Influence of Heating and Cooling Temperature on the Performance of Tensile Interfacial Bond between High Strength Concrete and Ultra High Performance Concrete (UHPC)

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Abstract: Ultra-high performance concrete (UHPC) is a cementitious material which has superior mechanical properties, ductility, and long-term durability. The exceptional performance and outstanding properties of UHPC makes it suitable as a grout material in connections between prefabricated bridge elements or an overlays for bridge rehabilitation. However, UHPC has high coefficient of thermal expansion (CTE) as compared to conventional or high strength concrete (HSC) due to its high cement content and the absence of the coarse aggregates. The difference in the CTEs between HSC, normally used in prefabricated bridge elements, and UHPC generates thermal stresses at the interface when subjected to heating and cooling temperature. The thermally induced stresses could lead to interfacial failure when the interface bond strength is exceeded or cause a reduction in the bond strength. In this study, the effect of heating and cooling temperature on the interface bond strength was investigated using a direct tension test. Composite specimens made of HSC and UHPC were subjected to heating and cooling temperature based on the AASHTO LRFD bridge design specifications and then tested under direct tension. The thermal strains generated under these conditions were also measured for both materials and then plotted against temperature. Composite specimens stored under room temperature conditions were also tested in the direct tension apparatus, and the results compared with those of composite specimens subjected to heating and cooling temperature. A bond versus slip relationship under room temperature that was investigated from the previous research was compared with relationship under temperature conditions. A reduction of 34% in the mean bond strength was observed when the composite specimens were subjected to heating and cooling temperature.

Keywords: UHPC, Heating, Cooling, Temperature, Thermal stresses, Bond, Bridge, Connection, Direct tension

1. Introduction

Temperature is considered one of the major factors significantly affecting the performance of bridges. Bridges just like other reinforced concrete structures are exposed to daily temperature fluctuations which may cause a significant effect on the mechanical properties of concrete as well as on the interfacial bond strength between bridge components. In practice, a bridge may be subjected to uniform temperature, temperature gradients or a combination of both. A uniform change in temperature can be calculated using either procedure A or B of the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD, 2016) depending on the type of bridge. Procedure A or B is applicable for concrete decks placed on concrete or steel girders whereas only procedure A is applicable for other types of bridges. In procedure A, the upper and lower uniform temperature changes were determined based on type of climate (moderate or cold) considering the number of the freezing days (average temperature less than 0°C [32°F]) per year (AASHTO LRFD 2016). The climate is considered moderate if the number of the freezing days is less than 14. Based on procedure A, the maximum extreme temperature is 49°C (120°F) whereas the minimum extreme temperature is -34°C (-30°F). Procedure B uses the contour map. if the bridge consists of concrete deck placed on concrete girder, the maximum temperature is 52°C (125°F) while the minimum temperature is -48°C (-55°F). In Minnesota for instance, the maximum and minimum temperatures are -40°C to 40.5°C (-40°F to 105°F), respectively. For steel girder bridges with concrete decks, the maximum and minimum temperatures are 54°C to -51°C (130°F to -60°F), respectively. The maximum and minimum temperature for Minnesota are -45.5 °C to 43°C (-50°F to 110°F), respectively. The state of Minnesota records relatively higher variations between maximum and minimum temperatures across the United States. For design purposes, the thermal deformation is calculated by taking the difference between the upper or lower limits (procedure A or B) and the base construction temperature. The linear or nonlinear changes in temperature over the height of the girder refers to the temperature gradients. Bending, deflections, and self-equilibrating stresses may develop for a bridge subjected to nonlinear temperature distribution across its height (Semendary et al.2018). Irrespective of the temperature type (e.g. uniform or gradients), the coefficient of thermal expansion (CTE) plays a significant role in the calculated deformation as well as the thermally induced stresses.

