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FATIGUE STRENGTH OF ANGLE-SHAPED TRANSVERSE CONNECTION FOR GFRP-REINFORCED PRECAST FULL-DEPTH DECK PANELS IN ACCELERATED BRIDGE CONSTRUCTION

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ABSTRACT

Prefabricated bridges elements and systems (PBES) are subjected to repeated truck loads while being exposed to weather conditions. Fatigue of the structural elements and corrosion of the reinforcement are the main reasons for bridge deterioration. This research investigates the fatigue strength of full-depth deck panels (FDDP) resting over steel cross-braced girders and reinforced with ribbed-surface, high-modulus (HM), glass fiber reinforced polymer (GFRP) bars. The precast FDDP has transverse panel-to-panel connection of angle-shape with female shear key, and panel-to-girder connection of V-shape, where both connections are filled with ultra-high performance fiber reinforced concrete (UHPFRC). Two different fatigue loading were conducted to simulate the Canadian Highway Bridge Design Code (CHBDC) truck loading, namely: constant amplitude fatigue (CAF) loading and variable amplitude fatigue (VAF) loading. The fatigue damage for all cycles is summed to obtain the cumulative fatigue damage (CFD) for the entire loading history. The reliability of the GFRP-reinforced precast FDDP subjected to high cycle fatigue is then evaluated based on load-cycle (P-N) damage accumulation approach. A simple life-span prediction model is proposed for the FDDP based on the CFD.

Keywords: Bridge, Fatigue Strength, Precast Full-Depth Deck Panel (FDDP), GFRP Bars, Angle Transverse Joint, Experimental testing, Design Standards.

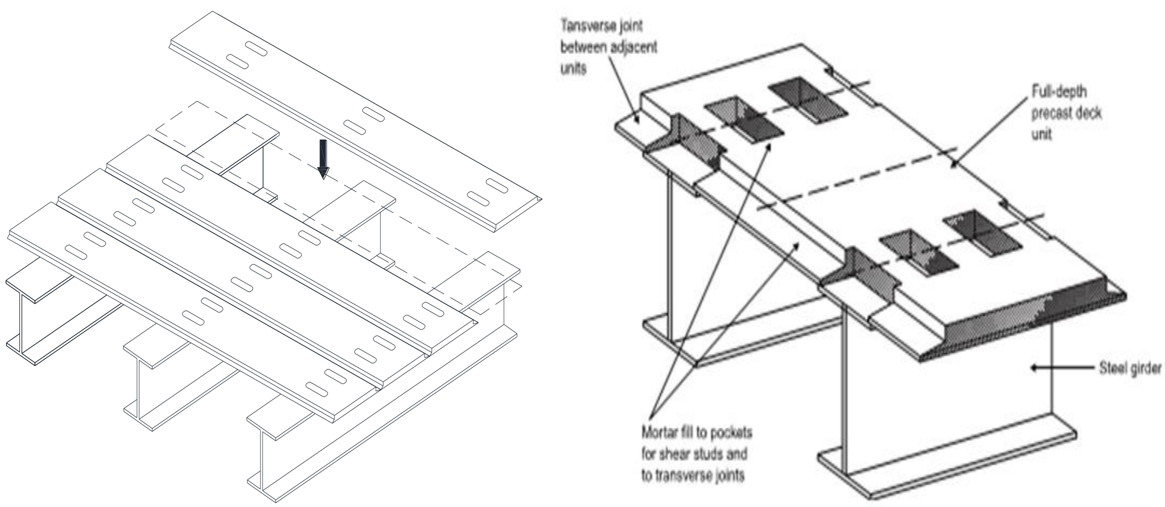
1. INTRODUCTIONS

Precast full-depth deck panels (FDDP) utilized in accelerated bridge construction are frequently subjected to complex sinusoidal variable amplitude fatigue (VAF) loading during their life-span. The most common forms of VAF test, namely: the multi-step test, subjects the specimen to a fixed number of cycles in the range of 10^4-10^5 at one constant amplitude fatigue (CAF) loading level (initial loading) after which different CAF levels (the runout load) are applied until failure occurs. The magnification factor (MF) for the fatigue limit state (FLS) determines the change of the cyclic loads. Results of VAF tests are presented into log-normal distribution to give a reasonable fit to the observed values. The cumulative fatigue damage (CFD) analysis is then conducted in terms of the specimen span-life defined as the sum of the cycles at the different CAF levels to the total runout cycles.

