

Methodology for Establishing Permanent Curl/Warp Temperature Gradients in Jointed Plain Concrete Pavements

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Abstract

Jointed plain concrete pavements curl due to temperature gradients and warp in the presence of drying shrinkage gradients. Since this deformation is restrained by factors, such as the slab self-weight, tensile stresses are generated. Slabs do not remain flat in the absence of daily gradients. This is due to the presence of permanent gradients in the slab. A portion of the permanent gradients is the temperature gradient that exists in the slab at the zero-stress time. The gradient present at the zero-stress time, known as built-in, locks into the slab, shifting the curling due to transient gradients up or down. The other component included in the built-in gradient is the permanent warp that develops due to irreversible drying shrinkage. The built-in gradient is reduced over time due to creep. A procedure is established in the study for defining the built-in gradient in the slab. First, the zero-stress time is established, so that the built in temperature gradient can be identified. This is performed using the data from four instrumented pavement sections in Western Pennsylvania. A temperature/hydration model is used to predict the temperature and degree of hydration within the pavement structure based on the ambient temperature and concrete mixture design. Next, the component of the built-in gradient attributed to irreversible drying shrinkage is determined. This is achieved by using long-term strain measurements in an instrumented pavement section in Pennsylvania, as well as two instrumented sections at the Minnesota Road Research Facility (MnROAD). The unrestrained drying shrinkage is then predicted using a numerical relative humidity model. The difference between the predicted and measured drying shrinkage is attributed to the effects of creep and base restraints. The temperature gradient at the zero-stress time is then added to the irreversible drying shrinkage component to obtain the total built-in gradient.

Introduction

Jointed plain concrete pavements (JPCP)s deform due to the presence of gradients in the slab. These gradients can be the result of a variation in the temperature and/or moisture content throughout the depth of the slab. When the slab is cooler at the top than the bottom (negative temperature gradient), the slab curls upward; and when the top of the slab is warmer than the bottom (positive temperature gradient), the slab curls downward. On the other hand, more-pronounced moisture loss occurs near the exposed surface of the slab, causing the slab to warp upward. The Mechanistic-Empirical Pavement Design Guide (MEPDG) considers the shape of the slab at the time each vehicle load is applied. The shape of the slab is defined as a function of the composite effects of the transient gradients and what will be referred to as the permanent gradient. As a result of the permanent gradient, researchers have shown that concrete slabs do not necessarily remain flat in the absence of transient gradients

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(Wells et al. 2006, Yu et al 2001). The permanent gradient ($\Delta T_{perm.curl/warp}$) consists of the combined effects of the built-in temperature/moisture gradient ($\Delta T_{built-in}$), the moisture gradient resulting from irreversible drying shrinkage (ΔT_{shri}) and the portion of these two gradients that dissipate due to creep (ΔT_{creep}). (NOTE: The “built-in temperature gradient” input required for the MEPDG is equivalent to $\Delta T_{perm.curl/warp}$ as it is defined here.) See Equation 1.

$$\Delta T_{perm.curl/warp} = \Delta T_{built-in} + \Delta T_{shri} - \Delta T_{creep} \quad (1)$$

The $\Delta T_{built-in}$ in Equation 1 is a linear temperature gradient equivalent to the nonlinear temperature/moisture gradients that are present in the slab at the zero-stress time (TZ). TZ is the time shortly after construction at which the slab begins to deform with change in temperature. TZ occurs after the placement of the slab, sometime during curing and following final set time. A previous field study (Wells et al. 2006) revealed that the slab remains fully saturated at TZ. Based on this observation, it is safe to define the $\Delta T_{built-in}$ in terms of only the temperature gradient present at TZ.

The second component of $\Delta T_{perm.curl/warp}$, as seen in Equation 1, is the irreversible drying shrinkage gradient. Part of the drying shrinkage that develops in the slab is recovered in the wet seasons. There is, however, an irreversible component, that gradually increases with time. The irreversible drying shrinkage typically reaches a plateau approximately four to five years after construction for most climates in the United States (Burnham and Koubaa 2001). The linear temperature gradient equivalent to the nonlinear long-term irreversible drying shrinkage gradient across the slab depth is the second component of $\Delta T_{perm.curl/warp}$ in Equation 1. The last component, ΔT_{creep} , quantifies the portion of $\Delta T_{perm.curl/warp}$ that is lost due to creep over time.

This study presents a simple, yet realistic procedure for estimating the $\Delta T_{perm.curl/warp}$ for a JPCP section. In establishing this parameter, the effect of all three influential factors, $\Delta T_{built-in}$, ΔT_{shri} and ΔT_{creep} , are considered. JPCPs are constructed with different structural design features and Portland cement concrete (PCC) mixture designs. They are also exposed to different climatic conditions, depending on their geographical location and the time of paving. This study provides an estimate of the $\Delta T_{perm.curl/warp}$ for a range of JPCPs with different design features and PCC mixtures, constructed in Pennsylvania and Minnesota. The proposed methodology can be easily extended and applied to other states across the nation.

Other researchers have provided approaches to quantify this parameter as well. One method used to establish this parameter was by selecting a value that minimized the error in the prediction provided by the cracking model incorporated into the MEPDG (Applied Research Associates (ARA) Inc. ERES Consultants Division 2004). Another approach used by ARA is through a statistical analysis. The ARA study suggests correlations for estimating the built-in gradient based on several influential factors. In this method, the built-in gradient is again selected by minimizing the error between the cracked predicted using the models within the MEPDG and that observed in the long-term pavement performance (LTPP) sections. Other efforts include using a combination of analyses performed using the finite element method (Khazanovich 1994) and tools for measuring the slab response, such as Dipsticks© (Yu et al. 1998; Rao et al. 2001; Schmidt 2001; Wells et al. 2006) and falling weight deflectometer (FWD testing) (Vandenbossche 2003; Rao 2005).

