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Mitigation of early-age cracking in concrete structures

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Abstract. Early-age cracking can adversely affect the behavior and durability of concrete elements. This paper will cover means to mitigate early-age cracking in concrete bridge decks and mass concrete elements. The development of in-place stresses is affected by the shrinkage, coefficient of thermal expansion, setting characteristics, restraint conditions, stress relaxation, and temperature history of the hardening concrete. The tensile strength is impacted by the cementitious materials, the water-cementitious materials ratio, the aggregate type and gradation, the curing (internal/external) provided, and the temperature history of the hardening concrete. In this study, restraint to volume change testing with rigid cracking frames (RCF) was used to directly measure and quantify the combined effects of all variables that affect the development of in-place stresses and strength in a specific application. The laboratory testing performed involved curing the concrete in the RCF under sealed, match-cured temperature conditions to simulate concrete placement in concrete bridge decks and mass concrete. Experimental results reveal that the use of low heat of hydration concretes, concretes that use fly ash and slag cement, and lightweight aggregate concretes (because of reduced modulus of elasticity and coefficient of thermal expansion), are very effective to reduce the risk of early-age cracking in these elements.

1 Introduction

Early-age cracking can adversely affect the behavior and durability of concrete elements. Darwin and Browning [1] reported that "by controlling early age cracking, the amount of cracking at later ages should remain low," and that early-age cracking can significantly increase the rate and amount of chloride penetration (from deicing salts), which may accelerate the corrosion rate of embedded reinforcing steel.

It has been reported that concretes that contain lightweight aggregates (LWAs) can attain higher early-age temperatures when compared to concrete with normalweight aggregates due to the insulating effect of the lightweight aggregates [2]. In some cases, higher early-age temperatures cause greater early-age stresses and can cause cracking [3,4]. However, lightweight aggregates are also known to have lower coefficient of thermal expansion and

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modulus of elasticity when compared to normalweight aggregates [5]. Reducing the coefficient of thermal expansion will result in less strain from a temperature change, and reducing the modulus of elasticity will result in reduced stress when volume change effects are restrained.

1.1 Paper Objectives and Research Methodology

The primary objective of the study documented herein is to cover means to mitigate early-age cracking in concrete bridge decks and mass concrete elements. Restraint to volume change testing with rigid cracking frames (RCF) was used to assess the stress development due to thermal and autogenous shrinkage. Thirteen concretes were tested under temperature conditions that simulate placement in either bridge deck or mass concrete conditions.

2 Background

2.1 Concrete Volume Change Effects

2.1.1 Thermal Effects

The development of thermal stresses (σ_T) can be calculated using the expression presented in Eq. 1. For an accurate estimate of thermal stress, creep effects during early ages and over the structure's life should be accounted for in Equation 1 [6].

$$Thermal\ Stress = \sigma_T = \Delta T \cdot \alpha_t \cdot E_c \cdot K_r \tag{1}$$

where,
ΔT = Temperature Change = $T_{zero\text{-}stress}$ - T_{min} (°C),
α_t = Coefficient of Thermal Expansion (strain/°C),
E_c = Creep-adjusted Modulus of Elasticity (MPa),
K_r = Degree of restraint factor,
$T_{zero\text{-}stress}$ = Concrete zero-stress temperature (°C), and
T_{min} = Minimum concrete temperature (°C).

An illustration of the development of concrete temperatures and thermal stresses over time under summer placement conditions for freshly placed concrete is presented in Fig. 1. In terms of stress development, the final-set temperature is the temperature at which the concrete begins to resist stresses that result from the restraint of external volume changes. In Fig. 1, it can be seen that due to hydration the concrete temperature increases beyond the setting temperature, line (A). Because the expansion of the concrete caused by the temperature rise is restrained, the concrete will be in compression when the peak temperature, line (B), is reached. When the peak temperature is reached, the hydrating paste is still developing structure, its strength is low, and high amounts of early-age relaxation may occur when the concrete is subjected to high compressive loads [7]. As the concrete temperature subsequently decreases, the compressive stress is gradually relieved until the stress condition changes from compression to tension, line (C). The temperature at which this transient stress-free condition occurs is denoted the "zero-stress temperature". Due to the effects of relaxation, the zero-stress temperature may be significantly higher than the final-set temperature [7]. If tensile stresses caused by a further temperature decrease exceed the tensile strength of the concrete, cracking will occur, line (D).

