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Prediction of Concrete Temperature and Its Effects on Continuously Reinforcement Concrete Pavement Behavior at Early Ages

초기재령에서 연속철근콘크리트포장 거동에 콘크리트 온도의 영향과 예측

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요 지

연속철근콘크리트포장에서 횡방향 균열은 초기의 온도와 습도변화로 인해 발생하며 횡방향 균열의 폭과 간격은 하중전달효율 및 펀치아웃과 같은 포장공용성에 직접 관련된다. 또한, 연속철근콘크리트포장에서 균열폭은 콘크리트의 건조수축 뿐만아니라 소위 특성 온도변화 시점으로 정의되는 "제로-스트레스 온도"에 의존한다. 연속철근콘크리트포장의 양호한 장기공용성을 위해서 횡방향 균열은 매우 작은 폭으로 유지될 필요가 있으며 초기재령에서 온도제어는 이를 위한 필수요소라 할 수 있다. 따라서, 본 연구에서는 페이브프로를 사용하여 콘크리트 온도를 예측하였으며 현장실험을 통한 실측온도와 비교하고 균열조사를 수행하였다. 본 연구로부터 도출된 결과는 첫째, 실측온도는 0.2~4.5℃의 범위에서 예측온도보다 큰 것으로 나타났으며 정확한 콘크리트 온도예측을 위해서는 수화특성치로 고려되는 활성화 에너지와 단열온도상수가 입력변수에서 정확히 고려되어야 할 것으로 판단되었다. 둘째, 콘크리트 타설 후 24시간 이내의 온도는 오후에 비하여 오전에 타설된 콘크리트에서 더 크게 나타났으며 최대온도는 정오에 타설된 온도에서 발생되었다. 마지막으로 재령 12일에 조사된 균열발생률은 오전에 타설된 구간에서 25% 증가되고 그 만큼 균열간격은 감소하는 것으로 나타났다. 이 결과로부터 최대 콘크리트 온도는 균열발생에 크게 영향을 미치며 콘크리트 온도제어는 연속철근콘크리트포장의 양호한 공용성을 위해 필요할 것으로 판단되었다.

핵심용어: 연속철근 콘크리트포장, 횡방향 균열, 제로-스트레스 온도, 페이브프로

Abstract

Transverse cracks in continuously reinforced concrete pavement (CRCP) occur at early ages due to temperature and moisture variations. The width and spacing of transverse cracks have a significant effect on pavement performance such as load transfer efficiency and punchout development. Also, crack widths in CRCP depend on "zero-stress temperature," which is defined as a temperature where initial concrete stresses become zero, as well as drying shrinkage of concrete. For good long-term performance of CRCP, transverse cracks need to be kept tight. To keep the crack widths tight throughout the pavement life, zero-stress temperature must be as low as practically possible. Thus, temperature control at early ages is a key component in ensuring good CRCP performance. In this study, concrete temperatures were predicted using PavePro, a concrete temperature prediction program, for a CRCP construction project, and those values were compared with actual measured temperatures obtained from field testing. The cracks were also surveyed for 12 days after concrete placement. Findings from this study can be summarized as follows. First, the actual maximum temperatures are greater than the predicted maximum temperature in the ranges of 0.2 to 4.5 ℃. For accurate temperature predictions, hydration properties of cementitious materials such as activation energy and adiabatic constants, should be evaluated and accurate values be obtained for use as input values. Second, within 24 hours of concrete placement, temperatures of concrete placed in the morning are higher than those placed in the afternoon, and the maximum concrete temperature occurred in the concrete placed at noon. Finally, from the 12 days of condition survey, it was noted that the rate of crack occurrence in the morning placed section was 25 percent greater than that in the afternoon placed section. Based on these findings, it is concluded that maximum concrete temperature has a significant effect on crack development, and better concrete temperature control is needed to ensure adequate CRCP performance.