Ultra-high performance concrete (UHPC) is a cementitious material, which has superior mechanical properties, ductility, and long-term durability. The exceptional performance and outstanding properties of UHPC makes it a suitable field cast-filling material in connections between prefabricated bridge elements or an overlays for bridge rehabilitation. However, UHPC has a higher CTE as compared to the precast concrete components due to its high cement content and the absence of coarse aggregates (Graybeal 2006). The differences in CTE can cause relative thermal movements between the UHPC connections and the surrounding components which can produce thermaly induced stresses at the interface when subjected to heating and cooling temperature. The thermally induced stresses may lead to interfacial failure when the interface bond strength is exceeded or results in less capacity to resist additional stress from other loads. The CTE of UHPC has been investigated and reported by other researchers as shown in Table 1. The CTE of normal concrete as specified by AASHTO LRFD (2016) is 10.8×10^{-6}/°C (6.0×10^{-6}/°F). The CTE depends on different parameters such as w/c ratio, age of the concrete, and moisture content (Mindess et al. 2003). Partially saturated concrete specimens have been reported to have CTE values of about 1.8 times higher than fully saturated concrete specimens whereas dry concrete specimens have been found to record CTE values that are 1.17 times larger than fully saturated concrete specimens (Emanuel and Hulsey 1977). Ahlborn et al. (2008) reported that thermally treated specimens had significantly higher CTE than air treated specimens as the CTE increases significantly with the age.

Table 1. Values of Coefficient of Thermal Expansion

Reference	Coefficients of thermal expansion (CTE)		Testing method
	x 10-6/°F	x 10-6 /°C	
Graybeal (2006)	8.3	15	AASHTO (TP60-00)
Ahlborn et al. (2008)	7.7	13.9	AASHTO (TP60-00)
JSCE (2006)	7.5	13.5	-
Hussein et al. (2016)	9.4	16.9	Ohio CTC Device

Despite the detrimental effect of these heating and cooling temperature on the bond strength and durability of UHPC and precast concrete interfaces, no comprehensive study has been conducted to investigate the effect of thermal stresses associated with heating and cooling temperature on the performance of bonds at the interfaces.

2. Literature Review

Field cast UHPC connections with high strength concrete (HSC) prefabricated bridge components can be exposed to temperature fluctuations that may rise to 52°C (125°F) in summer and decrease to -48°C (-55°F) in winter based on the extreme temperature range of AASHTO LRFD Bridge Design Specifications. As with any two bonded materials, the HSC-UHPC interface may be subjected to mechanical and/or environmental loads that may cause stresses at the interface. The bond performance at the HSC-UHPC interface under temperature effects has not yet been investigated. There is no applicable ASTM standard to investigate the effect of freeze and thaw cycles on bond performance (Lie et al. 1999). Furthermore, there is no ASTM standard that can be used to investigate the performance of bond strength under temperature changes. The performance of bond between two bonded materials was investigated using concrete prisms subjected to freeze-thaw cycles and tested using the split-tensile strength test method. According to the ASTM C666/C666M, procedure A or B can be used to determine the resistance of concrete to deterioration caused by rapid repeated cycles of freezing and thawing in the lab. In both procedures, the temperature of the specimens should be lowered from 4 to -18 °C (40 to 0 °F) and raised from -18 to 4 °C (0 to 40 °F)) between 2 to 5 hours. The maximum and minimum temperatures for both procedures are 4 °C and -18 °C, respectively. Therefore, the temperature limits from AASHTO LRFD (2016) is much higher /lower as compared with the ASTM standards.

