ESTABLISHING APPROPRIATE INPUTS WHEN USING THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE TO DESIGN RIGID PAVEMENTS IN PENNSYLVANIA

FINAL REPORT

Date: 03/31/2011

By: Dr. Julie Vandenbossche, P.E.
University of Pittsburgh

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

CONTRACT # 510601

WORK ORDER # PIT 013

Technical Report Documentation Page

1. Report No. FHWA-PA-2011-006-PIT013	**2. Government Accession No.**	**3. Recipient's Catalog No.**
4. Title and Subtitle Establish Inputs for the New Rigid Component of the Mechanistic-Empirical Pavement Design Guide (MEPDG)		**5. Report Date** March 2011 **6. Performing Organization Code**
7. Author(s) S. Nassiri and J. M. Vandenbossche		**8. Performing Organization Report No.**
9. Performing Organization Name and Address University of Pittsburgh 3700 O'Hara St., Department of Civil and Environmental Engineering, Benedum Hall, 15261, Pittsburgh, PA		**10. Work Unit No. (TRAIS)** **11. Contract or Grant No.** 510601 – PIT 013
12. Sponsoring Agency Name and Address The Pennsylvania Department of Transportation Bureau of Planning and Research Commonwealth Keystone Building 400 North Street, 6th Floor Harrisburg, PA 17120-0064		**13. Type of Report and Period Covered** Final Report: July 1, 2008 – March 31, 2011 **14. Sponsoring Agency Code**
15. Supplementary Notes None		
16. Abstract Each design input in the Mechanistic-Empirical Design Guide (MEPDG) required for the design of Jointed Plain Concrete Pavements (JPCPs) is introduced and discussed in this report. Best values for Pennsylvania conditions were established and recommended for each input by considering: 1-typical values suggested in the Pennsylvania Department of Transportation (PennDOT) publications and databases, 2-reccommendations in the MEPDG documentations-based on nationwide data and 3-laboratory/field test performed in different tasks over the duration of three years for this study. .		
17. Key Words MEPDG, design software, concrete, pavement, input parameter, pavement design, performance		**18. Distribution Statement** No restrictions. This document is available from the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report) Unclassified	**20. Security Classif. (of this page)** Unclassified	**21. No. of Pages** 101	**22. Price**

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

ACKNOWLEDGEMENTS

The financial and technical support provided by the Pennsylvania Department of Transportation (PennDOT) for this project is gratefully acknowledged. The authors would like to extend their sincere gratitude to the project managers Ms. Rebecca D. Lorah and also Ms. Lisa Tarson as well as the project's technical adviser, Ms. Lydia Peddicord. Mr. Joshua Freeman, Michael Long and Patrick Gardiner are also acknowledged for providing helpful comments and technical oversight. A special thanks is also extended to the following people from PennDOT, who assisted in execution of the various tasks performed under WO#13: Mr. David L. Jarvis, Ms. Margaret C. Thomson, Mr. Todd Rottet, Ms. Janice Arellano and Ms. Andrea Bahoric. Gratitude is also extended to the following people who assisted and cooperated with the Pitt research team in the completion of the extensive field work involved with WO#13:

- Mr. Scott Faieta, PennDOT, District 12, Construction Manager of Project 31477
- Mr. Theodore Mowen, PennDOT, District 12, Inspector in Charge of Project 31477
- Mr. Warner Barate, PennDOT, District 12, Inspector in Charge of Project 25833
- Mr. William Bromyard, PennDOT, District 12, Project Manager of Project 31478
- Mr. James H. Kyper , New Enterprise Stone & Lime Co., Inc., General Concrete Paving Superintendent
- Mr. Mark Kmetz, PennDOT, District 12, Project Manager of Project 25833
- Ms. Tania Wisor, PennDOT, District 12, Administrative Assistant of Project 25833
- Mr. Benjamin Shterengarts, PennDOT, District 12, Inspector in Charge of Project 25833
- Mr. Kurt Greenland, New Enterprise Stone & Lime Co., Inc., Inspector in Charge of Projects 31478 and 25833
- Mr. Jonathan Hopper, Golden Triangle Construction Co. Inc., Inspector in Charge of Project 31477
- Mr. James Beckwith, PennDOT, District 12, Inspector supervisor of Project 25833
- Mr. William L. Beaumariage, Support Services Engineer of Construction PennDOT, District 12
- Mr. Gary Mason, New Enterprise Stone & Lime Co., Inc., Job superintendent of Project 25833

- Mr. Robert Weist, New Enterprise Stone & Lime Co., Inc, Concrete technician at the batch plant
- Mr. Edward Colhoun, Golden Triangle Construction Co. Inc., Concrete technician at the batch plant

Contents

List of Figures

List of Tables

1. Introduction

The purpose of Work Order No. 13 (WO#13) is to assist the Pennsylvania Department of Transportation (PennDOT) in the implementation of the rigid pavement portion of the new Mechanistic-Empirical Pavement Design Guide (MEPDG). The PennDOT has been very active in regards to the MEPDG developmental process and also in taking steps towards implementing the MEPDG for pavement design in Pennsylvania. To date, several projects, such as the Superpave In-Situ Stress/Strain Investigation (SISSI), have been focused on the implementation of the hot mix asphalt (HMA) portion of the MEPDG. Therefore, WO#13 focused on the implementation of the rigid portion of the MEPDG throughout the state.

In contrast to the American Association of State Highway and Transportation Officials (AASHTO) 1993 Guide for Design of Pavement Structures (AASHTO 1993), the MEPDG requires more than 100 input parameters to be defined. When using the MEPDG, it is imperative that the design inputs be accurately defined for a more reliable final design. It is the overarching goal of this project, to assist the PennDOT in establishing the appropriate inputs for use in the MEPDG.

To accomplish this goal, WO#13 was divided into five different tasks. Task 1, as presented in a separate report submitted to PennDOT in May of 2009 (Gatti, Nassiri et al. 2009), focused on identifying and summarizing the already available and known values for the inputs required for the design when using the MEPDG for Pennsylvania conditions. These values for the design inputs were collected all across the state from various sources such as: PennDOT publications, personnel communication, databases and other sources. The accuracy and validity of this data was also investigated in this task. Task 2 for WO#13, presented in a report submitted to the PennDOT in August of 2009 (Nassiri, Vandenbossche et al. 2009), consisted of performing a sensitivity analysis of the design software (close to 6,000 runs of the MEPDG software). The purpose of this task was to define the critical design inputs that were both not yet well established under Task 1 and that had a significant influence on the MEPDG predicted distresses for jointed plain concrete pavements (JPCP). Some of the inputs that were identified as needing to be better defined include:

- Portland cement concrete (PCC) mechanical properties:
  - Modulus of rupture (MR)
  - Elastic modulus (Ec)
  - Compressive strength (f'_c)
- PCC thermal properties:
  - Coefficient of thermal expansion (CTE)
- PCC drying shrinkage
- The built-in gradient,

The design inputs that were not defined yet for Pennsylvania conditions, and identified as sensitive through the sensitivity analysis performed in Task 2, were established in the field or in the laboratory in Task 4 for WO#13. A comprehensive plan of the field and laboratory work was presented to the PennDOT in the Task 3 report (Nassiri, Ramirez et al. 2009). It was suggested in Task 3 that four different JPCP sections in Pennsylvania be instrumented to establish the built-in temperature difference in the slabs. The PCC material properties from these four sections was planned to be established thoroughly in the laboratory. This plan was executed in Task 4 for WO#13. Four pavement sections at different locations in western Pennsylvania, as listed in **Error! Reference source not found.**, were selected for instrumentation (in this table, the ATPB and the CTPB are abbreviations for asphalt treated permeable base and cement treated permeable base, respectively.)

Table 1 Four projects instrumented under Task 4 for WO#13.

Project	MPMS	Geographic Limits	Test Section Stationing	Pavement Structure	County	Date of Construction
1	31477	SR 22 (B09) from PA 982 to Auction Barn Rd T-968	From 1136+50 to 1147+60-Westbound	12" PCC/ 4" ATPB/ 6" 2A Subbase	Westmoreland	09/02/2009
2	31478	SR 22 (B10)from Auction Barn Rd T-968 to Indiana County Line	From 939+60 to 955+15-Westbound	14" PCC/ 4" CTPB/ 6" 2A Subbase	Westmoreland	10/8/2009
3	25833	US 22 from T-724/T-910 to SR 2024	From 377+96 to 399+45-Westbound	10" PCC/ 4" CTPB/ 6" 2A Subbase	Indiana	4/29/2010 and 4/30/2010
4	31477	SR 22 (B09) from PA 982 to Auction Barn Rd T-968	From 1115+00 to 1101+65-Eastbound	12" PCC/ 4" ATPB/ 6" 2A Subbase	Westmoreland	5/10/2010

A total of nine slabs were instrumented with thermocouples and vibrating wire (VW) static strain gages in each project. Details on the instrumentation and all the field work performed at the site for each of the projects was presented comprehensively in Task 4 report for WO#13 (Nassiri and Vandenbossche 2010).

The VW strain gages were used to record the temperature and strain changes in the slabs during set. The data from these gages was used in Task 5 for WO#13 to establish the built-in temperature difference in the slabs. Furthermore, the PCC mechanical/thermal properties established in the laboratory were presented for each project in the Task 5 report, delivered in August of 2010.

The current report focuses on establishing reccommended values for each design input in the MEPDG based on findings either in Task 1 or Task4/5. Every design input required for a rigid pavement design when using the MEPDG is discussed individually in this report. However, the primary emphasis for this project was on characterizing the PCC properties when using the MEPDG. The inputs are discussed in the same order that they need to be defined when using the MEPDG. The inputs can be categorized into four major groups: "general inputs", "traffic", "climate" and "pavement structure." Each of these categories will be discussed and expanded upon in the following sections starting with the "general inputs" category.

2. General Inputs/Site Identification and Analysis Parameters

2.1. General Information and Site Identification

General Information

A list of the inputs that need to be defined in this category is provided in *Table 2*. These inputs include the pavement design life, pavement type, construction month and the traffic opening month.

