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FATIGUE RESPONSE OF UHPC AS A CLOSURE STRIP MATERIAL IN PREFABRICATED BRIDGE APPLICATIONS

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ABSTRACT

Replacement of concrete bridge decks can be an expensive process. In order to decrease costs and time of construction, precast deck panels can be used with closure strips cast in place between the panels. In order to ensure that these connections can withstand the rigors of a bridge's life cycle, fatigue testing was completed on a specimen consisting of two precast concrete panels reinforced with GFRP and connected using a UHPC closure strip. These panels were subjected to 2,000,000 cycles of fatigue loading at three locations and subject to static failure loading at one location following fatigue loads. It was found that under fatigue loading the panels were able to maintain structural integrity while deflection values increased linearly following the initial cracking phase. At ultimate load, the panel failed in punching shear at levels less than those specified by bridge code. This is primarily due to the failure location adjacent to the closure strip failing on three punching shear planes and one plane along the interface between the UHPC.

Keywords: UHPC, GFRP, precast bridge deck panel

1. INTRODUCTION

Replacing a bridge deck can be a time consuming and expensive project to undergo. In order to decrease construction time, the use of full-depth precast concrete deck panels allows for the deck to be constructed or replaced more quickly than typical cast-in-place construction. This composite bridge system is shown schematically in Figure 1. The panels are placed on the steel girders, and composite behaviour is provided using grouped shear studs that coincide with pockets in the precast panels. The pockets are grouted after placing the panels on the girders to complete the connection. In order to ensure continuity of the deck system, the transverse joints (and longitudinal joints, if present) between panels are grouted. The panel reinforcement extends into the joints were a lap splice is used to provide reinforcement continuity. The shear pockets and joints between panels are typically filled with a non-shrink structural grout or concrete. More recently, ultra-high performance concrete (UHPC) has been used as a closure strip material between the precast concrete panels. This allows the joint width to be reduced. In order to understand how these UHPC joints will behave over the life cycle of the bridge deck, specimens replicating a transverse deck joint in a bridge deck system were created and fatigued at service load levels to simulate the loading life of a bridge. The deck panels used in this study were reinforced with glass fibre reinforced polymer (GFRP) bars based on guidance from the Ontario Ministry of Transportation (MTO). The specimen was loaded to failure following the fatigue cycles to compare the ultimate load capacity to the design load capacity specified in the Canadian Highway Bridge Design Code (CHBDC, 2013).

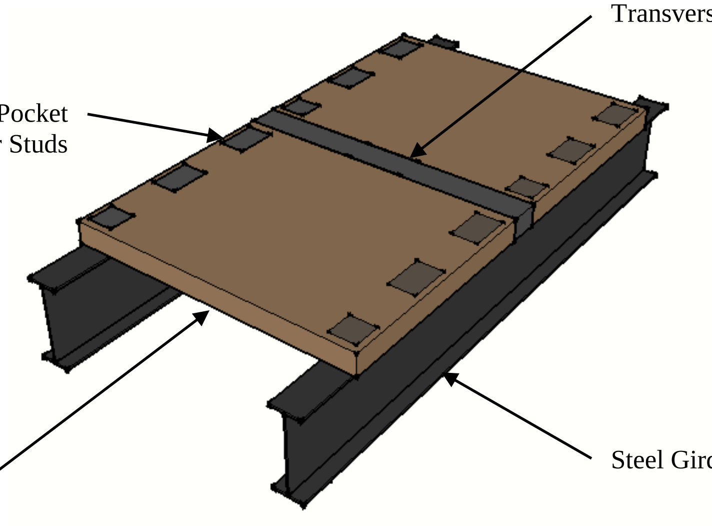

Figure 1: Precast Deck Panels Shear Connected to Steel Girders

2. LITERATURE REVIEW

Bridge decks are generally the first component of a bridge to experience deterioration requiring repair or replacement (Biswas, 1986). For this reason, the ability to replace the deck quickly is advantageous to transportation authorities. The UHPC used in the closure strip is a type of concrete consisting of portland cement, fine sand, silica fume, water-reducing admixture, ground quartz, steel fibres, and water (Graybeal, 2007). This concrete produces compressive strengths in excess of 100 MPa with 2-day strengths of approximately 70 MPa. It should also be noted that UHPC remains linear elastic up to 80% of its ultimate load capacity (Graybeal, 2007).

