especially at the mid-span underneath the loading area. Multiple cracks propagated gradually in UHPC only with a small reduction in stiffness of the specimen due to the stress redistribution of the PVA fibres. The maximum deflection at this stage was approximately 3.91 mm. Deck SUCS showed that after the occurrence of the first crack, the reinforced UHPC specimen could maintain its stiffness and sustain the applied load up to approximately 90 kN when yielding started.
- Phase three, the yielding phase, when the applied load increased, the primary cracks propagated upwards and widened. In contrast with Phase two, no noticeable increase in crack numbers was observed in this phase. A substantial increase in the mid-span deflection was observed due to yielding of longitudinal steel bars. The load rose slowly to 113.5 kN while deflection significantly increased to 9.57 mm. The peak load indicates the end of Phase three.
- Phase four, deflection softening behaviour, the crack width widened rapidly followed by a gradual drop of the applied load. The bottom steel bars could not resist higher stress after yielding and the specimen consequently failed in flexure mode with crushing of top concrete.

The deflection behaviour of Deck SUCF was slightly different from Deck SUCS. The linear elastic behaviour (Phase one) was observed up to approximately 30 kN. It can be seen that when a new crack was initiated, the load dropped slightly due to the release and redistribution of stress. Primary cracks occurred in this step and more cracks developed as the applied load increased, similar to the second phase of Deck SUCS. Linear behaviour before concrete cracking and non-linear behaviour with gradual stiffness reduction until reaching the peak load was also reported in previous studies for specimens reinforced with FRP bars (Tran et al. 2020b, Huang et al. 2021a). The yielding point, corresponding to the beginning of Phase three, was clearly seen at 87.2 kN and 146.5 kN for Decks SUCS and SUCF, respectively. It can be seen that the yielding load of Deck SUCF was much higher than that of Deck SUCS. This distinguished behaviour is attributed to the differences in mechanical properties of GFRP vs steel. It is noted that the yielding mechanism of these two decks is different. The yielding of Deck SUCS was governed by yielding of concrete and steel while only concrete governed the yielding of Deck SUCF. A similar yielding mechanism of FRP reinforced concrete beams was also reported in previous studies (Kim and Jang 2014, Huang et al. 2021b). Different from Deck SUCS, the ductile post-peak behaviour was not observed for Deck SUCF, for which the load suddenly dropped due to concrete crushing and shear failure. The higher peak load and different failure modes of Deck SNCF as compared to Deck SNCS were attributed to the higher tensile strength of the GFRP bars as compared to the steel bars which increased the flexural strength of the deck. While the low dowel action of GFRP bars in absence of shear reinforcements reduced the shear strength of Deck SUCF. Accordingly, the flexural capacity governed the failure of Deck SUCS while shear capacity dominated that of Deck SUCF.

Performance of decks with stay-in-place formwork

Unlike the control decks, flexural cracks were not observed in UHPC specimens with stay-in-place formwork until a high load level (approximately 50 kN) because the flexural strength of the decks was significantly improved by the structurally integrated stay-in-place formworks. This behaviour clearly shows that the surface treatment of GFRP stay-in-place formwork provided good bonding to sufficiently transfer bond stresses at the interface between the GFRP formwork and UHPC to distribute cracks. The peak load of the reinforced UHPC deck containing Y-shape stiffened stay-in-place formwork (Deck

SUYF) was 230.9 kN which was 19% and 103% higher than the control decks SUCF and SUCS, respectively. The response of Deck SUYF was governed by flexure mode up to roughly its peak load and the deck eventually failed by punching shear. The peak load of the UHPC deck containing SHS stiffened stay-in-place formwork (SUSF) was 149.1 kN, which dropped by 23% as compared to the control specimen SUCF. This phenomenon could be attributed to the 15% reduction in thickness of the specimen SUSF as compared to the control deck SUCF and the low shear capacity of SHS stiffeners while the failure of these decks was governed by shear. In addition, the hollow section created voids, considerably reduced the shear capacity of Deck SUSF and led to its punching shear failure.