The design life of the pavement is defined in terms of years. This parameter is important because the performance of the pavement is predicted over this period and is evaluated against the design requirements at the end of this period. The design life is suggested to be defined as 20 years when performing a rigid design using the MEPDG. This duration for the design life is implied by the PennDOT Pavement Policy Manual.

Pavement type can be selected as flexible, JPCP or continuously reinforced concrete pavement (CRCP) in the MEPDG. The JPCP design is the focus of this study.

The next input in the category of general information that needs to be defined is the "month of construction." This parameter is used in the MEPDG, to estimate the temperature of the slab at time of final set (TZ). This is a factor based on both the mean monthly temperature (MMT) for the month of construction and the cementitious material content. Considering the paving season in Pennsylvania, the MMT does not vary significantly. Additionally, the cementitious material content is relatively constant for the PennDOT PCC mixtures used for paving. Therefore, the rang of variation for TZ is not significant. Furthermore, this parameter was determined to be insensitive to the predicted performance in the sensitivity analysis performed under Task 2. So any month within the construction season can be selected to define this parameter and any difference between the assumed and actual construction month will not be significant.

The last parameter to be defined in this category is the traffic opening month. The traffic opening month can then be selected based on the amount of time it typically takes from paving to the opening to traffic based on the pavement construction month established above.

Table 2 General inputs to be defined when using the MEPDG for design.

Parameter	Value
Design Life	20 Years
Pavement Type	JPCP
Pavement Construction Month	Varies
Traffic Opening Month	Varies

Site Identification

Site identification is the title of the next group of inputs that needs to be defined. The information required in this category includes a project identification number, location and the milepost for the start and end of the section or section number.

2.2. Analysis Parameters

While the AASHTO 1993 design procedure is based on the drop in the level of serviceability, the design procedure in the MEPDG is based on predicted distress. These distresses or analysis parameters include the International Roughness Index (IRI); percent of slabs cracked and mean joint faulting. A maximum limit value needs to be defined for each of the distresses to establish the required criteria for the design. It is essential for the design criteria to be defined in accordance with the PennDOT requirements for construction and performance, which is based on serviceability. The initial and the terminal serviceability index (TSI) required by the PennDOT for each road classification can be found in the PennDOT's Pavement Policy Manual (Publication 242) and is presented in *Table 3*.

Table 3 TSI for different road classifications, the PennDOT *Pavement Policy Manual.*

Roadway Classification	Initial Serviceability	Terminal Serviceability
Interstate Highways	4.5	3.0
Limited Access and Major Arterial Highways	4.5	3.0
Minor Arterial Highways	4.5	2.5
Collector Highways	4.5	2.5
Local Access Highways	4.5	2.0

To be able to define the required design criteria by the MEPDG based on PennDOT serviceability requirements, the following empirical relation for the present serviceability index (PSI) can be used:

$$PSI=5.41-1.78 \log (1+SV) -0.9 (C+P) \qquad \text{Equation 1}$$

where, C = Medium and high severity transverse cracking (ft/1000 ft)

P = Patching (ft^2/1000 ft^2)

SV = Roughness which can be obtained from the following relation:

$$SV = 2.2704 (IRI)^2 \qquad \text{Equation 2}$$

IRI in this relation is in units of mm/m.

The first analysis parameter to be defined is the initial IRI. The initial IRI defines the as-constructed smoothness of the pavement and can have a significant impact on long-term ride quality of the pavement. This parameter highly depends on the agency specifications. Typical values for this parameter vary between 50 to 100 in/mile. By using Equation 1, for the initial serviceability of 4.5, while cracking and patching are assumed to be zero, the initial IRI is established as 63 in/mile. This typical value can be used for the design when using the MEPDG unless more accurate value is available for Pennsylvania conditions. The most appropriate way to define this parameter is by using the average initial IRI for paving projects in Pennsylvania measured as part of the QA/QC. This parameter needs to be established and documented for newly-constructed pavement sections in the state.

The terminal IRI, on the other hand, defines the maximum allowable roughness of the pavement at the end of its design life. Typically-used values are in the range of 150 to 250 in/mile for the terminal IRI in the MEPDG, depending on the functional class of the roadway. The default value of 172 in/mile is used for the terminal IRI in the MEPDG. This parameter is not predicted but estimated in the MEPDG. It relies on three different factors. These factors are the transverse cracking, the joint faulting and site specifics. See Equation 3. The site factor includes subgrade and climatic factors to account for the roughness caused by shrinking or swelling soils and frost heave conditions.

It has been found that the MEPDG, at times, will erroneously predict a higher IRI than observed in the field when the design includes a subgrade with a large amount of fines in an area with a high freezing index. In these cases, the predicted cracking and faulting can be negligible yet the predicted IRI will exceed the allowable criteria due to the site factor (see Equation 3) included in the prediction equation (Vandenbossche and Mu. 2008). Therefore, it is recommended that the design thickness be established based only on the allowable cracking and faulting criteria.

The empirical relation between the IRI, transverse cracking and joint faulting incorporated into the MEPDG can be used.

$$\mathrm{IRI}=\mathrm{IRI}_I+0.8203\times \mathrm{TC}+0.4417\times \mathrm{SPALL}+1.4929\times \mathrm{TFAULT}+25.24\times SF \qquad \text{Equation 3}$$

where, IRI_I = initial smoothness measured as IRI, in/mile

TC = percentage of slabs with transverse cracking (all severities)

$SPALL$ = percentage of joints with spalling (all severities)

$TFAULT$ = total joint faulting cumulated per mile, in

$$SF = \text{site factor} = \mathrm{Age}\times (1+0.5556\times \mathrm{FI}) \times (1+\mathrm{P}_{200})/1000000 \qquad \text{Equation 4}$$

where, *Age* = pavement age in years

FI = freezing index, ^{o}F days

P200 = percent subgrade material passing sieve #200

The next design criterion that needs to be defined is the percent of slabs with transverse cracking. The allowable level of transverse cracking depends on the individual highway agency's tolerance for the amount of slab cracking over the design period. According to the MEPDG documentation, typical values for the allowable cracking range from 10 to 45 percent, depending on the functional class of the roadway. The default value in the MEPDG is 15 percent. A value of 20 percent is probably more appropriate for lower volume roads. The allowable values must be established by the PennDOT as a function of the roadway classification.

The next parameter in the design criteria to be defined is the joint faulting. This distress in the pavements needs to be limited so that a desirable level of ride quality is achieved. Again, values for the maximum allowable faulting depend on the agency's tolerance for this distress. Typical values range between 0.125 and 0.25 in depending on the functional classification of the roadway. The default value in the MEPDG is 0.125

Reliability is the last parameter in the design criteria that needs to be defined. PennDOT recommends different levels of reliability for different classes of roadways. This is provided in *Table 4*. These values can be used to define the desired level of reliability for the design section when using the MEPDG.

Table 4 Reliability by functional classification from PennDOT Pavement Policy Manual.

Functional Classification	Value
Interstate and other Expressways	95
Arterials	90-95
Collectors	85-90
Locals	70-85

3. Traffic

Traffic data is one of the key data elements required in the MEPDG for the structural design and analysis of pavement sections. This data is used in the MEPDG to estimate the loads that are applied to the pavement and also the frequency of occurrence of loads throughout the design life. On the contrary to the AASHTO design guide, the MEPDG does not use the equivalent single axle load (ESAL) for traffic characterization. Traffic data is defined in the MEPDG in the following categories:

- Traffic volume—base year information
- Traffic volume adjustment factors:
  - Monthly Adjustment
  - Vehicle Class Distribution
  - Hourly Truck Distribution
  - Traffic Growth Factors
- Axle Load Distribution Factors
- General Traffic Inputs
  - Number Axles/trucks
  - Axle Configuration
  - Wheel Base

Typical values are recommended in the MEPDG for each of the above mentioned traffic inputs. These typical values have been established based on the traffic data available for the test sections constructed nationwide under the Long-Term Pavement Performance (LTPP) study.

The Weigh-In-Motion (WIM) and the Automatic Vehicle Classification (AVC) traffic data is collected by PennDOT for roadways in Pennsylvania. Traffic map data is readily available and easily accessible over the internet (PennDOT Internet Traffic Monitoring System [iTMS]). Traffic, data such as the two-way Annual Average Daily Truck Traffic (AADTT), directional factor, truck percent and the sections functional classification can be extracted from this valuable online database.

In an effort to use the traffic data collected by PennDOT to define the traffic requirements in the MPEDG, the University of Pittsburgh research team was supplied with the traffic data from a total of 10 traffic stations for a duration of one week in November of 2008. A list of these 10 stations and their corresponding locations can be found in *Table 5*. Furthermore, these stations were located on the map from Google, which is presented in *Figure 1*. This traffic data was analyzed under Task 1 for WO#13 and proper values were established for each traffic input parameter wherever possible. Based on these results, the values recommended in the MEPDG for each input parameter were evaluated.

Table 5 WIM/AVC stations and locations used in the traffic study.

Station	Symbol on the Map	Location
158	A	I-80E
317	B	I-99
324	C	PA-120
410	D	PA-49
501	E	PA-15
502	F	I-80W
505	G	US-22
506	H	SR-1002
600	I	I-76
700	J	US-422

Figure 1 Traffic stations that provided data for the traffic study.

However, this study was very preliminary and needs to be expanded upon. A significant amount of work is required to establish the proper traffic values for different traffic inputs in the MEPDG. As can be seen in *Figure 1*, the 10 traffic stations used in this preliminary study do not cover the entire state and the lack of stations in the eastern and western areas of the state is noticeable on the map.

Furthermore, the traffic data from the 10 stations used in the study corresponded to only one week in November of 2008. The results established based on this short period of data, can result in a lack of statistical confidence. Moreover, annual and monthly traffic parameters, such as monthly adjustment factors, could not be established based on this data.

Despite all the shortcomings of this preliminary traffic study, a complete comparison of the results with the recommended values used in the MEPDG at input Level 3 was performed in Task 1 for WO#13 (Gatti, Nassiri et al. 2009). The results from this study will be used here to suggest proper values for each traffic input when using the MEPDG.