Fatigue design criteria for bridges typically consists of three separate elements, namely: (i) a single-vehicle fatigue load model that is 5-axle truck-and-trailer vehicle weighing 625 kN; (ii) fatigue loading that is derived from analysis of effects on bridge spans of heavy vehicles recorded at weigh-in-motion (WIM), also known as vehicle loading spectrum; and (iii) material-specific fatigue life. In this paper, it is assumed that the Average Daily Traffic (ADT) in the travelling lane in a bridge is limited to 20,000 vehicles and maximum fraction of trucks of 0.20 that equals to the maximum Average Daily Truck Traffic (ADTT) of 4,000 trucks in one direction. For a two-lane bridge, the single $ADTT_{SL} = 0.85 \times ADTT$ = 3,400 trucks (AASHTO, 2012). The maximum number of five-axle trucks during the 75 years of bridge life is 3400 x 5 x 365 x 75 = 465 million. The P-N relationship for the rolling wheel load, P, and the number of cycles, N, was introduced in Equation 1 for both reinforced concrete and reinforcement-free deck slabs, where P_s is the static failure load. This equation is valid only for N is greater than 10,000 (Matsui, et al., 2001).

[1] $$\log(P/P_s) = -0.07835 \times \log(N) + \log(1.52)$$

High-modulus glass fiber reinforced polymer (HM-GFRP) reinforcement is a composite material made of polymer matrix reinforced with fibers. GFRP has high strength-to-weight ratio, and is corrosion free compared to conventional steel reinforcement. GFRP reinforcement is considered in this research to prolong the service life cycle of the bridge while significantly reducing the maintenance cost. Precast FDDPs are prefabricated off-site and assembled on-site. Precast FDDP has two types of connections, namely: (i) panel-to-panel connection, and (ii) panel-to-girder connection to achieve the full composite action for the bridge superstructure, as shown in Figure 1. Connections are joint-filled with cementitious materials that have higher compressive strength than that for FDDPs. The Canadian Highway Bridge Design Code (CHBDC) allows the use of GFRP-reinforced FDDPs in the construction of bridges, yet there is no code provision for the design (CSA, 2014), and experimental tests are required to determine the performance of such precast systems.

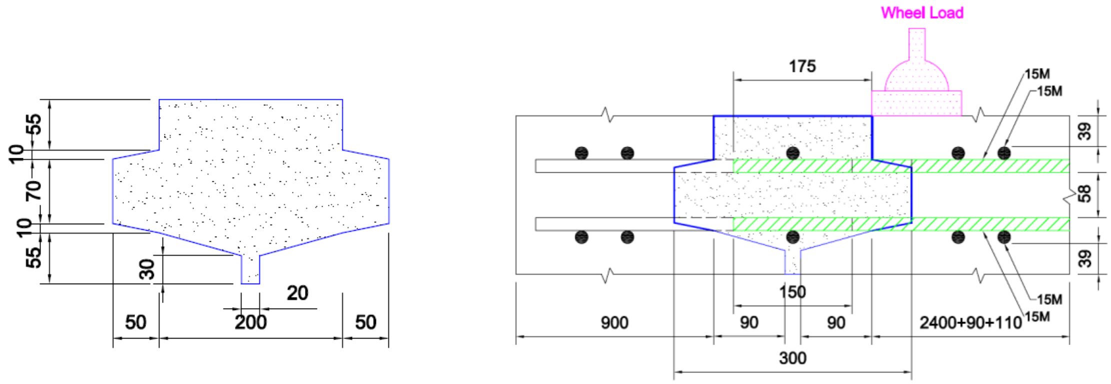


a) Isometric view (Source: (FHWA, 2014)) b) Photo (Source: (FHWA, 2012))
Figure 1: Views of full-depth, full-width, deck panels placed transversally on girders