The method proposed in the current study, although innovative and applicable to various JPCP sections, is empirical and has its own limitations. For instance, more field data is required to expand the application of the proposed methodology to states with extremely different climatic conditions than Pennsylvania and Minnesota and for pavement sections with different design features than the ones used in this study. This limitation can be overcome using the data already available in the LTPP database for different sections across the country. It should also be noted that the purpose of this paper is to introduce the procedure itself. The details regarding each task performed in the development of this procedure can be found in (Nassiri 2011.)

Research Approach

The complete procedure proposed in this study for estimating the permanent curl/warp temperature gradient for JPCPs is illustrated in the flowchart provided in Figure 1. As seen in Figure 1, the procedure is based upon two major numerical models, the temperature/hydration and the relative humidity prediction model. The temperature/hydration model is used to establish the permanent curling temperature gradient, while the moisture prediction model together with drying shrinkage measurements in the field for different pavement sections will be used to establish the permanent warping temperature gradient.

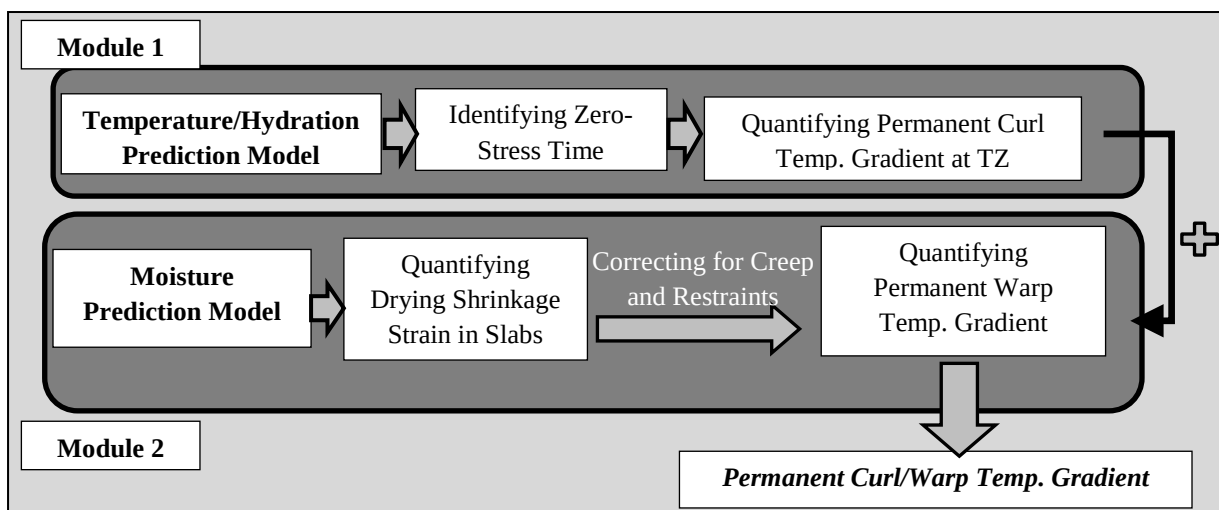


Figure 1. Schematic presentation of different tasks involved with estimating permanent curl/warp temp. gradient.

Module 1 is based upon short-term (couple of days) strain and temperature data from four instrumented JPCP sections constructed in the spring and summer of 2009 and 2010. The projects were all part of the State Route (SR) 22 renewal project, conducted by the Pennsylvania Department of Transportation (PennDOT). All projects are JPCPs with 3.6 by 5.7 m PCC slabs, including 38-mm dowel bars along the transverse joints and 16-mm tie bars along the longitudinal joints, shoulders and centerline. See Table 1 for details regarding the geographical location, structural design and date of paving for each project. A summary of the PCC mixture design obtained from the batch plant on the paving day for each project is provided in the appendix in Table A-1.

Each instrumented project consisted of three cells with three slabs in each cell. The three cells were located such that Cell 1 would be paved in the AM hours, Cell 2 would be paved around noon and Cell 3 would be paved in the PM hours. Strain data

was collected at the top and bottom of the slabs at midpanel, using Geokon Model 4200 vibrating wire (VW) static strain gages. Temperature was measured at different depths in the very first and the very last slab within each cell, using Type T Omega Engineering thermocouples. A portable weather station was assembled and installed at the site for each project to monitor and record the climatic conditions.

Table 1. Location and time of paving for four projects instrumented in this study (ATPB: Asphalt treated permeable base; and CTPB: cement treated permeable base).

Project No.	Geographic Limits	Test Section Stationing	Pavement Structure	County, State	Date of Construction
1	SR 22 (B09) from PA 982 to Auction Barn Rd T-968	From 1136+50 to 1147+60- Westbound	JPCP- 30.5 cm PCC/10 cm ATPB/ 15 cm 2A Subbase	Westmoreland, Pennsylvania	09/02/2009
2	SR 22 (B09) from PA 982 to Auction Barn Rd T-968	From 1115+00 to 1101+65- Eastbound	JPCP- 30.5 cm PCC/10 cm ATPB/ 15 cm 2A Subbase	Westmoreland, Pennsylvania	5/10/2010
3	SR 22 (B10) from Auction Barn Rd T-968 to Indiana County Line	From 939+60 to 955+15- Westbound	JPCP- 35.5 cm PCC/10 cm CTPB/15 cm 2A Subbase	Westmoreland, Pennsylvania	10/8/2009
4	US 22 from T-724/T-910 to SR 2024	From 377+96 to 399+45- Westbound	JPCP- 25.4 cm PCC/10 cm CTPB/15 cm 2A Subbase	Indiana, Pennsylvania	4/29/2010 and 4/30/2010

Module 2 in Figure 1 is based upon long-term (six-year) strain data collected in another instrumented JPCP section constructed in August of 2004. This section is also located on SR 22 in Western Pennsylvania. The pavement section will be discussed in detail in a subsequent section. The long-term strain data from two cells at the Minnesota Road Research Facility (MnROAD) is also used in Module 2.

