STRUCTURAL BEHAVIOR OF UHPFRC-FILLED, TRANSVERSE C-JOINT IN FULL-DEPTH, GFRP-REINFORCED, PRECAST BRIDGE DECK PANELS RESTING OVER STEEL GIRDERS

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Abstract: Precast full-depth deck panels (FDDP) with transverse joints, placed over steel or concrete girders, are efficient in rapid bridge replacement. In this system, grouted pockets are provided to accommodate clusters of shear connectors welded to steel girders or embedded in concrete girders. In this research, ultra-high performance fibre-reinforced concrete (UHPFRC) and high-modulus glass fibre reinforced polymer bars are utilized in the closure strip between the adjacent precast for enhanced strength and durability. Two actual-size, GFRP-reinforced, precast FDDPs were erected to perform fatigue tests using the foot print of the truck wheel loading specified in the Canadian Highway Bridge Design Code (CHBDC). Each FDDP had 200 mm thickness, 2500 mm width and 3700 mm length in the direction of traffic and rest over braced twin-steel girder system. The transverse closure strip between connected precast FDDPs has a width of 200 mm with female-to-female vertical shear key designated as C-shape joint to increase moment capacity along the interface between the UHPFRC and the precast FDDP along the joint. GFRP bars in the precast FDDPs project into the closure strip with a development length of 175 mm. Two types of fatigue tests were performed, namely: (i) high-cyclic constant amplitude fatigue loading followed by loading the slab monotonically to-collapse; and (ii) low-cyclic accelerated variable amplitude cyclic loading. Overall, the test results demonstrated the excellent fatigue performance of the developed closure strip details. In addition, the ultimate load carrying capacity of the slab was far greater than the factored design wheel load specified in CHBDC.

1 INTRODUCTION

Precast full depth deck panels (FDDPs) have recently used in new accelerated bridge construction (ABC) or for the rapid bridge replacement (RBR) of existing deteriorated bridge decks. FDDPs are produced off-site, quickly assembled on-site, reduce construction time, minimize lane closure and are considered as a good solution to minimize traffic disruption (Clumo, 2011). FDDPs are placed side by side as shown in Fig. 1, then the closure strips between them are filled with bonding material. FDDP closure strips should take the advantage of high quality concrete and non-corrosive reinforcement as glass fiber reinforced polymer (GFRP) bars for enhanced strength and durability. GFRP reinforcement is a composite material made of polymer matrix reinforced with fibers. It has high strength-to-weight ratio, is free of corrosion and lasts longer. Although the Canadian Highway Bridge Design Code, CHBDC (CSA, 2006) allows the use of GFRP-reinforced FDDPs in bridge construction, there is no code provision on the joint details between such precast systems. The behaviour of the FDDP monolithic concrete joint (MCJ), also known as moment resisting connection (MRC), accounts for the state of bond of the projected longitudinal bars anchored through the cast-field joints. The Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) is a relatively new class of cementitious matrix with steel fiber content that has high compressive strength (in order of 140 MPa) and relatively large tensile strength (in order of 8 MPa), with strain hardening behavior in tension that ensure crack opening remain very small (Russell and Graybeal, 2013). The use of UHPFRC as a filling material of the closure strip between connected FDDPs have numerous benefits, including reduction of joint size, improving durability, speed of construction and prolonging usage life.

Deflection and vibration play an important role on the serviceability of bridges. The current AASHTO-LRFD Bridge Design Specification (AASHTO, 2012) specifies the bridge deflection limits at L/800 for vehicular bridges and L/1000 for pedestrian bridges as optional criteria where L is the span of the structural element. Traditionally bridges are designed using static loads that include the dynamic load allowance (DLA) due to passing trucks at the ultimate, serviceability and fatigue limit states. It is important to examine the structural behaviour of the jointed precast FDDPs under different fatigue loading conditions which lead to progressive, internal and permanent structural changes in the materials. After the crack initiation and propagation, failure is caused by the deterioration of the bond between coarse aggregate, reinforced bars and the binding matrix. Two types of fatigue loading are considered in testing, namely: constant amplitude fatigue loading (CAF) and variable amplitude fatigue loading (VAF). Fatigue loading is known to reduce the life span for the bridge deck (Karunananda et al., 2010). The constant amplitude fatigue (CAF) is the classical method for fatigue analysis of the materials to obtain the three fatigue resistance components and structures, namely: stress-life (S-N) known as Wöhler curve, strain-life (ε-N) and fatigue crack growth (FCG). CAF limit is the safe stress level under elastic deformation for design that can take a very large number of cycles, longer than one-million cycles. The Average Daily Truck Traffic (ADTT) of 100 trucks per lane over 25 years produce number of cycles exceeds the CAF limit of two million cycles. The variable amplitude fatigue (VAF) limit investigates the effect of periodic overloading cycles. VAF is based on the same concepts with addition of cycle counting and damage summation due to increasing step loading. However, the resulting stresses are high enough for plastic deformation to occur within the number of cycles very much less than one-million cycles.