A recent research program (El-Ragaby, 2007), involved GFRP-Reinforced FDDPs that were constructed and subjected to fatigue loading. Punching shear was the main mode of failure of these slabs after sustaining different numbers of load cycles at the peak loads for mid-span loading. The failure occurs suddenly and accompanied by a big noise. No rupture of GFRP bars was observed with failure of the punching area. GFRP bars exhibit good bond with concrete outside the failure zones, with no anchorage loss. GFRP-Reinforced slab-on-girders and precast FDDP with longitudinal joint connections were tested in Ryerson University (Khalafalla, 2014). Precast FDDP with transverse joint connections were investigated having c-shape (Sayed-Ahmed & Sennah, 2015a) and zigzag shape (Sayed-Ahmed & Sennah, 2015b), and their fatigue resistances have passed requirement set by the CHBDC (Sayed-Ahmed & Sennah, 2015c). The object of this research is to (i) investigate the fatigue strength for the HM GFRP-reinforced FDDP with developed angle-shaped transverse joint filled with ultra-high performance fiber reinforced concrete (UHPFRC), and (ii) plot the P-N curves to predict the maximum lifetime per axle loads.

2. CONNECTION DETAILS

Figure 2 depicts the angle-shape for the panel-to-panel connection with vertical female-to-female shear key. The top clear joint width between the ends of the jointed panels is 200 mm. The projected GFRP bar from the end of one panel into the joint is 175 mm as the development length, where the splice length is 150 mm. Connections were joint-filled with UHPFRC. The UHPFRC is field-cast joint fill solution for precast deck panel bridges, its high strength allows for reduced joint widths. The characteristic design strength values of UHPFRC can be reached within 96 hours of casting – as long as ambient temperatures above 16˚C are ensured. UHPFRC can achieve a 28-day compressive strength of 140 MPa, direct tensile strength of 8 MPa, and modulus of elasticity of 50 GPa (Lafarge Canada Inc., 2009).

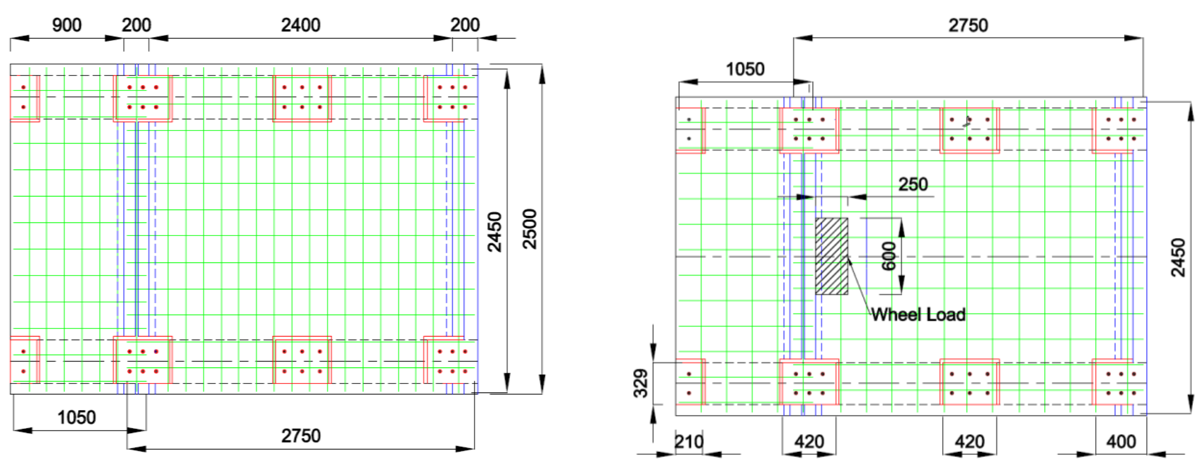


a. Cross-section b. Reinforcement detailing

Figure 2: Schematic diagrams of the developed transverse angle-shape joint details