2.1.2 Chemical and Autogenous Shrinkage

The reaction products formed from cement hydration are smaller than the initial components. The reduction of the absolute volume of the reactants due to hydration is chemical shrinkage. Before setting this phenomenon results in a volumetric change but generates no stress due to the viscoelastic nature of fresh concrete. After setting, chemical shrinkage leads to the creation of internal water-filled voids. As water is consumed by the ongoing hydration process, the voids empty and capillary stresses are generated, which results in a volumetric shrinkage. Autogenous shrinkage is the concrete volume change that occurs without moisture transfer to the environment [8]. Chemical shrinkage and autogenous shrinkage are equal before setting [8]. The addition of saturated LWA helps mitigate stress due to autogenous shrinkage because water is desorbed from the aggregate particles into the hydrated cement paste pore structure, which reduces the capillary tension [9].

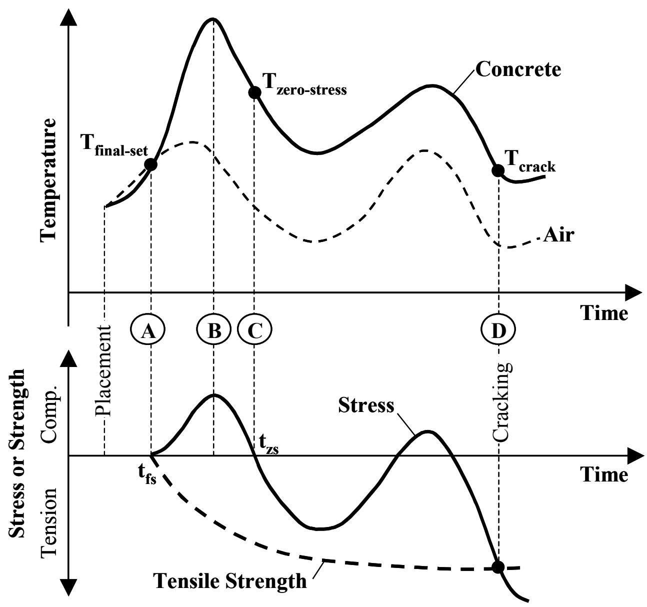

Fig. 1. Development of early-age thermal stresses [6].

2.2 Lightweight Aggregate Concrete

The α_t of concrete is primarily affected by the α_t of the aggregate, because the aggregate makes up the bulk of the concrete [10]. LWA are reported to have a lower α_t compared to siliceous gravel; therefore, concrete made with LWA has a lower α_t than its normalweight counterpart [11]. Like α_t, the modulus of elasticity of the concrete depends heavily on the stiffness of the aggregate. Eq. 2 [12] can be used to estimate the modulus of elasticity from a known unit weight and compressive strength. This expression indicates that the modulus of elasticity is directly proportional to the unit weight to the 1.5 power and the square root of the compressive strength. Consequently, lightweight aggregate concrete has a lower modulus of elasticity compared to normalweight concrete [10]. Due to its high porosity, LWA is known to have a lower thermal conductivity (or greater insulating ability) compared to normalweight concrete [5,10,11]. It has been observed that LWA concrete has a greater temperature rise due to hydration compared to normalweight concrete with identical cementitious materials, water, and fine aggregate contents [2].

$$E_c = 0.043 w_c^{1.5} \sqrt{f_c} \quad (2)$$

where, E_c = Modulus of elasticity (MPa),
w_c = Unit weight of normal concrete or equilibrium density of lightweight concrete (kg/m^3), and
f_c = concrete compressive strength (MPa).

3 Experimental work

3.1 Measurement of early-age stress development with rigid cracking frames

The rigid cracking frame (RCF), shown in Fig. 2, is made up of two mild-steel crossheads and two 100-mm diameter Invar side bars. The test setup was adapted from the configuration developed by Dr. Rupert Springenschmid [13]. The formwork shown includes 12.7-mm diameter copper tubing throughout. A mixture of water and ethylene glycol is circulated from a temperature-controlled water bath through the formwork to control the curing temperature of the concrete sample. The formwork of the RCF is lined with plastic to reduce friction between the concrete and the form and to seal the concrete specimen on all surfaces. Because of the presence of the sealed plastic layer around the concrete specimen, no moisture is lost and drying shrinkage effects do not develop while the forms are in place on the RCF.

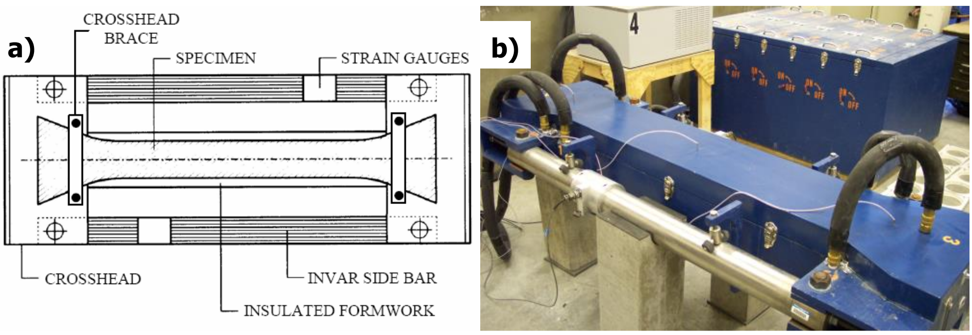

Fig. 2. Rigid cracking frame test: a) schematic of test [4] and b) actual equipment used.