3.1. Traffic Volume-Base Year Information

When defining the traffic inputs in the MEPDG, the following base year traffic information is required:

- AADTT
- Number of lanes in the design direction
- Percent trucks in the design direction (DDF)
- Percent trucks in the design lane (LDF)
- Vehicle (truck) operational speed

Each of these input parameters will be discussed individually in the following sections.

3.1.1. AADTT

Two-way AADTT is the total volume of truck traffic (the total number of heavy vehicles in federal highway association [FHWA] Classes 4 to 13) passing a point or segment of a road facility to be designed in both directions during a 24-hour period (ARA 2004). As mentioned earlier, this information can be extracted from the iTMS for roadways in Pennsylvania. The AADTT in the design lane for the 10 WIM/AVC used in the traffic study was extracted from the same source and is provided in *Table 6*, as examples.

Table 6 AADTT for each WIM/AVC station.

Location	AADTT
I-80E	6,669
I-99	808
PA-120	867
PA-49	94
PA-15	930
I-80W	6,606
US-22	1,154
SR-1002	73
I-76	7,841
US-422	1,692

When performing a design using the MEPDG, it is suggested that the AADTT be characterized for the design section based on the data available in the iTMS database for this parameter. The AADTT from the iTMS database must be adjusted so that the AADTT reflects the year it is estimated construction will occur.

3.1.2. *Design Direction Factor*

Percent trucks in the design direction, or the DDF, is used to quantify any difference in the overall volume of trucks in two directions. It is usually assumed to be 50 percent when the AADT and AADTT are given in two directions. However, this is not always the case (ARA 2004). The value selected for design should represent the predominant type of truck using the roadway. If detailed site-specific or regional/statewide truck traffic data are unavailable, the truck DDF for the most common truck type (e.g., vehicle Class 9) is suggested for use as the default value for all truck traffic.

The required data to define this factor is available online in the iTMS for roadways in Pennsylvania and can be pulled from this source for any individual project. It is also recommended by the PennDOT in the Pavement Policy Manual (Page 7-2) to use the value of 50 percent for this parameter unless stated otherwise in the iTMS or advised otherwise by the Bureau of Planning and Research. It is suggested to use the iTMS database for defining DDF when using the MEPDG.

3.1.3. *Lane Distribution Factor*

Percent trucks in the design lane, or LDF, accounts for the distribution of truck traffic between the lanes in one direction (ARA 2004). The recommended values in the MEPDG for the LDF representing the most common vehicle class (vehicle Class 9) are as follows:

- Single-lane roadways in one direction, LDF = 1.00.
- Two-lane roadways in one direction, LDF = 0.90.
- Three-lane roadways in one direction, LDF = 0.60.
- Four-lane roadways in one direction, LDF =0.45.

LDF is also defined in the PennDOT Pavement Policy Manual, (Page 7-1), as follows:

- Two-lane highway, LDF = 1.00.
- Four-lane highway, LDF = 0.90.
- Six or more- lane highways, LDF = 0.8.

It is suggested to use the values recommended by PennDOT for designing when using the MEPDG.

3.1.4. Vehicle (Truck) Operational Speed

The vehicle operational speed of trucks or the average travel speed generally depends on many factors, including the roadway classification (Interstate or otherwise), terrain, percentage of trucks in the traffic stream, etc. A value of 60 mph is used as default in the MEPDG design software. However, it is suggested that this parameter be defined for each roadway based on the posted speed for that roadway. This factor is not influential for the design of rigid pavements.

3.2. Truck Traffic Volume Adjustment Factors

3.2.1. Monthly Adjustment Factor

Truck traffic monthly adjustment factors simply represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month (ARA 2004). Traffic volume can vary over different months due to several reasons, such as existence of industry such as, agriculture, manufacturing or mining in the area. As presented in *Table 7*, an equal distribution is defined in the MEPDG for all the vehicles throughout the year.

Table 7 Monthly adjustment factors, default values in the MEPDG.

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12
Jan.	1	1	1	1	1	1	1	1	1
Feb.	1	1	1	1	1	1	1	1	1
Mar.	1	1	1	1	1	1	1	1	1
Apr.	1	1	1	1	1	1	1	1	1
May	1	1	1	1	1	1	1	1	1
June	1	1	1	1	1	1	1	1	1
July	1	1	1	1	1	1	1	1	1
Aug.	1	1	1	1	1	1	1	1	1

Oct.	1	1	1	1	1	1	1	1	1
Nov.	1	1	1	1	1	1	1	1	1
Dec.	1	1	1	1	1	1	1	1	1

Monthly adjustment factors could not be established based on the available WIM/AVC traffic data since the data corresponded to only one week. Therefore, it is suggested that the uniform distribution recommended in the MEPDG be used for the design unless more detailed regional data is available.

In the future, this parameter should be established for different roadways based on long-term traffic data when there is concern that there might be seasonal variability due to local industry.

3.2.2. Vehicle Class Distribution

The next traffic factor to be defined in the MEPDG is the vehicle class distribution for the stream of traffic on the roadway. Just like other factors, typical values are available for this factor in the MEPDG based on nationwide data from the LTPP database.

This factor can be defined based on the functional class of the roadway and the Truck Traffic Classification (TTC) group that best describes the traffic stream on for that roadway classification. The definition for each TTC group can be found in *Table 8*. The default values for the vehicle class distribution for each TTC can be found in *Table 9*.

Using the WIM/AVC data provided by PennDOT, the vehicle class distribution was estimated for each roadway under Task 1 for WO#13. Out of 10 roadway sections, 2 were classified as minor arterial and the remainder of the sites was classified as interstate or principal arterials. The TTC was then designated for each roadway based on the functional class and suggested TTCs in the MEPDG, as listed in *Table 10*. It was concluded that the recommended values used in the MEPDG were able to characterize the vehicle class distributions on the PennDOT roadways.

This study needs to be extended so that the TTC groups specifically represent vehicle/truck traffic for Pennsylvania roadways. This might include modification of the TTC groups used in the MEPDG, as presented in *Table 8*, together with production of new TTC groups that would include all classes of truck used in different regions in the state that are supporting different types of industry, such as mining or farming.

It is also noteworthy that PennDOT uses 10 different traffic pattern groups (TPGs) to classify roads based on their functional classification, geographic area, and urban/rural characteristics. A list of these 10 TPGs are provided in *Table 11*.

As part of future research work, truck class distributions can be established and studied for the TPGs used by the PennDOT. The established truck class distribution for each TPG can then be compared to the corresponding TTC group from the MEPDG. If the nationwide-established distributions from the MEPDG differ from the ones common in Pennsylvania, TTCs in the MEPDG need to be adjusted accordingly.

It is suggested that when performing a design with the MEPDG, use the default values in the design software which are based on the nationwide traffic data until more representative data can be established for the state.

Table 8 Definition and description of each TTC group.

Busses in Traffic Stream	Commodity Being Transferred by the Type of Truck		
	Multi-Trailer	Single Trailers and Single Units	TTC Group No.
Low to none (<2%)	Relatively high amount of multi-trailer trucks (>10%)	Predominantly single-trailer trucks	5
		High percentage of single-trailer trucks, but some single-unit trucks	8
		Mixed truck traffic with a higher percentage of single-trailer trucks	11
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	13
		Predominantly single-unit trucks	16
	Moderate amount of multi-trailer trucks (2-10%)	Predominantly single-trailer trucks	3
		Mixed truck traffic with a higher percentage of single-trailer trucks	7
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	10
		Predominantly single-unit trucks	15
Low to moderate (>2%)	Low to none (<2%)	Predominantly single-trailer trucks	1
		Predominantly single-trailer trucks,	2

		but with a low percentage of single-unit trucks	
		Predominantly single-trailer trucks with a low to moderate amount of single-unit trucks	4
		Mixed truck traffic with a higher percentage of single-trailer trucks	6
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	9
		Mixed truck traffic with a higher percentage of single-unit trucks	12
		Predominantly single-unit trucks	14
Major bus route (>25%)	Low to none (<2%)	Mixed truck traffic with about equal single-unit and single-trailer trucks	17

Table 9 Vehicle class distribution used in the MEPDG at input Level 3.

TTC Group	TTC Description	Vehicle/Truck Class Distribution									
		4	5	6	7	8	9	10	11	12	13
1	Major single-trailer truck route (type I)	1.3	8.5	2.8	0.3	7.6	74.0	1.2	3.4	0.6	0.3
2	Major single-trailer truck route (Type II)	2.4	14.1	4.5	0.7	7.9	66.3	1.4	2.2	0.3	0.2
3	Major single- and multi- trailer truck route (Type I)	0.9	11.6	3.6	0.2	6.7	62.0	4.8	2.6	1.4	6.2
4	Major single-trailer truck route (Type III)	2.4	22.7	5.7	1.4	8.1	55.5	1.7	2.2	0.2	0.4
5	Major single- and multi- trailer truck route (Type II).	0.9	14.2	3.5	0.6	6.9	54.0	5.0	2.7	1.2	11.0
6	Intermediate light and single-trailer truck route (I)	2.8	31.0	7.3	0.8	9.3	44.8	2.3	1.0	0.4	0.3
7	Major mixed truck route (Type I)	1.0	23.8	4.2	0.5	10.2	42.2	5.8	2.6	1.3	8.4
8	Major multi-trailer truck route (Type I)	1.7	19.3	4.6	0.9	6.7	44.8	6.0	2.6	1.6	11.8
9	Intermediate light and single-trailer truck route (II)	3.3	34.0	11.7	1.6	9.9	36.2	1.0	1.8	0.2	0.3
10	Major mixed truck route (Type II)	0.8	30.8	6.9	0.1	7.8	37.5	3.7	1.2	4.5	6.7
11	Major multi-trailer truck route (Type II)	1.8	24.6	7.6	0.5	5.0	31.3	9.8	0.8	3.3	15.3
12	Intermediate light and single-trailer truck route (III)	3.9	40.8	11.7	1.5	12.2	25.0	2.7	0.6	0.3	1.3
13	Major mixed truck route (Type III)	0.8	33.6	6.2	0.1	7.9	26.0	10.5	1.4	3.2	10.3
14	Major light truck route (Type I)	2.9	56.9	10.4	3.7	9.2	15.3	0.6	0.3	0.4	0.3
15	Major light truck route (Type II)	1.8	56.5	8.5	1.8	6.2	14.1	5.4	0.0	0.0	5.7
16	Major light and multi-trailer truck route	1.3	48.4	10.8	1.9	6.7	13.4	4.3	0.5	0.1	12.6
17	Major bus route	36.2	14.6	13.4	0.5	14.6	17.8	0.5	0.8	0.1	1.5

Table 10 Suggested guidance for selecting appropriate TTC groups for different highway functional classifications.