In previous tests completed by El-Ragaby and El-Salakawy (2011), it was found that GFRP reinforced deck slabs were able to withstand fatigue loading of 70 kilonewtons (kN) for 3,000,000 cycles at various thermal conditions and still fail through punching shear at a load similar to that for the control unfatigued specimen. Steel reinforced panels with UHPC closure strips were tested by Au, Lam and Tharambala (2011) which concluded that U-shaped, L-shaped, and welded bars provided continuity within the closure strip. It has also been shown that transverse UHPC filled joints with shear keys and CZ shapes reinforced with GFRP survived up to 4,000,000 cycles under continuous amplitude loading and had punching failures which exceeded the code required punching load (Sayed-Ahmed & Sennah, 2015).

3. EXPERIMENTAL PROGRAM

The research presented in this paper is part of a larger study of the behaviour of full-depth precast panel deck systems for composite bridges. The primary objective is to study the effect of fatigue loading on the punching shear capacity of full-depth precast panels reinforced with GFRP bars and with UHPC used to complete the transverse panel joint. The research variables investigated include the load position (mid-panel, adjacent to UHPC joint and centred on UHPC joint) and the type of GFRP bars (bars from two manufacturers were used).

3.1 Slab Layout

The configuration and details of the test specimen were adapted from typical details used for this type of composite bridge system in Ontario highway bridges. The test specimen utilized two 150 mm deep slabs with dimensions of 1500 mm x 1965 mm supported by two W530x101 steel beams spaced at 1565 mm on center. The slab dimensions and beam spacing were selected based on an approximately 70% scale of a full size bridge deck. In order to connect the slabs to the beams, shear pockets were cast into the slabs to allow for 19 mm diameter bolts to be installed at a height of 115 mm within the shear pocket. These shear pockets were then filled with UHPC to complete the connection between the slab and the beams. Figure 2 shows a diagram of the shear pockets on the support girders.