When concrete in the RCF starts to hydrate and volume changes due to temperature and autogenous shrinkage effects develop, the Invar bars provide restraint against movement, and stress develops in the concrete. Concrete stress development is monitored using strain gauges mounted on the Invar bars, which are calibrated to the bar forces, which equilibrate the concrete stresses. The concrete stresses generated are a function of the relaxation, coefficient of thermal expansion, modulus of elasticity, temperature history, and maturity of the concrete. The RCF thus captures the combined effect of all these variables under programmable concrete temperature conditions.

3.2 Measurement of the development of concrete properties

The development of mechanical properties of each concrete was assessed by using, 150×300 mm cylindrical specimens cast and match-cured to the RCF temperature profile for the first 96 hours or until the onset of cracking of the RCF specimen. Thereafter, they were cured in standard moist-curing conditions per ASTM C192. The compressive strength (ASTM C39), splitting tensile strength (ASTM C496), and modulus of elasticity

(ASTM C469) were measured at 0.5, 1, 2, 3, 7, and 28 days. The coefficient of thermal expansion was also measured in accordance with a modified AASHTO T336 procedure [14].

3.3 Concrete temperature modeling for match curing in rigid cracking frame

The temperature modeling for all concretes was performed with the ConcreteWorks [15] software. Initial semi-adiabatic calorimetry was performed on all concretes and the hydration parameters [16] determined for each mixture. The inputs used for modeling the temperature profile in ConcreteWorks were the hydration parameters along with the mixture proportions and other factors including the placement date, placement time, type of formwork, etc. The elements to which the concrete in the rigid cracking frame was match cured in this study include either a bridge deck or mass concrete element. By using this approach, the unique heat of hydration of each concrete was accounted for and curing temperatures representative of in-place elements were achieved. In order to ensure that most RCF tests are completed within a reasonable period, a duration of seven days was selected for temperature modeling and early-age concrete testing. If cracking did not occur after 96 hours, the concrete was artificially cooled at a rate of 1.0˚C/hr or 0.5˚C/hr until the onset of cracking for the bridge deck and mass concrete simulation, respectively.

3.4 Concretes evaluated in three phases

Concretes were tested in three different phases. In Phase I, concretes typically used for bridge deck construction were evaluated [17]. All concretes in this phase were match-cured under temperature conditions simulating the center of a 200-mm thick bridge deck. A reference concrete (REF 0.44) using only portland cement and three concretes using various SCMs were tested. The mixture proportions for this phase are shown in Table 1. Since cracking in bridge decks is more severe when placed under summer conditions [18], the influence of using supplementary cementing materials (SCMs) on early cracking tendency was only evaluated under summer placement conditions. The SCM mixtures had 20, 30, and 50 percent replacement of cement by mass of Class F fly ash, Class C fly ash, and slag cement, respectively. A mixture with only portland cement and a water-cement ratio (*w/c*) of 0.36 was also tested at two placement temperatures to evaluate the effect of *w/c* on the cracking tendency. The reference concrete was tested at three different temperatures (10, 23, and 35 °C) to evaluate the effect of placement and curing temperature on early-age cracking.

Table 1. Bridge deck concrete proportions for Phase I testing [17].

Item	REF 0.44	30C Ash	20F Ash	50 Slag	REF 0.36
Water Content, kg/m^3	162	162	162	162	148
Cement Content, kg/m^3	368	258	294	184	413
Class F Fly Ash Content, kg/m^3	0	0	74	0	0
Class C Fly Ash Content, kg/m^3	0	110	0	0	0
Slag Cement, kg/m^3	0	0	0	184	0
SSD Normalweight Coarse Aggregate, kg/m^3	117	1100	1100	1100	1157
SSD Normalweight Fine Aggregate, kg/m^3	682	720	717	726	680
Water-Reducing Admixture, ml/m^3	841	841	841	841	0
High-Range Water-Reducing Admixture, ml/m^3	0	0			787
Target Total Air Content, percent	2	2	2	2	2
Water-to-Cementitious Materials Ratio (*w/cm*)	0.44	0.44	0.44	0.44	0.36

Note: SSD refers to saturated-surface dry condition.