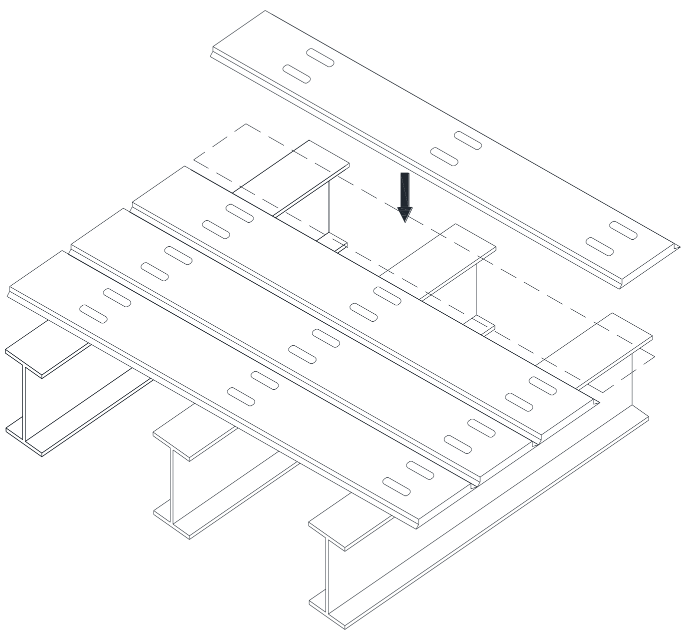


Fig. 1. Isometric view of a precast full-depth, full width, deck panels placed transversally over girders

The use of the precast FDDPs in bridge construction started in United States in early 1960s, with the purpose to shorten the deck construction in areas with high traffic volumes. The deck-girder system was primarily non-composite, and the panel-to-panel connections exhibit partial failures. By 1974, FDDPs were made composite with the superstructure by extending the steel shear stud into the deck. The spacing of the shear pockets ranged from 457 mm to 610 mm and the number of studs per pocket ranged from 4 to 12. Two sizes of steel studs are typically used, namely: 19 mm and 22 mm. FDDP were supported on girders and secured to it using the shear studs embedded in the shear pockets that are normally filled with non-shrink grout to eliminate stress concentrations in the panels (Badie and Tadros, 2008). The transverse panel-to-panel connection is provided with shear keys to protect adjacent panels from relative vertical movement due to traffic load. This type of joint has two types of forces, namely: (i) vertical shear force between the panel and the field-casted joint; and (ii bending moment that puts the top half of the joint in compression and the bottom half in tension. The panel-to-panel connection has several shapes available in the literature including male-female (tongue/groove) shear key. However cracking, spalling and leakage were observed in such joints in practise. Other panel-to-panel connections included female-to-female shear key which comes into bulb shape, and diamond shape. Splicing longitudinal reinforcement was introduced into overlapping U-bars or using HS spirals, or using open or closed steel tubes (PCI, 2011a and 2011b). Grouting materials to fill the shear pockets and transverse joints have common properties as: (i) high strength at young age, (ii) small shrinkage deformation, (iii) superior bonding and (iv) low permeability (Badie et al., 2006). The steel reinforcement lap-splice joints exploit bonding performance with the joint-field materials made of UHPFRC (Hwang and Park, 2014). The direct tension of GFRP bars were pulled out from UHPFRC blocks to determine the development length (Sayed-Ahmed and Sennah, 2014c). This led to developing few precast panel connection details that were considered for qualifying tests. Three developed female-to-female connections for the GFRP-reinforced precast FDDPs with lap-spliced bars were constructed in full scale in the laboratory to examine their strength and serviceability under increasing static loading with the use of normal strength concrete (Sayed-Ahmed and Sennah, 2014a) and high-performance concrete (Sayed-Ahmed and Sennah, 2014b) in the precast deck slabs. This research led to the current research reported in this paper. This paper reports the experimental program to test one of these developed joints in real-world situation. Two precast deck slabs were constructed over twin-steel girder system, with one of them tested under CAF loading followed by loading it monotonically to-collapse, and the other one was tested under VAF loading directly to-collapse. Test results are analyzed to examine the fatigue performance and the ultimate load carrying capacity of the developed jointed precast slabs.

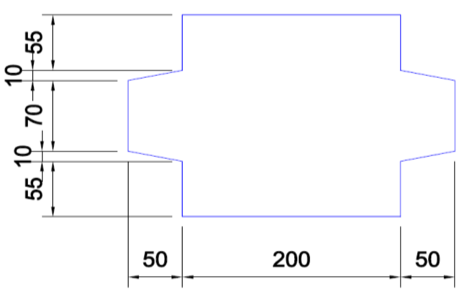
2 NEW CONNECTION DETAILS

Figure 2 depicts the C-shape panel-to-panel connection with vertical female-to-female shear key. The clear joint width between the ends of the jointed panels is 200 mm. So a projecting GFRP bar from the end of one panel will project into the joint with a length of 175 mm. The pullout strength of the embedded GFRP in the joint will be resisted by the bond between its surface and the surrounding UHPFRC. The slope of the inner side of the shear key has a slope of 1vertical to 5 horizontal. The vertical shear key is expected to provide vertical shear friction resistance between the precast concrete and the UHPFRC filling to allow for vertical shear continuity of the slab across the joint.