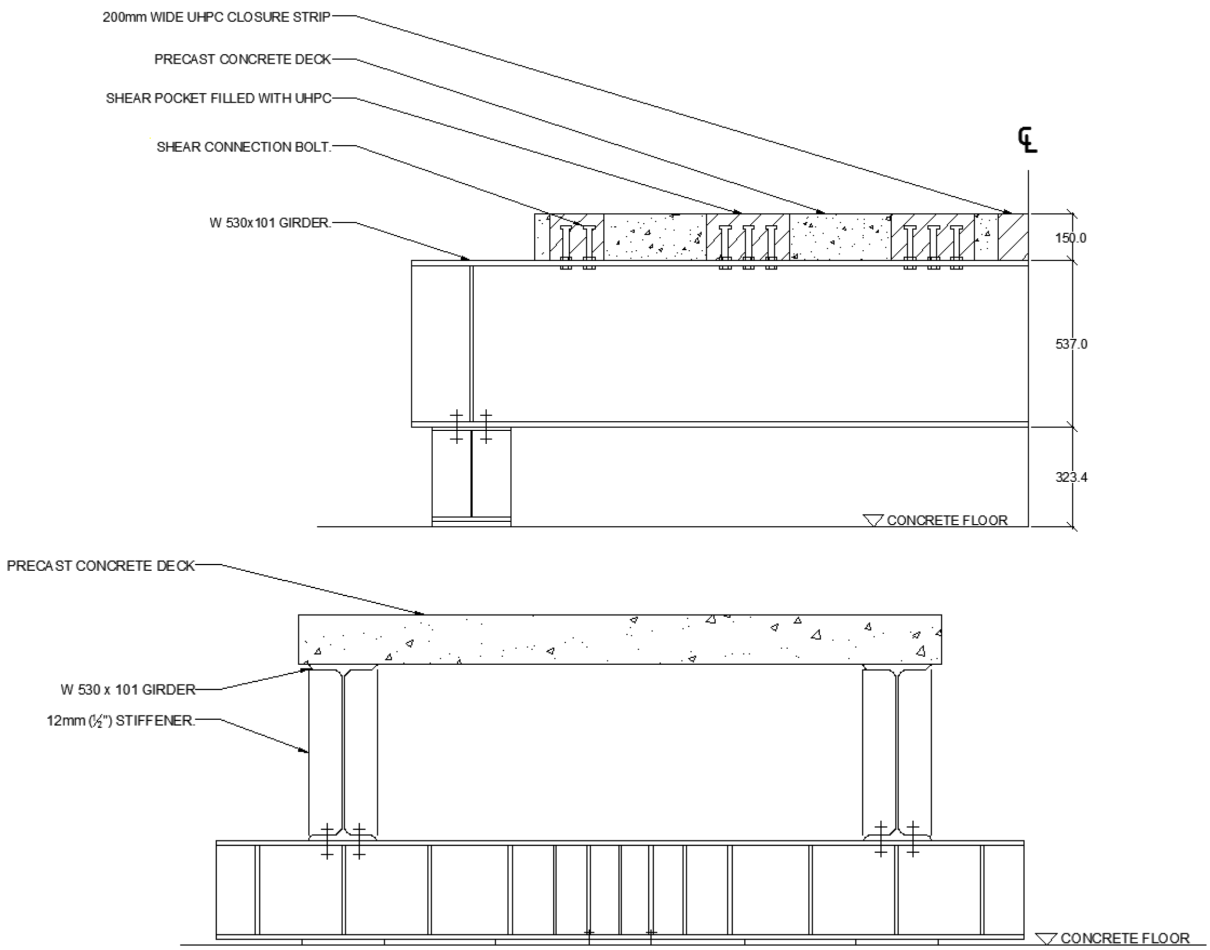

Figure 2: Diagrams of experimental setup with shear pocket locations.

The transverse deck joint between the precast panels consisted of a 200 mm wide closure strip. During casting of the precast slabs the longitudinal reinforcing bars were positioned to allow for 150 mm of GFRP bar to protrude from the interior face of the precast panel. These bars overlapped the bars from the opposite slab over a distance of 100 mm within the closure strip. The interior faces of the slabs were sandblasted prior to placement to allow for a better bond between the UHPC and precast concrete. Figure 3 shows the closure strip prior to casting of UHPC.

Figure 3: Closure strip prior to UHPC placement.

3.2 Material Properties

The precast concrete slabs were reinforced with sand-coated GFRP #5 bars. The precast slabs utilized self-consolidating concrete with a compressive strength of 46.5 MPa at the time of testing. The UHPC used to join the panels was a premix provided by a manufacturer with a 28-day compressive strength of 173 MPa. Table 1 shows a summary of all of the materials utilized within the slabs.

Table :. Material Properties

Property	GFRP Reinforcement	UHPC	Concrete
Elastic Modulus (GPa)	52.5	50 *	30.7
Compressive strength (MPa)	N/A	173.2	46.5
Tensile strength (MPa)	1130*	8*	N/A

*Indicates an estimated value provided by the manufacturer

3.3 Test Procedure

The specimen underwent fatigue loading at a load level of 100 kN for 2,000,000 cycles at three locations. The load of 100 kN was applied on an area of 150 mm x 400 mm. This load level and area were selected based on analysis which concluded that these dimensions and load provided equivalent reinforcement and concrete stress to a full depth, 225 mm deep concrete panel subjected to the load from the maximum loaded wheel with dynamic load allowance of the CL-625-ONT design truck loading (CHBDC, 2013). The reduced load and loaded area sizing were selected based on the 70% scale of the slab design. This initial sizing was confirmed based on analysis in SAP2000.

The fatigue loading was applied at three locations on the test specimen: the center of one panel, directly adjacent to the strip on the non-loaded panel, and on the centerline of the transverse UHPC joint. The center of the panel acted as a control location to compare the discontinuity of UHPC joint to a continuous concrete slab. The loading directly adjacent to the transverse joint was selected to create the maximum bar stress in the longitudinal bars within the closure strip, and to examine the effect of the discontinuity of the joint and the differing concrete material properties on the deck response. The loading on the center of the closure strip was selected to determine the effect of loading on the UHPC directly and its ability to transfer load into the adjacent panels. Following fatigue loading, the specimen was subject to static loading to failure directly adjacent to the closure strip. This location was deemed to be the most severe from a punching shear perspective as the UHPC joint partially intersects the punching shear cone. Figure 4 shows a plan view of the slab and the three fatigue load locations and the static failure locations.