A study by Li et al. (1999) examined the effect of freeze-thaw cycles on the bond performance by exposing prisms made of two bonded materials to temperature cycles before testing them in the split-tensile strength test apparatus. Composite prismatic specimens with dimensions 102 × 76 × 406 mm (4×3×16 in.) in accordance with the ASTM standard dimensions were used in the freeze-thaw test. After the prisms were subjected to freeze-thaw cycles, the specimens were saw-cut into four smaller prisms 76×76×102 mm (3×3×4 in.) and tested by splitting. The specimens were subjected to 300 cycles following the ASTM C 666 Procedure A. The results indicated that the bond performance under freeze-thaw cycles depends on numerous factors, namely material types, mix design, casting conditions, and curing conditions. The bond strength after 300 freeze-thaw cycles had reduced between 5 to 30 % as compared to specimens without freeze-thaw cycles. Geissert et al. (1999) reported that under dry or wet curing conditions, the freeze-thaw cycles had no significant effect on bond strength in the split-tensile strength test after exposure to 300 freeze-thaw cycles when the results were compared to unexposed specimens. The effects of temperature on the performance of the bond between polymer cement mortar (PCM) and concrete was investigated at temperatures 20, 40 and 60 °C (68°F, 104°F, and 140°F) by Tamon et al. (2017). After the specimens were exposed to temperature effects, both split-tensile and bi-shear strength test methods were used to investigate the interface bond strengths. The specimens were exposed to a temperature of 40 or 60 °C (104 °F or 140 °F) for more than 16 hours after a significant time from casting and then tested under a temperature of 20 °C (68°F). Special strategies were used while moving the specimens from the chamber to the testing machine to minimize temperature changes. Furthermore, the testing machine was equipped with heaters to maintain the desired temperature level during the test. The mechanical properties for concrete as well as PCM were also subjected to different temperature levels. The effects of temperature on compressive strength of concrete was insignificant whereas a large reduction in compressive strength of PCM was observed. The tensile bond strength under splitting test reduced by 26% and 30% at temperatures 40°C (104°F) and 60°C (140°F), respectively as compared to control specimens tested at 20°C (68°F). The shear bond strength reduced by 30% and 70% at temperatures of 40°C (104°F) and 60°C (140°F), respectively as compared to control specimens. The authors reported the high cement content and small water to cement ratio used in the mix of PCM as the reasons for the reduction in compressive strength and deformation at the interface. The difference in the coefficient of thermal expansion between concrete and PCM generated high internal stresses and micro cracks that caused weakness at the interface.

The effects of temperature on the performance of the bond between normal strength concrete (NSC) and ultra-high-performance concrete (UHPC) has been investigated by Carbonell Muñoz et al. (2014) using a combination split-tensile test and freeze-thaw cycles based on ASTM C666/C66M (2008b). Different surface preparations ranging from sandblasted, brushed, smooth, chipped, and grooved surfaces were tested. The same test configuration used by Li et al. (1999) and Geissert et al. (1999) were used by casting prisms with dimensions of 102 × 76 × 406 mm (4×3×16 in.). The large prisms were further cut down into four smaller prisms. The specimens were subjected to 300, 600, and 900 freeze-thaw cycles based on Procedure B to provide more severe conditions. All specimens with the exception of the grooved ones failed during the cutting process when the dry surface preparation was utilized, which indicated a weak bond strength under dry conditions. Excellent bond strength was observed using the saturated surface dried (SSD) substrate. The freeze-thaw cycles had no effect on the interface bond strength. The interface bond strength increased as the number of cycles increased because UHPC had a significant amount of un-hydrated cement particles at the interface that hydrates when water is present. Therefore, the effects of the freeze-thaw cycles on the bond performance with UHPC was unreliable as the water presents in both procedures which has a positive effect on the bond performance by allowing to the un-hydrated cement particles to hydrate. An investigation of bond performance under dry conditions to study the effects of temperature on the interface bond strength is, therefore, needed for a complete characterization of the effect of freeze-thaw cycles on HSC-UHPC interface bond strength.