In Phase II, the effect of using lightweight aggregate (LWA) on the early-age cracking tendency of bridge deck concrete was evaluated [19]. Normalweight concrete, internally cured concrete (ICC), sand-lightweight concrete (SLWC), and all-lightweight concrete (ALWC) were evaluated. The mixture proportions evaluated during this phase are shown in Table 2. The reference concrete (REF 0.42) for this phase was made with a siliceous sand and river gravel. The LWA used herein was an expanded shale, which had absorption capacities of 32.0% and 19.3% for the coarse and fine aggregate, respectively. The SLWC was proportioned using the LWA as the coarse aggregate and normalweight siliceous sand. The ICC is similar to the normalweight concrete, except a fraction of the normalweight sand was replaced with lightweight fine aggregate. The ICC was proportioned by adding as much lightweight fine aggregate as possible to obtain a calculated equilibrium density of no less than 2162 kg/m^3 (135 lb/ft^3). This was done to ensure that this ICC could be classified as "normalweight" concrete in accordance with ACI 318 [12].

Table 2. Bridge deck concrete proportions for Phase II testing [19].

Item	**REF 0.42**	**ICC 0.42**	**SLWC 0.42**	**ALWC 0.42**
Water Content, kg/m^3	154	154	164	164
Cement Content, kg/m^3	368	368	390	390
SSD Normalweight Coarse Aggregate, kg/m^3	1044	1044	0	0
SD Shale Lightweight Coarse Aggregate, kg/m^3	0	0	610	562
SSD Normalweight Fine Aggregate, kg/m^3	718	521	780	0
SD Lightweight Fine Agg. (0 to 9.5mm), kg/m^3	0	136	0	0
SD Lightweight Fine Aggregate (0 to #4), kg/m^3	0	0	0	592
Water-Reducing Admixture, ml/m^3	1200	1200	0	0
High-Range Water-Reducing Admixture, ml/m^3	0	0	2036	1335
Rheology-Controlling Admixture, ml/m^3	0	0	0	1018
Air-Entraining Admixture, ml/m^3	31	31	762	97
Target Total Air Content, percent	5.5	5.5	5.5	5.5
Water-to-Cement Ratio (*w/c*)	0.42	0.42	0.42	0.42

Note: SD refers to pre-wetted surface dry condition, and SSD refers to saturated-surface dry condition.

In Phase III, the effect of using LWA on the early-age cracking tendency of mass concrete was evaluated [20]. For the concretes tested in Phase III, the rigid cracking frame specimens were match-cured to temperature conditions simulating the edge of a 2.4×2.4 m mass concrete column. The LWA used in this phase was also an expanded shale, which had absorption capacities of 18.0% and 20% for the coarse and fine aggregate, respectively. Note that the two expanded shales tested in Phase II and III are from different US regions. The reference concrete (REF 0.45) for this phase is representative of mass concrete mixtures commonly used in the Southeastern United States and thus contained 30 percent Class F fly ash. The following concrete types were also made at *w/cm* of 0.45: ICC, SLWC, and ALWC. The mixture proportions for this phase are shown in Table 3. The cementitious material content and paste volume were held constant for all these concretes.

4 Results and discussion

4.1 Influence of water-to-cement ratio (Phase I)

The curing temperature and RCF results for the reference concrete (REF-0.44) and the concrete with *w/c* of 0.36 (REF-0.36) tested in Phase I under summer and fall conditions are presented in Fig. 3. As shown in Table 1, the *w/c* was decreased by increasing the cement content and decreasing the water content to maintain a constant paste volume. The increase in cement and increased placement temperature cause an increase in peak temperatures as shown in Fig. 3a. The increased peak temperature decreased the time to cracking. These results confirm that using lower cement content concretes will improve the resistance to early-age cracking of concretes used in bridge deck applications.

Table 3. Mass concrete proportions for Phase III testing [20].

Item	REF 0.45	ICC 0.45	SLWC 0.45	ALWC 0.45
Water Content, kg/m^3	156	156	156	156
Cement Content, kg/m^3	243	243	243	243
Class F Fly Ash Content, kg/m^3	104	104	104	104
SSD Normalweight Coarse Aggregate, kg/m^3	1032	1032	0	0
SD Lightweight Coarse Aggregate, kg/m^3	0	0	540	508
SSD Normalweight Fine Aggregate, kg/m^3	724	593	706	0
SD Lightweight Fine Aggregate, kg/m^3	0	83	0	486
Water-Reducing Admixture, ml/m^3	619	542	0	0
High-Range Water-Reducing Admixture, ml/m^3	0	0	851	696
Rheology-Controlling Admixture, ml/m^3	0	0	0	1238
Air-Entraining Admixture, ml/m^3	39	39	135	135
Target Total Air Content, percent	5.0	5.0	5.0	5.0
Water-to-Cementitious Materials Ratio (*w/cm*)	0.45	0.45	0.45	0.45

Note: SD refers to pre-wetted surface dry condition, and SSD refers to saturated-surface dry condition.