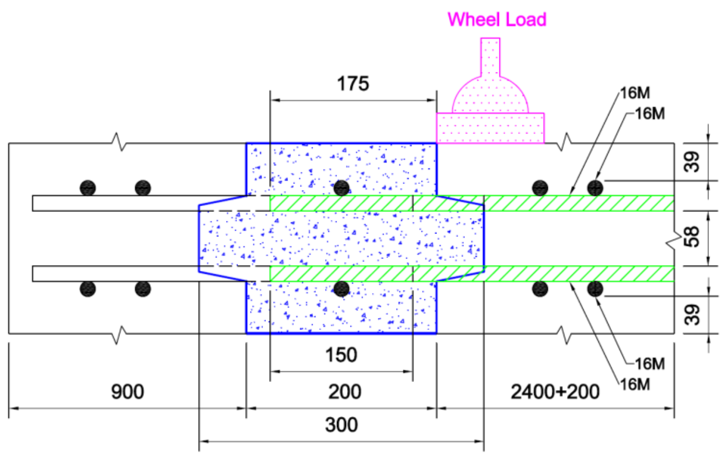
3 EXPERIMENTAL PROGRAM

The experimental program included testing two laterally restrained precast FDDPs supported over /twin-steel girder bridge system, using the available force-control hydraulic actuator system. The width of the cast slab is 2500 mm so that it can be supported over the twin girders to produce slab span of 2000 mm as depicted in Fig. 3(a). The precast slabs were of 200 mm thickness and were made of 35 MPa normal strength concrete (NSC) with 10 mm nominal size aggregate, 150 mm slump with added super plasticizer, and no air-entrant. Straight-ended, 16M ribbed-surface, high-modulus HM GFRP bars was used to reinforce the precast slab per CHBDC requirements. The bottom and top transverse reinforcement of the slab was taken 16M@140 mm and 16M@200 mm respectively. While the slab was reinforced with 16M@200 mm HM GFRP bars in the bottom and top longitudinal direction (i.e. parallel to the girder). The specified modulus of elasticity and ultimate tensile strength of the GFRP bars were 64 GPa and 1188 MPa, respectively (Schoeck, 2012). To form the joint between the precast FDDPs, two precast FDDPs were formed first. The first precast FDDP was of 200 mm thickness, 2400 mm length in girder direction and 2500 mm, while the second precast FDDP was of 200 mm thickness, 1000 mm length in girder direction and 2500 width. This made the final dimensions of the jointed slab of 3700 mm in the direction of traffic as depicted in Fig. 3(a). It should be noted that the short precast FDDP of 1000 mm was introduced beside the large precast FDDP to ensure deck slab continuity beyond the joint. Figure 3(c) show views of the formwork, HM GFRP bar arrangement and Styrofoam used to form the joints and the shear pockets before casting concrete. The panel-to-girder connection was made using shear pockets to achieve the full composite action. Shear studs were used to establish such full composite action between the girder and the precast panel every 1200 mm. High tensile headed shear studs (structural bolts) of 25 mm diameter were used. The UHPFRC (Ductal Joint Fill JS1000) was used for the cast-in-place of the panel-to-panel connection. The ultimate strengths of the UHPFRC were 140 MPa, 30 MPa and 8 MPa in compression, flexural and direct tension, respectively, while its modulus of elasticity was 50 GPa. More details about the experimental program can be found elsewhere (Sayed-Ahmed, 2014).

The experimental program included testing two precast FDDPs supported over /twin-steel girder bridge system, using the available force-control hydraulic actuator system. The steel I-girders were 7500 mm in length and made of W610x241. They were placed over 330x330x25 mm elastomeric pads that were supported over steel pedestals, making the clear spacing of the girder equal 7000 mm. Transverse cross-type bracings were installed at the two ends of the steel girders to provide lateral restraints to the deck slab as specified into the CHBDC empirical design method. The spacing of the twin girders was 2000 mm measured center-to-center of the girders. The first precast FDDP system was tested under high-cycle constant-amplitude fatigue (CAF) loading followed by increasing monotonic loading to-collapse, while the second precast FDDP system was tested under low-cycle incremental step fatigue loading of variable amplitude (VAF) to collapse. The actuator system generates sinusoidal harmonic force, $p_t = p_{avg} + p_o \sin 2\pi f t$, where p_{avg} is the average load of the max and min load, p_o is the amplitude of applied load equal to FLS/2, f is the frequency and t is the time. Before performing fatigue tests, a crack was initiated in the tested slab by applying monotonic loading equal to 3 times the applied wheel load for serviceability limit state (SLS_1 = 87.5 kN x 1.4 x 0.9 = 110.25 kN) design per CHBDC. This applied wheel load equals the heaviest wheel load in the specified CHBDC per CL-625-ONT truck model, multiplied with the 1.4 to include the dynamic load allowance (DLA) and 0.9 as the load factor.(i.e. 3 times SLS_1 = 110.25 x 3 = 330.75 kN). The footprint of the applied wheel load on top of the tested slab measures 600 mm wide by 250 mm long as depicted in Fig. 3(b). It was decided to locate it just beside the joint as depicted in Fig. 3(b).