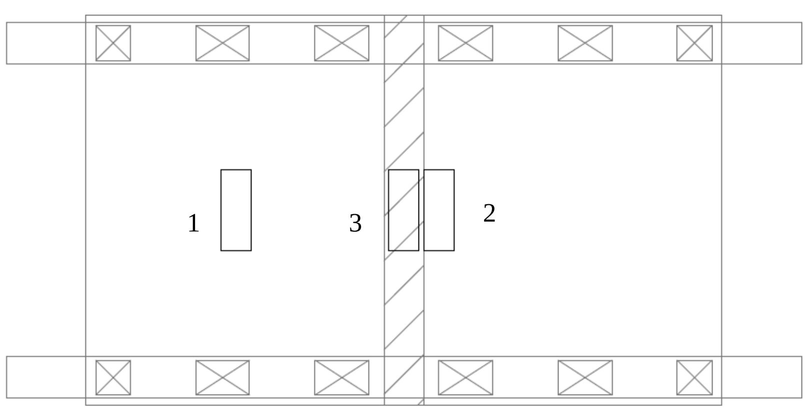

Figure 4: Load location layout. Fatigue loading proceeded from 1 to 2 to 3. Failure loading was applied at location 2 first and location 1 second.

4. EXPERIMENTAL RESULTS

4.1 Deck Response During Fatigue Loading

The bridge deck was subjected to constant amplitude fatigue loading ranging from 10 kN to 110 kN. For the load locations at the center of the panel and adjacent to the closure strip, it was clear that during the initial 200,000 cycles the deflection of the slabs significantly increased due to the formation and initial growth of cracks under the repeated loading. After this initial cracking period, the deflection increased gradually at an approximately linear rate for the remaining 1,800,000 cycles. It is also valuable to note that the slab did not return to the initial deflection value, but that the minimum deflection value (at 10 kN) and the maximum deflection value (at 110 kN) follow similar curves at different magnitudes over the duration of the fatigue loading. The slab response for the loading applied at the centreline of the closure strip did not exhibit the rapid increase in deflection over the first 200,000 cycles. It is assumed that since both deck panels were already cracked from loading at the first two locations, the initial rapid softening of the deck system did not occur at this load location. Figure 5 shows the deflection versus cycle count curves for the three fatigue loading locations.