Keywords: CRCP, transverse crack, zero-stress temperature, PavePro

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1. INTRODUCTION

Most of the transverse cracks in continuously reinforced concrete pavement (CRCP) occur at early ages due to the temperature and moisture variations. Transverse cracks in CRCP relieve stresses in both concrete and steel and do not necessarily indicate any defects in pavement structure. However, for CRCP to be able to provide good long-term performance, transverse cracks need to be kept tight throughout the pavement life. It is postulated that the crack widths in CRCP depend on zero-stress temperature and subsequent temperature variations as well as drying shrinkage of concrete. Accordingly, it is hypothesized that the zero-stress temperature has significant effect on CRCP performance, and temperature control at early ages is a key component in ensuring good CRCP performance. For example, high zero-stress temperature will result in larger crack widths, which will reduce load transfer efficiency (LTE) at transverse cracks. Transverse cracks with low LTE will increase wheel load stress and the probability of further cracking and punchouts. Substantial efforts have been made at the Center for Transportation Research (CTR) of the University of Texas at Austin to evaluate the effects of early-age concrete temperatures on CRCP performance. As part of the effort, a concrete temperature prediction program, called PavePro, was developed at CTR. PavePro utilizes predicted ambient temperature and other environmental condition, hydration characteristics of cementitious materials, and geometric layout of concrete pavement structure to compute heat of hydration as well as heat exchange with surrounding environment to predict temperatures in various locations in the concrete slab. In this study, concrete temperatures were predicted using PavePro for a CRCP construction project, and those values were compared with actual measured temperatures. Also studied is the development of transverse cracking.

2. THEORETICAL BACKGROUND

The thermal stress σ_T , depends on the magnitude of ΔT, which is the temperature change. For an accurate estimate of the thermal stresses, stress relaxation due to creep effects during early-ages and over-the-pavement life should be accounted for as shown in Equation 1.

$$\sigma_T = \Delta T \cdot \alpha_C \cdot E_C \cdot K_r \qquad \text{Eq. 1}$$

In this equation, ΔT = concrete temperature change (℃),
α_C = concrete coefficient of thermal expansion (strain/℃),
E_C = creep adjusted modulus of elasticity (Pa), and
K_r = degree of restraint factor.

Figure 1 represents the development of concrete temperatures at early-age and thermal stresses over time in a fully restrained specimen. The displayed temperature development is typical for concrete placed under hot weather field conditions. The concrete is plastic at placement and stresses do not start to develop until enough hydration products have formed to cause final setting, which occurs at time t_{fs}. The hydration of cement with water is exothermic in nature, and this causes the concrete temperature to increase beyond the setting temperature. The restrained expansion of the concrete caused by the increase of temperature leads to the development of compressive stresses until the peak temperature, T_{max} is reached at time t_a. During this phase, the hydrating paste is still developing the structure, the strength is low, and most of the early-age compressive stresses are relaxed. As concrete temperature starts to fall, compressive stresses are relieved until the concrete temperature drops below the zero-stress temperature, T_{ZS} · This condition is reached at time t_{ZS}, where the stress condition changes from compression to tension for the first time. The zero-stress temperature is generally significantly higher than the final-set temperature, due to the relaxation of early-age compressive stresses and the rapid gain in concrete stiffness during early-ages. If the temperature continues to decrease, additional tensile stresses will develop. When these tensile stresses exceed the tensile strength, cracking will occur at time t_c · The effective temperature change that caused cracking is the difference between the zero-stress temperature and the temperature at cracking (Schindler et al, 2002).

Also, as shown in equation 2, crack width after cracking is a function of ΔT_ζ which is the difference between "zero-stress" temperature and temperature of concrete at a time of interest.. It is well accepted that crack width is the most significant performance indicator of CRCP since LTE depends primarily on crack widths, and keeping crack widths as tight as possible is one of the most important factors that need to be considered in the design and construction of CRCP. There are a number of equations available to estimate crack widths in concrete. The difficulty of accurate prediction of crack widths in CRCP comes from complexity of estimating concrete stress in CRCP, especially in the vicinity of longitudinal steel due to large variations in bond stress distribution. The most widely used equation is the one used in NCHRP 1-37, as shown in equation (2).

$$cw = Max\left[L\cdot\left(\varepsilon_{SHR}+\alpha_{PCC}\Delta T_\zeta-\frac{c_2 f_\sigma}{E_{PCC}}\right)\cdot 1000\cdot CC,\ 0.001\right] \quad \text{Eq. 2}$$