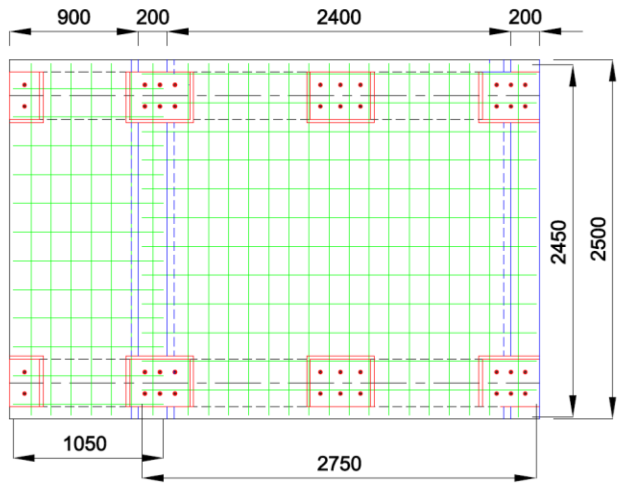
a. C-Joint details



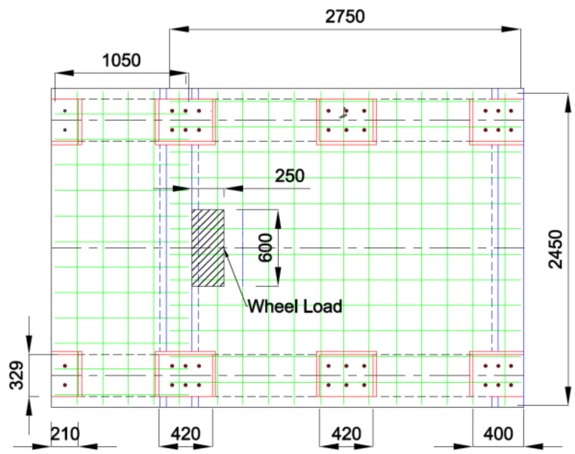
b. Panel-to-panel connection details

Figure 2. Plan views of the developed C-shape joint details

The constant amplitude fatigue (CAF) loading was applied under force control with sinusoidal shape to represent the fatigue limit state (FLS) load specified into the CHBDC as FLS = 87.5 x 1.4 x 1.0 = 122.5 kN at the frequency of 4 Hz for 4 million cycles. To prevent rattling of the test setup under cyclic loading, the loading cycle started with 15 kN applied load that increased by 122.5 kN. Thus, the sinusoidal cyclic CAF ended up with loading range of upper and lower absolute values of 137.5 kN and 15 kN, respectively with sample rate of 20.013 Hz. Monotonic test at 1.5 time the applied FLS load (i.e. 122.5 kN x 1.5 = 183.75 kN) was conducted after each 250,000 cycles to assess the degradation of the FDDP system due to fatigue loading. The force-control monotonic test had a ramp segment shape at loading and unloading rate of 5 kN/min. and 10 kN/min., respectively, with collecting data points every 0.049967 sec. After the end of the 4 million cycles, the FDDP system was monotonically loaded to-collapse using a hydraulic jack with 1,300 kN capacity. The incremental step variable amplitude fatigue (VAF) loading was applied under force control with sinusoidal shape to different 7 absolute peak levels of 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, and 4.0 times the FLS load of 122.5 kN plus 15 kN as the absolute load lower level. The corresponding peak loads of the 7 incremental step VAF loadings were 137.5, 198.75, 260, 321.25, 382.5, 443.75 and 505 kN. Each load level was applied for 100,000 cycles at the range of 2 Hz to 0.5 Hz depending on the stiffness of the FDDP system, and the steel loading frame system, with lowest frequency when approaching failure of the slab. Data was collected at a sample rate of 20.013 Hz. Monotonic test was performed after each 100,000 cycles with the same setting of the CAF monotonic test. After finishing with 7 absolute peak levels mentioned earlier, the VAF loading testing continued with the highest peak value up to the failure of the specimen. Figure 4 shows view of the test setup during fatigue testing, while Fig. 5 shows view of the test setup during the monotonic testing.



