

Effect of Extreme Temperatures on the Coefficient of Thermal Expansion for Ultra-High Performance Concrete

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Abstract: During the last decade, ultra-high performance concrete (UHPC) has been used in highway bridge connections due to its superior strength and durability. Specifically, UHPC shear keys have been used to reduce joint cracking and enhance transverse load transfer between high strength concrete (HSC) girders of adjacent box-girder bridges. In the calculation of thermal stresses between UHPC and HSC components, the thermal expansion coefficients of the materials are required. In current analysis/design of bridge structures with normal strength concrete, the coefficient of thermal expansion (CTE) is assumed to be constant for temperatures between 32 and 140°F (0 to 60°C). However, recent field analysis of an adjacent box-girder bridge with HSC girders connected using UHPC shear keys has shown that the actual temperature range experience by the UHPC-HSC was 14 to 100.4°F (-10 to 38°C). Furthermore, according to the bridge design procedure specified by AASHTO, temperature ranges between -55 to 125°F (-48.3 to 51.6°C) should be used in the analysis/design of bridges depending on their geographic location within the United States. The main objective of this study was to establish thermal expansion-temperature curves for UHPC and HSC for temperatures ranging from -76 to 140°F (-60 to 60°C). The data was obtained using the Ohio CTE testing method so that the thermal strain could be measured in the air-dry condition. The experimental results revealed the CTE of UHPC tested in this study to be constant for the temperature range considered, with an approximate value of $9.4 \times 10^{-6}/^{\circ}\text{F}$ ($16.9 \times 10^{-6}/^{\circ}\text{C}$). Also, the CTE of HSC is approximately linear in the temperature range from 35 to 70°F (1.7 to 21.1°C), and constant for temperature changes outside this range. An average value of HSC CTE was determined to be $6.0 \times 10^{-6}/^{\circ}\text{F}$ ($10.8 \times 10^{-6}/^{\circ}\text{C}$). The findings of this study are expected to be useful in the design/analysis of bridges to calculate the thermal stresses between UHPC and HSC composite structures.

Keyword: Thermal strain; Coefficient of thermal expansion; CTE; Ultra-high performance concrete; High strength concrete.

1. Introduction

1.1 UHPC in Highway Bridge Joints

Recently, ultra-high performance concrete (UHPC) has been used increasingly in highway bridges such as in precast waffle panels for bridge decks, in shear keys as grout material, as well as in precast pre-stressed girders. A comprehensive list of UHPC applications in highway bridges can be found in the literature (Russell and Graybeal, 2013). UHPC has enhanced material properties such as increased strength, exceptional durability, and long-term stability. Unlike traditional normal strength concrete (NSC) and high performance concrete (HPC), UHPC tends to have low water content, typical gradation of granular materials, and minimal usage of coarse aggregates. UHPC mix consists of steel fibers, fine sand, Portland cement, accelerating admixture, high-range water reducer, ground quartz, and other additives. Superior mechanical properties of UHPC are a tensile strength of 1.3 ksi (9.0 MPa), a significant tensile toughness, a compressive strength of 28 ksi (193 MPa), an elastic modulus of 7600 ksi (52.4 GPa), and minimal long-term creep or shrinkage (Graybeal, 2006). Hussein et al. (2016) reported that UHPC has superior adhesion strength to high strength concrete (HSC) for surfaces with varying roughness.

One of the recent applications of UHPC for highway bridges is in the joints between girders and deck panels. Figure 1 shows a typical cross-section of an adjacent box and T-section girder (Rajlic et al., 2010; Perry and Mathew Royce, 2010). Both joints experience high stresses due to the environmental load, which is mainly caused by daily and seasonal fluctuations in temperature. The effect of these fluctuations in temperature becomes significant after the joint experiences a large number of temperature cycles (Perry and Weiss 2009). Due to the difference in thermal strain between two different materials, the interface between UHPC and HSC may experience high-level stresses leading to longitudinal cracking. These cracks allow water and chemical agents to penetrate through the joints; therefore, causing further damage to the overall bridge superstructure. Therefore, the coefficient of thermal expansion (CTE) of concrete plays an important factor in the bridge design, and should be considered when two or more materials are used in the bridge superstructure (AASHTO 2010).