The load-deflection curves of the decks with stay-in-place formwork (Decks SUSF and SUYF) were linear up to almost failure. This behaviour meant that the stiffness of these two decks did not change up to almost failure and thus low deflection was observed at failure. The pre-cracking stiffness of the control decks was slightly higher than that of the decks with stay-in-place formwork. This was attributed to the 15% larger thickness of the control decks which has a direct influence on the initial cracking moment before tensile cracks appeared in concrete on the tension side. The control decks had a larger overall depth due to the existence of the bottom concrete cover (10 mm), which did not exist in the decks with GFRP stay-in-place formwork. The tensile strength of concrete in the tension side before cracking contributed to the initial flexural resistance therefore the cracking moment of the reinforced decks (Decks SUCS and SUCF) was greater than that of the decks with stay-in-place formwork (Decks SUSF and SUYF). It is worth mentioning that the initial cracking load was identified by the change in the slope of the load-deflection curves and/or the occurrence of the first crack. However, because there is no clear change in the slope of the stress-strain curve of Decks SUSF and SUYF, their cracking load is determined when visual observation noticed the occurrence of the first crack.

Although the control deck reinforced with FRP bars (Deck SUCF) failed in shear with the load dropping suddenly upon reaching the peak load, Deck SUSF showed a reduction in stiffness at 97% of peak load (approximately 145 kN) due to the high dowel action of stiffeners and progressive tearing of the SHS GFRP stay-in-place formwork.

The stiffness of Deck SUYF was relatively similar to Deck SUSF up to the applied load of approximately 100 kN but with a significantly higher load-carrying capacity because the Y-shaped stiffener increased the shear capacity of the deck and mitigated the shear failure. As shown in Fig. 8, even though the peak load of Deck SUYF was significantly higher than that of the control decks, the deflection at the peak load of the Deck SUYF was significantly lower than those of the control decks. The higher peak load and smaller deformation of the deck with Y-shaped GFRP stay-in-place formwork was a result of higher shear resistance of the formwork and lower deformability of stiffeners of Deck SUYF.

Effect of stiffener configuration

As shown in Fig. 8, the initial stiffness of the decks with stay-in-place formwork was similar before the applied load reached approximately 85 kN regardless of the type of stiffeners. The first crack of both the decks occurred at approximately 20 kN with the corresponding deflection of 0.30 mm. Although both Decks SUSF and SUYF failed in punching shear, the peak load of the deck comprising Y-shape stiffeners was 55% higher than the deck comprising SHS stiffeners. Considering that both decks used the same amount of FRP material, the results indicated the higher efficiency of using Y-shape FRP stiffeners. It is noted that Deck SUSF had voids inside SHS which also reduced the shear resistance from UHPC. Therefore, this observation should be confirmed again with Deck SUSF in which SHS is filled with UHPC, which requires more labour cost in construction.

To investigate the failure behaviour of stiffeners, the decks with GFRP stay-in-place formwork were sectioned through the longitudinal (a-a) and transverse (b-b) centre line as demonstrated in Fig. 2. As it can be seen in Fig.11, both Y-shape and SHS stiffeners experienced tension-shear-coupling failure associated with the tension shear cracks within the shear zone. Deck SUYF failed through rupture of the Y-shape stiffeners at the junction of the web and the bottom plate or web and the top flanges. In addition, the bottom plate delaminated from the rest of the deck and concrete damaging at the top of the deck was also observed. Visual inspections showed that a thin layer of concrete was attached to the surface to Y-shape stiffeners and the concrete shear crack occurred very close to the interface. Therefore, it can be concluded that no debonding between Y-shape stiffeners and concrete occurred. It indicated sufficient bonding between the stiffeners and concrete resulted from a good combination performance of surface treatment and high mechanical interlock. As shown in Fig. 11, SHS stiffeners failed by deforming and rupturing the hollow section due to its low shear capacity followed by critical debonding between concrete and stiffeners, as well as top concrete damage. In contrast with Deck SUYF, debonding between SHS stiffeners and concrete is a result of the slip and high shear stress in the interface area which occurred due to deformation of SHS stiffeners and lack of decent mechanical interlock. It can be concluded that after the failure of the SHS stiffeners, the effective depth of the deck was reduced by 63% and only the concrete from the top of the stiffener to the loading pad provided punching shear resistance. As a result, Deck SUSF showed a 30% lower peak load as compared to the control deck SUCF.