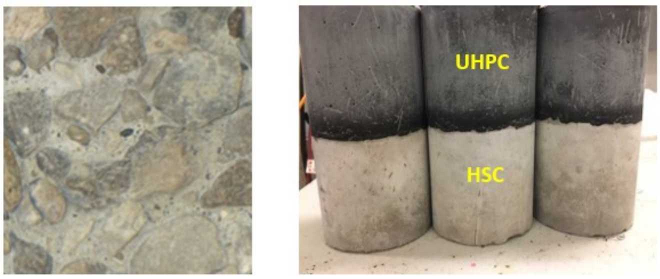
3. Experimental Program

The experimental program consisted of two parts. The first part describes the testing procedures that were used to measure the thermal strains as well as the methodology for applying the heating and cooling temperature to the specimens. The second part describes the test method that was used to measure the bond strength between high strength concrete (HSC) and UHPC under a direct tension test state.

3.1. Specimen preparation

The thermal strain of HSC /UHPC was measured using specimens obtained by cutting three 37.5 mm (1.5 in.) thick test specimens from 150 mm (6 in.) diameter by 300 mm (12 in.) long cylindrically shaped concrete samples. The HSC test specimens were made using Type I Portland cement, slag cement, coarse aggregate, natural sand, and admixtures. The concrete cylinders were prepared and tested for compressive strength according to ASTM C 39/C 39M – 05 test protocol (ASTM 2015). The concrete reached a compressive strength of 63 MPa (9.14 ksi) at the time of the test (after approximately 120 days). The split-tensile strength samples were tested in accordance with the ASTM C 496/C 496M – 04 test procedure (ASTM 2004) and was found to be 5.52 MPa (0.8 ksi). Commercially available UHPC was used in this research. The mix comprised of premix, steel fibers, water, and superplasticizer. The steel fibers formed 2% of the mix volume. The concrete cylinders were prepared and tested for compressive strength using the ASTM C 39/C 39M – 05 (ASTM 2015) test protocol with the exception of UHPC where the loading rate was increased to 1 MPa/s (150 psi/s) due to its high strength (Graybeal 2006). The compressive strength reached 136 MPa (20 ksi) during testing.

The specimens that were used to measure the interface bond strength under direct tension stress state were prepared in accordance with the ASTM C1404/C1404M (ASTM 2003) protocol. In this test, the exposed aggregate surface preparation as recommended by Graybeal (2014) for the UHPC bridge connections was used. The exposed aggregate surface was prepared by casting 75 mm x 150 mm (3 in. by 6 in.) HSC concrete cylinders using a form retarder. The surface was power washed after 24 hours to expose the aggregate. After completing the curing cycle, the cylinders were cut into two parts and only the part with the exposed aggregate end was used. The UHPC part was cast at approximately 70 days after casting the HSC to limit dimensional changes of substrate such as shrinkage in the HSC which may cause stresses at the interface. A dry surface preparation was used to determine a more conservative value of bond strength. The saturated surface dry interface preparation has been reported providing better bond (Carbonell Muñoz et al. 2014). The test was performed at approximately 90 days after casting UHPC part. The surface preparations and the composite specimens are shown in Figure 1. Three specimens were tested without temperature effects and the other three specimens were subjected to temperature for 14 hours before testing.

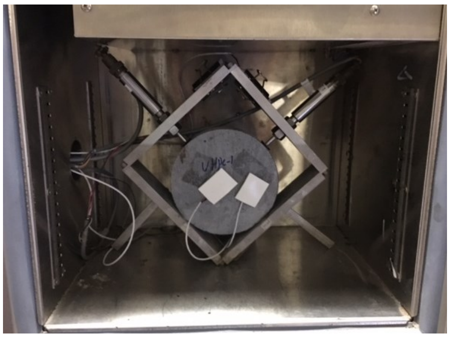


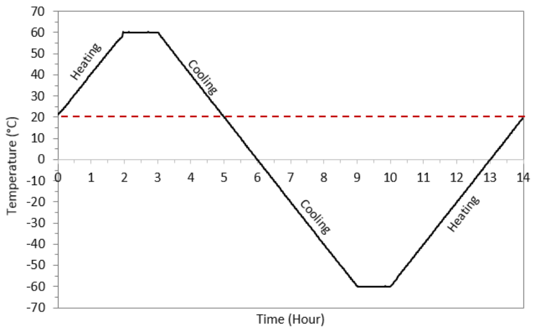
(a) (b)