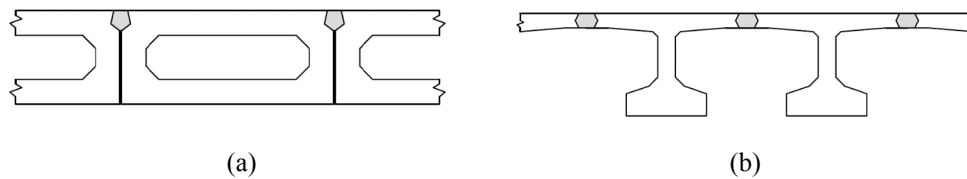


Figure 1. Typical cross-section of (a) adjacent box girder and (b) t-section girder.

1.2 AASHTO Specifications for Temperatures in Concrete Bridges

According to AASHTO (2010), provisions for loads due to temperature changes should be made by considering the fluctuations in effective bridge temperature. These fluctuations can cause longitudinal and transverse stresses in bridge superstructures. Such stresses may generate joint cracks that could affect the performance of the bridge and reduce the bridge service life. Therefore, the effect of longitudinal and transverse stresses should be included in the analysis and design of bridge components. In order to calculate the internal stresses in the longitudinal and transverse direction, both temperature gradient and uniform temperature should be considered. Temperature gradient is defined as the nonlinear differential temperature between the top and bottom surfaces of bridge superstructure (AASHTO, 2010). The temperature gradient value depends on the depth

and location of bridge. In the State of Ohio, the top and bottom surface temperatures for the temperature gradient are 41°F (22.8°C) and 0°F (0°C) for concrete girder bridges, respectively. These temperature values are not the absolute temperatures that a bridge girder may experience. The effect of temperature gradient on the shear key of precast pre-stressed concrete bridge is minimal due to the partial depth of the shear keys.

According to AASHTO (2010), a uniform temperature is generally assumed for the design of bridges, although these bridges do experience temperature changes through the depth. In section 3.12.2 in AASHTO (2010), procedure A and procedure B specify the minimum and maximum uniform temperatures as T min-design and T max-design, respectively. The T min-design and T max-design should be used to calculate thermal deformation by assuming a reference construction temperature. For procedure A, the temperature range for concrete is from 0 to 80°F (-17.8 to 26.7°C). For procedure B, the location of the bridge superstructure in the United States is considered. The minimum temperature across the states ranges from -55 to 50°F (-48.3 to 12.8°C), and the maximum temperature ranges from 75 to 125°F (23.9 to 51.7°C). It is important to consider these ranges in the measurement of the thermal expansion of the concrete so that the obtained thermal expansion value would be representative for all temperature fluctuations in the United States. Therefore, it is preferred to test the concrete for thermal expansion in range from -55 F to 125°F (-48.3 to 51.7°C) to cover all of the United States.

1.3 Coefficient of Thermal Expansion Test Methods

In past research, the CTEs are obtain using ASTM C531-00 (2000), CRD-C 39-81 (1981), AASHTO TP60-00 (2000), and Protocol-P63 by Simpson, Peter, and Gonzalo (2009). There are limit on the temperature range for testing methods such as ASTM C531-00, CRD-C 39-81, AASHTO TP60-00, and Protocol-P63. ASTM C531-00 testing method specifies the oven-dry condition to obtain the CTE. In this test, the sample length should be measured at 73° F (22.8°C) and 201°F (93.9°C). Then, the CTE is the length change divided by temperature change (73 to 201°F (22.8 to 93.9°C)). CRD-C 39-81 specifies a saturated condition by immersing a sample in water for 48 hours. Then, the sample length should be measured at 40°F (4.4°C) and 140°F (60°C). Therefore, ASTM C531-00 and CRD-C 39-81 do not measure the thermal strain continually to obtain the behavior of the CTE. AASHTO TP60-00 and Protocol-P63 testing methods also specify a saturated condition to obtain the CTE with the temperature ranging from 50 to 122°F (10 to 50°C). However, as discussed previously, the actual temperatures experienced by bridges in the United States range between -55 to 125°F (-48.3 to 51.7°C). Therefore, there is need to test the concrete for thermal expansion in this range to cover all of the United States.