3. EXPERIMENTAL PROGRAM

The experimental program included testing two laterally-restrained FDDPs supported over cross-braced twin-steel girder bridge system made of W610x241 steel beams. Each FDDP had a thickness of 200 mm and was made of 35 MPa normal strength concrete (NSC) with 10 mm nominal size aggregate. A 150-mm-slump concrete with added superplasticizer, and no air-entrant, was uses. Straight-end, 15M ribbed-surface, high modulus GFRP bars was used for reinforcement. The bottom and top transverse reinforcement of the slab was taken 15M@140 mm and 15M@200, mm respectively. The slab was reinforced with 15M@200 mm in the bottom and top longitudinal direction parallel to the girder. The specified modulus of elasticity and ultimate tensile strength of the GFRP bar were 64 GPa and 1188 MPa respectively (Schoeck Canada Inc., 2013). To form the transverse panel-to-panel connection, two precast FDDPs were constructed. The first FDDP was of 200 mm thickness, 2400 mm length and 2500 mm width. The second FDDP was of 200 mm thickness, 900 mm length in the direction of the girder and 2500 mm with. This made the final dimension of the jointed precast FDDPs of 3700 m in the direction of traffic, as shown in Figure 3.



(a) Bottom reinforcement (b) Top Reinforcement and load location

Figure 3: Schematic diagrams of the constructed FDDP with angle-shape connection and wheel load location

All FDDPs were tested up to failure under either monotonic or cyclic loading using one concentrated load at the center spacing between the twin girders. The load was applied through a steel pedestal that have a foot print of 250x600 mm to simulates the footprint of the standard CL-625 truck wheel load, as specified by clause 3.8.3.1 in the

CHBDC. Fatigue involves the failure of the materials under cyclic loading, where the maximum load can be significantly lower than that required load to cause static failure. Fatigue life of FDDPs can be expressed as the number of loading cycles after starting after initiation of crack, to propagate the crack to critical size, followed with rapid fracture as demonstrated in the diagram shown in Figure 4. The wheel footprint of 600x250 mm was applied eccentric to the joint as shown in Figures 2.b and 3.b.

The first FDDP system was tested under high-cycle constant-amplitude fatigue (CAF) loading followed by increasing monotonic loading to-collapse, while the second FDDP system was tested under low-cycle incremental step fatigue loading of variable amplitude (VAF) to collapse. The actuator system generated sinusoidal harmonic force, $p_t = p_{avg} + p_o \sin 2\pi f t$, where p_{avg} is the average load of the maximum and minimum load, p_o is the amplitude of applied load equal to FLS/2, f is the frequency and t is the time. Before performing the 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 design per CHBDC (SLS_1 = 87.5 kN x 1.4 x 0.9 = 110.25 kN; 3 times SLS_1 = 110.25 x 3 = 330.75 kN). This applied wheel load (87.5 kN) equals the heaviest wheel load in the specified CHBDC per CL-625-ONT truck model (Clause 3.8.3.1), multiplied with 1.4 to include the dynamic load allowance (DLA) in Clause 3.8.4.5.3.b and 0.9 as the load factor listed in Clause 3.5.1a. The footprint of the applied wheel load on top of the tested slab measures 600 mm wide by 250 mm long. It was decided to locate it just beside the joint as depicted in Figure 4. The factored design load was taken as P_f = 87.5 x 1.4 x 1.7 = 208.25 kN, where the live load factor equals to 1.7 as per CHBDC Clause 3.5.1a.

The CAF loading was applied using 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 a frequency of 4 Hz for 4 million cycles, as shown in Figure 5.a. 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 rate of 5 kN/min, 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.