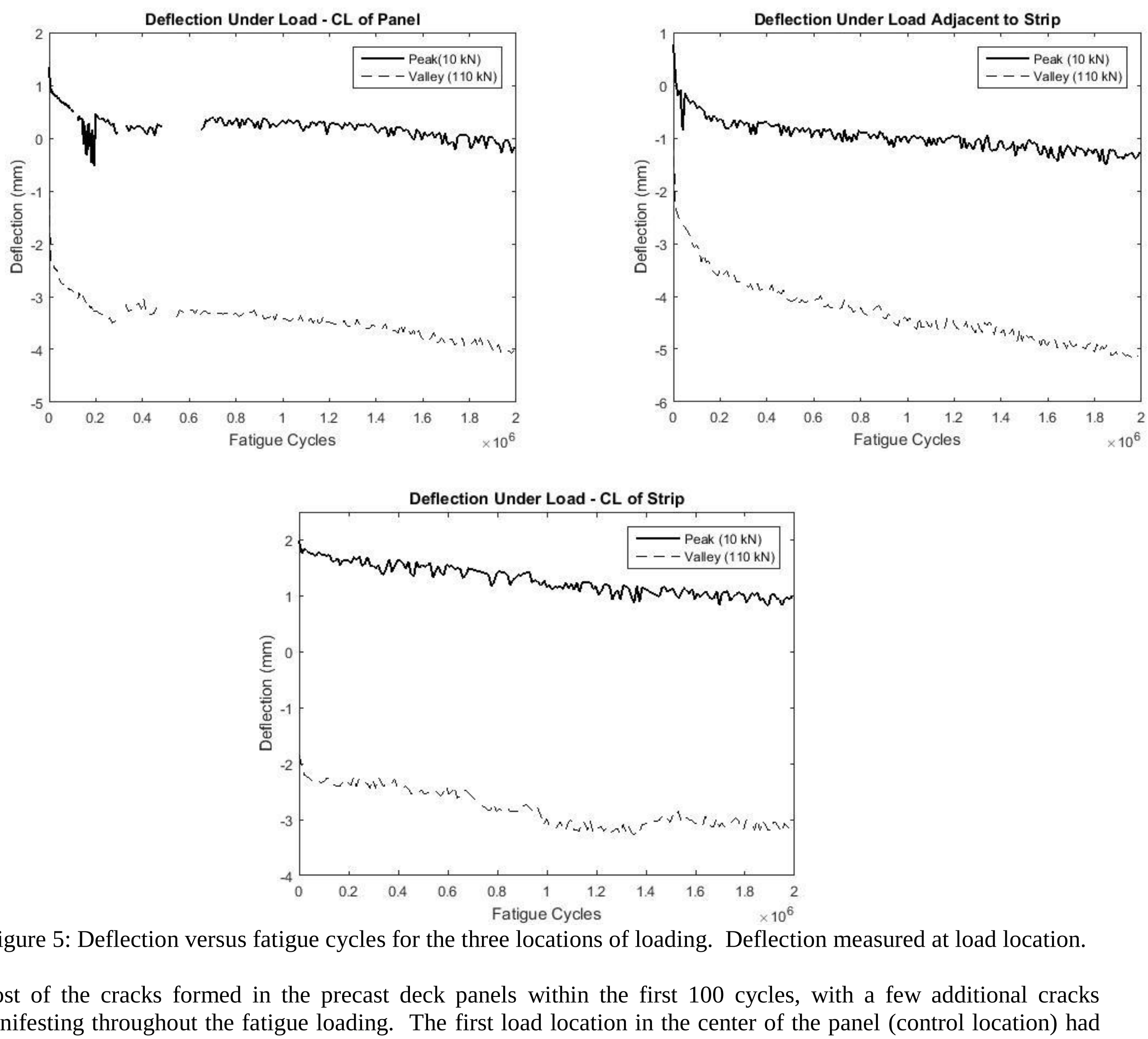

Figure 5: Deflection versus fatigue cycles for the three locations of loading. Deflection measured at load location.

Most of the cracks formed in the precast deck panels within the first 100 cycles, with a few additional cracks manifesting throughout the fatigue loading. The first load location in the center of the panel (control location) had cracks that reached the UHPC closure strip but did not extend into the strip. However, immediately after fatigue loading began at the second load location, adjacent to the closure strip, a crack extended across the width of the UHPC joint at the approximate midpoint of the joint. It was clear that the crack was wide enough to break the steel fibres within the UHPC, thus significantly reducing the tensile properties in that portion of the UHPC. Loading at the third location directly on the closure strip did not increase the number of cracks significantly, however fatigue in this location did increase the size of the crack through the closure strip. Figure 6 shows the crack in the UHPC closure strip.

Figure 6: Crack through UHPC during fatigue loading adjacent to the closure strip

4.2 Static Loading to Failure

Once fatigue loading was completed at the three load locations, the slab was loaded to failure at location 2 directly adjacent to the closure strip as this was the point of greatest interest due to the influence of the UHPC-to-precast interface. The slab experienced a punching shear failure, however it exhibited a significant amount of strength post-failure. It should also be noted that the punching shear failure plane did not pass through the UHPC (following the typical punching shear cone geometry), but rather was truncated at the interface between the UHPC and precast panel. Figure 7 shows the underside of the loaded area after failure loads were applied.

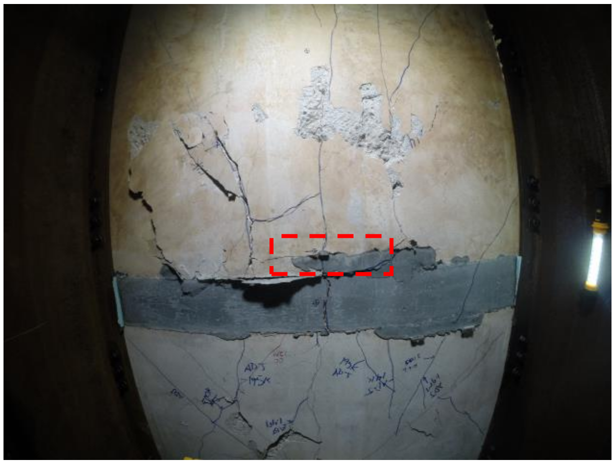

Figure 7: Underside of slab after loading to failure adjacent to panel (load location 2). Red rectangle indicates the approximate load location.

Figure 8 shows the load deflection curves for the static loading to failure adjacent to the closure strip. Note that the specimen was unloaded and reloaded several times during the static loading history as the interlock limits on the controller were triggered. Punching failure occurred adjacent to the strip at a load of approximately 285 kN as indicated by a sudden drop in the load carrying capacity coincident with the formation of a typical punching shear crack failure pattern. Applied loading was continued after the punching shear failure had occurred. During this time, the applied load increased to 325 kN as the load-carrying mechanism changed to arch action.