1.4 Coefficient of Thermal Expansion for NSC

The effect of thermal expansion for concretes is a function of a number of variables. The most significant factors are aggregate type, aggregate proportion, and concrete degree of saturation. Due to the effect of concrete degree of saturation, the concrete sample should be tested as completely dry or saturated. Other factors have less effect on the thermal expansion such as mix proportions, age, and cement type (ACI 209R-92, 2008). According to ACI 209R-92 (2008), the thermal coefficient is affected by moisture content, and the thermal expansion values should be corrected for the expected degree of saturation. According to ACI 209R-92 (2008), the range of NSC thermal expansion is from 4.7×10^{-6} to $6.5 \times 10^{-6}/^{\circ}\text{F}$ (8.5×10^{-6} to $11.7 \times 10^{-6}/^{\circ}\text{C}$), depending on the thermal expansion of the aggregate. Also, the CTE is assumed to be constant within a temperature range

from 32 to 140°F (0 to 60°C). AASHTO (2010) assumes $6.0 \times 10^{-6}/^{\circ}\text{F}$ ($10.8 \times 10^{-6}/^{\circ}\text{C}$) as the thermal coefficient of NSC due to the lack of precise data.

1.5 Coefficient of Thermal Expansion for UHPC

Past researchers have used AASHTO test specification (AASHTO TP60-00) to measure the CTE of UHPC. Other researchers have used different testing method and/or different temperature ranges. Graybeal (2006) and Ahlborn, Erron, and Donald (2008) used the AASHTO TP60-00 testing method to measure the CTE of UHPC with some modifications to the testing protocol. These modifications were made in both the specimen preparation and testing procedures. For instance, the researchers completely coated the UHPC specimens with epoxy to limit saturation, except for four locations on each specimen. These locations corresponded with the linear variable differential transformer sensor (LVDT) location and the steel support buttons that came into contact with the specimen. However, this modification may have affected the concrete degree of saturation, and therefore the thermal expansion of concrete measured. The Japanese specification (JSCE, 2010), the French recommendations (AFGC, 2002), and Graybeal (2014) specify the CTE of UHPC as $7.5 \times 10^{-6}/^{\circ}\text{F}$ ($13.5 \times 10^{-6}/^{\circ}\text{C}$), $6.1 \times 10^{-6}/^{\circ}\text{F}$ ($11 \times 10^{-6}/^{\circ}\text{C}$), and $8.2 \times 10^{-6}/^{\circ}\text{F}$ ($14.75 \times 10^{-6}/^{\circ}\text{C}$), respectively. These CTE values are higher than NSCs of $4.1 \times 10^{-6}/^{\circ}\text{F}$ to $6.5 \times 10^{-6}/^{\circ}\text{F}$ ($7.4 \times 10^{-6}/^{\circ}\text{C}$ to $11.7 \times 10^{-6}/^{\circ}\text{C}$) (ACI 209R-92, 2008). To date, the CTE of UHPC has not been tested in the air-dry condition.

2 Objective

The aim of this study was to determine the CTE for UHPC and HSC to be used in the analysis and design of UHPC connection in bridges. A modified version of the Ohio-CTE testing method was used in order to measure the CTE in the air-dry condition. Furthermore, temperature range from -76 to 140°F (-60 to 60°C) was considered in order to account for the temperature variations found throughout the United States.