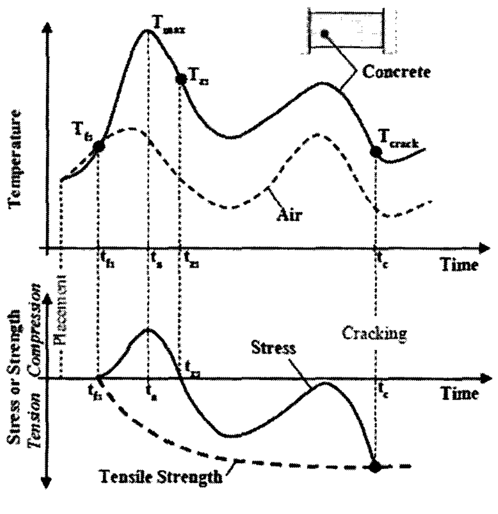


Figure 1. Thermal Stresses and Strength of Concrete at Early-Age

In this formula, cw = average crack width at the depth of the steel, mils, L = crack spacing based on design crack distribution, in., ε_{SHR} = unrestrained concrete drying shrinkage at the depth of the steel, $\times 10^{-6}$, α_{PCC}= PCC coefficient of thermal expansion, /°F, ΔT_ζ = drop in PCC temperature from the concrete "zero-stress temperature" at the depth of the steel, °F, c_2. = second bond stress coefficient, f_σ = maximum longitudinal tensile stress in PCC at the steel level, psi, E_{PCC} = PCC elastic modulus, psi, and CC = local calibration constant (CC = 1 for the national calibration).

3. TEMPERETURE PREDICTION PROGRAM DESCRIPTION

3.1 PROGRAM DESCRIPTION

The PavePro computer program was developed by the Center for Transportation Research of the University of Texas at Austin under research project 0-1700 (Schindler, et al, 2002). Input variables of the PavePro program are divided into the following groups: general, mix proportion, material, environmental, and construction inputs.

General inputs : Variables in this category include pavement slab and subbase thicknesses and type (asphalt concrete, cement stabilized, asphalt stabilized, granular, and existing PCCP), prediction reliability level(50, 75, 90, 95%), geographical location in Texas, construction date, and time of placement.

Mixture proportion inputs : Information available in normal concrete mix design is required. They include mixture proportions such as w/c ratio, the amount of cementitious materials, aggregates and chemical admixtures.

Material inputs : Listed below are the material properties required for PavePro:

(1) cement and other supplementary cementing

materials (SCM): type, chemical composition, fineness, surface area, CaO content for fly ash

(2) coarse aggregate: type of coarse aggregate, thermal coefficient of concrete

(3) cementitious materials: activation energy, adiabatic constants (α_u, τ, β).

The most significant input variables are the hydration properties defined by the activation energy and the hydration parameters of the cement used. These values are determined from the semi-adiabatic testing. In this program, the following exponential function has been employed to represent the degree of hydration development:

$$\alpha(t_e) = \alpha_u \cdot \exp\left(-\left[\frac{\tau}{t_e}\right]^{\beta}\right) \qquad \text{Eq. (3)}$$

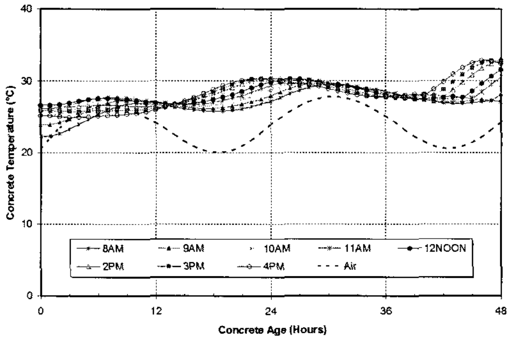
In this formula, the hydration time parameter(τ) corresponds to the time at which 37% of the degree of hydration has progressed. Higher values of τ are anticipated for more reactive cementitious materials such as Type Ⅲ cements, whereas, lower τ values are expected for cements that contain fly ash or slag. The hydration slope parameter, β, predominantly changes the slope of the hydration curve. An increase in β is associated with more reactive cementitious materials. However, because the hydration time is simultaneously delayed, a coinciding change in the τ parameter is also required. The ultimate degree of hydration parameter, α_u, is a factor affecting the magnitude of the degree of hydration. The higher α_u, the higher the final degree of hydration will become, and additional total heat will become available for the hydration process (Schindler 2002).