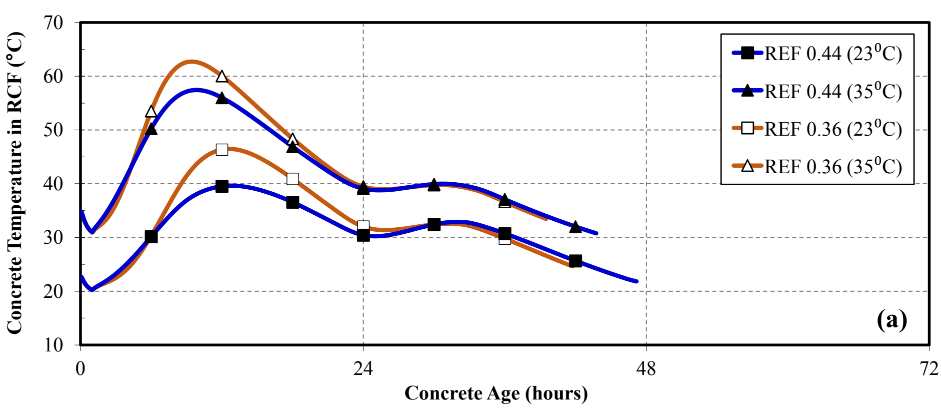

Fig. 3a. Measured Phase I results for Control and 0.36 concretes placed under summer and fall conditions - temperature profiles [17].

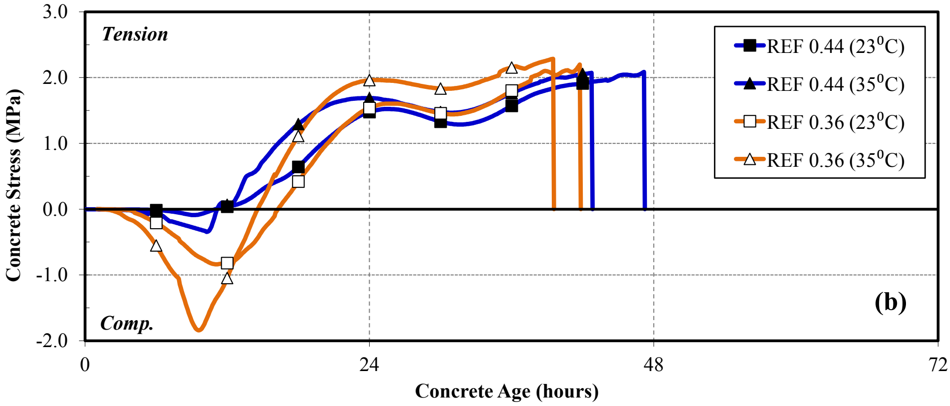

Fig. 3b. Measured Phase I results for Control and 0.36 concretes placed under summer and fall conditions - stresses from RCF [17].

4.2 Influence of fly ash and slag cement (Phase I)

The curing temperature and RCF results for the reference concrete (REF-0.44) and the SCM concretes for the summer placement condition tested in Phase I are presented in Fig. 4. Increasing the replacement of cement with an SCM decreased the rate of hydration and maximum temperature reached, as shown in Fig. 4a. The reduced rate of temperature development and stiffness development that accompanied the use of SCMs increased the time to zero stress, decreased the zero-stress temperature, and increased the time to cracking [17].

Springenschmid and Breitenbücher [21] found fly ash to reduce the cracking temperature. In the present study, it was found that the use of fly ash or slag cement caused a reduction in the cracking temperature (i.e. later time to cracking). In their study, Breitenbücher and Mangold [4] concluded that slag cement reduces temperature rises and tensile stresses. These findings are also valid for the slag cement concrete evaluated in this study. From the results of this study, it may be concluded that the use of SCMs under hot weather conditions can substantially reduce the development of tensile stresses due to thermal shrinkage effects.

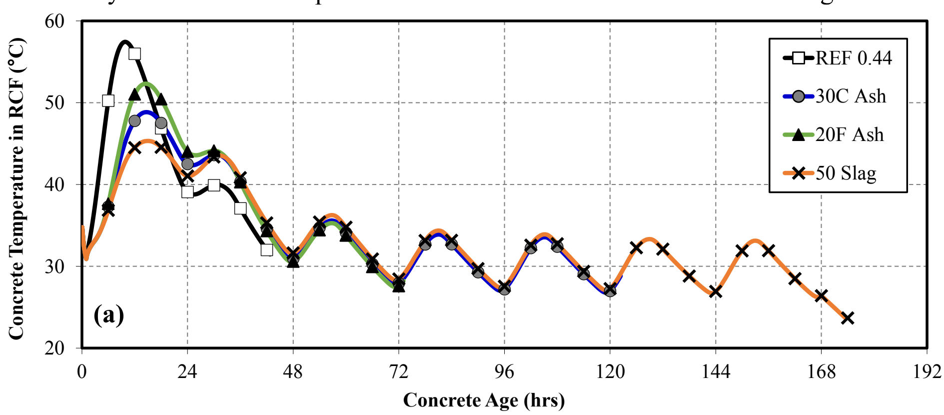

Fig. 4a. Measured Phase I results for the control and SCM concretes placed under summer conditions - temperature profiles [17].