Module 1-Establishing Permanent Curl Temperature Gradient

Identifying TZ

A method developed in a previous study (Wells et al. 2006) based upon the strain-temperature behavior of the slab was used to identify TZ in the four instrumented JPCPs introduced earlier. In this method, TZ is defined as the transition point in the strain-temperature response of the slab, as the concrete gradually transforms from a thixotropic material capable of flow to a hardened, rigid material. TZ was established for all four projects following this methodology.

To be able to establish TZ for any JPCP with the same structure as the four instrumented projects, the degree of hydration at TZ was computed for each project. It is known that for a particular mixture, setting takes place at a certain degree of hydration (Byfors 1980). This concept has been previously used to identify the final time of set for PCC mixtures (Schindler 2004) and is adopted in the current study to identify TZ. The best-fit mathematical model for estimating the degree of hydration has the following form (Schindler and Folliard 2005):

$$\alpha(t_e) = \alpha_u \exp\left(\frac{\tau}{t_e}\right)^\beta \quad (2)$$

Where, $\alpha(t_e)$ = degree of hydration; α_u = ultimate degree of hydration, t_e = equivalent age and τ and β are the time and hydration shape parameters. Schindler and Folliard (2005) provided equations for estimating these parameters, which have not been provided here for the sake of brevity. The average degree of hydration at which TZ was established for all four instrumented projects was 45 percent. Therefore it is hypothesized that doweled 23- to 38-cm thick JPCPs constructed on stabilized bases will reach TZ at a degree of hydration of approximately 45 percent.

Establishing Permanent Curl Temp Gradient

Once TZ is established, the temperatures throughout the depth of the slab at TZ are used to establish the built-in temperature gradient. When establishing this parameter, it is essential to consider the nonlinearity in the temperature distribution throughout the slab depth. To do so, the equivalent temperature gradient (ELTG) concept was adopted. In 2000, Janssen and Snyder developed a method for estimating the ELTG based on the temperature moment in the slab. This method is summarized in the following relations.

$$TM_0 = -0.25 \left\{ \sum_{i=1}^n [(t_i + t_{i+1})(d_i^2 - d_{i+1}^2)] - 2(d_1^2 - d_n^2)T_{WAT} \right\} \quad (3)$$

$$T_{WAT} = \sum_{i=1}^n \left[\frac{0.5(t_i + t_{i+1})(d_i - d_{i+1})}{(d_1 - d_n)} \right] \quad (4)$$

In the above equations, t_i = temperature at location i , °C; d_i = depth at location i , cm; T_{WAT} = weighted average temperature, °C. The temperature moment can be converted into an ELTG by determining the linear gradient that produces the same magnitude of temperature moment as the measured surface profile, using Equation 5.

$$ELTG = \frac{-12(TM_0)}{h^3} \quad (5)$$

In this equation, h is slab thickness in cm. A summary of the built-in temperature gradient (the ELTG at TZ) established for each of the three cells for all four projects is presented in Figure 2. In this figure, the cells are grouped based on the time of paving. According to Figure 2, the lowest variability in the magnitude of the built-in temperature gradient is seen for the slabs paved in the morning and at noon. The largest variability is seen among the slabs paved in late afternoon. This could be because in the case of morning/noon paving, similar climatic conditions were present on the days the four projects were constructed. In the case of afternoon and evening paving, the climatic conditions varied between projects. Cold conditions present in the afternoon decreased the rate of hydration of the cement. Depending on the rate of hydration, TZ can occur anytime during the night, resulting in a negative built-in gradient, or the following day, in which case a positive temperature gradient would be present.

This study aims at establishing the built-in temperature gradient without the need to instrument the slab with thermocouples. To achieve this goal, a numerical temperature model is developed, based on the general heat transfer equation. The details regarding the temperature model are discussed in the following section.

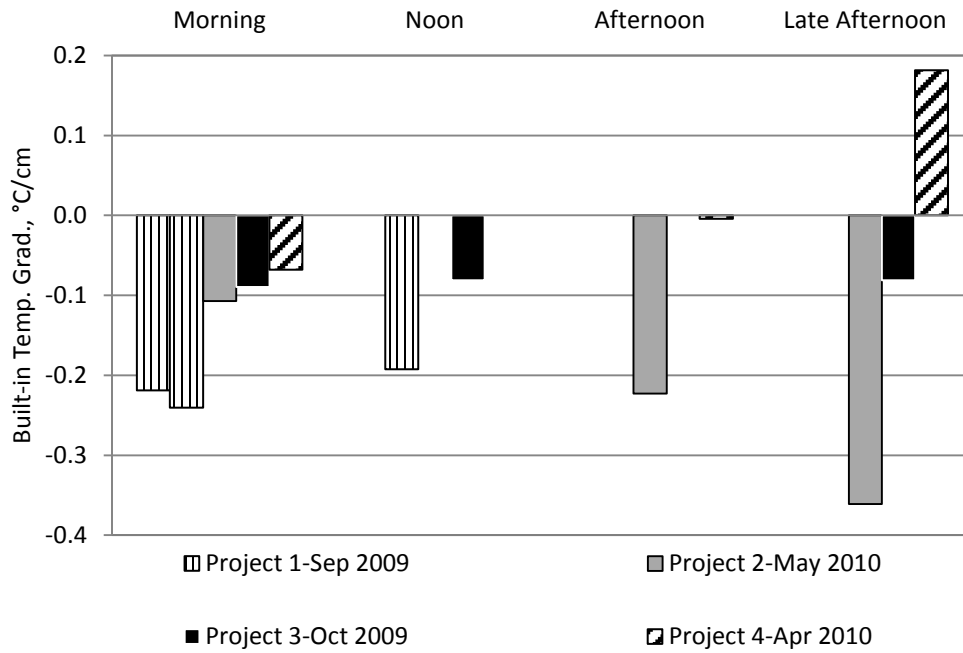


Figure 2. Built-in temperature gradients established for the four instrumented projects.