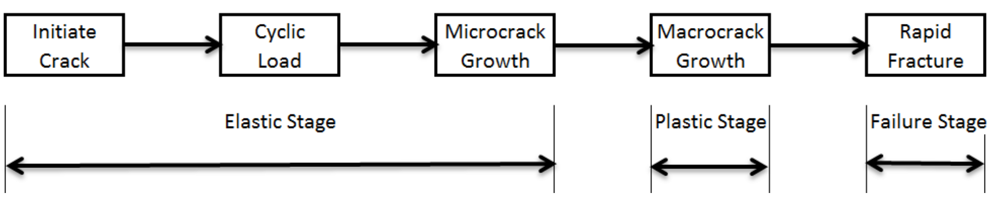
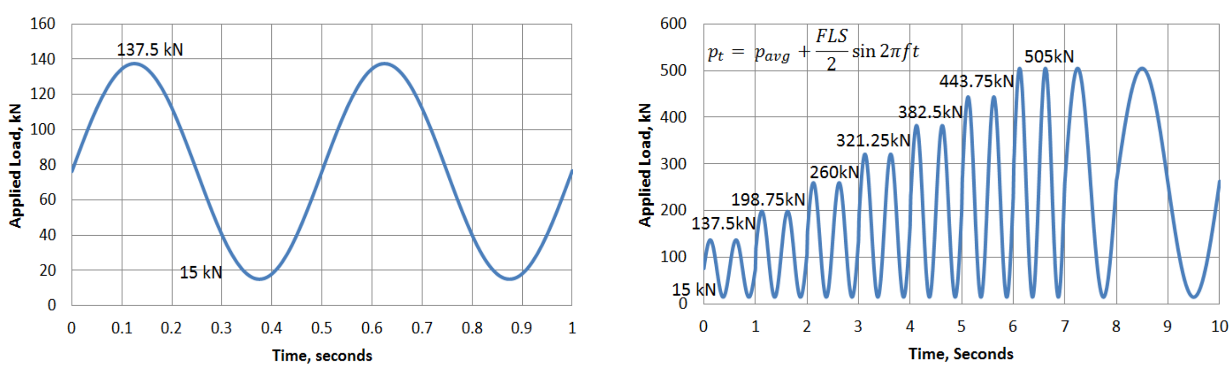


Figure 4: Different phases of fatigue life of concrete deck slab



(a) Typical high cyclic frequency (HCF) (b) Typical low cyclic frequency (LCF)

Figure 5: Typical fatigue loading

The VAF loading was applied using 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 loading were 137.50, 198.75, 260.00, 321.25, 382.50, 443.75 and 505.00 kN, as shown in Figure 5.b. Each load level was applied for 100,000 cycles at the range of 2 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 from sensors was collected at a sample rate of 20.013 Hz. Monotonic tests were performed after each 100,000 cycles with the same setting as that for the CAF monotonic test. After finishing with 7 absolute peak levels mentioned earlier, the VAF loading testing continued with the highest peak value till collapse.

4. TEST RESULTS FOR THE A-JOINTED PRECAST FDDP

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. The applied load was increased to 3 times the FLS load (330.75 kN) to increase crack propagation. 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 dynamic load allowance (DLA) of 0.40. This makes the factored design applied load ULS1 = 87.5 x 1.4 x 1.7 = 208.25 kN.

4.1 Constant amplitude fatigue loading

The first specimen was tested under CAF. Its compressive strength of the concrete cylinders taken from the concrete mix were 60.76, 59.83, and 54.26 MPa, with an average value of 58.28 MPa. The tested cylinders for the UHPFRC resulted in compressive strengths of 161.94, 163.30, 170.54 and 159.20 MPa, with an average value of 161.48 MPa. During the initiation of fatigue precracking procedure, the static load reached 330.75 kN. After each 250,000 cycles, the slab was subjected to monotonic loading to observe the change in slab flexural stiffness through deflection measurements. After the 4-million fatigue cycles, the slab was subjected to monotonic load to-collapse. It failed due to punching shear at a jacking load of 930.92 kN equals to 4.47 P_f and maximum deflection of 23.05 mm and 23.88 mm at failure. After the strength loss, the precast FDDP continued to gain strength after punching shear, but at a reduced stiffness.

4.2 Variable amplitude fatigue loading

The second specimen was tested under VAF and its compressive strengths of concrete cylinders for the NSC used to cast this slab were 54.29, 57.22, 59.98, 46.54, 65.84, 64.7 MPa, with an average value of 58.10 MPa. The splitting tensile test for the NSC resulted in tensile strength of 3.53, 5.73, 5.31, 4.3 and 4.7 MPa, with an average value of 4.71 MPa. The compressive strengths of the concrete cylinder for the UHPFRC used to fill the joints were 154.17, 188.12, 184.61 and 181.91 MPa, with an average value of 179.52 MPa. The splitting tensile test for the UHPFRC resulted in tensile strength of 15.12, 12.14, 15.76 MPa, with an average value of 14.42 MPa. The first 501,002 fatigue load cycles were performed at a frequency of 2 Hz, then followed by 160,242 cycles at 1 Hz, and finally followed by 130,139 cycles at 0.5 Hz leading to punching shear failure at a total number of cycles of 809,493. This precast FDDP failed at a jacking load of 487.50 kN and a maximum slab deflection of 32.46 mm. It is interesting to mention that such failure load is about 2.34 times the CHBDC factored design wheel load.