Highway Functional Classification Descriptions	Applicable Truck Traffic Classification Group Number
Principal Arterials – Interstate and Defense Routes	1,2,3,4,5,8,11,13
Principal Arterials – Intrastate Routes, including Freeways and Expressways	1,2,3,4,6,7,8,9,10,11,12,14,16
Minor Arterials	4,6,8,9,10,11,12,15,16,17
Major Collectors	6,9,.12,14,15,17
Minor Collectors	9,12,14,17

Table 11 Description of PennDOT TPGs.

TPG	TPG Definition
1	Urban State
2	Rural Interstate
3	Urban Principal Arterial
4	Rural Principal Arterial
5	Urban Minor Arterial or Collector
6	North Rural Minor Arterials
7	Central Rural Minor Arterials
8	North Rural Collectors
9	Central Rural Collectors
10	Special Recreational

3.2.3. Hourly Distribution Factor

The hourly truck distribution factor (HDF) represents the percentage of the AADTT within each hour of the day (ARA 2004). The values recommended in the MEPDG for this factor are the same for all TTC groups and roadways and are presented in *Figure 2*.

Using the traffic data from the 10 WIM/AVC stations in the state, the hourly distribution factors were estimated for each roadway under Task 1 for WO#13. This study required some minor corrections. It is noteworthy that the corrections were regarding graphing and did not have an effect on the analysis. However, to avoid any confusion, it is provided in complete form here again.

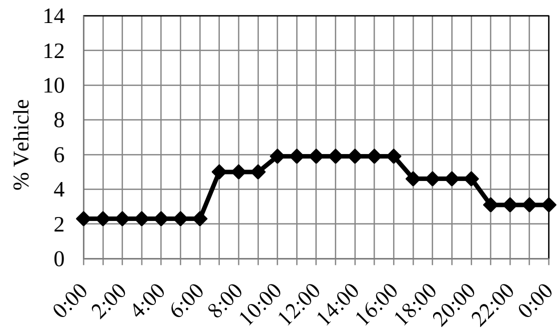

Figure 2 HDF used in the MEPDG as default based on nationwide traffic data.

The 10 roadways in the study were grouped together based on their TTC. The HDF established for each roadway is provided in *Figure 3* to *Figure 7*. Based on *Figure 3* and *Figure 4*, the MEPDG values for the HDF match relatively well with the estimated values for this factor for TTC 1 and 2.

For TTC 11, on the other hand, as seen in *Figure 5*, the MEPDG values and the estimated values agree relatively well except for the duration of between 10:00- and 11:00 AM at which a peak value of 11 percent is seen for the estimated values. This could be an outlier in the data. A longer duration of data can be very beneficial in further investigation of this anomaly.

As seen in *Figure 6*, the estimated and the default values agree very well for TTC 4. Eventually, according to *Figure 7*, the established values for TTC 9 and the default values agree relatively well for most of the hours of the day except for the duration of 2:00- and 3:00 PM at which the MEPDG underestimated the percent of vehicles. Again, this could be an outlier in the data and needs to be further investigated.

Overall, it can be concluded that based on *Figure 3* to *Figure 7*, the default values used for the hourly distribution factors in the MEPDG, estimated based on the traffic data from the LTPP database, agree relatively well with the estimated values for this factor for most of the TTCs.

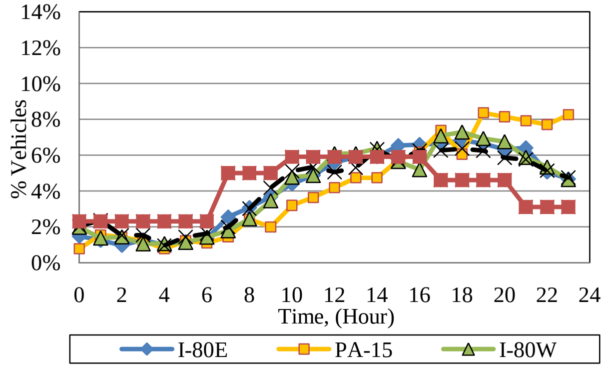

Figure 3 Comparison of HDF established for TTC 1 with the MEPDG default values.

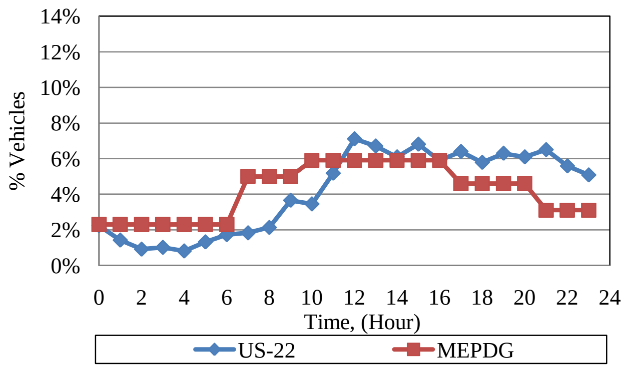

Figure 4 Comparison of HDF determined for TTC 2 with the MEPDG default values.

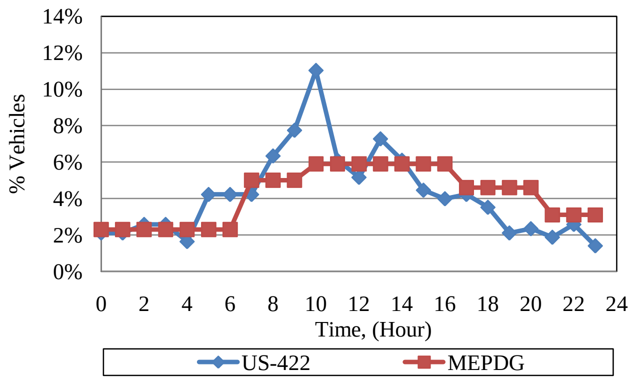

Figure 5 Comparison of HDF determined for TTC 11 with the MEPDG default values.

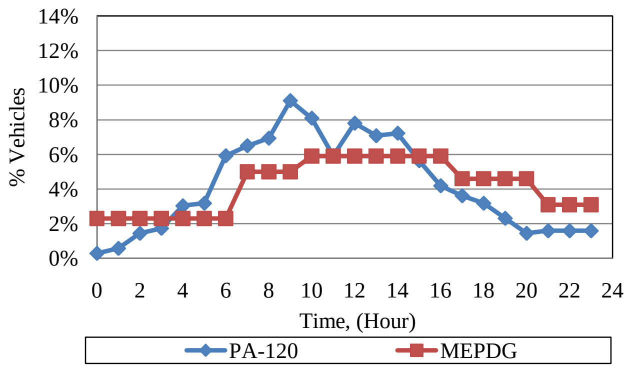

Figure 6 Comparison of HDF determined for TTC 4 with the MEPDG default values.

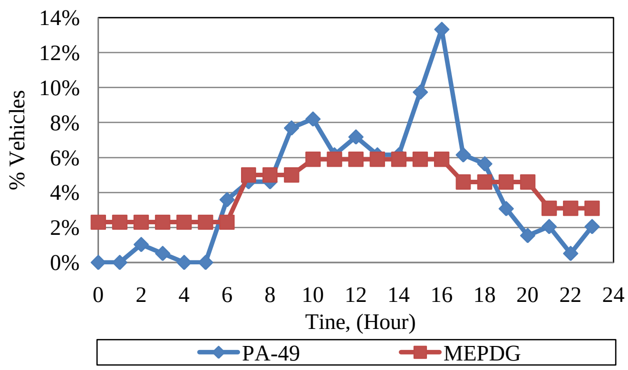

Figure 7 Comparison of HDF determined for TTC 9 with the MEPDG default values.

The HDF is also available in the PennDOT online traffic database, iTMS, for different TPGs. This information is provided in *Table 12*. *Figure 8* presents this data together with the HDF from the MEPDG. Based on this figure, the HDF from the MEPDG appears to be lower than the HDFs recommended by the PennDOT for all the TPGs except for TPG 1 and 2. The average HDF recommended by the PennDOT for all TPGs is also presented in *Table 12* and *Figure 8*. It is suggested that this average value be used for the design if the corresponding TPG is unknown for the section, otherwise the HDF corresponding to the TPG of the design section can be used.

Table 12 PennDOT HDF for each TPG.