Environmental inputs : There are two options available for environment inputs. In the first option, a user provides the geographical location of the project and the program generates environmental data needed for the analysis. The data generated is based on 30-year weather information from the NOAA database. The other option is to obtain local weather information for the next three days from the day of concrete paving and input this information into the program. It should be noted that the environmental input values have a significant effect on the accuracy of the predicted concrete temperatures and efforts should be made to obtain accurate information on the environmental conditions during concrete paving. The following link is the source of the weather data used in this study.
<http://www.srh.noaa.gov/ifps/MapClick.php?FcstType=digital&textField1=30.3153839111328&textField2=-97.7663345336914&site=ewx>

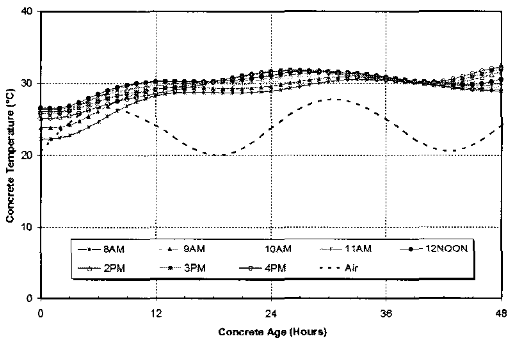
Construction inputs : Finally, construction inputs include fresh concrete temperature, base temperature, and curing method. Information needed for the curing method includes time of the curing compound application, number of curing applications such as single coat or double coat, the application rate defined by square feet per gallon, and color of curing compound.

3.2 TEMPERETURE PREDICTION BY PAVEPRO

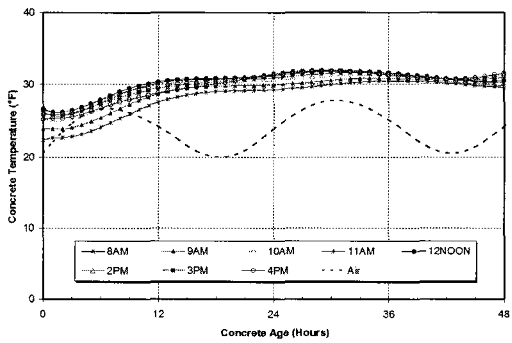
Figure 2 presents three different graphs showing predicted air and concrete temperatures at different depths of the slab with the air temperatures, predicted by PavePro. Each graph shows the relationship between concrete temperatures, and its age, for concrete placed at different times of the day. As shown in Figure 2(a), the maximum temperature predicted at one inch below the surface of the concrete slab was 30.4 and 32.8℃ during the first 24 hours and 48 hours of concrete placed at 2 p.m. Likewise, Figure 2(b) shows that the maximum temperature predicted in the middle of the slab was 31.7 and 32.3℃ during the first 24 hours and 48 hours of concrete placed at 2 to 4p.m. Lastly, Figure 2(c) shows that the maximum temperature predicted at one inch



(a) Top



(b) Middle



(c) Bottom

Figure 2 Slab Temperatures at Each Depths Predicted by PavePro

above the bottom of the slab was 31.4 and 31.9℃ during the first 24 hours and 48 hours of concrete placed at 2~3p.m. The maximum temperatures predicted by PavePro take place at concrete placed in the afternoon.

4. FIELD TESTING AND RESULTS

4.1 FIELD TESTING

The test section was located on US 183 between IH35 and US290 in Austin, Texas. The placement of concrete began at 7:00 a.m. and was completed at 3:30 p.m. on November 11, 2005. The slab thickness of this test section is 13 inches, while the width is about 25 feet. The coarse aggregate used was dolomite. Concrete mix designs used on the three test sections are given in Table 1.

Table 1. Concrete Mix Designs used on Test Sections

Water-Cement Ratio	Maximum Aggregate Size	Unit Weight(lbs/cy)					Air Entraining (oz)	Reducer Retarding (oz)
		W	C	C.A	F.A	F/a		
0.52	lin.	229	358	1916	1368	100	2.64	10.58

4.2 TEMPERATURE MEASURING DEVICE

The devices used for concrete temperature measurements were I-buttons manufactured by Dallas Semiconductor. These devices can be programmed to record the temperature at any desired time interval. They can also internally store up to 2048 temperature readings. The data can be downloaded into a computer using software provided by the manufacturer. Since the I-buttons are inexpensive and simple to use, they are effective tools to measure concrete temperatures.