3 Experimental Program

In this research, the Ohio-CTE method has been used in an attempt to overcome some limitations with the ASTM C531, CRD 39, AASHTO TP60-00, and FHWA-Protocol-P63, in applying a temperature range from -76 to 140°F (-60 to 60°C) and using air-dry condition. It should be noted that the Ohio-CTE method is typically used to measure CTE of asphalt mixtures by Kim et al., 2015, and has been modified in this research to measure CTE of UHPC and HSC.

3.1 Apparatus

The Ohio CTE device consists of a square frame made from four 8.25 inches (209.5 mm) 6061 aluminum bars, each with a width of 2 inches (50.8 mm) and thickness of 0.5 inch (12.7 mm). The CTE test specimen, 1 inch (25.4 mm) thick with 6 inches (152.4) in diameter, is placed at the bottom corner of the frame, as shown in Figure 2. Two high resolution LVDTs are employed to measure the dimensional change of the test sample temperature change, as shown in Figure 2 (a and b). Temperature sensors (resistance temperature detector (RTD) sensor) are used to monitor the temperatures of the frame, LVDTs, and concrete disk, as shown in Figure 2 (a). The apparatus is placed in an air-dry environmental chamber, as shown in Figure 2(b). This chamber is capable of manipulating cooling and heating rates for a temperature range of -76 to 374°F (-60 to 190°C) with $\pm 0.9^{\circ}\text{F}$ ($\pm 0.5^{\circ}\text{C}$) stability. It should be noted that the frame and LVDTs contract or expand

together with the concrete disk during cooling or heating inside the environmental chamber. In order to determine the deformation of the concrete sample, the frame and LVDTs are calibrated using a square invar cross with Ohio CTE device. Ohio-CTE test is validated by testing two 6 inches (152.4 mm) diameter disk metals (6061 aluminum and stainless steel 316).

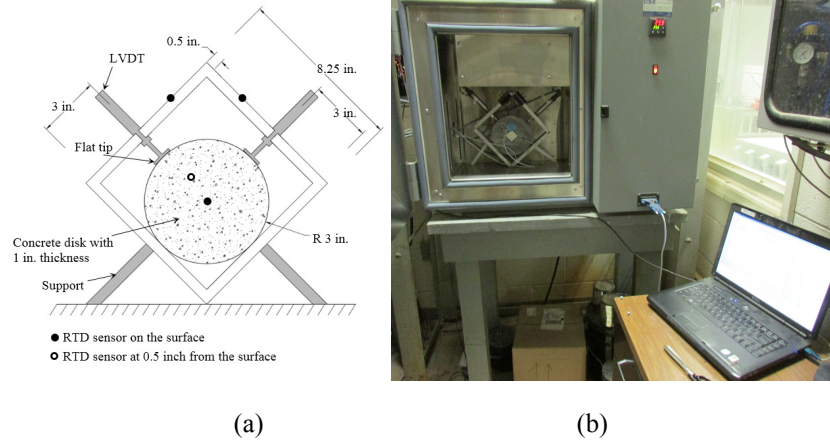


Figure 2. (a) Ohio CTE device (b) Ohio CTE device set-up with a test specimen

3.2 HSC and UHPC Materials and Sample Preparation

Both HSC and UHPC were tested using Ohio-CTE testing method. The concrete cylinders were cured in accordance with ASTM C31/C31M (2012), and the compressive strength of the concrete cylinders was determined according to the test method ASTM C39/C39M (2014). Table 1 shows the mix proportions for the HSC. The compressive strength of the HSC cylinders after 28 days was 11.5 ksi (79.3 MPa). Four concrete discs were cut to 1 inch (25.4 mm) thickness from the 6 x 12 inch (152.4 x 304.8 mm) concrete cylinder. The first disc at the edge was discarded due to low aggregate density. The proportions of the UHPC mix are shown in Table 1. The compressive strength of the UHPC cylinder after 28 days was 22.9 ksi (158.5 MPa). The UHPC was casted in a 6 x 120 inch (152.4 x 3048 mm) plastic pipe in order to obtain a fiber orientation consistent with the shear key of an adjacent box beam bridge. Four concrete disc were cut to 1 inch (25.4 mm) thickness from the end of UHPC cylinder. The first disc at the edge was discarded due to random fiber orientation.