Temperature/Hydration Model

The governing equation of heat transfer, presented in Equation 6, was solved throughout the depth of the slab over time, using the finite difference method (FDM).

$$\frac{d}{dx} \left(k \cdot \frac{dT}{dx} \right) + Q_H = \rho \cdot C_p \cdot \frac{dT}{dt} \quad (6)$$

In this equation, T = temperature, °C; t = time, hours; ρ = density, kg/m³; c_p = specific heat, J/kg/°C; Q_H = generated heat per unit time and volume, W/m³; k = thermal conductivity, W/m/°C. The temperature in the PCC is determined by the balance between the heat generation in the PCC and the heat exchange with the environment. Heat transfer was accounted for in four different modes: conduction, convection (q_c), irradiation (q_r) and solar absorption (q_s). The boundary conditions defined at the top of the pavement section is presented in Equation 7 (Jeong, Wang et al. 2001). At the bottom, a constant temperature of 12 °C was assumed at a depth of 2.5 m (Hermansson 2001). Models already available in literature for defining each of the four heat transfer mechanisms at the boundary conditions were used in this study.

$$-k \nabla T \cdot n + q_c + q_r - q_s = 0 \quad (7)$$

The hydration reaction of Portland cement is an exothermic process. Therefore, when using the general heat transfer equation, presented in Equation 6, to predict the temperature variations across the pavement structure, the rate of heat liberation, Q_H , needs to be considered in each time step. The following regression model was used for this purpose (Schindler and Folliard 2005).

$$Q_H = H_u C_c \left(\frac{\tau}{te} \right)^\beta \left(\frac{\beta}{te} \right) \alpha(te) \exp \left[\frac{-E}{R} \left(\frac{1}{273 + T_c} - \frac{1}{273 + T_r} \right) \right] \quad (8)$$

$$H_u = H_{cem} P_{cem} + 461P_{slag} + 1800P_{Cao-FA} \cdot P_{FA} \quad (9)$$

$$H_{cem} = 500P_{C3A} + 260P_{C2S} + 866P_{C3S} + 420P_{C4AF} + 624P_{So3} + 1186P_{free-Cao} + 85P_{Mgo} \quad (10)$$

Where, Q_H = rate of heat liberation, W/m^3 ; C_c = cementitious materials content, kg/m^3 , H_u = total heat of hydration of cementitious materials at 100 percent hydration, J/kg , H_{cem} = total heat of hydration of cement, P_i = weight ratio of i^{th} compound in terms of total cement content.

Temperature throughout the depth of the slabs in all four instrumented projects was predicted over the first 72 hours after construction, using the numerical temperature model. In doing so, the ambient climatic data collected at the site for each project was incorporated into the model. To validate the accuracy of the model, the predicted temperatures at different depths were compared against the temperatures measured at the same depth. Figure 3 provides an example of a comparison between the ELTG estimated using the predicted values and the measured temperatures. TZ is also indicated in this figure with a vertical line. As seen in the figure, the temperature model predicts the ELTG in the slab at TZ with good accuracy. The same accuracy was achieved when comparing the predicted and measured ELTG at TZ for the other three projects as well. In conclusion, the temperature/hydration model can be used to identify TZ as the time when the slab reaches 45 percent of the ultimate degree of hydration. The ELTG at TZ is then established based on the temperature predictions throughout the slab depth and is used as the $\Delta T_{built-in}$.

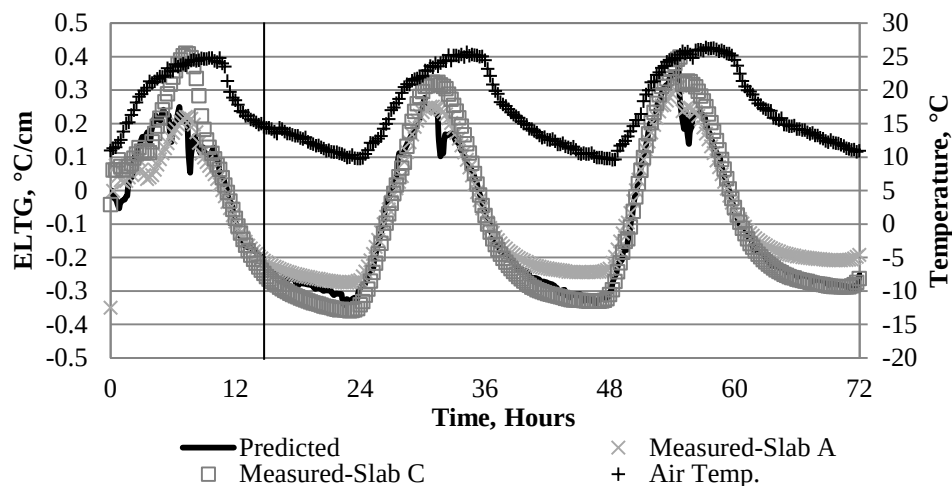


Figure 3. ELTG estimated based on field measurements and model predicted temperatures for Cell 1 on Project 1 paved at 8:00 AM.