Figure 1. Specimens Preparation: (A) An Exposed Surface; (B) Composite Specimens

3.2. Thermal Strain Measurements and the Application of Thermal Cycles

The Ohio CTC device (OCD) was used to measure the thermal strains generated in the HSC and UHPC specimens associated with thermal cycles and apply heating and cooling temperature to the HSC-UHPC composite specimens. The OCD was initially developed for the measurement of the Coefficient of Thermal Contraction (CTC) of asphalt mixtures (Akentuna et al. 2017). The OCD was constructed with 6061 general purpose aluminum bars with dimensions 50.8 mm (2 in.) wide and 12.7 mm (0.5 in.) thick. They were assembled into a square frame with 209.6 mm (8.25 in.) by 209.6 mm (8.25 in.) outer dimensions. During testing, the test frame was placed on its corner with the aid of supports as shown in Figure 2a. Each 150 mm (6 in.) diameter and 37.5 mm (1.47 in.) thick HSC and UHPC specimen was placed at the bottom corner of the Ohio CTC device frame during testing. Two LVDTs with flat tips were fixed at the top sides of the device frame in such a way that they were mutually perpendicular and coincided with the diameter of the test specimen. As temperature changed, the LVDTs took two independent measurements of the change in diameter of the test sample. The change in temperature of the sample and the frame was measured using four resistance temperature detectors (RTDs). The test specimen was then subjected to the thermal cycle illustrated in Figure 2b. The maximum and minimum temperatures were 60 °C and -60 °C (140 °F to -76 °F), respectively, which slightly exceeded the limits from the AASHTO LRFD (2016). The room temperature was assumed to be 20 °C (68 °F). Three specimens were tested for both HSC and UHPC samples. Each specimen was kept in the chamber for 14 hours to complete the one temperature cycle. The strain readings from the two LVDTs were corrected to account for the deformation of the aluminum frame. The strains were then averaged and plotted against temperature for the determination of thermal contraction/expansion coefficient (CTC/CTE) of the HSC and UHPC concrete. The thermal cycle profile illustrated in Figure 2b was also applied to composite HSC-UHPC samples placed in the OCD chamber before testing them in the direct tension apparatus.





(a) (b)

Figure 2. (a) OCD Test Setup; (b) Temperature Profile used in The Test

3.3. Direct Tension Test

Direct tension tests were conducted using a MTS testing machine (Q Test/25) with an ultimate capacity of 25 kN (5.6 kips), which satisfied the requirements of ASTM C1404/C1404M test protocol (ASTM 2003). After subjecting the HSC-UHPC composite specimen to heating and cooling temperature, they were placed vertically on a flat and clean surface. An O-ring with a 73 mm (2.8 in.) inside diameter, 83 mm (3.3 in.) outside diameter, and a thickness of 5 mm (0.2 in.) was placed at the HSC-UHPC interface. Type V, Grade 1 epoxy resin was mixed and placed on the outside surface of the HSC. A steel-pipe nipple with 75 mm (3 in.) inside diameter and 75 mm (3 in.) long was installed from the top of the HSC until it reached the O-ring. The specimen was then inverted and the outside surface of the UHPC was covered with epoxy. Another steel pipe nipple was installed over the UHPC using the same procedure as used for the HSC half. The epoxy in the specimen was then allowed to cure in accordance with the manufacture's specifications. Two steel-pipe caps 75 mm (3 in.) in diameter were threaded to fit the steel-pipe nipple molds. The caps were then connected to the MTS machine as shown in Figure 3. The load was applied by setting the cross-head speed at 1.0 mm/min (0.04 in./min) until failure. The load and displacement data together with the failure mode were reported for each specimen. The tensile/bond strength was calculated as shown in Eq. 1.