It is worth mentioning that the surface treatment for both decks with SHS and Y-shape stiffeners was similar. However, Y-shape stiffeners showed significantly better bonding performance which led to higher composite action. The composite action between concrete and stay-in-place formwork not only relies on the mechanical interlock and adhesive bonding between concrete and formwork (Keller et al. 2007, He et al. 2012) but also depends on the bonding between stiffeners and the plate. In other words, integrity is an important factor in the performance of stay-in-place formwork to act as a one-piece component. This integrity is directly related to the bonding interface area between stiffeners and GFRP plates. As shown in Fig. 3, the stiffeners to the GFRP plate interface area in SHS stiffened formwork was approximately 60% larger than that of Y-shape stiffened formwork. As a result, sufficient bonding was observed for Decks SNSF and SUSF which were stiffened by SHS stiffeners. While the bottom delamination was observed in Deck SUYF after large deformation due to a smaller bonding interface area between stiffeners and GFRP plates.

Additionally, it is widely accepted that the total nominal shear strength of an FRP reinforced concrete cross-section is the sum of the shear resistance provided by concrete and the shear reinforcement. In this study, the tensile reinforcements were replaced by integrated permanent stay-in-place formwork. The better performance of the deck with Y-shape stiffeners is not only attributed to the higher contribution of concrete but also attributed to the higher contribution of FRP stiffeners. The most effective design is to place shear reinforcement at the centre of a section to resist the maximum shear stress of the shear flow. As shown in Fig. 12, the cross-section area of concrete and Y-shape stiffeners in the middle region of the deck were respectively 1.6 and 1.2 times higher than that of the deck with SHS stiffeners, considering that both decks used the same amount of FRP material. Therefore, the shear resistance of Deck SUYF was significantly higher than Deck SUSF.

The tensile strain at the mid-span of the GFRP plates of Decks SUSF and SUYF is shown in Fig. 10. The tensile strain of the GFRP plate at bottom of the decks increased almost linearly from the beginning until the peak load. The maximum GFRP longitudinal strain in Deck SUSF and Deck SUYF was approximately 4,000 μm/m and 16,700 μm/m, respectively, which were 24% and 98% of the 17,000 μm/m nominal rupture strain of the GFRP plate. This very substantial difference was attributed to the low utilization of GFRP plate capacity in SUSF due to its low modulus and over-reinforced section. The strain value of the GFRP plate for Deck SUYF linearly increased up to approximately 5,300 μm/m as the applied load increased up to approximately 200 kN. Thereafter, the strain value dramatically increased from 5,300 μm/m to 16,700 μm/m while the applied load increment was insignificant. This is attributed to the transferring considerable stress to the GFRP bottom plate after crack propagation in conjunction with initiating the Y-shape FRP stiffeners rupture. However, no significant stiffness reduction was observed in the deck deflection response due to an excellent contribution of GFRP stay-in-place formwork. It should be mentioned that the strain gauge installed on the bottom GFRP plates of Deck SUYF was damaged after a large strain corresponding to a 205 kN loading. This observation is interesting and it requires further investigation to unveil clearly the mechanism behind it. Meanwhile, the brittle post-peak behaviour of Deck SUYF was due to the shear failure of stiffeners at a high loading level. The longitudinal strain of the GFRP plate was still below the ultimate material strain and no rupture was observed. The extremely high utilization of GFRP plate in Deck SUYF has clearly proven the advantages of using Y-shape stiffeners as compared to SHS stiffeners in these structures.

To further investigate the behaviour of the stiffeners, the relation between the applied load and strain on the stiffeners is presented in Fig. 13. It is clear that the strain of GFRP stiffeners in all the decks linearly increased from the beginning until the first crack. Strain at midspan (SG V1-3) was much greater than that at SGs L1-3 and SGs R1-3. Considering the shear damage surface, these strain gauges should exhibit similar results. The substantial difference in the strain of SGs V1-3 vs SGs L1-3/R1-3 indicated strain induced by flexural response dominate the total measured strain. After UHPC cracking, the contribution of UHPC was lost and considerable stress was transferred to the stiffeners that led to a reduction of composite action. Accordingly, the slope of the curve significantly reduced after the first crack, indicating a rapid increase of strain. SG L1 and SG R1 showed the highest level of strain redistribution as there were located near the loading area and the first flexural crack occurred underneath the loading area. For Deck SUSF, as the applied load increased, more flexural cracks appeared near SGs L2 and R2. A noticeable reduction in the slope of curves was observed at approximately the 30 kN loading level due to critical cracking and transferring large stresses to the stiffeners which led to an increase of strain. Before the peak load, SGs L2-3 showed reversal strain due to the rupture of SHS stiffeners. SG R2 also showed a reversal strain at 84% of peak load due to shear failure of the stiffener in the vertical direction, however, the stiffener was able to carry more stress as the applied load increased. A similar reversal strain was also reported in previous studies (Nelson and Fam 2013, 2014). The ultimate tensile strength of GFRP in this study was 4 times lower than its ultimate tensile strength in the longitudinal direction. Hence, utilizing FRP stiffeners with bi-directional fibres with higher transverse tensile strength can be an alternative solution to prevent premature failure in the transverse direction.