a. Bottom reinforcement



b. Top reinforcement and wheel load position

c. View of GFRP bars and Styrofoam to form the connection

Figure 3. Construction of the precast FDDP with C-shape connection

4 TEST RESULTS

This section discusses the structural behavior of the tested specimens in the form of slab vertical deflection, and crack pattern. As mentioned earlier, fatigue precracking was conducted under force control. The first hair flexural crack was observed at 2.5 times the FLS loading (275.625 kN) underneath the wheel footprint area at the mid-span in the longitudinal direction (parallel to the supporting girders). Load was increased to 3 times the FLS load (330.75 kN) to increase the crack propagation beyond the wheel footprint area. The flexural crack width was found to be 80 µm at that static load. CHBDC specifies that design factored ultimate limit state (ULS) load of the deck slab is the multiplication of CHBDC truck wheel load of 87.5 kN, load factor of 1.7 and DLA of 0.40. This makes the factored design applied load ULS_1 = 87.5 x 1.4 x 1.7 = 208.25 kN. It is interesting to mention that at the precracking monotonic load of 330.75 kN, at which a minor flexural crack appeared, is about 59% greater that the CHBDC factored design load.

4.1 Constant Amplitude Fatigue Loading

For the tested specimen under CAF loading, the compressive strength of the concrete cylinders taken from the concrete mix were 58.10, 57.08, and 54.87 MPa, with an average value of 56.68 MPa. The tested cylinders for the UHPFRC, that were cast 10 days before the start of the fatigue testing, resulted in compressive strengths of 168.03, 162.33, 170.54 and 149.22 MPa, with an average value of 162.53 MPa.

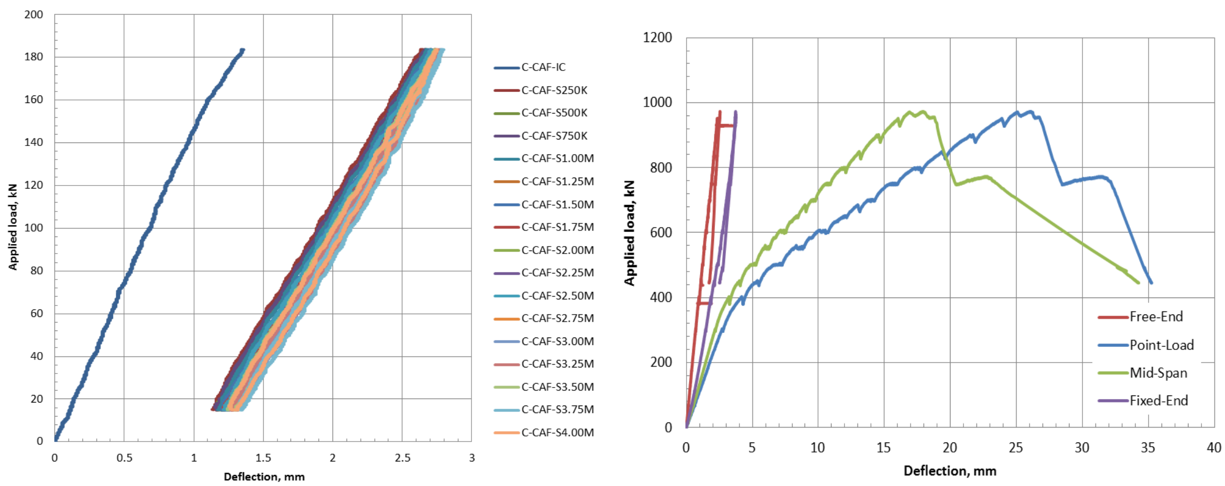
a. Slab-girder bridge system during CAF and VAF loading b. close-up view for the actuator