Table 1. HSC and UHPC Mixed Designs for one cubic yard.

HSC		UHPC (Ductal JS1000)	
Constituents	Amount	Constituents	Amount
Coarse aggregate	1600 Ib	Premix	3699.8 Ib
Fine aggregate	1189 Ib	Water	219.1 Ib
Cement type III	775 Ib	Premi 150	50.6 Ib
Water	283 Ib	Steel fiber	262.9 Ib
Air entraining admixture	2.7 oz	-	-
High range water reducing admixture	99.2 oz	-	-

3.3 Test Procedure

After preparing the UHPC and HSC specimens, the specimens were air-dried by applying 120°F (48.9°C) for 24 hours to ensure that the moisture level in the specimens was low. Next, the Ohio-CTE device and the concrete disk were placed at the middle of the environmental chamber. At the beginning of the Ohio-CTE test, different cooling rates were tried to obtain the optimum cooling rate for the test setup. Figure 3 shows the temperature profile for the optimum cooling rate. The temperature for the first hour was held constant at 140°F (60°C) to ensure that the frame, LVDTs, and the specimen have the same temperature. Then, the specimen was cooled from 140°F to -76°F (60 to -60°C) at a 36°F/hr. (20°C/hr.) rate under the air-dry condition while collecting temperature and deformation data every 60 seconds. After completion of the test, the collected data was corrected using calibration factors to determine the thermal strain of the tested specimen.

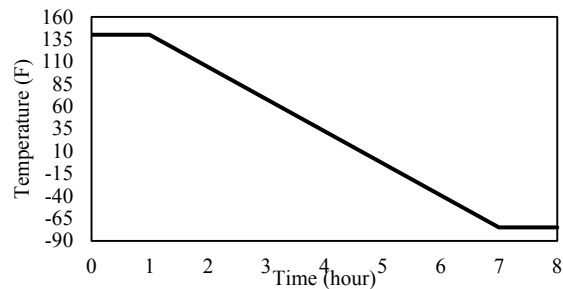


Figure 3. Temperature profile

4 Results and Discussions

4.1 UHPC and HSC CTEs

The thermal strain versus temperature curves for HSC and UHPC are shown in Figure 4(a) and Figure 4(b), respectively. The thermal strain-temperature behavior of HSC is approximately nonlinear in the temperature range from 35 to 70°F (1.7 to 21°C), and linear for temperature changes outside this range. For UHPC, the thermal strain-temperature behavior is approximately linear over the entire temperature range from -76 to 140°F (-60 to 60°C). Figure 5(a and b) represent the CTE versus temperature for the HSC and UHPC, respectively, where the CTE is the slope of the thermal strain versus temperature. The CTE curves were plotted using a moving average approach, which involves taking the slope of the thermal strain-temperature curves in 15°F (8.3°C) temperature increment (Figure 4). The CTE-temperature behavior of HSC is approximately linear in the temperature range from 35 to 70°F (1.7 to 21.1°C), and constant for temperature changes outside of this range. For UHPC, the CTE-temperature behavior is approximately constant in temperature range from -76 to 140°F (-60 to 60°C). The average CTEs were calculated to be $9.4 \times 10^{-6}/^{\circ}\text{F}$ ($16.9 \times 10^{-6}/^{\circ}\text{C}$) and $6.0 \times 10^{-6}/^{\circ}\text{F}$ ($10.8 \times 10^{-6}/^{\circ}\text{C}$) for UHPC and HSC, respectively. The higher CTE in UHPC can be attributed to high cement content and no coarse aggregate in the mix design. Instead, the UHPC mix consists of 29 percent cement and 40 percent fine sand. Therefore, the UHPC CTE value is governed by CTE value of the fine sand and cement. The lower HSC CTE value is due to the low thermal expansion of the coarse aggregate (ACI 209R-92, 2008).