TPG 1&2	TPG 3&4	TPG 5&6&7	TPG 8&9	Average of All TPGs
2.86%	1.19%	0.82%	0.96%	1.46%
2.63%	1.12%	0.74%	0.84%	1.33%
2.55%	1.20%	0.80%	0.89%	1.36%
2.64%	1.48%	0.99%	1.10%	1.55%
2.87%	2.03%	1.47%	1.59%	1.99%
3.28%	3.17%	2.68%	2.78%	2.98%
3.94%	4.93%	4.97%	4.91%	4.69%
4.37%	6.24%	6.82%	6.66%	6.02%
4.65%	6.74%	7.15%	6.81%	6.34%
4.93%	6.66%	6.78%	6.55%	6.23%
5.15%	6.75%	6.78%	6.62%	6.33%
5.23%	6.78%	6.90%	6.67%	6.40%
5.20%	6.71%	6.83%	6.65%	6.35%
5.26%	6.75%	6.90%	6.74%	6.41%
5.33%	6.87%	7.27%	7.13%	6.65%
5.31%	6.67%	7.39%	7.39%	6.69%
5.16%	5.81%	6.39%	6.37%	5.93%
4.88%	4.80%	5.14%	5.12%	4.99%
4.58%	3.67%	3.73%	3.88%	3.97%
4.32%	2.84%	2.84%	3.03%	3.26%
4.07%	2.39%	2.29%	2.50%	2.81%
3.89%	2.04%	1.82%	1.99%	2.44%
3.63%	1.71%	1.41%	1.56%	2.08%
3.26%	1.45%	1.11%	1.25%	1.77%
100.00%	100.00%	100.00%	100.00%	100.00%

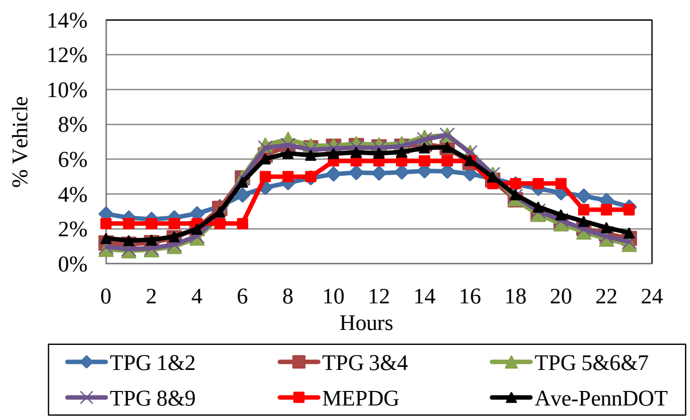

Figure 8 PennDOT HDF for different TPGs vs. the MEPDG HDF.

3.2.4. Traffic Growth Factor

Traffic growth factors (TGF) at a particular site or segment are best estimated when data from a continuous traffic count is available. Three different functions, no-growth, linear growth and compound growth, are incorporated into the MEPDG for estimating the growth factor. These functions are as follows:

$$AADTT_x = 1.0 * AADTT_{base\text{-}year} \quad \text{Equation 5}$$

$$AADTT_x = GR * AGE + AADTT_{base\text{-}year} \quad \text{Equation 6}$$

$$AADTT_x = AADTT_{base\text{-}year} * (GR)^{AGE} \quad \text{Equation 7}$$

GR in Equation 6 and 7 is the growth rate. GR is defined on a yearly base in the iTMS for different TPGs. This is presented in *Table 13*.

Table 13 Multi-year percent change in the annual traffic extracted from iTMS.

TPG	Average Percent Yearly Change from Year 2004 to 2009
1	2.42%
2	2.56%
3	1.01%
4	1.15%
5	1.01%
6	1.16%
7	1.16%
8	1.16%
9	1.16%
10	1.11%
Statewide	1.39%

It is suggested that the compound growth function, Equation 7, with the GR corresponding to the TPG of the design section from *Table 13* be used.

3.3. Axle Distribution Factors

The axle load distribution factors represent the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (Classes 4 through 13). A definition of load intervals for each axle type is provided below:

- Single axles – 3,000 lb to 40,000 lb at 1,000-lb intervals.
- Tandem axles – 6,000 lb to 80,000 lb at 2,000-lb intervals.
- Tridem and quad axles – 12,000 lb to 102,000 lb at 3,000-lb intervals.

The default values used in the MEPDG only for the single and the tandem axles were summarized in *Table 14* and *Table 17* for each truck classification and also for each load level.

The data provided by the PennDOT for the 10 traffic stations in Pennsylvania were broken down and analyzed thoroughly to establish the axle distribution factors for single, tandem and tridem axles and also for all different truck classifications. This study was performed under Task 1 for WO#13 and the details can be found in (Gatti, Nassiri et al. 2009). To avoid repetition, only the results of this study will be provided here.

It was concluded in this study, that the established axle load distributions for the majority of the 10 stations followed the same trend as the one used in the MEPDG. The distributions established using the PennDOT data also showed a frequency peak at around the same load as the default values in the MEPDG. For some of the cases, the values established using the PennDOT data were very close to the MEPDG distributions, except for the fact that the estimated values were mostly higher both in the frequency and the load value. It should be noted that for the same vehicle class, some sites, such as PA-49 and SR-1002, did not follow the distribution trends seen for the other stations. It is suspected that these anomalies are due to the limited data available and/or the quality of the AVC sensors in service at the site. Furthermore, the MEPDG default values for load axle distribution is the same for all the TTCs. This could also explain some of the high values seen in the percentage of loads in different TTCs. The values recommended by the MEPDG should be used when performing the design using the MPEDG until this factor is further investigated and established properly for the state.

Table 14 Single-axle load distribution values (percentages) for each vehicle/truck class recommended in the MEPDG.

Mean Axle Load, (lbs)	4	5	6	7	8	9	10	11	12	13
3000	1.80	10.03	2.47	2.14	11.62	1.74	3.64	3.55	6.68	8.88
4000	0.96	13.19	1.78	0.55	5.36	1.37	1.24	2.91	2.29	2.67
5000	2.91	16.40	3.45	2.42	7.82	2.84	2.36	5.19	4.87	3.81
6000	3.99	10.69	3.95	2.70	6.98	3.53	3.38	5.27	5.86	5.23
7000	6.80	9.21	6.70	3.21	7.98	4.93	5.18	6.32	5.97	6.03
8000	11.45	8.26	8.44	5.81	9.69	8.43	8.34	6.97	8.85	8.10
9000	11.28	7.11	11.93	5.26	9.98	13.66	13.84	8.07	9.57	8.35
10000	11.04	5.84	13.55	7.38	8.49	17.66	17.33	9.70	9.95	10.69
11000	9.86	4.53	12.12	6.85	6.46	16.69	16.19	8.54	8.59	10.69
12000	8.53	3.46	9.47	7.41	5.18	11.63	10.30	7.28	7.09	11.11
13000	7.32	2.56	6.81	8.99	4.00	6.09	6.52	7.16	5.86	7.34
14000	5.55	1.92	5.05	8.15	3.38	3.52	3.94	5.65	6.58	3.78
15000	4.23	1.54	2.74	7.77	2.73	1.91	2.33	4.77	4.55	3.10
16000	3.11	1.19	2.66	6.84	2.19	1.55	1.57	4.35	3.63	2.58
17000	2.54	0.90	1.92	5.67	1.83	1.10	1.07	3.56	2.56	1.52
18000	1.98	0.68	1.43	4.63	1.53	0.88	0.71	3.02	2.00	1.32
19000	1.53	0.52	1.07	3.50	1.16	0.73	0.53	2.06	1.54	1.00
20000	1.19	0.40	0.82	2.64	0.97	0.53	0.32	1.63	0.98	0.83
21000	1.16	0.31	0.64	1.90	0.61	0.38	0.29	1.27	0.71	0.64
22000	0.66	0.31	0.49	1.31	0.55	0.25	0.19	0.76	0.51	0.38
23000	0.56	0.18	0.38	0.97	0.36	0.17	0.15	0.59	0.29	0.52
24000	0.37	0.14	0.26	0.67	0.26	0.13	0.17	0.41	0.27	0.22
25000	0.31	0.15	0.24	0.43	0.19	0.08	0.09	0.25	0.19	0.13
26000	0.18	0.12	0.13	1.18	0.16	0.06	0.05	0.14	0.15	0.26
27000	0.18	0.08	0.13	0.26	0.11	0.04	0.03	0.21	0.12	0.28
28000	0.14	0.05	0.08	0.17	0.08	0.03	0.02	0.07	0.08	0.12
29000	0.08	0.05	0.08	0.17	0.05	0.02	0.03	0.09	0.09	0.13
30000	0.05	0.02	0.05	0.08	0.04	0.01	0.02	0.06	0.02	0.05
31000	0.04	0.02	0.03	0.72	0.04	0.01	0.03	0.03	0.03	0.05
32000	0.04	0.02	0.03	0.06	0.12	0.01	0.01	0.04	0.01	0.08
33000	0.04	0.02	0.03	0.03	0.01	0.01	0.02	0.01	0.01	0.06
34000	0.03	0.02	0.02	0.03	0.02	0.01	0.01	0.01	0.01	0.02
35000	0.02	0.02	0.01	0.02	0.02	0.00	0.01	0.01	0.01	0.01
36000	0.02	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01
37000	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.00	0.01	0.01
38000	0.01	0.01	0.01	0.01	0.00	0.00	0.01	0.02	0.01	0.01
39000	0.01	0.00	0.01	0.01	0.01	0.00	0.01	0.01	0.00	0.01
40000	0.01	0.00	0.01	0.01	0.00	0.00	0.04	0.02	0.00	0.00
41000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 15 Tandem-axle load distribution values (percentages) for each vehicle/truck class recommended in the MEPDG.