Module 2-Establishing Permanent Warp Temperature Gradient

Predicting Drying Shrinkage Strain in the Slab

The method developed by Mohamed and Hansen in 1997 is employed herein for estimating a linear temperature gradient equivalent to the irreversible drying shrinkage gradient. In this method, the relative humidity (RH) profile throughout the slab depth is used to estimate a humidity difference coefficient. A third-degree polynomial function is then fit through the humidity difference coefficients across the

slab depth. See Equations 11 and 12. The regression coefficients from the polynomial function are then used to establish an equivalent linear humidity difference coefficient. The product of this coefficient and the ultimate drying shrinkage of the concrete, provides an estimate of the drying shrinkage strain in the slab at any time of interest (Jeong and Zollinger 2005). This strain, when divided by the coefficient of thermal expansion (CTE) of the concrete and the slab thickness, produces an equivalent temperature gradient.

$$\Delta[1 - (RH/100)^3] = A + Bz + Cz^2 + Dz^3 \quad (11)$$

$$\Delta[1 - (RH/100)^3]_{eq} = -12\left(\frac{Bh}{12} + \frac{Dh^3}{80}\right) \quad (12)$$

$$ELG = \frac{-\varepsilon_{\infty} \Delta[1 - \frac{RH}{100}]_{eq}^3}{\alpha_c h} \quad (13)$$

In the above equations, $\Delta[1 - (RH/100)^3]_{eq}$ = equivalent linear humidity difference coefficient, z = coordinate defined as zero at mid-depth of the slab, where upward is negative and downward is positive, A, B, C, D = Regression coefficients, h = thickness of the concrete slab, cm. A model was developed to predict the RH distribution in the slab required in Equation 11.

Section B01 on SR 22 Test Section

The RH was measured in a JPCP instrumented and constructed in Western Pennsylvania in August of 2004. A total of six JPCP slabs were instrumented with thermocouples, VW static strain gages and humidity/temperature sensors. Cell 1 consisted of three consecutive unrestrained (no dowel or tie bars) PCC slabs. Cell 2 consisted of three restrained PCC slabs. The restrained slabs feature 16-mm tie bars and 3.8-cm diameter dowel bars placed at every 30.5 cm along the transverse joints. Slab Z is a transition slab that separates the two cells. The pavement section consists of 30.5-cm thick PCC slabs dimensioning 3.66 m by 4.6 m placed on a layer of ATPB.

The variation in RH throughout the depth of slabs was measured for the first two years after the construction of the test section. A total of 24 Sensirion SHT75 RH and temperature sensors were installed in Cell 2 and the transition slab (Slab Z) at various depths. However, out of 24, only the sensors installed at the center of Slab C and at the corner of Slab Z remained active after the construction of the test section. The active sensors at the center of Slab C include those installed at depths 2-, 2.6-, 15- and 25.5 cm. The sensors installed at depths 4- and 16.7 cm at the corner of Slab Z also remained active. Procedures developed at the University of Illinois for using these sensors in concrete applications were followed (Grasley et al. 2003). The data from these sensors will be used to calibrate the numerical RH model.

Numerical Model to Predict RH in the Slab

Fick's second law based on diffusion theory was used in this study for predicting RH throughout the slab depth over time. See Equation 14.

$$\frac{\partial H}{\partial t} = \frac{\partial}{\partial x} \left(C(H) \frac{\partial H}{\partial x} \right) \quad (14)$$

In this equation, H is the concrete internal RH, t = time in hours, x = slab depth in cm and $C(H)$ = diffusivity in cm^2/hour . $C(H)$, in Equation 14, is a key parameter, which determines the moisture transport “speed” inside the material. This parameter is a function of the internal RH, porosity and concrete age (McCullough and Rasmussen 1999). Several relations have been proposed for the diffusivity by different researchers (Sakata 1983; Ayano and Wittman 2002). The most widely-accepted relationship for estimating diffusivity is by far the one developed by Bazant and Najjar in 1972, presented in Equation 14. In this relationship, diffusivity is defined based on the level of RH in the concrete.

$$C(H) = C_1 \left(\alpha_0 + \frac{1 - \alpha_0}{1 + \left(\frac{1 - H}{1 - H_c} \right)^n} \right) \quad (15)$$

C_1 , in this equation, is diffusivity of the saturated concrete in cm^2/hour . Other parameters in Equation 15 include α_0 = ratio of $C_{\min}/C_{\max}$, n = spread of the drop in C and H_c = humidity at which C drops halfway between its maximum and minimum and is defined as 0.75 for most concrete mixtures. Equation 15 was solved using finite difference method. In doing so, proper boundary conditions needed to be defined at the top and bottom of the slab. The relationship provided in Equation 16 was used to define the boundary condition at the surface of the slab. At the slab/base interface, the PCC slab was considered relatively saturated based on RH measured in the slabs in the field.

$$C(H) \frac{\partial H}{\partial x} = f(H_{\text{ambient}} - H_{\text{surface}}) \quad (16)$$

The best values for the inputs within the model, including C_1 , α_0 and n , were established through iterations until the predicted and measured RH converged. Foremost, the ambient RH measured by an onsite weather station at the SR 22 test section was incorporated into the numerical model to predict the RH within the slab over the first two years after construction. The best agreement between the predicted RH and measured values were obtained with C_1 , n and α_0 equal to 0.8, 6 and 0.1, respectively. The RH predicted using these values together with the measured RH at different depths are presented in Figure 4.