$$T = \frac{P}{A} \tag{1}$$

Where: *T*=tensile strength or bond strength at failure, *P*=maximum applied load, and *A*=cross area of concrete. The failure in this test method is anticipated to occur in the HSC, UHPC, or at the interface. If the failure occurs at the interface, the measured bond strength will be the adhesive force between UHPC and HSC. However, the bond strength will be considered cohesive if the failure occurs away from the interface.

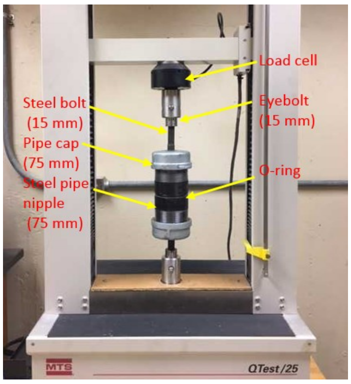


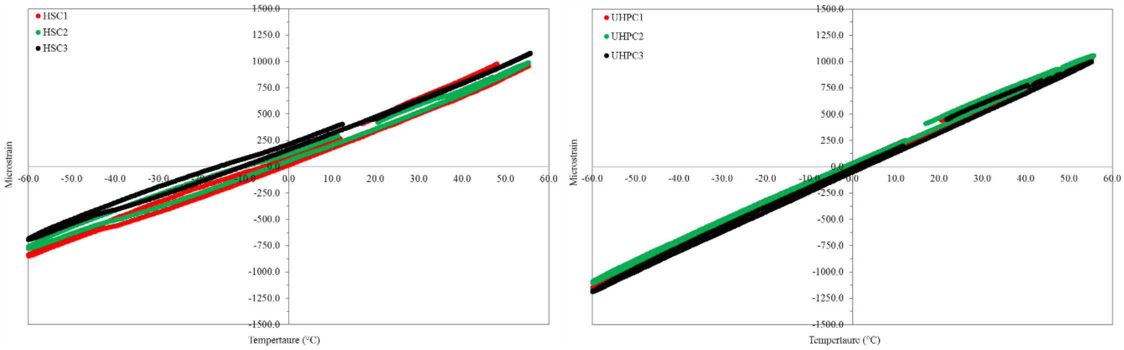
Figure 3. Testing Machine with Composite Specimen

4. Experimental Results and Discussions

The results from this study was presented in two sections. The first section discusses the behavior of HSC and UHPC under temperatures by measuring the thermal strains and CTC/CTE using the OCD. The second part of the results comprised of the analyses of the effect of thermal cycle on the tensile bond strength under direct tension.

4.1. Effects of Temperature on the thermal strains and CTC/CTE of HSC, UHPC, and HSC-UHPC composite concrete materials

The thermal strains generated in the HSC and UHPC specimens under temperature conditions were plotted against temperature as shown in Figure 4. The UHPC exhibited relatively similar thermal strains for the heating and cooling temperature within a temperature range of 20°C to -60°C. For the 20°C to 60°C temperature range, there was a slight difference in the strains recorded in the UHPC during the heating and the cooling temperature. The HSC, however, exhibited different strain values for heating and cooling temperature for the temperature ranges (20°C to-60°C and 20°C to 60°C) considered. The relatively higher difference between the strains recorded in HSC during the heating and cooling temperature may be attributed to micro cracks caused by the temperature conditions. Table 2 presents a summary of the CTC/CTE of the HSC and UHPC materials at different temperature ranges. The CTE values from this study were higher than reported values from literature. The difference in CTE values can be attributed to the difference in the testing method, temperature range as well as the saturated conditions of the specimens during testing. Dry concrete specimens were used in the current study. The literature indicates dry concrete specimens have been found to record CTE values that are 1.17 times larger than fully saturated concrete specimens (Emanuel and Hulsey 1977). For a temperature range of 20°C to 60°C, there was no noticeable difference between the CTC/CTE of HSC and UHPC for the heating and cooling temperature. At temperatures between 20°C to -60°C however, the UHPC exhibited higher CTC/CTE than the HSC for the heating and cooling temperature. The difference in CTC/CTE between the UHPC and HSC within this temperature range may generate stress which can compromise the strength of the bond between UHPC and HSC.