The results of CTE tests are summarized in Table 2, and include the average CTE for the UHPC and HSC and the coefficient of variation (COV) for different categories of temperature

range. The averages in Table 2 indicate a decreasing HSC CTE value as the temperature drops under 40°F (4.4°C) while the average UHPC CTE remains almost constant over the temperature range from -76 to 140°F (-60 to 60°C). For overall CTE, the average UHPC CTE was $9.4 \times 10^{-6}/^{\circ}\text{F}$ ($16.9 \times 10^{-6}/^{\circ}\text{C}$) with a COV of 0.6 percent, and the average HSC CTE was $6.0 \times 10^{-6}/^{\circ}\text{F}$ ($10.8 \times 10^{-6}/^{\circ}\text{C}$) with a COV of 0.4 percent. The UHPC CTE value is 1.55 times higher than the HSC value for the temperature range from 40 to 105°F (4.4 to 40.6°C), and this difference increases to 1.68 times for the temperature range of -55 to 40°F (-48.3 to 4.4°C). The difference in CTE will generate high stresses in HSC bridges using UHPC joints due to the difference in thermal strain between the materials. The axial thermal strain due to a change in temperature is expressed by:

$$\epsilon_u = \alpha(T_o - T_i) \quad (1)$$

where α is the CTE, T_i is initial temperature or construction temperature, and T_o may be taken as the minimum effective temperature described in section 1.2. For a temperature increase, T_o may be taken as the maximum effective temperature described in section 1.2. Relative movement between the shear keys and girders induces either a tensile or compressive stress on the UHPC and HSC, depending on whether the temperature increases or decreases, respectively, from construction temperature. The difference in CTE between UHPC and HSC is larger at lower temperatures, and the UHPC-HSC composite section will experience high thermal stresses. The most likely place for a high stress to take place is at the interface between HSC and UHPC in the bridge superstructure. The difference in thermal expansion between UHPC and HSC is generally not considered in the joint design.