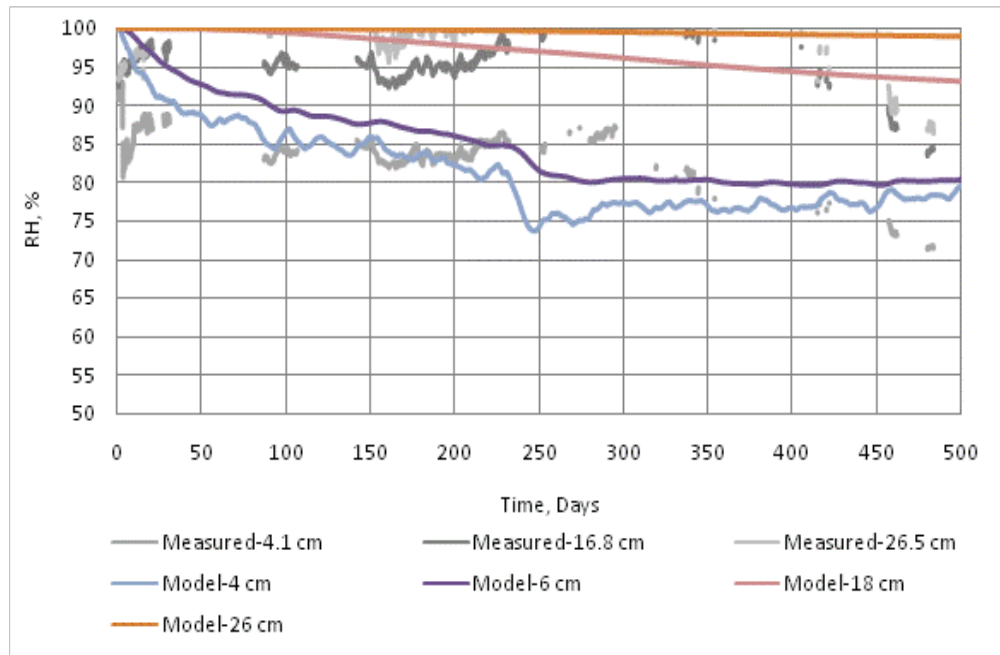


Figure. 4. Predicted RH with respect to measured RH at the center of Slab C.

The next step in Module 2 for establishing the ΔT_{shri} is to use the RH profile to establish the humidity difference coefficient and convert that into drying shrinkage strain. In doing so, as described earlier, the ultimate drying shrinkage, ϵ_{∞} , of the paving mixture is required. The ϵ_{∞} can be established by either performing AASHTO T 160 test procedure or employing the relation provided in Equation 17 (Bazant and Baweja 2000 and Bazant 2000).

$$\epsilon_{\infty} = C_1 C_2 \{26w^{2.1} (f'_c)^{-0.28} + 270\} \quad (17)$$

Where, w = water content for the PCC mixture in cm^3/kg^3 ; f'_c = 28-day PCC compressive strength in MPa; C_1 = cement type factor defined as 1.0, 0.85 and 1.1 for Types I, II, and III, respectively and C_2 = curing factor defined as 0.75, 1.0 and 1.2 if steam cured, cured in water (or 100 percent RH) and sealed (curing compound,) respectively.

The ϵ_{∞} for the PCC mixture used to pave the test section was established as an average value of $920 \mu\epsilon$ by performing the AASHTO T 160 test in the laboratory on three samples cast from the paving mixture. This parameter was also established as approximately $700 \mu\epsilon$, when using the relation provided in Equation 16. Between the two, the value established using the Bazant model was adopted for establishing the drying shrinkage strain, since the moist-curing period of 28 days advised by the standard was not followed while performing the test in the laboratory. The drying shrinkage strain predicted for the test section over a five-year period is presented in Figure 5. To eliminate the seasonal fluctuations in the drying shrinkage throughout each year and to establish the irreversible drying shrinkage strain for the slab, the mean monthly strain for the month of October in each year was used. These values are indicated by the black squares connected by the black line in Figure 5.

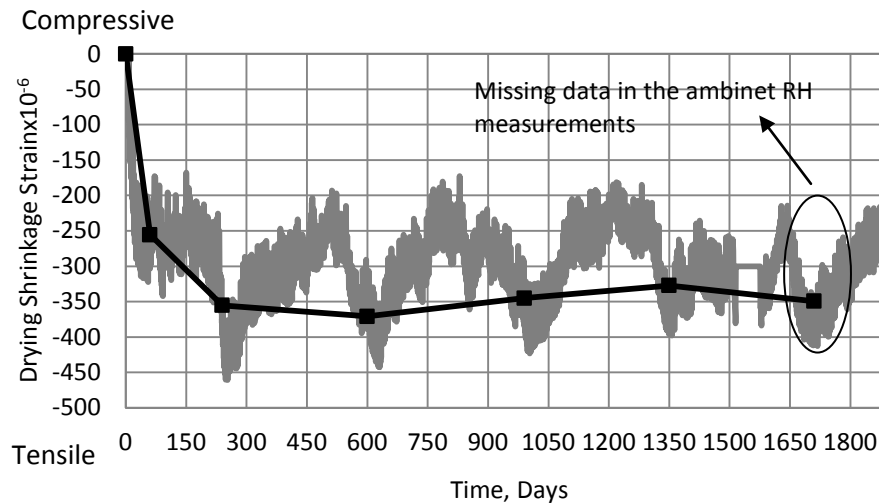


Figure 5. $\Delta [1 - (\frac{RH}{100})^3]_{eq}$ over five years predicted by the numerical model for the test section.

Effects of Restraining Factors and Creep

The procedure carried out in Module 2 allows for the effects of environmental conditions, slab thickness and the PCC mixture on the development of drying shrinkage in the slab with age. This procedure, however, does not consider the resistance contributed by several factors in the field against the tendency of the slab to shrink due to drying. Among these factors are the friction between the slab and the underlying layer and the dowel and tie bars. In addition to these restraining factors, the development of creep reduces the effects of the drying shrinkage and relaxes a portion of the warping gradient. The predicted drying shrinkage in Module 2 needs to be corrected for the effects of the restraining factors and creep relaxation. To accomplish this goal, drying shrinkage measured in the field for the two cells (restrained and unrestrained) on Section B01 on SR 22 (described previously) and two cells at MnROAD are considered.