Mean Axle Load, (lbs)	4	5	6	7	8	9	10	11	12	13
6000	5.88	7.06	5.28	13.74	18.95	2.78	2.45	7.93	5.23	6.41
8000	1.44	35.42	8.42	6.71	8.05	3.92	2.19	3.15	1.75	3.85
10000	1.94	13.23	10.81	6.49	11.15	6.51	3.65	5.21	3.35	5.58
12000	2.73	6.32	8.99	3.46	11.92	7.61	5.40	8.24	5.89	5.66
14000	3.63	4.33	7.71	7.06	10.51	7.74	6.90	8.88	8.72	5.73
16000	4.96	5.09	7.50	4.83	8.25	7.00	7.51	8.45	8.37	5.53
18000	7.95	5.05	6.76	4.97	6.77	5.82	6.99	7.08	9.76	4.90
20000	11.58	4.39	6.06	4.58	5.32	5.59	6.61	5.49	10.85	4.54
22000	14.20	2.31	5.71	4.26	4.13	5.16	6.26	5.14	10.78	6.45
24000	13.14	2.28	5.17	3.85	3.12	5.05	5.95	5.99	7.24	4.77
26000	10.75	1.53	4.52	3.44	2.34	5.28	6.16	5.73	6.14	4.34
28000	7.47	1.96	3.96	6.06	1.82	5.53	6.54	4.37	4.93	5.63
30000	5.08	1.89	3.21	3.68	1.58	6.13	6.24	6.57	3.93	7.24
32000	3.12	2.19	3.91	2.98	1.20	6.34	5.92	4.61	3.09	4.69
34000	1.87	1.74	2.12	2.89	1.05	5.67	4.99	4.48	2.74	4.51
36000	1.30	1.78	1.74	2.54	0.94	4.46	3.63	2.91	1.73	3.93
38000	0.76	1.67	1.44	2.66	0.56	3.16	2.79	1.83	1.32	4.20
40000	0.53	0.38	1.26	2.50	0.64	2.13	2.24	1.12	1.07	3.22
42000	0.52	0.36	1.01	1.57	0.28	1.41	1.69	0.84	0.58	2.28
44000	0.30	0.19	0.83	1.53	0.28	0.91	1.26	0.68	0.51	1.77
46000	0.21	0.13	0.71	2.13	0.41	0.59	1.54	0.32	0.43	1.23
48000	0.18	0.13	0.63	1.89	0.20	0.39	0.73	0.21	0.22	0.85
50000	0.11	0.14	0.49	1.17	0.14	0.26	0.57	0.21	0.22	0.64
52000	0.06	0.20	0.39	1.07	0.11	0.17	0.40	0.07	0.23	0.39
54000	0.04	0.06	0.32	0.87	0.06	0.11	0.38	0.13	0.20	0.60
56000	0.08	0.06	0.26	0.81	0.05	0.08	0.25	0.15	0.12	0.26
58000	0.01	0.02	0.19	0.47	0.03	0.05	0.16	0.09	0.07	0.18
60000	0.02	0.02	0.17	0.49	0.02	0.03	0.15	0.03	0.19	0.08
62000	0.10	0.01	0.13	0.38	0.06	0.02	0.09	0.06	0.09	0.14
64000	0.01	0.01	0.08	0.24	0.02	0.02	0.08	0.01	0.04	0.07
66000	0.02	0.01	0.06	0.15	0.02	0.02	0.06	0.01	0.02	0.08
68000	0.01	0.00	0.07	0.16	0.00	0.02	0.05	0.01	0.04	0.03
70000	0.01	0.02	0.04	0.06	0.00	0.01	0.11	0.00	0.12	0.01
72000	0.00	0.01	0.04	0.13	0.00	0.01	0.04	0.00	0.01	0.04
74000	0.00	0.00	0.02	0.06	0.00	0.01	0.01	0.00	0.01	0.02
76000	0.00	0.00	0.01	0.06	0.00	0.00	0.01	0.00	0.01	0.04
78000	0.00	0.00	0.00	0.02	0.00	0.00	0.01	0.00	0.01	0.02
80000	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.08
82000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

3.4. General Traffic Data

General traffic inputs include mean wheel location, traffic wander standard deviation, and design lane width, number of axles per truck, axle configuration and wheelbase. The values recommended to be used in the MEPDG, established based on nationwide traffic data, is suggested to be used for most of these parameters. These values are presented in *Table 16*, I, *Table 18* and *Table 19* and will be discussed individually as follows.

Starting from *Table 16*, the first variable to be discussed is the mean wheel location. This factor is defined as the distance from the outer edge of the wheel to the pavement marking. Based on nationwide data, this factor is defined as 18 inch in the MEPDG.

The next parameter is the traffic wander standard deviation. This is the standard deviation of the lateral traffic wander. The wander is used in the MPEDG to determine the number of axle load applications over a point for predicting distresses. A national value of 10 in is used in the MEPDG at input Level 3. Since more accurate data representing Pennsylvania conditions is unavailable for this parameter, a value of 10 in should be used when performing design using the MEPDG.

The next factor in *Table 16* is the design lane width. This parameter refers to the actual traffic lane width, as defined by the distance between the lane markings on either side of the design lane. This factor may or may not be equal to the slab width. The default value for standard-width lanes is 12 ft. It is suggested that this value be used for the design in Pennsylvania.

Table 16 Typical values for general traffic inputs used in the MEPDG.

Parameter	Value
Mean Wheel Location	18
Traffic Wander Standard Deviation	10
Design Lane Width	Typically 12 ft

The next input parameter is the number of axle types per truck class. This input represents the average number of axles for each truck class (Class 4 to 13) for each axle type (single, tandem, tridem, and quad). The values used in the MEPDG at input Level 3 established based on analyzing the data from the LTPP database is provided in Table 18. Until more accurate data established based on direct analysis of site-specific traffic data is available for Pennsylvania, the values provided in *Table 17* should be used in design.

Table 17 Typical values for general traffic inputs used in the MEPDG, continued.

Truck Class	Single	Tandem	Tridem	Quad
Class 4	1.62	0.39	0	0
Class 5	2	0	0	0
Class 6	1.02	0.99	0	0
Class 7	1	0.26	0.83	0
Class 8	2.38	0.67	0	0
Class 9	1.13	1.93	0	0
Class 10	1.19	1.09	0.89	0
Class 11	4.29	0.26	0.06	0
Class 12	3.52	1.14	0.06	0
Class 13	2.15	2.13	0.35	0

A series of inputs are needed to define the configurations of the typical tire and axle loads. These inputs together with their corresponding values based on the MEPDG recommendations are provided in *Table 18*. The values from this table should be used in design when using the MEPDG, until more accurate data is established for each.

Table 18 Typical values for general traffic inputs used in the MEPDG, continued.

Parameter	Value
Average Axle Width (ft)	8.5
Dual Tire Spacing (in)	12
Tire Pressure (psi)	120
Tandem Axle Spacing (in)	51.6
Tridem Axle Spacing (in)	49.2
Quad Axle Spacing (in)	49.2

A series of data elements are needed to describe the details of the vehicle wheelbase for use in computing pavement responses in the MEPDG. These data elements can be obtained directly from manufacturer's databases or measured directly in the field. Typical values are provided for each of the following elements at input Level 3 in the MEPDG. See *Table 19*. However, site-specific values may be used, if available. It is suggested that the recommended value in the MEPDG, as provided in *Table 19*, be used for the design in Pennsylvania until more accurate data is available.

Table 19 Typical values for general traffic inputs used in the MEPDG, continued.

Parameter	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent Trucks (%)	33	33	34

4. Climatic Stations in the MEPDG

Environmental conditions have a significant impact on the performance of rigid pavements. Factors such as ambient temperature, relative humidity, precipitation, freeze-thaw cycles and depth to the water table can influence the performance of the pavement at different time of the year. The change in temperature and moisture profiles in the pavement structure is considered in the MEPDG through a comprehensive climatic modeling tool named the Enhanced Integrated Climatic Model (EICM).

The EICM is a one-dimensional coupled heat and moisture flow program that simulates changes in the behavior and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation (ARA 2004). To do so, the EICM uses climatic databases containing hourly data for sunshine, rainfall, wind speed, air temperature, and relative humidity. The data from more than 800 climatic stations in the United States is already incorporated into the MEPDG to be selected based on the project location. As it was studied thoroughly in Task 1 for WO#13, 23 climatic databases are available in the MEPDG for the state of Pennsylvania.

One issue that needs to be considered about the climatic databases is the duration of the available data. Since the data available in the database will be used to generate the required climatic data for the whole design life of the pavement, it is essential that the available database include a minimum of 5 years worth of climatic data. *Table 20* presents a list of all the climate stations in Pennsylvania that have provided climatic data for the MEPDG. The duration of the data from each of these stations is also provided in this table. Based on *Table 20*, all the climatic stations appear to include at least 5 years of data. A record of climatic data for duration of 5 years is essential and is the minimum requirement for performing a design using the MEPDG. However, 10 years of climatic data is ideal and provides more confidence in the accuracy of the design.

Table 20 Climatic stations available in the MEPDG for Pennsylvania.

Climate Station	Start Date of Data	End Date of Data	Duration of Data, Years
Allentown-Lehigh Valley International Airport	7/1/1996	2/28/2006	9.67
Altoona-Blair International Airport	7/1/1999		6.67
Bradford Regional Airport	1/1/1997		9.16
Clearfield-Lawrence Airport	2/1/2000		6.08
Doylestown Airport	8/1/1999		6.58
Du Bois-Jefferson County Airport	7/1/2000		5.67
Erie International/T. Ridge Field Airport	7/1/1996		9.67
Harrisburg International Airport	2/1/2001		5.08
Harrisburg-Capital City Airport	1/1/2000		6.16
Johnstown-Cambria County Airport	9/1/2000		5.50
Lancaster Airport	3/1/1999		7.00
Port Meadville Airport	7/1/1997		8.67
Pocono Mountains Municipal Airport	1/1/1999		7.16
Philadelphia International Airport	7/1/1996		9.67
NE Philadelphia Airport	7/1/1996		9.67
Pittsburgh-Allegheny County Airport	2/1/1999		7.08
Pittsburgh International Airport	7/1/1996		9.67
Pottstown Limerick Airport	3/1/1999		7.00
Reading Regional Airport	2/1/1999		7.08
Selinsgrove-Penn Valley Airport	9/1/1997		8.50
Wilkes-Barre/Scranton International Airport	7/1/1996		9.67
Williamsport Regional Airport	7/1/1996		9.67
York Airport	9/1/1997		8.50

It is suggested that when performing a design in Pennsylvania using the MEPDG, the closest climatic station that would be most representative of the climatic conditions of the site for the design section be selected. If no climatic station is available close to the site of the design section, the interpolation option from the MEPDG can be used. With this option, a virtual climatic database is generated based on interpolation of the data from several climatic stations in the vicinity of the location of interest.

5. Pavement Structure

The pavement structure section will consist of three major discussions including "pavement design features," "slab/base interface properties" and "pavement layers characteristics." The "pavement design features" category focuses on defining the joint and shoulder properties, while the "slab/base interface properties" category includes defining properties of the base such as erodibility and also the level of friction at the base/slab interface. Other material properties of the base layer will be defined under the category of "pavement layers properties." This order is followed because it is the same order that the inputs need to be defined when using the MEPDG. The input parameters in each of these three categories will be discussed individually starting with the "pavement design features."