(a) (b)

Figure 4. Thermal Strain Versus Temperature Under Heating and Cooling Temperature Cycles: (a) HSC; (b) UHPC

The results agreed with field data for both HSC and UHPC which showed that the UHPC contracted more under lower temperature than HSC did (Semendary et al. 2018).

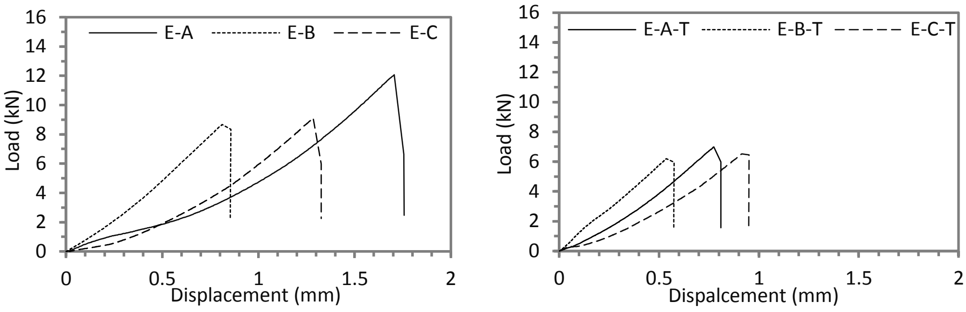
Table 2. Coefficient of Thermal Expansion Values Based on Data Analysis

Specimens Identification	Coefficient of Thermal Expansion (Length/Length/°C)							
	Temperature range							
	20°C to 60°C				20°C to -60°C			
	Heating		Cooling		Heating		Cooling	
	10^{-6}/°F	10^{-6}/°C	10^{-6}/°F	10^{-6}/°C	10^{-6}/°F	10^{-6}/°C	10^{-6}/°F	10^{-6}/°C
HSC-A[a]	9.78	17.61	9.76	17.56	8.42	15.15	8.06	14.50
HSC-B	8.92	16.06	9.85	17.73	7.99	14.39	7.76	13.97
HSC-C	9.63	17.34	9.32	16.78	8.12	14.61	7.96	14.32
Mean	9.4	17.0	9.6	17.4	8.2	14.7	7.9	14.3
COV (%)	4.86		2.92		2.7		1.93	
UHPC-A[b]	9.46	17.02	10.73	19.31	10.73	19.31	10.47	18.85
UHPC -B	9.22	16.59	10.40	18.72	10.26	18.48	10.34	18.61
UHPC -C	8.98	16.18	10.65	19.17	10.63	19.14	11.50	20.70
Mean	9.2	16.6	10.6	19.1	10.5	19.0	10.8	19.4
COV (%)	2.6		1.63		2.35		5.9	

[a]HSC (High Strength Concrete)-A, B and C (replicated)
[b]UHPC (Ultra High Performance Concrete)-A, B and C (replicated)

4.2. Effect of Thermal Cycle on Interface Bond Strength

The interface bond strengths under direct tension states were investigated for the specimens with and without temperature effect The load versus displacement plots as well as the failure modes are shown in Figure 5 and 6.