Table 1: Summary of test results

Slab	Test type *	Peak cyclic load (kN)	Frequency (Hz)	No. of load cycles	Ultimate load (kN)	Ultimate deflection (mm)	Failure Mode
A-CAF	CAF + SUL	137.5	4	4,000,000	930.92	23.47	Punching
A-VAF	VAF	500.0	2 – 0.5	809,493	487.50	32.46	Punching

CAF: constant amplitude fatigue; SUL: static ultimate load VAF: variable amplitude fatigue

5. LIFE ESTIMATION OF FATIGUE OF GFRP-REINFORCED FDDPS

Realistic representation of the service loads is usually of variable amplitude that should consider the accurately measure of the applied load on the existing structure and predicted loads on the structure that doesn't exist yet. Loads can be obtained from real-life histories or through simplified segmental loading. The fatigue cycle counting methods is to compare the effect of the variable amplitude fatigue load histories to the fatigue data and curves obtained with the simple constant amplitude fatigue loading cycles. The more general sort of fatigue testing adds a mean load P_m on which a sinusoidal cycle is superimposed, a common alternating load P_a and the load ratio $R > 0$ that corresponds to tension-tension cycle.

[5.1] $$P_{alternating} = P_a = \frac{P_{max} - P_{min}}{2}$$

[5.2] $$P_{mean} = P_m = \frac{P_{max} + P_{min}}{2}$$

[5.3] $$P_{max} = P_m + P_a$$

[5.4] $$P_{min} = P_m - P_a$$

[5.5] $$Load\ Ratio = R = \frac{P_{min}}{P_{max}}$$

[5.6] $$Amplitude\ Ratio = A = \frac{P_a}{P_m}$$

Applying linear damage rule where D = 1.0 requires the knowledge of the mean and amplitude of load to which the damaging event is compared. One approach to the variable load histories is the concept of the damage, known as fraction life or cycle ratio. These fractions are added together with the sum of 1.0 as defined into Equation 5.7 by the linear damage rule as proposed by Palmgren (Palmgren, 1924) and later again by Miner (Miner, 1945).

[5.7] $$D = \sum \frac{n_i}{N_{fi}} = 1$$

Where n is the number of cycles, and N_f is the number of repetitions of the same cycle that equals life to failure. The damaging effect of n_1 cycles at P_1 load amplitude is assume to be $n_1 D_1 = n_1/N_{f1}$, while the damaging effect of n_2 cycles at P_2 load amplitude is assumed to be $n_2 D_2 = n_2/N_{f2}$. Similarly, the cycle ratio or damage caused by n_i cycles at P_i load amplitude is $n_i D_i = n_i/N_{fi}$. Failure is predicted when the sum of all ratios becomes 1 or 100%. Although the assumption of the linear damage depends on the rate of damage accumulation and load amplitude, this equation leads to $n_i/N_{fi} \neq 1$ for a low-to-high or a high-to-low loading sequences. However, it is widely used because of simplicity and hardly to achieve better agreement with the current experimental data. Nonlinear damage theories proposed $D = \sum (n_i/N_{fi})^{\alpha_l}$ where α_l depends on the load level. When considering the change of load level to be P/P_u, the *authors propose the nonlinearity of the damaging* effect to the step loading through Equation 5.8, keeping the linearity of $D = 1$ and solving for the η using the nonlinear least square regression analysis.

[5.8] $$N_f = e^{\eta\,(1 - P/P_u)}$$

Where η equals to 25.86 for the A-Joint. Tables 1 and 2 illustrate the cumulative fatigue damage (CFD) data where D = 1 for both type of FDDPs. The *proposed model* to determine P-N effect is shown in Equation 5.9, where K equals to 0.039 (or 1/K = 25.64) for the A-Joint.