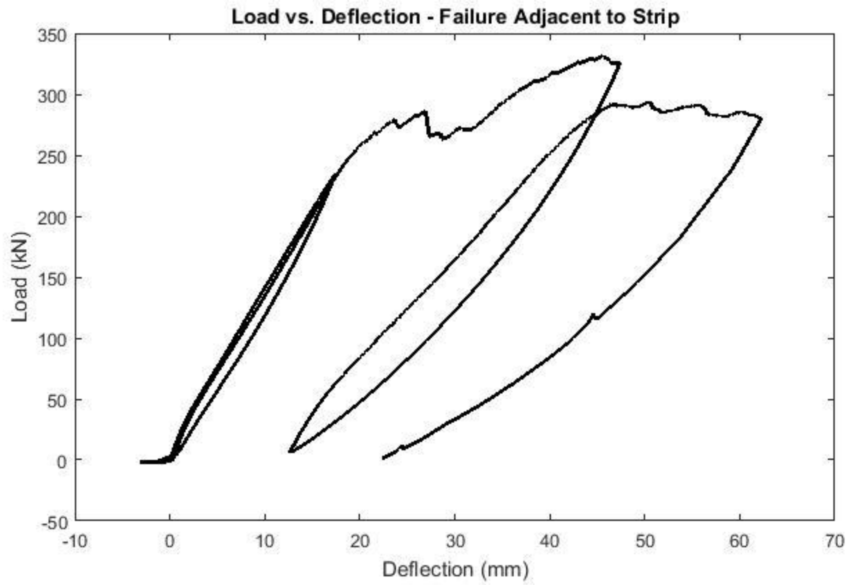

Figure 8: Load versus deflection curves for specimen loaded to failure adjacent to strip.

4.3 Comparison of Failure Load and Predicted Strength

The measured punching shear failure load was compared to the strength predicted using the CAN/CSA S6 (CHBDC, 2013) Clause 8.9.4.3 provisions as given by Equation 1.

[1] $$V_r = \left(\phi_c f_{cr} + 0.25 f_{pc}\right) b_o d + V_p$$

Where f_{cr} is the cracking strength of concrete (MPa), f_{pc} is the compressive stress in concrete after prestress losses (MPa), b_o is the critical section perimeter (mm), d is effective depth of reinforcement (mm), and V_p is the shear component from prestressing. For this case, it was assumed that b_o would be measured on the four sides of the loaded area and that d would be measured from the top fibre of the slab to the centroid of the bottom reinforcement layer. Using a resistance factor for concrete of 1.0 (for behaviour prediction rather than design strength) and omitting the prestressing contribution, V_p, since the deck was not prestressed, it was found that the calculated shear resistance of the slab should be 486.5 kN. This predicted value is greater than the punching load experienced, however the predicted failure load does not account for pre-existing damage to the panel (from fatigue loading) and assumes that the failure plane is symmetrical and is centred on the load point. For the case being considered, the load adjacent to the panel only had three standard failure planes since the fourth plane was truncated along the face of the UHPC. That is, the failure plane did not extend into the UHPC to follow the geometry normally assumed for a punching shear failure. It is assumed that the very high strength and toughness of the UHPC relative to the precast concrete panel resisted extension of the punching shear failure plane into the UHPC. Figure 9 shows a diagram of the approximate failure plane based on a top view of the slab after punching failure.


Figure 9: Approximate failure plane location indicated by red dashed line.

A more accurate prediction for the failure load of slab adjacent to the closure strip can be obtained using the punching shear provisions of CSA A23.3 (CSA, 2010), Clauses 13.3.3 and 13.3.4. These provisions differ from the CAN/CSA S6 provisions in that situations for edge and corner columns are addressed. In these conditions, the typical four-sided punching shear cone is truncated by the slab free edges, thus reducing the failure perimeter and punching shear capacity. Clause 13.3.4 predicts the failure shear stress as the minimum of Equation 2, Equation 3, and Equation 4.