5.1. Pavement Design Features

The design features for a JPCP structure include inputs such as the joint spacing, sealant type, dowel diameter and spacing and the edge support. Each of these parameters will be discussed individually in the following sections.

5.1.1. Joint Spacing

A 15-ft joint spacing should be used in accordance with PennDOT Design Standards.

5.1.2. Sealant Type

The next parameter to be defined in the category of design features is the sealant type. According to the PennDOT Publication 408, Section 705, neoprene seal or silicon joint sealing can be used for joint filling. When defining this parameter in the MEPDG, two options are available: "preformed" and "other" (including liquid, silicone and no sealant). The sealant type selected has little effect on the predicted performance. Therefore, a difference between the design and as-built conditions would not affect the design thickness of the slab.

5.1.3. Dowel Bars

Two input parameters regarding dowel bars need to be defined when using the MEPDG: 1. dowel diameter and 2. dowel spacing. For dowel diameter, based on Publication 72M, RC-20M, Page 1 of 3, value of 1.25 in should be used for slabs thinner than 10 in, while value of 1.5 in should be used for slabs thicker than 10 in. Dowel spacing, on the other hand, should be defined as 12 in, based on this publication.

5.1.4. Edge Supports

The type of shoulder also needs to be defined in this section. Tied PCC shoulders and widened slabs can significantly improve JPCP performance by reducing critical deflections and stresses along the edge. The shoulder type also affects the amount of moisture infiltration into the pavement structure (ARA 2004). When the edge support is defined as a tied PCC shoulder, long-term load transfer efficiency (LTE) of the shoulder is required to be defined as well. This parameter has been recommended in the MEPDG to be defined as 40 percent unless more accurate data is available.

5.2. Base/Slab Interface Properties

Pavement structure characteristics such as base/slab friction and base or subbase erosion can influence the initiation and propagation of distresses in the pavement according to the MEPDG documentation (ARA 2004). These parameters will be discussed individually in the next two sections.

5.2.1. Erodibility

The base material is categorized into 5 different classes in the MEPDG. This is done based on the material long-term erodibility behavior; starting from Class 1, defined as extremely erosion resistant and ending with Class 5, defined as very erodible. Base material such as a lean concrete and an asphalt treated permeable base (ATPB) fall under Class 1, cement treated permeable base is under Class 2 and finally unbounded material such as gravel and untreated subgrade soil fall into the last categories and are among the poorest base materials.

A PennDOT ATPB and a PennDOT concrete treated permeable base (CTPB) would fall under Class 1: extremely erosion resistant.

5.2.2. PCC/Base Interface Friction and Loss of Full Friction (Age in Months)

The stabilized base, (an ATPB or CTPB), is either considered fully bonded with full friction contact or fully unbonded with zero friction contact. The stabilized base layer, if bonded to the PCC slab, can provide significant structural contribution to the performance of the slab. Furthermore, the JPCP design procedure includes the modeling of the changes in the interface bond condition over time. This is accomplished by specifying the pavement age at which the debonding occurs. Up to the debonding age, the slab-base interface is assumed fully bonded; after the debonding age, the interface is assumed fully unbonded. The design input is the pavement age at debonding, in months. In general, specifying the debonding age greater than 5 years (60 months) is not recommended and has not been used in calibration of the MEPDG (ARA 2004).

It is suggested that, as recommended in the MEPDG, the slab/base interface be defined as full friction for treated base layers and the age of debonding be defined as 60 months since these values were used when the national calibration of performed. Thes value should be used by PennDOT until more accurate data representing the typical bond life for pavements in Pennsylvania can be further evaluated and a local calibration has been performed.

5.3. Pavements Layers Properties

Next design inputs to be defined in the MEPDG are the pavement structure and each layer's corresponding material properties. Rigid pavement structures in Pennsylvania commonly consists of a PCC slab over a stabilized base on top of a 2A subbase (separator) on a subgrade with a resilient modulus of higher than 7500 psi.

The typical material properties used for each of these layers in the MEPDG at input Level 3 will be presented below following the same order that they are defined when using the MEPDG. The validity and accuracy of the default values for the material properties will be evaluated based on the corresponding established values in the laboratory or in the field, wherever possible.

5.3.1. JPCP Slab

Design inputs in the JPCP category that need to be defined include three different groups of inputs: “general properties,” “thermal properties” and “mix properties.” Since the primary focus of WO#13 was to establish the required PCC properties, most of these parameters were established in the laboratory for specimens from the four different paving projects included in this study. Comparisons between the laboratory-established values and the typical values available in the MEPDG, established based on data from the LTPP sections, will be made in the following sections. Based on this evaluation, proper values will be suggested for use in the MEPDG for each parameter.

5.3.1.1. General Properties

Layer Thickness/Unit Weight and Poisson’s Ratio (υ)

The PCC layer thickness is the first JPCP design input in the MEPDG that needs to be defined. According to the PennDOT Pavement Policy Manual, Table 8.3, Page 8-6, the PCC slab minimum thickness can vary between 7 in. and 9 in. The minimum thickness of the slab depends on the roadway functional class. This minimum thickness is 7 in. for local roads, 8 in. for collectors and 9 in. for interstates. The maximum allowable PCC slab thickness based on this table is 20 in. It is suggested that the minimum thicknesses recommended by the PennDOT for each road class be used for the initial run when using the MEPDG for the design.

The unit weight is another PCC property that needs to be defined in the MEPDG. According to the MEPDG documentation, this property of the PCC can have a value of between 140 and 160 lb/ft^3. This property of the PCC was measured and established in the field for the PCC mixtures used for the four instrumented projects. This test was performed in the field and in accordance to AASHTO T121/ASTM C138. The test results showed that the average value for this parameter was established at 152-, 150-, 149- and 148 lb/ft^3 for Projects 1, 2, 3 and 4, respectively. The corresponding standard deviation for each of the projects was 0-, 1-, 1.2- and

1.5 lb/ft^3, respectively. The design thickness is not sensitive to this parameter and did not show significant variability in different PCC mixtures from the instrumented projects. Therefore, when using the MEPDG, a typical value of 150 lb/ft^3 is suggested to be used for the design.

The Poisson's ratio of the PCC is the next PCC property that needs to be defined when using the MEPDG. This parameter is an input to the structural response computation models in the MEPDG, although its effect on computed pavement responses is not significant. The υ for the PCC may be determined simultaneously with the determination of the elastic modulus, in accordance to ASTM C469. Based on the MEPDG documentation (ARA 2004), this parameter can have a value of anywhere between 0.15 and 0.25 for newly constructed or existing pavements. This parameter was established in the laboratory for the PCC mix used in each of the instrumented projects. The average υ was established as 0.22, 0.19, 0.2 and 0.22 for Projects 1, 2, 3 and 4, respectively. However, it should be noted that the values established in the laboratory for this parameter correspond to saturated specimens. This can result in a slight overestimation of υ for the actual PCC slab exposed to field conditions. Therefore, it is suggested that the value of 0.18, that is very typical for PCC slabs, be used in the design when using the MEPDG.

5.3.1.2. Thermal Properties

Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) of the PCC is a critical parameter that needs to be defined properly when performing a design using the MEPDG. This parameter is defined as the change in unit length per degree of temperature change (ARA 2004). The MEPDG uses this parameter to calculate the unrestrained change in the slab length. The CTE of the concrete is based primarily on the type and quantity of the coarse aggregate used in the concrete. The values recommended in the MEPDG for this parameter are based on CTEs established for cores pulled from different LTPP sections across the state. A list of LTPP sections in Pennsylvania for which the PCC CTE was measured is provided in *Table 21*. These sections are also projected on the map of the state in *Figure 9*.

Based on *Table 21*, the most commonly-used aggregate in the state is limestone. As seen in *Table 21*, the CTE for the PCC with this type of rock varies between 4.8 and 6.2 με/ºF. In addition to limestone, dolomite has also been used at two sites across the state. The CTE established for these two sites are 6.3 and 6.2 με/ºF.

*Table 21 Measured CTE for LTPP sections in Pennsylvania (**NOT CORRECTED!**).*

SHRP_ID	Symbol on the Map	Aggregate Type	Average CTE (10^{-6}/ºF)	Number of Tests	Standard Deviation
1598	A	Crushed stone, Sedimentary, Limestone	5.4E-06	3	5.6E-08
0659	B	NA	5.6E-06	1	0
1606	C	NA	6.6E-06	2	4.3E-07
1610	D	Crushed stone, Sedimentary, Limestone	5.3E-06	6	6.3E-07
1613	E	Crushed stone, Sedimentary, Dolomite	6.3E-06	4	1.8E-07
1614	F	Crushed stone, Sedimentary, Limestone	5.0E-06	7	2.2E-07
1623	G	Crushed stone, Sedimentary, Limestone	5.3E-06	8	5.5E-07
1627	H	Crushed stone, Sedimentary, Limestone	4.8E-06	2	3.9E-07
1690	I	Crushed stone, Sedimentary, Dolomite	6.2E-06	3	6.3E-07
1691	J	Crushed stone, Igneous (extrusive), Basalt	5.9E-06	5	5.2E-07
3044	K	Crushed stone, Sedimentary, Limestone	6.4E-06	7	4.2E-07
7037	L	Crushed stone, Sedimentary, Limestone	5.0E-06	5	7.3E-07

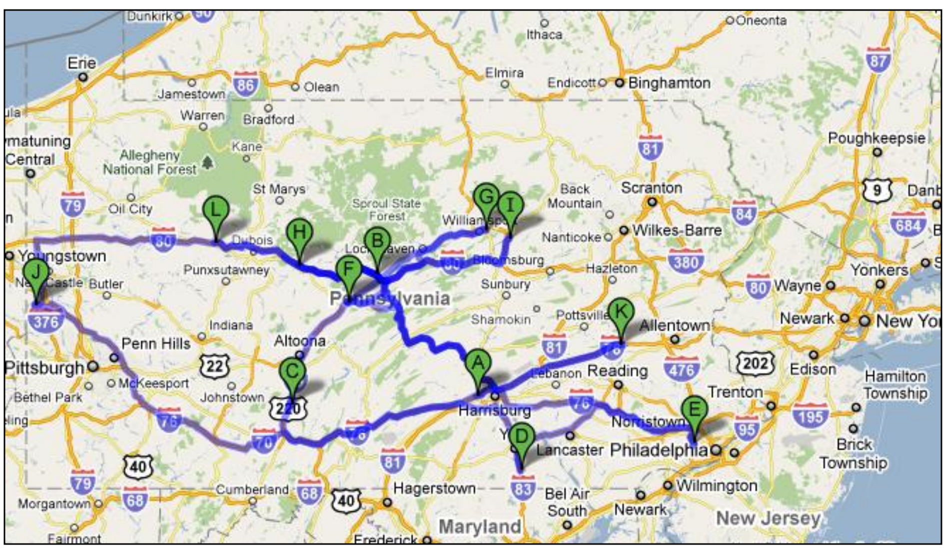

Figure 9 LTPP sites used in PCC CTE characterization for Pennsylvania.