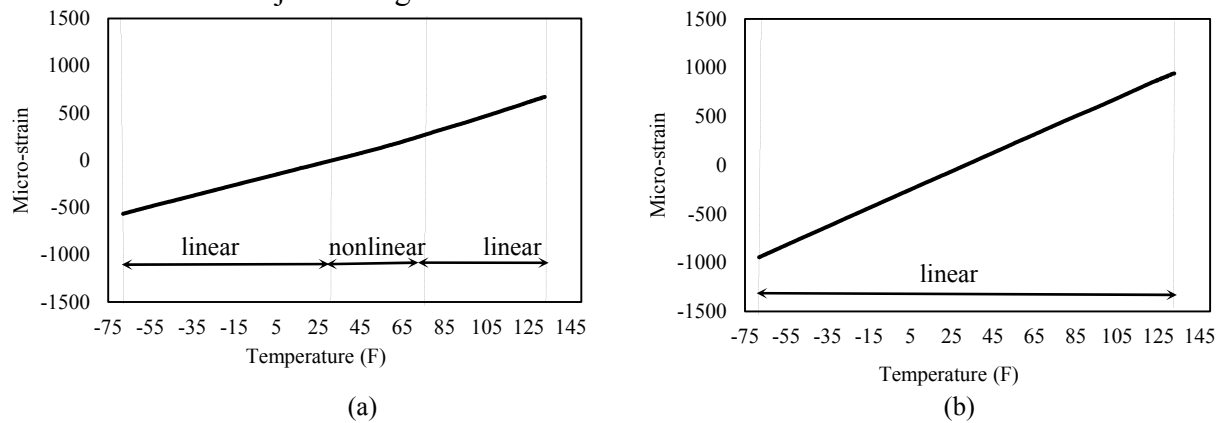


Figure 4. Thermal strain-temperature of (a) HSC (b) UHPC

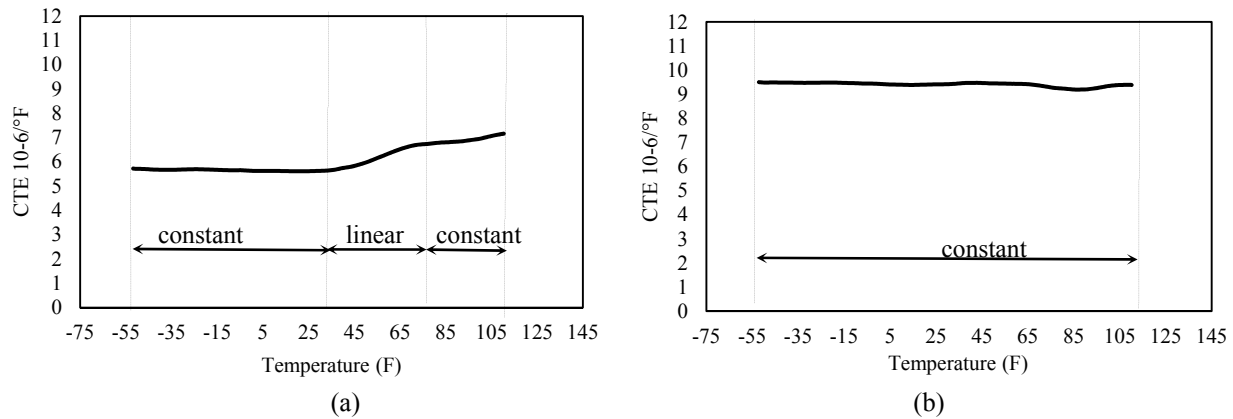


Figure 5. CTE-temperature of (a) HSC (b) UHPC

Table 2. Test Results for All Composite Specimens.

	Overall CTE 10 ⁻⁶	COV %	CTE 10 ⁻⁶ (40 to 105F)	COV %	CTE 10 ⁻⁶ (40 to -55F)	COV %
HSC-1	6.042		6.809		5.661	
HSC-2	6.060		6.738		5.684	
HSC-3	6.091		6.775		5.692	
	Ave=6.064	0.4	Ave=6.774	0.5	Ave=5.679	0.3
UHPC-1	9.514		9.329		9.619	
UHPC-2	9.425		9.442		9.517	
UHPC-3	9.404		9.321		9.449	
	Ave=9.447	0.6	Ave=9.364	0.7	Ave=9.528	0.9

4.2 Comparison with past research

To the best of the authors' knowledge, the laboratory testing performed in this study represents the first time the air-dry condition test has been applied to investigate the CTE of UHPC within a temperature range from -76 to 140°F (-60 to 60°C). Therefore, a comparison of past research on measuring the CTE of UHPC is warranted. A summary of the results from the present study and past research is presented in Table 3. Most researchers reported that the steam treated UHPC specimens have a higher CTE value than air cured specimens (untreated specimens). The results from past research using modified AASHTO TP60-00 of untreated specimens show that the average UHPC CTE range from $7.7 \times 10^{-6}/^{\circ}\text{F}$ to $8.2 \times 10^{-6}/^{\circ}\text{F}$ ($13.9 \times 10^{-6}/^{\circ}\text{C}$ to $14.8 \times 10^{-6}/^{\circ}\text{C}$). Furthermore, it was reported that the measured UHPC CTE was dependent of the saturated level of specimens during the testing process and temperature control (air-dry, oven-dry, and saturated conditions). The results highlight a limitation of AASHTO TP60-00 and other test methods for the measurement of UHPC CTE, where the UHPC CTE value is limited by water temperature range and the saturation level of specimens. The average UHPC CTE of $7.7 \times 10^{-6}/^{\circ}\text{F}$ ($13.9 \times 10^{-6}/^{\circ}\text{C}$) determined by Ahlborn, Erron, and Donald (2008) using a modified AASHTO TP60-00 test is not far removed from the $8.2 \times 10^{-6}/^{\circ}\text{F}$ CTE ($14.8 \times 10^{-6}/^{\circ}\text{C}$) value determined from the same method by Graybeal (2006). Also, the Japanese recommendations (JSCE, 2010) report that a UHPC CTE value of steam treated specimen is $7.5 \times 10^{-6}/^{\circ}\text{F}$ ($13.5 \times 10^{-6}/^{\circ}\text{C}$).