Figure 4. View of the test setup for fatigue loading

a. Slab-girder bridge system during monotonic loading b. View of loaded area

Figure 5. View of the test setup for the monotonic loading

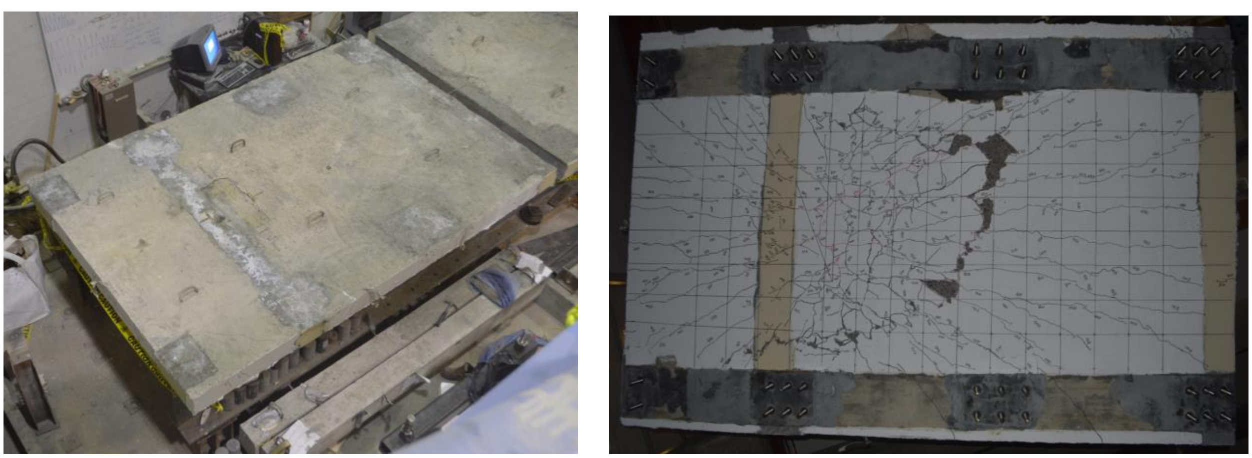
During the initiation of fatigue precracking procedure, at a static load of 220.5 kN, flexural crack propagated from underneath the mid-point of wheel footprint about 100 mm towards the middle shear pockets shown at the middle of the precast slab segment shown in Fig. 3(a). When the applied load increased to 275.625 kN, the flexural crack propagated further another 300 mm. However, when the applied load reached 330.75 kN, the flexural crack propagated diagonally from underneath the mid-point of the wheel footprint to the closest corner of the middle shear pocket. The maximum recorded flexural crack width at that point measured 80 µm. No more flexural cracks were observed during the CAF test that last over 16 days. After each 250,000 cycles, the slab was subjected to monotonic loading to observe the change in slab flexural stiffness through deflection measurements. Figure 6(a) depicts the load-deflection relationship for the slab at the centre of the footprint of the wheel load. It can be observed that the slope of the curves after each group of fatigue cycles appeared unchanged and maintained linear. After the 4-million fatigue cycles, the slab was subjected to monotonic load to-collapse. The precast FDDP failed due to punching shear at a jacking load of 973 kN. It is interesting to mention that such failure load is about 4.67 times the CHBDC factored design wheel load. Figure 7(a) shows top view of the slab showing punching shear failure at the footprint of the wheel load. While Fig. 7(b) shows bottom view of the slab showing crack pattern after failure. One may observe the radial cracks starting from the location of the footprint of the wheel load and propagating towards the support line in a fan shape. At failure, concrete spalling appeared in some parts of the bottom side of the slabs as signs for punching shear failure. Figure 6(b) depicts the load-deflection relationship for the tested slab under static loading to-collapse. Deflections values were recorded at the mid-length of the free edge of the short slab shown in Fig. 3(a), noted as "Free Eng" curve in Fig. 6(b). Such deflection reached 2.54 mm at failure. On the other hand, the deflections under the wheel footprint, denoted as "Point Load" in Fig. 6(b) were recorded as 25.98 mm at failure. The deflection at the centre of the long precast slab, denoted as "Mid-span" in Fig. 6(b) was recorded as 17.83 mm. The maximum deflection of the long precast slab at the mid length of the edge joint, denoted as "Fixed End" in Fig. 6(b) was recorded as 3.73 mm at failure.



a. Load-deflection curves under static load after Each 500,0000 fatigue cycles

b. Load-deflection cures under static load up-to-collpase

Figure 6. Monotonic load-deflection history for the first specimen tested under CAF loading



a. Top view of the slab showing punching shear failure at the footprint of the wheel load

b. Bottom view of the slab showing crack pattern after failure

Figure 7. Crack pattern after failure of the first slab tested under CAF loading

4.2 Variable Amplitude Fatigue

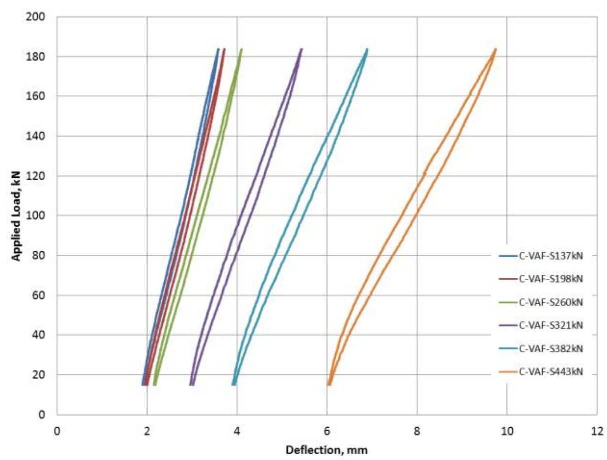
The second precast FDDP specimen underwent sinusoidal waveform fatigue load cycles with incremental step low cycle fatigue loading. The compressive strengths of concrete cylinders for the NSC used to cast this slab were 67.78, 64.93, 65.63, and 67.81 MPa, with an average value of 66.54 MPa. The compressive strengths of the concrete cylinder for the UHPFRC used to fill the joints were 147.60, 150.69, and 151.45 MPa, with an average value of 149.91 MPa. The first 500,000 fatigue load cycles were performed at a frequency of 2 Hz, then followed by 100,000 cycles at 1 Hz, and finally followed by 92,866 cycles at 1 Hz leading to punching shear failure at a total number of cycles of 692,866. Figure 8 depicts the punching shear failure at wheel footprint on top of the slab. While Fig. 9(a) depicts the crack pattern at the bottom surface of the slab at failure. A fan-shape crack pattern was observed at the bottom surface similar to those developed for the slab tested to collapse after passing the CAF loading. However, Fig. 9(a) shows greater concrete spalling along the perimeter on the punching shear plane at the bottom of the slab but only from one side of the closure strip. This slab failed at a jacking load of 495.69 kN and a maximum slab deflection of 40.89 mm. It is interesting to mention that such failure load is about 2.38 times the CHBDC factored design wheel load. Figure 9(b) depicts the monotonic load-deflection relationship of the slab after each 100,000 fatigue load cycles. It can be observed the slope of the curve decreased, leading to a reduction in slab flexural stiffness, with increase in number of VAF load cycles.