Measured Drying Shrinkage Strain in Section B01 on SR 22

Drying shrinkage strain was measured in the slabs at Section B01 on SR 22 for a period of approximately 65 months. For this purpose, the axial strains measured with VW gages in the longitudinal direction at the corner location adjacent to the shoulder were used. These gages are located at approximately 2.5 cm from the surface of the slab. The reason these gages were selected is because the slabs are more prone to drying/wetting at the corner adjacent to the shoulder than in the middle of the slab. When using the strain measured by the embedded gages, the strains due to thermal expansion/contraction of the slab were isolated from the total measured strain using Equation 18.

$$\varepsilon_{Mechanical} = (R_t - R_0)B + (T_t - T_0)(\alpha_s - \alpha_c) \quad (18)$$

In this equation, $\varepsilon_{Mechanical}$ = strain in the concrete due to environmental loads in $\mu\epsilon$; R_0 = raw strain at time zero (time of final set for the concrete); R_t = raw strain at the time of interest; T_0 = temperature at time zero in $^{\circ}\text{C}$; T_t = temperature at the time of interest in $^{\circ}\text{C}$; α_s = CTE of the steel in the strain gauge and is equal to $12.2 \mu\epsilon/^{\circ}\text{C}$; α_c = CTE of the concrete and B is the batch calibration factor provided by the manufacturer. It must be noted that establishing drying shrinkage in the slabs using Equation 18 includes some errors. This is because it is assumed in this method that the

slab is free to expand and contract due to thermal changes. The average $\epsilon_{\text{Mechanical}}$ in three consecutive slabs during the month of October in each year is presented in Figure 6 for both the restrained and unrestrained cells. Based on Figure 6, the unrestrained slabs consistently show more drying shrinkage in comparison to the restrained slabs. This agrees with logic, since the dowel bars are expected to restrict the shrinkage of the slab in the longitudinal direction.

Correcting for Restraining Factors and Creep

The predicted drying shrinkage as a function of the ultimate drying shrinkage of the PCC mixture, presented previously in Figure 5 for SR 22 is superimposed onto Figure 6. This strain is compared to the “measured” drying shrinkage strain that developed in the same slabs. The difference between the two (predicted and measured drying shrinkage) is introduced as the field correction factors at different ages. These correction factors can be applied to the predicted drying shrinkage for any slab with the same design as SR 22 to establish the in-situ drying shrinkage. The percent difference between the measured drying shrinkage and that predicted is presented in Table 3 for both the doweled and undoweled slabs.

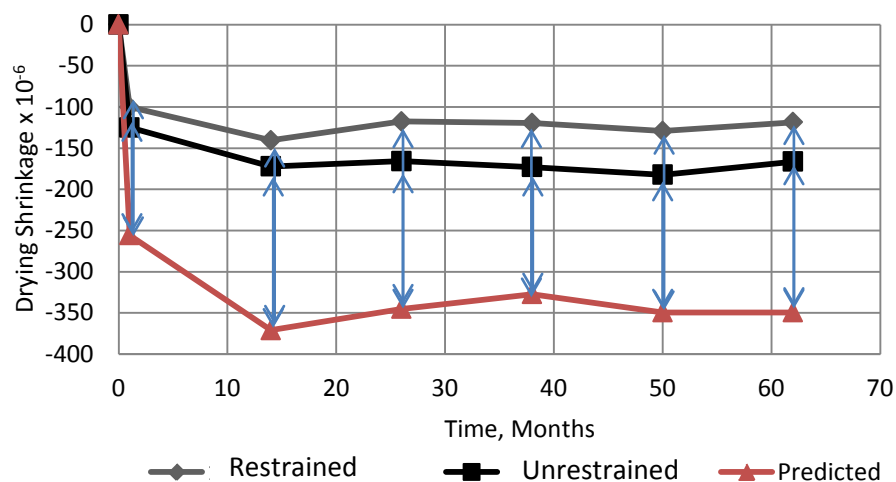


Figure 6. Comparison of the measured and predicted drying shrinkage for two cells at SR 22.

In an attempt to expand the field factors to include a larger range of design features, the field correction factors were also established for Cells 8 and 9 at MnROAD. These cells at MnROAD were constructed in Sep. of 1992 and both consist of 4 m by 4.6 m, 19-cm thick slabs on a the ATPB base, while only the slabs in Cell 8 include 2.54-cm diameter dowels and tie bars. The PCC mixture for these cells is provided in the Appendix in Table A-3.

First, the drying shrinkage strain was predicted for each cell using the RH model. In doing so, the climatic database available in the MEPDG for MnROAD was adopted. This compromise was inevitable, since the ambient data collected using the weather station at MnROAD showed discontinuities. The average ultimate drying shrinkage for the MnROAD mixture was estimated using the Bazant empirical relation to be approximately 610 $\mu\epsilon$. As discussed previously, a product of this value and the linear equivalent humidity difference coefficient established for each cell using the RH model provides an estimate of the drying shrinkage for each cell. Both predicted and measured drying shrinkage strains are presented in Figure 7 for Cells 8

and 9. The field correction factors at different ages for both cells are summarized in Table 4 as a percent difference.

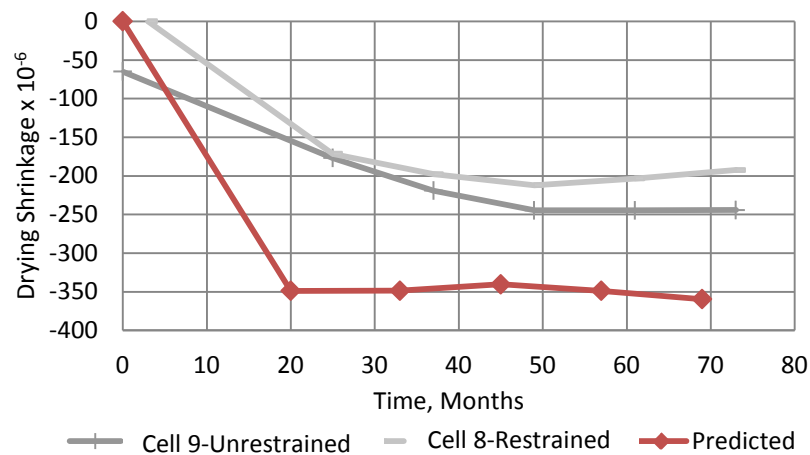


Figure 7. Comparison of the measured and predicted drying shrinkage for Cells 8 and 9 at MnROAD.