(a) (b)

Figure 5. Load Versus Displacement Plots Under Direct Tension: (a) Without Temperature; (b) With Temperature

(a) (b)

Figure 6. Failure Mode: (A) Without Temperature Effects; (B) With Temperature Effects

There was a relatively linear relationship between the applied load and the displacement until the peak load after which a sudden reduction in capacity was observed. The maximum and minimum loads for the specimens that were cured under room temperature conditions and tested after approximately 90 days were 12.1 kN for E-A and 8.7 kN for E-B, respectively (Semendary et al. 2019). However, the maximum and minimum loads for the specimens that were cured under room temperature and then subjected to temperature profile (Figure 3b) before testing were 6.9 kN for E-A-T and 6.2 kN for E-B-T, respectively. A significant reduction was observed due to the temperature effect. This reduction was also supported by the observed difference in the failure modes shown in Figure 6. Interface failure modes were observed in both cases. The failure mode of the specimens not exposed to temperature condition was fracture in the aggregate and cement paste, whereas only fracture in aggregates was observed in the specimens that were exposed to temperature cycles. This indicates that the difference in the thermal expansion between UHPC and HSC created stress at the interface that caused the failure mode to shift up toward the HSC side or the micro cracks that occurred due to temperature in HSC caused a damage in the cement paste at the interface or reduced the bond between aggregate and cement paste. The difference in the coefficient of the thermal expansion between aggregate and surrounding concrete at extreme cooling temperatures may produce a high internal stresses that may weaken the bond between aggregate and hydrated cement paste (Rashid et al. 2014). This phenomenon may account for UHPC because it has high CTC/CTE as compared to the HSC at relatively lower temperatures. The mean interface bond strength for the specimens without temperature was 2.23 MPa with a coefficient of variation (COV) of 18.7%. For the specimens exposed to temperature effects the mean interface bond strength was 1.47 MPa with a COV of 6.04% as shown in Table 3. The maximum reduction in the mean interface bond strength was 34 % and occurred under heating and cooling temperature.

Table 3. Summary of Direct Tension Test Results for The Specimens with/out Temperature Effects

Sample No.	Load (kN)	Maximum displacement (mm)	T, Tensile strength at failure (MPa)
E-A[a]	12.1	1.76	2.70
E-B	8.7	0.86	1.94
E-C	9.1	1.33	2.04
			Mean= 2.23
			COV (%)=18.7
E-A-T[b]	6.9	0.81	1.56
E-B-T	6.2	0.58	1.38
E-C-T	6.5	0.95	1.46
			Mean= 1.47
			COV (%)=6.04

[a]E (Exposed)-A, B and C (replicated)
[b] E(Exposed)-A, B and C (replicated), T (Temperature)

5. Conclusions

The effects of a heating and cooling temperature on the performance of the interface bond between HSC and UHPC was investigated. Based on the analyses of the results, the following conclusions were drawn:

- The UHPC exhibited relatively similar thermal strains for the heating and cooling within a temperature range of 20°C to -60°C and a slight difference in strains between the heating and cooling for the 20°C to 60°C temperature range.
- The HSC exhibited different strain values for heating and cooling for the temperature ranges 20°C to-60°C and 20°C to 60°C.
- The relatively higher difference between the strains recorded in HSC during the heating and cooling may be attributed to micro-cracks generated by the temperature condition.
- The measured CTC/CTE of HSC and UHPC from this study were higher than the reported values from literature. The difference may be attributed to the difference in the testing method, materials and/or the saturated conditions of the specimens during the test.
- There was no noticeable difference between the mean CTC/CTE of HSC and UHPC for the heating and cooling within a temperature range of 20°C to 60°C.
- For the temperature range of 20°C to -60°C, the UHPC exhibited higher mean CTC/CTE than the HSC for the heating and cooling. The difference in CTC/CTE between the UHPC and the HSC within this temperature range may result in the generation of thermal stresses at the interface of the UHPC-HSC composite bond.
- The maximum reduction in the mean interface bond strength was 34 % and occurred under heating and cooling temperature. This reduction was also supported by the observed difference in the failure modes.

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