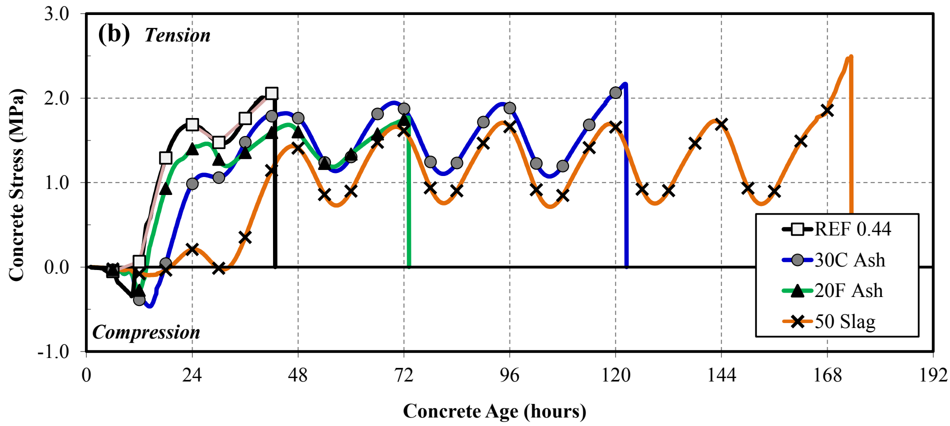

Fig. 4b. Measured Phase I results for the control and SCM concretes placed under summer conditions - stresses from RCF [17].

4.3 Influence of temperature (Phase I)

The curing temperature and RCF results for the reference concrete (REF-0.44) of Phase I placed at three temperature conditions are presented in Fig. 5. In all cases, the temperatures and stresses are shown until the time of cracking. Decreasing the placement and curing temperatures delay and decrease the temperature peak shown in Fig. 5a, which decreased the zero stress temperature and increased the zero stress time, which reduced stresses and delayed cracking as shown in Fig. 5b. Breitenbücher and Mangold [4] also found that decreasing the temperature of the fresh concrete significantly increased the time to cracking. These results confirm that the thermal stresses that develop during summer placements are much higher than those that develop during winter placements.

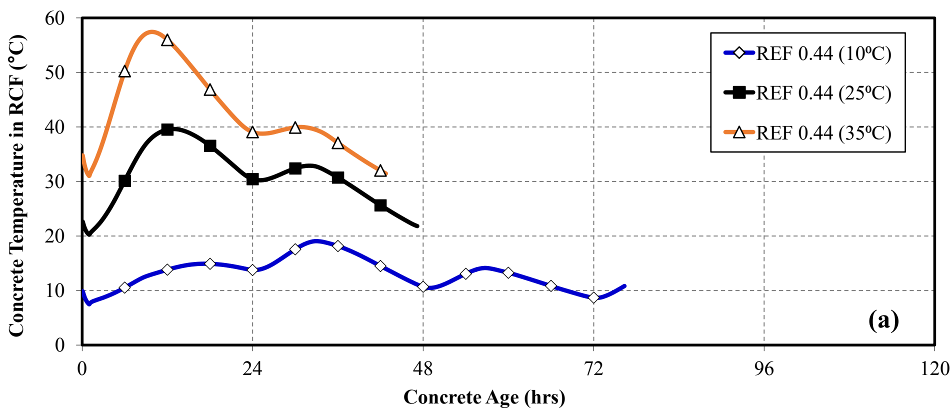

Fig. 5a. Measured Phase I results for the control concrete placed under summer, fall, and winter placement conditions - temperature profiles [17].

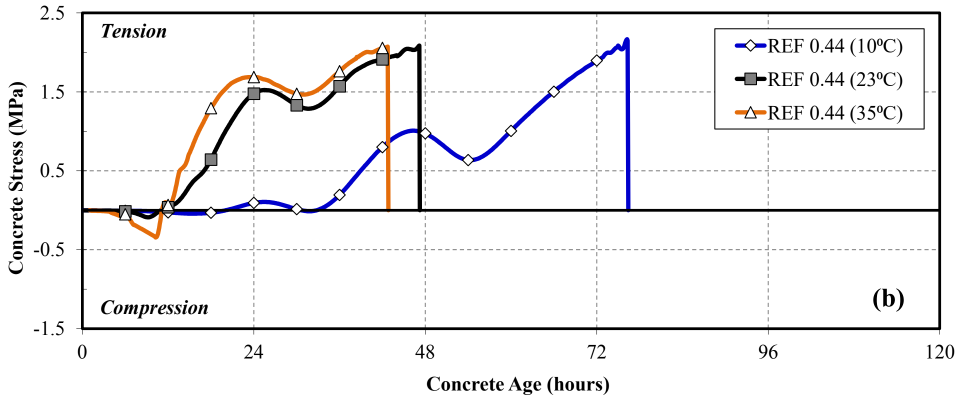

Fig. 5b. Measured Phase I results for the control concrete placed under summer, fall, and winter placement conditions - stresses from RCF [17].