In the present study, the average value of the CTE for UHPC with a low moisture content in the specimens was found to be $9.4 \times 10^{-6}/^{\circ}\text{F}$ ($16.9 \times 10^{-6}/^{\circ}\text{C}$), and corresponded to the air-dry condition. This condition is consistent with the highest temperature that the concrete could experience in the field according to AASHTO LFRD (2012). Comparison of this value with those reported in the literature reveals that the UHPC CTE of untreated specimens measured in this study is larger than the average CTE for all of the UHPC specimens measured from previous studies regardless of the treatment method. These results highlight the benefit of using the Ohio-CTE testing method, where the air-dry condition was applied to obtain the actual value of UHPC CTE. This method allows the UHPC specimen to expand/shrink freely under the completely air-dry condition.

Table 3. Comparison of UHPC CTE with Past Research

Past research	CTE Micro-strain/°F	COV %	Method	Temperature range F
Graybeal, (2006)	8.2 ^b	2.5	AASHTOTP60-00 ^a	50 to 122°F
	8.7 ^c	1.7		
Fehling, Bunje, and Leutbecher (2004)	6.7	-	-	-
Simon (2009)	5.6 to 6.7	-	-	68 to 1472
Behloul et al. (2002)	6.7	-	-	68 to 1112
AFGC (2002)	6.1	-	-	-
Graybeal, (2014)	8.2	-	AASHTOTP60-00	50 to 122°F
Ahlborn, Erron, and Donald, (2008)	7.7 ^b	1.9	AASHTOTP60-00 ^a	50 to 122°F
	8.1 ^c	2.1		
JSCE, (2010)	7.5 ^c	-	-	-
Present study	9.4 ^b	0.6	Ohio-CTE method	-55 to 130

^a modified AASHTO TP60-00 testing method; ^b untreated specimen; ^c Steam treated specimen

5 Conclusions

An experimental approach was developed to determine the CTE of UHPC and HSC for a temperature range from -76 to 140°F (-60 to 60°C) under the air-dry condition. This temperature range covers temperature variations for different geographic locations throughout the United States. The air-dry condition was applied in the experiment to obtain the actual values of CTEs for UHPC and HSC. Based on the results of the testing performed in this study, the following conclusion are drawn:

- The CTE of UHPC can be considered constant for temperature range from -76 to 140°F (-60 to 60°C). For HSC, the CTE is approximately linear in the temperature range from 35 to 70°F (1.7 to 21.1°C), and constant for temperature changes outside of this range.
- The UHPC CTE value is 1.68 times higher than the HSC value for the temperature range from -55 to 40°F (-48.3 to 4.4°C), and this difference decreases to 1.55 times for the temperature range of 40 to 105°F (4.4 to 40.6°C). The difference in CTE between UHPC and HSC will generate high longitudinal stresses in HSC bridges with UHPC joints, and this effect will be more significant for temperature variations occurring at low temperatures.
- The average CTE for UHPC and HSC is found to be equal to $9.4 \times 10^{-6}/\text{°F}$ ($16.9 \times 10^{-6}/\text{°C}$) and $6.0 \times 10^{-6}/\text{°F}$ ($10.8 \times 10^{-6}/\text{°C}$), respectively. The CTE value for UHPC determined by the Ohio-CTE test is greater than the average CTE values previously reported for UHPC using AASHTO TP60-00.
- The higher CTE value of UHPC obtain in the present study can be attributed to the use of the Ohio-CTE test, where the air-dry condition is applied.

The CTE values of UHPC and HSC may now be used to calculate the thermal stresses between the materials in the analysis and design of UHPC-HSC bridge connections.

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