[5.9] $$\frac{P}{P_u} = -K \ln(N_f) + 1$$

The fatigue coefficient (1/K) was predicted to be 30 and 33.6 for cast-in-place GFRP-reinforced bridge deck as suggested per (Mufti, et al., 2002) and (Khalafalla, 2014) respectively. The 1/K was found to be 29 for the precast GFRP-reinforced bridge deck slabs with longitudinal joints (Khalafalla, 2014).

Table 2: The A-Precast FDDP loading history for the equivalent constant amplitude fatigue load segments

Segment	P_u	FLS			P_{min}	P_{max}	P_{amp}	P_{mean}	R	A	P_{max}/P_u	n	N_f	n/N_f
		MF	WL	FLS_1										
1	930.92	1	87.5	122.5	15	137.50	61.25	76.25	0.109091	0.803279	0.1477033	100,000	3,740,491,266	2.673E-05
2	930.92	1.5	87.5	183.8	15	198.75	91.88	106.88	0.075472	0.859649	0.2134985	100,000	682,217,686	0.0001466
3	930.92	2	87.5	245	15	260.00	122.50	137.50	0.057692	0.890909	0.2792936	100,000	124,427,766	0.0008037
4	930.92	2.5	87.5	306.3	15	321.25	153.13	168.13	0.046693	0.910781	0.3450887	100,000	22,694,030	0.0044064
5	930.92	3	87.5	367.5	15	382.50	183.75	198.75	0.039216	0.924528	0.4108839	100,000	4,139,100	0.0241598
6	930.92	3.5	87.5	428.8	15	443.75	214.38	229.38	0.033803	0.934605	0.476679	100,000	754,919	0.1324646
7	930.92	4	87.5	490	15	505.00	245.00	260.00	0.029703	0.942308	0.5424741	115,381	137,688	0.8379916
8	930.92	4	87.5	490	15	505.00	245.00	260.00	0.029703	0.942308	0.5424741	94,112	137,688	0.6835187
											Total	809,493	Σn/N	0.9999995

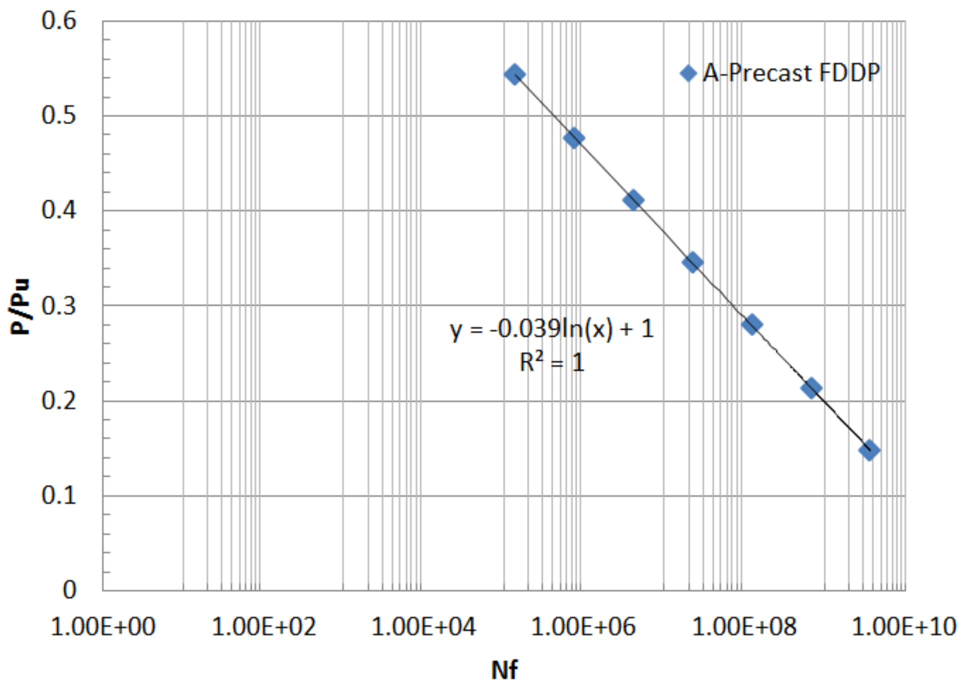


Figure 6: P-N curves for the GFRP-reinforced FDDPs

6. CONCLUSIONS

Fatigue tests were conducted on the developed precast transverse FDDPs supported over steel girders. Experimental results for the angle-shape jointed FDDPs 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 FDDP under CAF loading followed with increasing monotonic wheel load to-collapse sustained a failure load about 4.47 times the CHBDC factored design wheel load. On the other hand, the tested FDDP under low-cyclic incremental step VAF loading sustained a failure load about 2.34 times the CHBDC factored design wheel load. The two FDDPs failed in punching shear mode.