The three I-buttons were assembled to measure temperatures at three different depths of a concrete slab. In this study, the three depths are defined as top (1-in from the slab surface), middle (at the mid-depth), and bottom (1-in from the bottom of the slab). These I-button assemblies were installed at desired locations in the concrete pavement. Figure 3 shows an assembly of I-buttons before being inserted into the concrete pavement.

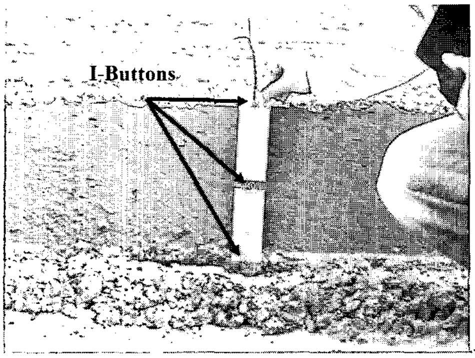


Figure 3. Geometric Configuration of I-button Assembly

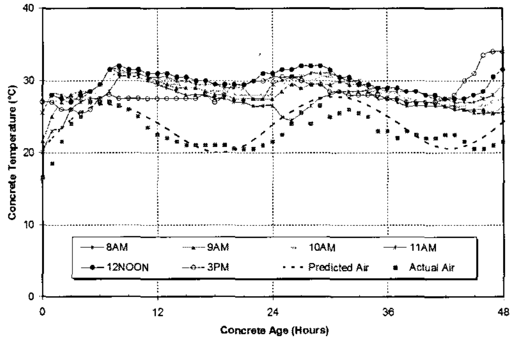
4.3 TEMPERATURE MEASUREMENT

I-Button assemblies were installed at 8, 9, 10, 11, 12, 2, and 3 p.m. The I-Buttons installed at 2 p.m. were broken and hence the data could not be obtained. Figure 4 shows the actual temperature variation of the test sections. As shown in Figure 4 (b) and (c), the actual maximum temperature shows 35.5℃ in the middle and 1 inch above the bottom of slab for the concrete at age of 8 to 15 hours and placed at noon. Also, Figure 4(a) shows that the maximum temperature at top occurred in 48 hours after concrete placement. This is due to the combined effect of

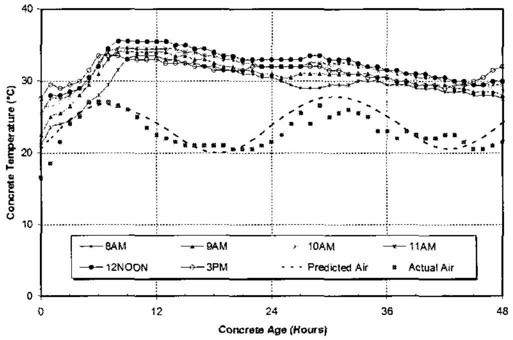
Table 2. Predicted and Actual Maximum Temperatures during 24 and 48 hours after placement

Depth	Time	24 Hours			48 Hours		
		Prediction	Actual	Difference	Prediction	Actual	Difference
Top	8AM	27.1	30.5	3.4	29.5	30.5	1.0
	9AM	27.9	31.5	3.6	29.7	31.5	1.8
	10AM	28.7	31.0	2.3	30.0	31.0	1.0
	11AM	29.4	31.5	2.1	30.5	31.5	1.0
	12NOON	29.9	32.0	2.1	31.6	32.0	0.4
	3PM	30.3	30.5	0.2	32.8	34.0	1.2
Middle	8AM	29.0	33.5	4.5	30.5	33.5	3.0
	9AM	29.6	34.0	4.4	30.9	34.0	3.1
	10AM	30.2	34.5	4.3	31.2	34.5	3.3
	11AM	30.7	34.5	3.8	31.5	34.5	3.0
	12NOON	31.2	35.5	4.3	31.7	35.5	3.8
	3PM	31.7	33.5	1.8	32.0	33.5	1.5
Bottom	8AM	29.2	32.5	3.3	30.5	32.5	2.0
	9AM	29.9	33.5	3.6	30.9	33.5	2.6
	10AM	30.5	34.0	3.5	31.3	34.0	2.7
	11AM	30.9	34.5	3.6	31.6	34.5	2.9
	12NOON	31.2	35.5	4.3	31.8	35.5	3.7
	3PM	31.4	33.5	2.1	31.9	33.5	1.6