Stress redistribution of the Y-shape stiffeners in the longitudinal direction was very small while the corresponding strain in the transverse direction was considerable in Deck SUYF. The results showed that the maximum tensile strain of SHS stiffeners of Deck SUSF at SGs V2-3 in the transverse direction was approximately 600 μm/m and 550 μm/m at the peak load, respectively. While the corresponding strain of the Y-shape stiffeners of Deck SUYF at SGs V2-3 at the same load level (149.1 kN) was approximately 980 μm/m and 800 μm/m, respectively. Considering both the decks failed by punching shear and the concrete strength of both decks was the same, the higher strain value in Y-shape stiffeners (50% increase) as compared to SHS stiffener was the result of the remarkable contribution of Y-shape stiffener and good mechanical bonding between stiffener and concrete which increased the contribution of stiffeners to the shear resistance. The maximum strain of the Y-shape stiffeners was approximately 2,100 μm/m in the transverse direction, which demonstrated the stiffeners resisted well the shear stress in the UHPC deck. The higher strain in the transverse direction as compared to the longitudinal direction in both decks is attributed to the governing of the flexural behaviour of the decks until the peak loads. Accordingly, SGs V1-3 were located at midspan and thus the elongation due to bending governed the reported strain which was much greater than those in the longitudinal direction.

Effect of concrete strength

The effect of concrete strength on the performance of decks cast on stay-in-place formworks is discussed via comparing Decks SNSF vs SUSF. Both the decks were identical in all aspects, except the concrete strength. PVA-UHPC with the compressive strength of 140 MPa and splitting tensile strength of 12 MPa was used to cast Deck SUSF, while normal concrete with the compressive strength of 34 MPa and splitting tensile strength of 3 MPa was utilized in Deck SNSF. Even though both the decks failed in punching shear, Deck SUSF clearly showed a higher loading capacity due to the higher concrete strength. The maximum load-carrying capacity of Deck SUSF was 149.1 kN, which was 63% higher than that of Deck SNSF (91.4 kN). This is attributed to a higher splitting tensile strength of UHPC vs normal strength concrete (12 MPa vs 3 MPa). The flexural crack was observed on the tension zone of both the decks and the number of flexural cracks in Deck SNSF was significantly lower (see Fig. 9) due to the absence of fibre in concrete while Deck SUSF exhibited more number of cracks with smaller width, indicating more uniform crack distribution and thus better material's utilization.

Finally, Deck SNSF failed by sudden punching shear cracks as Deck SUSF. As shown in Fig. 8, the stiffness of Decks SNSF and SUSF was linear and almost similar before cracking. This was because the influence of PVA fibres had not been activated yet at the early stage. Similar behaviour was also reported in the previous study (Yoo and Yoon 2015). After the initial cracking, new cracks developed in Deck SUSF. This cracking behaviour was different from Deck SNSF that experienced only one major flexural crack with wider crack width. This phenomenon demonstrated the multiple cracking behaviour of UHPC due to the ability to redistribute the stress before tensile failure. Deck SNSF exhibited a slightly better ductility beyond punching shear while the post-shear failure of Deck SUSF was not ductile (see Fig. 8). This phenomenon is due to the gradual deforming, rupturing and partial debonding between concrete and GFRP stay-in-place formwork after concrete failure in lower load level. The deformation of the SHS stiffeners was observed upon inspecting the deck after testing.

Meanwhile, Fig. 10 indicates a low utilization of the bottom GFRP plate of Decks SUSF and SNSF (maximum longitudinal strain of approximately 4,000 μm/m as compared to the maximum rupture strain of the GFRP plate of 17,000 μm/m). This low utilization of GFRP plate capacity is not merely because of the over-reinforced section, but also a result of premature rupture failure of SHS stiffeners which led to the failure of the deck before fully engaging the bottom plate. Therefore, it can be recommended that the capacity of the bottom GFRP plate with 3.2 mm thickness is significantly more than the required capacity of the decks with SHS stiffeners and GFRP plates with smaller thickness can be used in case of using SHS stiffeners to save the material cost.