a. Top view showing punching shear failure at wheel footprint

b. Close-up view for the punching shear failure at the wheel footprint

Figure 8. Views of punching shear failure of the slab tested under VAF loading

a. Bottom view showing crack pattern



b. Monotonic load-deflection curves after each 100,000 cycles

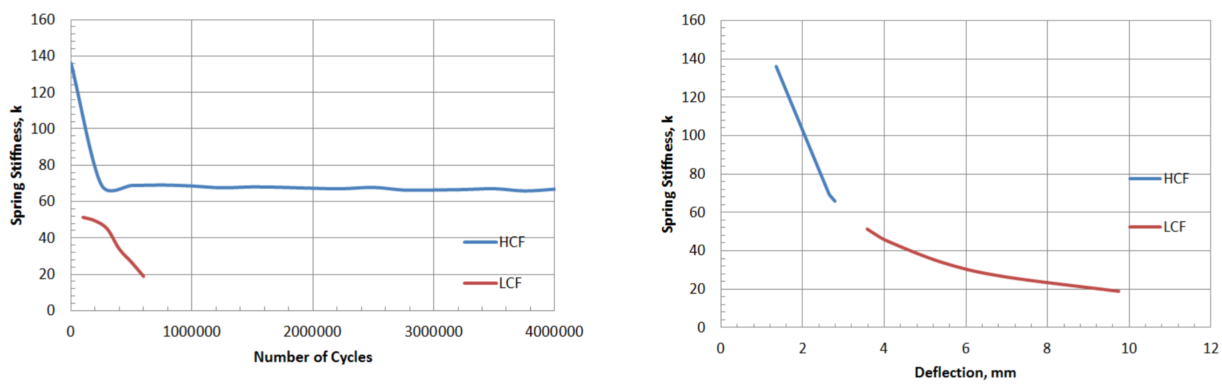
Figure 9. Crack pattern and monotonic load-deflection history for the slab tested under VAF loading

4.3 Stiffness degradation

The stiffness degradation of precast FDDPs under flexural loading was calculated in the form of spring stiffness (k). k is considered as the ration between the applied monotonic load, F, in kN and corresponding slab deflection, d, in mm. Table 1 summarizes the results for the CAF loading (high cycle fatigue, HCF) and for the VAF loading (low cycle fatigue, LCF). Figures 10(a) and 10(b) depict the relationship between the spring stiffness and the number of fatigue cycles and slab deflection, respectively. One may observe that the first specimen's stiffness degraded by about 50.73% after 4 million cycles of constant amplitude fatigue (CAF) loading. On the other hand, the second specimen's stiffness degraded by 63.24% when subjected to variable amplitude fatigue (VAF) loading before complete collapse.

Table 1. Stiffness degradation of the precast FDDP slabs

HCF - CAF				LCF – VAF			
Cumulative Cycles	Load, kN	Deflection, mm	k = F/d	Cumulative Cycles	Load, kN	Deflection, mm	k = F/d
0	183.7544	1.35	136.1144	100,000	183.7591	3.58	51.32936
500,000	183.7645	2.65	69.34509	200,000	183.7669	3.72	49.3997
1,000,000	183.7605	2.67	68.82416	300,000	183.75	4.09	44.92665
1,250,000	183.767	2.66	69.08534	400,000	183.7521	5.43	33.84017
1,500,000	183.7614	2.68	68.56769	500,000	183.7675	6.89	26.67163
1,750,000	183.7655	2.72	67.56085	600,000	183.7523	9.74	18.86574
2,000,000	183.7605	2.7	68.05944	**692,866**			
2,250,000	183.7634	2.71	67.80937				
2,500,000	183.7568	2.73	67.31018				
2,750,000	183.7636	2.74	67.06701				
3,000,000	183.7537	2.71	67.80579				
3,250,000	183.7577	2.77	66.33852				
3,500,000	183.7682	2.77	66.34231				
3,750,000	183.7695	2.76	66.58315				
4,000,000	183.7603	2.74	67.0658				



a. Spring stiffness-number of cycles curves b. Spring stiffness-deflection curves