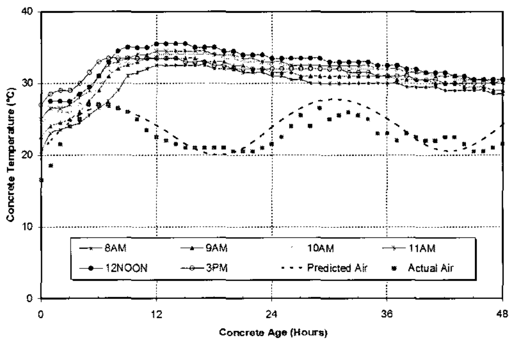
* Average Difference : 3.2℃ ** Average Difference : 2.2℃



(a) Top



(b) Middle



(c) Bottom

Figure 4. Actual Temperature of Test Section

cement hydration heat and solar radiation. Table 2 compares predicted with actual maximum temperatures during 24 and 48 hours after placement. The averages of the difference between the actual and predicted maximum temperature during 24 and 48 hours after placement are 3.2 and 2.2℃, respectively. The actual maximum temperatures are greater than the predicted maximum temperature. The difference ranges from 0.2 to 4.5 ℃. As explained in the previous section, these differences might have been caused by inaccurate estimation of hydration properties such as activation energy and adiabatic constants. For the most accurate temperature prediction, it is recommended that the hydration proerties be accurately evaluated. Also, it is noted that within 24 hours of concrete placement, temperatures of concrete placed in the morning are higher than those placed in the afternoon. In this project, the maximum concrete temperature occurred in the concrete placed at noon.

4.4 CRACK SURVEY

The test section consisted of six total sections, four in the morning and two in the afternoon, in which each small section was100 feet long. The cracks were surveyed for 12 days after concrete placement. The average numbers of cracks for the morning and afternoon placed sections were 7.5 and 6, respectively. The average crack spacing for the morning section was 13.2 feet and 16.5 feet for the afternoon section. The rate of crack occurrence increased at 25 percent greater in the morning placed section than the afternoon section, while crack spacing decreased at a rate of 25 percent during the same time period. Therefore, in the case of jointed concrete pavement, concrete placed in the morning would require earlier saw cutting.

Table 3. Number of Crack and Average Crack Spacing

	Morning Sections				Afternoon Sections	
	Section1	Section2	Section3	Section4	Section5	Section6
Number of Crack	8	7	7	8	6	6
	Average 7.5(12.5%)				6(100%)	
Average Crack Spacing	13.2feet(100%)				16.5feet(125%)	

5. CONCLUSIONS

High temperatures of concrete at early ages are believed to have a significant influence on the long-term performance of continuously reinforcement concrete pavement. Based on these premises, the program PavePro was developed by the Center for Transportation Research of the University of Texas at Austin under research project 0-1700 in order to predict concrete pavement temperatures more accurately by taking factors such as material properties and environmental conditions into account. In this study, test sections were selected and field testing was performed. The measured data were compared with the results predicted by PavePro. Findings from this study can be summarized as follows:

1) The average values of the difference between actual and predicted maximum temperature during 24 and 48 hours after placement are 3.2 and 2.2℃, respectively. The actual maximum temperatures are greater than the predicted maximum temperature. The difference ranges from 0.2 to 4.5 ℃.
2) For accurate temperature predictions, activation energy and adiabatic constants (α_u, τ, β) , with the hydration properties of cementitious materials, should be evaluated and accurate values be obtained for use as input values.
3) Whthin 24 hours of concrete placement, temperatures of concrete placed in the morning are higher than those placed in the afternoon. In this project, the maximum concrete temperature occurred in the concrete placed at noon..

4) The rate of crack occurrence increased at 25 percent greater in the morning placed section than the afternoon section, while crack spacing decreased at a rate of 25 percent during the same time period. For JCP, concrete placed in the morning would require earlier saw cutting.

Based on these findings, it is concluded that maximum concrete temperature has a significant effect on crack development, and better concrete temperature control is needed to ensure adequate CRCP performance.

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