Based on experimental findings, a mathematical model was proposed to determine the cumulative fatigue damage (CFD) and fatigue resistance (P-N effect) for the GFRP-reinforced FDDPs with transvers joint. The cumulative linear damage, D, was successfully maintained to be 1 for transverse jointed FDDPs. It can be noticed that the magnification factor to the fatigue loading is inversely proportional to the number of the repetitions of the same cycle that equals life to failure.

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REFERENCES

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

CSA, 2014. Canadian Highway Bridge Design Code, CAN/CSA-S6-14, Mississauga, Ontario: Canadian Standards Association.

El-Ragaby, A., 2007. *Fatigue Behaviour of Concrete Bridge Deck Slabs Reinforced with Glass FRP Bars, PhD Thesis,* Sherbrooke, QC: Sherbrooke University.

FHWA, 2012. *CHAPTER 4. DECK REPLACEMENT.* [Online] Available at: https://www.fhwa.dot.gov/hfl/pubs/hif13032/chap04.cfm [Accessed 2016].

FHWA, 2014. Chapter 2: Findings on Prefabricated Bridge Systems. [Online] Available at: http://international.fhwa.dot.gov/prefab_bridges/chapter_two_a.cfm [Accessed 2012].

Khalafalla, I. E., 2014. Development of Sustainable concrete bridge deck slab systems using corrosion-resistant GFRP bars, PhD Thesis, Toronto, ON: Ryerson University.

Lafarge Canada Inc., 2009. Ductal JS1000, field-cast joint fill solutions for precast deck panel bridges. [Online] Available at: http://www.ductal.fr/JS1000_2009.pdf [Accessed Dec. 2011].

Matsui, S., Tokai, D., Higashiyama, H. and Mizukoshi, M., 2001. Fatigue durability of fiber reinforced concrete decks under running wheel load. Vancouver, BC, Canada, Proceedings, Third International Conference on Concrete under Severe Conditions, pp. 982-991.

Miner, M. A., 1945. Cumulative Damage in Fatigue. Journal of Applied Mechanics, Volume 67, pp. A159-164.

Mufti, A., Memon, A., Bakht, B. and Banthia, N., 2002. Fatigue Investigation of the Steel-Free Bridge Deck Slab. ACI International, Volume SP-206, pp. 61-70.

Palmgren, A., 1924. Die Lebensdauer von Kugellagern. Verfahrenstechinik. Volume 68, pp. 338-341.

Sayed-Ahmed, M. and Sennah, K., 2015a. Structural Behavior of UHPFRC-Filled, Transverse C-Joint in Full-Depth, GFRP-Reinforced, Precast Bridge Deck Panels Resting Over Steel Girders. Regina, SK, CSCE 2015 Annual Conference/ 4th International Conference on Engineering Materials and Mechanics.

Sayed-Ahmed, M. and Sennah, K., 2015b. Ultimate and fatigue strength of GFRP-reinforced, full-depth, precast bridge deck panels with zigzag-shape transverse joints filled with UHPFRC. Vancouver, BC, CSCE 5th International / 11th Construction Specialty Conference (ICSC).

Sayed-Ahmed, M. and Sennah, K., 2015c. Fatigue Resistance of GFRP-Reinforced Precast Full-Depth Deck Panel with Transverse Joints. Cairo, Egypt, International Conference on Structural and Geotechnical Engineering, Ain Shams University.

Schoeck Canada Inc., 2013. Schock ComBar Technical Information. [Online] Available at: http://www.schoeck.ca/upload/files/download/Technical_Information_Schoeck_ComBAR_[1513].pdf [Accessed April 2013].