4.4 Influence of using lightweight aggregate in concrete (Phase II and III)

The curing temperature and RCF results for each concrete tested in bridge deck applications during Phase II are shown in Fig. 6. It can be seen in Fig. 6a that the peak temperature is the greatest for the ALWC, followed by the SLWC, and then the ICC and reference concrete. This is due to the decreased thermal conductivity of the lightweight concrete [19,20]. After comparing the results shown in Fig. 6a and 6b, it can be concluded that temperature history is not the only factor affecting the cracking tendency of these concretes. While peak temperature is important, the systematic decrease in both coefficient of thermal expansion and modulus of elasticity of the concrete containing an increased proportion of LWA reduce the early-age stress development in accordance with Eq. 1 [19]. Although the ALWC and SLWC experience greater peak temperatures, the decreased coefficient of thermal expansion and modulus of elasticity lead to an overall increased time to cracking as compared to the reference concrete. The use of lightweight aggregates is thus very effective in delaying the occurrence of cracking at early ages in bridge deck concrete applications.

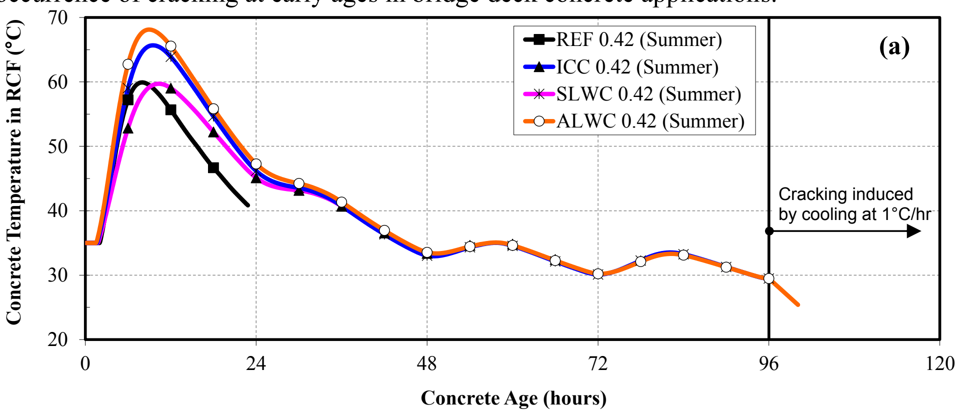

Fig. 6a. Measured Phase II results for control and expanded shale concretes - summer placement temperature profiles [19].

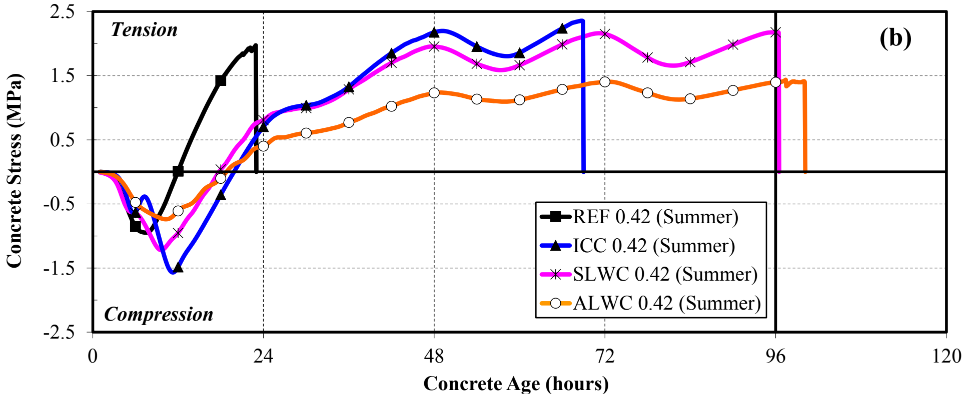

Fig. 6b. Measured Phase II results for control and expanded shale concretes - stresses from RCF [19].