The results indicate that the maximum tensile strain of stiffeners in the longitudinal direction for Deck SNSF was 650 μm/m at the peak load (91.4 kN), which was almost twice the corresponding strain of Deck SUYF. The reinforcements for these two decks were the same and both failed in shear so that the shear response governed the behaviour of these specimens. Accordingly, the lower longitudinal strain value in Deck SUSF as compared to Deck SNSF was the result of the notable contribution of UHPC in shear resistance. The fibre bridging mechanism assisted UHPC to carry a big portion of loads after cracking (Pournasiri et al. 2018). The maximum strain of the SHS stiffeners of Deck SUSF in the longitudinal direction was approximately 660 μm/m at 149.1 kN, which was almost similar to the maximum stiffeners strain of Deck SNSF at failure. This result also confirmed the better contribution of the UHPC to the shear resistance of the decks with stay-in-place formworks. Similar results were also observed in the transverse direction. The maximum tensile strain of SGs V2 and V3 in the transverse direction of Deck SNSF at the peak load was significantly greater (3~4 times) than those of Deck SUSF. This observation further demonstrated the better influence of UHPC on the shear resistance of Deck SUSF by transferring fewer stresses to the stiffeners after cracking.

In general, UHPC increased the system performance due to higher stiffness after the initial cracking and punching shear capacity as compared to normal concrete. However, the specimen with UHPC suffered brittle failure due to the sudden rupturing of GFRP stiffeners at a high applied load.

Analysis of shear strength of decks with stay-in-place formworks

An analytical model was developed by Noël and Fam (2016) to predict the shear strength of decks with stay-in-place formworks. The ultimate nominal shear strength (P_n) is given as follows:

$$\frac{P_n}{b_0 d_{ave}\sqrt{f'_c}} = 0.375\left(\omega_{eff} + 0.0334\right)(B + 4.0)(5.6 - L) \quad (1)$$

where b_0 is the perimeter of the punching shear failure plane, d_{ave} is the weighted average effective depth, ω_{eff} is the reinforcement ratio normalized with respect to the ratio of FRP and steel modulus, B is the deck width and L is the span length. f'_c is the compressive strength of concrete. b_0, d_{ave} and ω_{eff} can be calculated as follows:

$$d_{ave} = \frac{d_x c_x + d_y c_y}{c_x + c_y} \quad (2)$$

$$b_0 = 546 + 2d_x + 2d_y \quad (3)$$

$$\omega_{eff} = \frac{\omega_x c_x + \omega_y c_y}{c_x + c_y} \quad (4)$$

where d_x is the concrete thickness to the bottom GFRP plate, d_y is the concrete depth measured to the centroid of the FRP repeated pattern, c_x and c_y are the ratios of perimeters in each of the orthogonal directions to the total perimeter and are GFRP reinforcement indices in each of the orthogonal directions. ω_x and ω_y are FRP reinforcement indexes in the traffic and normal to traffic directions. c_x, c_y, ω_x and ω_y are given by

$$c_x = \frac{192 + d_y}{273 + d_x + d_y} \quad (5)$$

$$c_y = \frac{81 + d_x}{273 + d_x + d_y} \quad (6)$$

$$\omega_x = \rho_x \frac{E_x}{E_S} \tag{7}$$

$$\omega_y = \rho_y \frac{E_y}{E_S} \tag{8}$$

where ρ_x and ρ_y are the GFRP reinforcement ratios in the two orthogonal directions, E_x and E_y are Young's modulus of the GFRP section in the two directions and E_s is Young's modulus of steel.

The comparison of the predicted values and experimental results is summarized in Table 1. Predicted to experimental ratio for Deck SNSF was 97% which showed a good arrangement in the case of using SHS stiffened stay-in-place formwork in conjunction with normal concrete. The predicated punching shear capacity of the Decks SUYF and SUSF showed reasonable agreement where the average predicted-to-experimental load ratio was 81% with a standard deviation of 19%. It is worth mentioning that the model was intended for decks with normal concrete, and it showed the best agreement in conjunction with stay-in-place structural formworks stiffened with SHS stiffeners. This variation is because of the presence of fibres in UHPC and the different mechanical properties of UHPC as compared to the normal concrete.

The predicted punching shear confirmed that decks with Y-shaped stiffeners can carry more load as compared to the deck with SHS stiffeners. Modification is required for predicting decks with the proposed Y-shape stiffeners and UHPC.