Figure 10. Degradation of the precast FDDP under CAF and VAF loading

5 CONCLUSIONS

This paper investigates the fatigue behavior and ultimate load carrying capacity of laterally-restrained precast FDDP reinforced with high-modulus GFRP bars with developed transverse C-Joint combined with vertical shear key, filled with UHPFRC, and subjected to CHBDC wheel loading. Based on the experimental results, it can be concluded that the developed transverse panel-to-panel C-shape joint with projecting straight-end HM GFRP bars can provide a continuous force transfer in the transverse joint for precast FDDPs. Experimental results also indicated that precast high-modulus, GFRP-reinforced, FDDP showed high fatigue performance as there was no observed fatigue damage after being subjected to 4,000,000 cycles of high-cyclic CAF loading of 122.5 kN specified in CHDBC. The tested precast FDDP under CAF loading followed with increasing monotonic wheel load to-collapse sustained a failure load about 4.67 times the CHBDC factored ULS_1 design wheel load. While the tested precast FDDP under low-cyclic incremental step VAF loading sustained a failure load about 2.38 times the CHBDC factored ULS_1 design wheel load. The two precast FDDPs failed in punching shear mode. Finally, the first precast FDDP specimen's stiffness degraded by about 50.73% after 4 million cycles of high-cyclic constant amplitude fatigue (CAF) loading. On the other hand, the second precast FDDP specimen's stiffness degraded by 63.42% when subjected to low-cyclic variable amplitude fatigue (VAF) loading before complete collapse.

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REFERENCES

AASHTO. 2012. AASHTO LRFD Bridge Design Specifications, Fifth Edition. American Association of State Highway and Transportation Officials, Washington D.C.

Badie, S., and Tadros,M. 2008. Full-Depth Precast Concrete Bridge Deck Panel Systems, NCHRP Report 584. Transportation Research Board, Washington D.C.

Badie, S., Tadros, M., and Girgis, A. 2006. Full-Depth, Precast-Concrete Bridge Deck Panel Systems, Report No. NCHRP 12-65. Transportation Research Board, Washington, D.C.

CSA. 2006. Canadian Highway Bridge Design Code, CAN/CSA-S6-06. Canadian Standards Association, Mississauga, Onatrio, Canda.

Clumo, M. 2011. Accelerated Bridge Construction - Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems, FHWA-HIF-12-013. Office of Bridge Technology, HIBT-10, Federal Highway Administration,McLean, VA.

Hwang, H., and Park, S. 2014. A Study on the Flexural Behavior of Lap-spliced Cast-in-place Joints under Static Loading in Ultra-high Performance Concrete Bridge Deck Slabs. Canadian Journal of Civil Engineering, 41: 615-623.

Karunananda, K., Ohga, M., Dissanayake, R., and Siriwardane, S. 2010. A Combined High and Low Cycle Fatigue Model to Estimate Life of Steel Bridges. Journal of Engineering and Technology Research, 2(8): 144-160.

PCI. 2011a. Full Depth Deck Panels Guidelines For Accelerated Bridge Deck Replacement Or Construction, Report Number PCINER-11-FDDP, Precast/Prestressed Concrete Institute, USA.

PCI Committee on Bridges and the PCI Bridge Producers Committee. 2011B. PCI State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels (SOA-01-1911). Precast/Prestressed Concrete Institute, USA.

Russell, H., and Graybeal, B. 2013. Ultra-High Performance Concrete: A State-of-the-Art Report for the Bridge Community, Publication No. FHWA-HRT-13-060. Federal Highway Administration, McLean, VA.

Sayed-Ahmed, M. 2014. Structural Design Issues of Accelerated Bridge Construction Incorporating High-Performance Materials. Ph.D. Dissertation, expected September 2014, Civil Engineering Department, Ryerson University, Toronto, Ontario, Canada.

Sayed-Ahmed, M., and Sennah, K. 2014a. Development of Transverse Joint for Full-Depth Precast Normal-strength concrete deck panels incorporating ribbed-surface GFRP and UHPFRC. Proceedings of the 9th International Conference on Short and Medium Span Bridges. Canadian Society for Civil Engineering, Calgary, Alberta, pp. 1-10.

Sayed-Ahmed, M., and Sennah, K. 2014b. Development of Transverse Joints for Full-depth Precast Deck Panels Incorporating Ribbed-surface GFRP Bars and UHPFRC. Proceedings of the PCI Convention and National Bridge Conference. M.D., USA, pp. 1-10.

Sayed-Ahmed, M., and Sennah, K. 2014c. Pullout Strength of Sand-coated GFRP Bars Embedded in Ultra-High Performance Fiber Reinforced Concrete. Proceedings of the 4th International Structural Specialty Conference. Canadian Society for Civil Engineering, Halifax, NS, pp. 1-10.

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