[2] $$v_r = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c}$$

[3] $$v_r = \left(\frac{\alpha_s d}{b_o} + 0.19\right) \lambda \phi_c \sqrt{f'_c}$$

[4] $$v_r = 0.38 \lambda \phi_c \sqrt{f'_c}$$

Where β_c is the ratio of the long to short side of the load, α_s is a factor that adjusts the shear strength of concrete based on support dimensions, d is the distance from the extreme compression fibre to centroid of the tension reinforcement (mm), b_o is the critical section perimeter (mm), λ is factor for concrete density (1.0 for normal density concrete), ϕ_c is the resistance factor of concrete (1.0 for behaviour prediction), and f'_c is the compressive strength of concrete (MPa). For this case, $b_o$is assumed to only occur on three sides of the loaded area at a distance of one-half of d from the load edge.

In order to apply the CSA A23.3 provisions to the deck loading positon adjacent to the UHPC joint, it was assumed that the truncation of the failure cone by the UHPC had a similar effect to that of an edge column. As well, the bond between the precast panel and UHPC joint was assumed to be negligible, and the dowel action of the panel reinforcement extending into the UHPC joint was neglected. Using Equation 2 with βc of 0.75 the shear stress was found to be 2.23 MPa. Using Equation 3 α_s taken as 3 for a case with three planes of failure, the predicted failure shear stress for the location adjacent to the panel would be 3.68 MPa. Finally, Equation 4 finds a shear stress of 2.54 MPa. For a four sided failure plane, the failure load is calculated to be 377.8 kN. However, assuming the failure area is the three sides of a punching failure cone defined by the depth of reinforcement and critical perimeter, b_o, the predicted punching shear failure load in this case is 225.7 kN using the shear stress from Equation 2. Therefore, the failure load of 285 kN is between the three and four sided failure plane assumptions. This difference may be caused by the contribution of the shear or bond on the interface between the UHPC and precast panel which is insufficient to transfer the entire shear stress through the UHPC. Once the punching shear failure cone developed and slip occurred at the UHPC to panel interface, the GFRP did not appear to provide significant shear resistance in dowel action. Upon removal of the top layer of panel concrete, the top layer GFRP bar under the load position was found to have sheared near the UHPC to precast concrete interface. Figure 10 shows the shear failure through this bar.

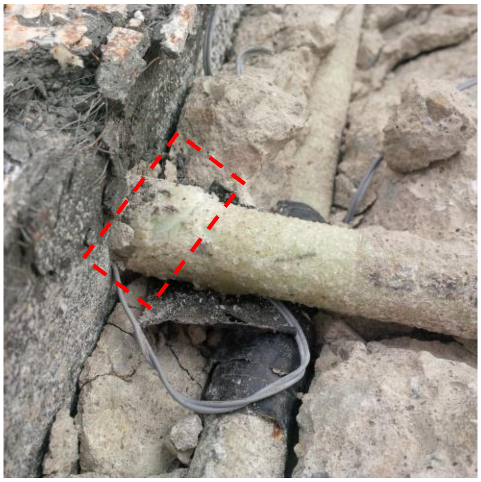
Figure 10: Shear failure of top GFRP bar after failure load adjacent to the closure strip.

5. FUTURE TESTING

Further tests will be conducted using a similar method of construction and loading. Two additional slabs will be created utilizing GFRP bars from a second manufacturer and one additional slab with same bar type used in the experiment described above. One difference for these tests is the precast concrete interface will have a greater surface roughness through an exposed coarse aggregate finish, rather than simply sandblasted. Figure 11 shows the difference between the tested sandblasted surface and the new coarse aggregate surface.


Figure 11: Previous sandblasted surface (left) and the new coarse aggregate finish (right).

6. CONCLUSIONS

In conclusion, a slab consisting of two precast panels joined by a UHPC closure strip was tested under fatigue loading followed by static loading to failure. Under fatigue loads it was found that slab deflections increased significantly for the 200,000 cycles followed by gradually increasing deflections for the remaining 1,800,000 cycles of fatigue loading. During fatigue loading, cracks were noticed primarily during the initial loading phase with few cracks propagating during the latter stages of fatigue loading. The slab was loaded to failure directly adjacent to the UHPC closure strip which resulted in punching failure. The failure load of 285 kN was less than the punching shear capacity predicted by CAN/CSA S6 of 485 kN. The reason for this difference is likely due to truncation of the punching shear failure cone along the precast to UHPC interface, rather than the assumed full, four-sided cone of failure. Future testing is required to identify if this interface can be improved to provide greater shear resistance and increase the ultimate failure load of the specimen. Future investigation will be conducted to determine what effect fatigue has on the ultimate capacity of the slab.

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