Table 3. Field correction factors for cells on SR 22 and Cells 8 and 9 at MnROAD.

Test Section	Age (Month)	Percent Difference between Predicted and Measured Drying Shrinkage	
		Restrained Slabs	Unrestrained Slabs
SR 22	0	0	0
	1	48	34
	14	62	54
	26	66	52
	38	63	47
	50	63	48
	62	66	52
MnROAD	0	0	0
	25	51	49
	37	43	37
	49	38	28
	61	42	30
	73	46	32
	85	42	31

Considering Figures 6 and 7 together, one can note that the 30.5-cm slabs with the ATPB layer at SR 22 show the lowest drying shrinkage of all slabs. This could be because the slabs on SR 22 are thicker than those at MnROAD and also because the slabs at MnROAD experience drier climatic conditions than the SR 22 slabs in Pennsylvania. The effect of other variables such as the difference in the PCC mixture designs, slab length and month of construction are believed to be negligible. Also, after between 60 and 65 months from the time of paving, drying shrinkage remains relatively constant and the correction factor is 32 and 46 percent for the unrestrained and restrained 19-cm slabs, respectively. These values are 52 and 66 percent for the unrestrained and restrained 30.5-cm slabs, respectively. It is very interesting to note that the effect of dowel bars on the field factors for both sets of slabs is almost the same, even though the dowel diameter for the 30.5-cm slabs is larger by approximately 1 cm.

The effect of slab thickness is almost constant between the doweled and undoweled slabs. An increase of 10 cm in slab thickness resulted in a 30 percent increase in the field factors for the restrained slabs and 38 percent for the unrestrained slabs. The field factors established in this study can be interpolated to be used for doweled and undoweled PCC slabs with a stabilized base and a slab thickness ranging approximately between 19 and 31 cm.

In conclusion, in Module 2 of the procedure, long-term in-situ irreversible drying shrinkage was estimated for doweled and undoweled JPCPs with a slab thickness ranging between 19 and 31 cm. This strain when divided by the CTE of the concrete and the slab thickness to obtain ΔT_{shri} . This parameter needs to be added to the $\Delta T_{built-in}$ established in Module 1 to produce the $\Delta T_{perm.curl/warp}$ for the slab.

Conclusions

A procedure is proposed for establishing the permanent curl/warp temperature gradient in JPCPs. This procedure includes two major modules including a temperature/hydration prediction model and a relative humidity prediction model. Module 1 aims at establishing permanent curl temperature gradient based on the progression of hydration in the slab. Module 2 focuses on predicting the drying shrinkage in the slab using a numerical model and field data. Overall, similar to any empirical procedure, this procedure can be shaped and expanded more with the inclusion of additional field data. Both Modules 1 and 2 can be expanded to include a variety of JPCPs, representing a broader range of design features. The significance of variables such as PCC mixture design and month of construction on the field factors developed in Module 2 should also be further investigated.

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Table A-1. PCC mixture design, fresh and hardened concrete properties for four projects used in Module 1.

	Project 1			Project 2			Project 3			Project 4		
	Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3
PCC Mixture Design (kg/m³)												
Cement-Type I	297	297	297	326	326	326	297	297	297	297	297	297
Fly Ash	52	52	52	59	59	59	52	52	52	52	52	52
Fine Aggregate	791	784	772	688	688	690	691	691	691	780	762	772
Coarse Aggregate	1101	1099	1101	1091	1091	1091	1091	1091	1091	1103	1103	1099
Water Content	95	104	102	85	85	85	107	115	111	89	97	102
Cement Composition (%)												
SiO ₂	20.2			19.5			20.7			20.1		
Al ₂ O ₃	5.3			4.8			4.5			5.35		
Fe ₂ O ₃	4.3			3.6			3.2			4.4		
CaO	64.4			52			63.1			63.4		
MgO	1.0			2.7			2.8			1.0		
SO ₃	2.95			2.9			2.7			2.85		
C ₃ S	59			69			59			54.9		
C ₂ S				12			12					
C ₃ A	6.7			1			6					
Alkalis	0.55						0.66			0.53		
Blaine (m ² /kg)	380			403			390.9			358		
Fresh Mix Properties												
Slump (mm)	42	38	38	38	32	22	38	44	32	44	32	32
Entrained Air (%)	6.2	5.25	6.5	5.5	5.9	5.8	6.8	7.2	5.8	6	5.75	6
Unit weight (kg/m ³)	2435.04	2435.04	NA	2386.98	2411.01	2394.99	2370.96	2403	2370.96	2354.94	2403	2370.96
w/cm Ratio	0.41	0.475	0.47	0.48	0.5	0.46	0.47	0.46	0.46	0.46	-	0.48
Initial Set Time (Hours after Placement)	4.5				5.6			6.8			5.6	
Final Set Time (Hours after Placement)	5.7				7.6			9.4			7.3	
28-day Strength Properties												
Elastic Modulus, MPa	36883	34125	32402	27921	29300	33091	28955	30678	29644	35849	33781	34125
Poisson's Ratio	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Compressive Strength, MPa	39.9	39.5	33.2	33.8	33.4	36.3	31.6	34.5	35.1	40.6	39.5	44.4
Flexural Strength, MPa	6.4	6.5	5.4	5.3	4.9	6.0	5.9	5.1	5.4	5.9	5.0	5.8

Table A-2. Mixture design for the paving PCC used for SR 22.

Weight in the Batch (kg/m ³)				
Cement	Ground Granulated Blast Furnace Slag	Water	Fine Aggregate	Coarse Aggregate
Type I, 226.5	122	169.5	740	1115.5
Fresh Concrete Properties				

Table A-3. PCC mixture design used to pave MnROAD concrete cells.

Weight in the Batch (kg/m ³)				
Cement	Fly Ash	Water	Fine Aggregate	Coarse Aggregate
Type I, 267.5	Class C, 47	145	711	1145