The curing temperature and RCF results for each concrete tested in mass concrete applications during Phase III are shown in Fig. 7. As was the case in Phase II, it can be seen in Fig. 7a that the peak temperature is the greatest for the ALWC, followed by the SLWC, and then the ICC and reference concrete. Care should be taken when using LWA concrete in mass concrete to make sure that the threshold for Delayed Ettringite Formation (DEF) to occur is not exceeded. It can be observed from Fig. 7b that despite an increase in the maximum concrete temperatures for all LWA concretes in comparison to the reference concrete, all concretes made with LWA exhibit an improved resistance to early-age cracking. Increasing the amount of pre-wetted LWA in concrete systematically decreases the coefficient of thermal expansion and modulus of elasticity of the concrete [19,20]. As shown in Figure 7(b), sand-lightweight concrete provided the best overall resistance to early-age cracking. The all-lightweight concrete did not perform as well as the sand-lightweight concrete, and this is attributed to its reduced splitting tensile strength when compared to the sand-lightweight concrete [20]. The presence of LWA in concrete delays the time to cracking with SLWC having the latest time to cracking followed in order of decreasing time to cracking by ALW and IC concretes.

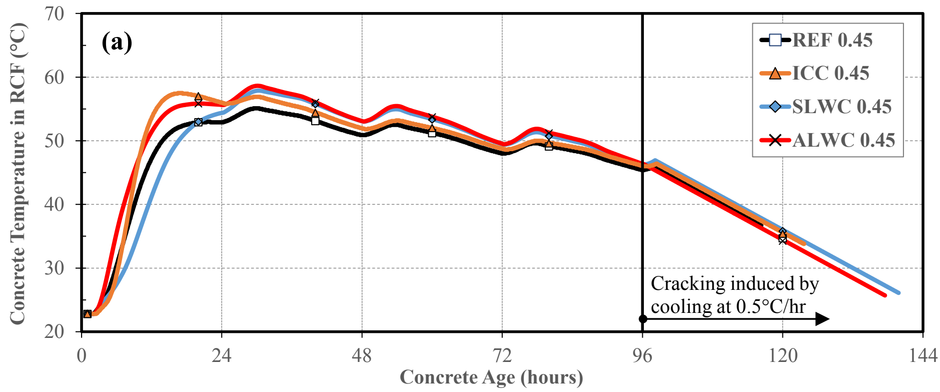

Fig. 7a. Measured Phase III results for control and expanded shale concretes - curing temperature profile for 0.45 w/cm concretes [20].

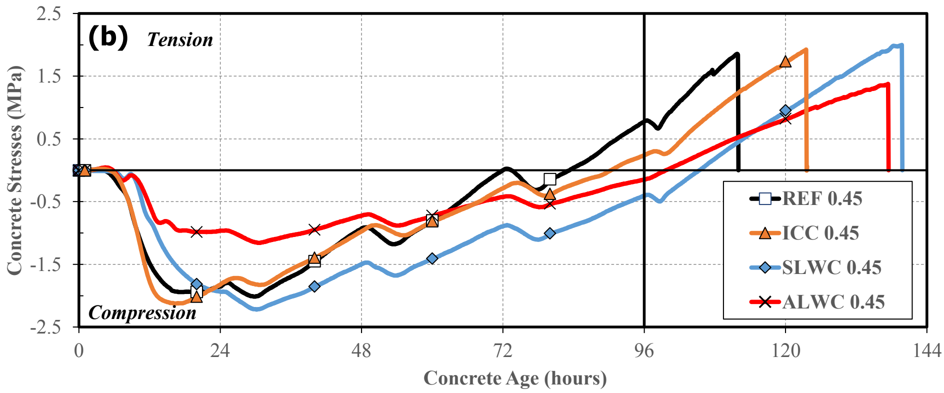

Fig. 7b. Measured Phase III results for control and expanded shale concretes - restrained stress profiles for 0.45 w/cm concretes [20].

5. Conclusions

The results of this work reported in this paper support the following conclusions:

- Decreasing the *w/c* while maintaining a constant paste content leads to an increase in cement content, an increase in peak temperature, and a decrease resistance to early-age cracking.
- The use of SCMs can be very effective in delaying cracking under summer placement conditions. Increasing the replacement of cement with an SCM decreases the rate of hydration and maximum temperature. The concretes made with 30% Class C Fly ash and the 50% slag cement replacements were most effective in delaying cracking under summer placement conditions when thermal stresses are most severe.
- Higher placement and curing temperatures result in higher thermal stresses. Decreasing the placement and curing temperature can reduce stresses and delay cracking.
- The maximum in-place concrete temperatures increased as more LWA were used in the mixture. The all-lightweight concrete reached the highest maximum in-place concrete temperature followed by the sand-lightweight concrete and then the internally cured concrete and normalweight concrete. Care should be taken when using LWA in mass concrete to make sure that the threshold for DEF to occur is not exceeded.
- Although an increasing amount of LWA in concrete will increase the maximum concrete temperature, the increasing use of LWA will reduce the modulus of elasticity, reduce the coefficient of thermal expansion, and eliminate autogenous shrinkage effects, which all result in an overall improvement in resistance to early-age cracking.

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