Conclusion

This study investigated the structural performance of PVA-UHPC cast on a GFRP stay-in-place formwork using a new configuration. The following findings have been drawn based on the results discussed in this paper:

1. Unlike from the control decks SNCS and SNCF, the loading capacity of the decks with GFRP stay-in-place formwork was governed by shear capacity. Their ultimate capacity depended on the configuration of stay-in-place formwork and concrete strength.

2. The loading capacities of the decks with stay-in-place formwork were 3.8 to 9.5 times higher than the established equivalent design service load level based on the half-axle load of the CL-625 design truck considering the dynamic loading effect. All the decks tested in this study also met deflection limits of $L/1,600$ at equivalent service load.
3. The load-carrying capacity of the decks was enhanced due to utilizing the structurally integrated GFRP stay-in-place formworks. The first crack occurred at a high loading level due to good transferring stresses from concrete to the GFRP stay-in-place formwork.
4. The peak load of the deck with UHPC was 63% higher than that of the corresponding deck with normal concrete, increasing from 91 kN to 149 kN. UHPC significantly improved the shear resistance of the deck with normal strength concrete.
5. The proposed new Y-shape stiffened stay-in-place formwork can be the alternative replacement for the conventional tensile bar reinforcement by achieving a 55% higher loading capacity as compared to SHS stiffened formworks due to the increased shear resistance.

The use of UHPC and Y shape stiffeners have significantly improved the shear capacity of the decks with stay-in-place formwork up to 153%. Hollow formwork with SHS stiffeners can be used to reduce the self-weight of the structure with a lower shear resistance. Concrete-filled SHS can be used to improve the shear resistance but it requires additional filling step and hence increases the labour cost. Future work is also required to improve the design to take advantage of the improved properties of the UHPC.

Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

Acknowledgements

The financial support from the Australian Research Council Laureate Fellowships FL180100196 is acknowledged. The authors wish to thank Sika Australia Pty Ltd and Silicon Metal Company of Australia (SIMCOA) for providing the required material for this study.

Notation

B = deck width;

L = span length;

b_0 = perimeter of the punching shear failure plane;

c_x = ratios of punching shear perimeters in in direction of traffic to the total perimeter;

c_y = ratios of punching shear perimeters in in direction of normal to traffic to the total perimeter;

d_{ave} = weighted average effective depth;

d_x = the concrete thickness to the bottom GFRP plate;

d_y = the concrete depth measured to the centroid of the FRP repeated pattern;

E_s = Young's modulus of steel;

E_x = Young's modulus of the GFRP section in in direction of traffic;

E_y = Young's modulus of the GFRP section in in direction of normal to traffic;

f'_c = compressive strength of concrete;

P_n = ultimate nominal shear strength;

ρ_x = the GFRP reinforcement ratios in the in direction of traffic;

ρ_y = the GFRP reinforcement ratios in the in direction of normal to traffic;

ω_{eff} = reinforcement ratio normalized with respect to the ratio of FRP and steel modulus;

ω_x = the GFRP reinforcement index in the direction of traffic;

ω_y = the GFRP reinforcement index in the direction normal to traffic.

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Table 1. Summary of the test matrix.

ID	f'_c (MPa)	Depth (mm)	Reinforcements		P_{exp} (kN)	P_{pred} (kN)	P_{pred} /P_{exp}	Failure mode
			Bottom	Top mesh				
SUCS	140	75	Steel mesh	Steel	113.5	-	-	Flexural
SUCF	140	75	GFRP mesh	GFRP	194.3	-	-	Punching Shear
SUSF	140	65	SHS SIP formwork	GFRP	149.1	179.7	1.21	Punching Shear
SUYF	140	65	Y-shape SIP formwork	GFRP	230.9	190.6	0.83	Punching Shear
SNSF	34	65	SHS SIP formwork	GFRP	91.4	88.3	0.97	Punching Shear

Table 2. Mixture proportions of normal concrete.

UHPC constituent	weight (kg/m^3)
Cement	248.5
Slag	106.5
10 mm aggregate	1015
Sand	809
Water	172
Superplasticizer	1.4

Table 3. Mixture proportions of UHPC.

UHPC constituent	weight (kg/m^3)
Cement	1,000
Silica fume	250
Silica sand	1,100
Water	170
Superplasticizer	65
PVA fibre (2% vol.)	26