Due to its significance, this parameter was established in the laboratory for the PCC used for each of the instrumented projects. The aggregate used in all four projects was limestone. This aggregate was from a pit in Blairsville for three of the projects and for one of the projects was from a pit in New Alexandria. Both these pits are located in Allegheny, East of Pittsburgh. Therefore, not much variability is expected in the CTE for limestone from these two pits. The length changes of cylindrical PCC test samples were established by embedding Geokon Model 4200 vibrating wire (VW) static strain gages. The test was performed based on AASHTO TP60. The average CTE established for all the projects was 5.2 με/ºF with a standard deviation of 0.09 με/ºF.

The average CTE value established for the PCC mixtures used for the instrumented projects is significantly lower than the range of CTE values established for the LTPP sections in Pennsylvania for the same type of rock. This is because the CTE values used in the MEPDG are most likely erroneous and need adjustments since the CTE of the calibration specimen (304 SS) used to perform the AASHTO TP60 CTE test on the cores from the LTPP sections, has been underestimated and needs to be adjusted to a higher value (Crawford 2010).

However, since the performance models incorporated into the MEPDG are calibrated based on the erroneous CTE values, until the models are recalibrated and updated based on new CTE values, the older and erroneous CTE values should be used when using the MEPDG. The version addressing this issue in scheduled to be released in January 2011. The values in *Table 21* should not be used in the versions of the MEPDG released after January 2011 and the new corrected CTE values should be extracted and used in the design.

Therefore, it is suggested that when using the MEPDG, the values for the CTE be selected from *Table 21* based on the location of the design section. Recommendations for establishing CTE in regions that are not covered in *Table 21* will be provided in Section 5.3.1.3, Aggregate Types.

Furthermore, it is important to establish the CTE values for PCC made of aggregate from different quarries across the state in the future.

Thermal Conductivity and Heat Capacity

The other two thermal properties that need to be characterized for the concrete when using the MEPDG include the thermal conductivity and the heat capacity. Values of 1.25 BTU/hr/ft/°F and 0.28 BTU/lb/°F are recommended in the MEPDG to be used for the thermal conductivity and the heat capacity of the PCC, respectively. These parameters are not significant to the final design when considering variability typically found between different mixes for these two parameters. Therefore, the values suggested in the MEPDG, as mentioned above, should be used to define these parameters.

5.3.1.3. Mix Properties

Cement Type

Three different cement types are allowed to be defined when using the MEPDG. This includes Type I, II and III. According to the PennDOT Publication 408, Section 704, Portland cement, conforming to the optional chemical requirement in AASHTO M 85, and also blended hydraulic cements of Types IS and IP can be used for paving. It is suggested that Type I be used for the design when using the MEPDG. This cement type is most commonly used across the state for paving mixes and the exact cement type that will be used is not typically known at the time of the design.

Cementitious Material Content

The cementitious material content, including the cement plus the fly ash/slag content, is the next parameter that needs to be defined in the MEPDG. According to the PennDOT Publication 408, Section 704.1, Table A, slip form concrete Class AA with a cement factor of between 6.25 and 8.0 bags/yd^3 is recommended for paving.

A typical concrete mixture used for paving in Pennsylvania is Class AA 500 slip form concrete, which includes 500 lbs/yd^3 cement and 88 lb/yd^3 fly ash (equal to 15 percent of the total cementitious materials).

In addition to this concrete mix, Class AA 550 slip form concrete with 100 lb/yd^3 fly ash is occasionally used for paving. Class AA 550 is typically only used if cold weather conditions exist or early breaks are required to allow early loading for accommodating an expeditied construction schedule.

Therefore, it is suggested that the typical value of 588 lb/yd^3 be used for the design when using the MEPDG.

w/c Ratio

The w/c ratio is used in the MEPDG to establish the drying shrinkage of the PCC. The drying shrinkage of the PCC is a significant parameter that affects the performance of the slabs.

According to the PennDOT Publication 408, Table A, the w/c ratio needs to be reduced to under 0.477 when using concrete Class AA. Due to its significance, this parameter was measured in the field for the PCC mixtures used to construct the instrumented projects. This was achieved by performing the microwave oven w/c ratio test. This test was performed at least twice on two different samples from the PCC used to pave each cell in each project. The average w/c ratio established for Project 1, 2, 3 and 4 were 0.45, 0.47, 0.49 and 0.46, respectively. The corresponding standard deviation for each project was 0.04, 0.01, 0.03 and 0.01. It is suggested that the typical value of 0.47 be used for this parameter when using the MEPDG.

Aggregate Type

The aggregate type needs to be defined based on the aggregate sources in different regions of the state. A complete list of the aggregate producers together with their corresponding products was compiled based on the PennDOT Publication No. 34, Bulletin 14. This was provided in Task l report for WO#13 (Gatti, Nassiri et al. 2009).

It is suggested that the aggregate type typical for the region where the pavement is being constructed be used for the design input. The producers' list mentioned above can be used for identifying the regional aggregate sources and consequently the aggregate type for the design section.

It is noteworthy that the proper value of CTE needs to be defined based on the selected aggregate type. Typical ranges are available for CTE of aggregates made of different rock types. The CTE of the aggregate is then combined with the CTE of the paste to produce the CTE of the PCC mixture. This is performed by employing an empirical relation embedded within the MEPDG. More details on this can be found in Task 1 report for WO#13 (Gatti, Nassiri et al. 2009).

PCC Zero-Stress Temperature

The zero-stress temperature is the average temperature in the slabs at the time the concrete sets.. This parameter is significant since it is used to estimate the joint and crack opening. The joints and cracks open when the temperature within the slabs falls below the zero-stress temperature. This parameter is estimated in the MEPDG through an empirical model, which is based on PCC cementitious material content and also the mean monthly temperature of the month of construction.

The accuracy of this model was investigated thoroughly in Task 5 for WO#13 using the PCC mixture properties and also the mean monthly temperature of the month of construction for each of the instrumented projects. The results of this study showed that the empirical model incorporated into the MEPDG was able to predict the temperature at time zero relatively well in all four projects. Therefore, it is suggested that this model be used to estimate the temperature at time zero in the slabs. This means, when using the MEPDG, the box next to this variable needs to be remained unchecked. Refer to the snap shots of the software provided in the appendix for further clarification.

Ultimate Drying Shrinkage

The ultimate drying shrinkage of the PCC mixture is characterized in the MEPDG by using an empirical model. This model was discussed and evaluated in Task 5 for WO#13 in detail. To verify the accuracy and validity of this model, the ultimate drying shrinkage of the PCC mixtures used for each of the instrumented projects was established in the laboratory in accordance to AASHTO T 160/ASTM C 157. The latest test results for all the projects together with the predicted values by using the model incorporated in the MEPDG are summarized in *Table 22*.

It is interesting to notice that the values established in the laboratory for Project 1 is consistent with the ones predicted by the empirical model incorporated into the MEPDG. The model, however, underestimates the ultimate drying shrinkage for the other projects. It is noteworthy that both Projects 1 and 4 included Class AA 500 slip form concrete mixture with 15 percent Class C fly ash. The PCC mixture used for Project 2 and 3, on the other hand, included Class AA 550 slip form concrete with 15 percent Class F fly ash. Furthermore, the air-entrainment and water reducer admixtures used in the two sets of projects were different from each other. This empirical model does not include the effects of the supplementary cementitious material (SCMs), such as fly ash, or the chemical admixtures. Therefore, it is not surprising that the model better predicted the drying shrinkage when Class C fly ash (which sets in a more similar manner to Portland cement) was used as compared to Class F fly ash.

Overall, based on the results from *Table 22*, one can conclude that the empirical model is able to provide a rough estimate of the ultimate drying shrinkage for the PennDOT concrete Class AA 500.

Table 22 PCC drying shrinkage for all four projects.

Project	Drying Shrinkage (με)						
	Cured @ RH = 100% & Temp.=73 3 ºF			Cured @ RH = 50 3% & Temp.= 73 3 ºF			Predicted by the MEPDG
	Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3	Average of all Cells
1 at 430 Days	408	463	--				440
2 at 418 Days	629	--	--	706	753	733	480
3 at 312 Days	560	570	620	780	760	900	500
4 at 308 Days	520	--	--	720	--	--	440

Reversible Drying Shrinkage

Drying shrinkage develops over time when the PCC is subjected to drying. Upon rewetting, PCC expands to reverse a portion of the drying shrinkage. The main factor that affects the reversible portion of the drying shrinkage is the ambient relative humidity (ARA 2004). The value of 50 percent is recommended to be used in the MEPDG for the reversible drying shrinkage. This value should be used in the design by the MEPDG since no further data is available on this parameter.

Time to Develop 50 Percent of the Ultimate Drying Shrinkage

The next parameter to be discussed is the time to develop 50 percent of the ultimate shrinkage. According to the MEPDG documentation, 35 days should be used as the time required to develop 50 percent of the ultimate drying shrinkage, unless more reliable information is available. This is based on the recommendation provided by the ACI Committee 209. This value was used in calibrating the pavement performance models (ARA 2004).

The drying shrinkage data measured in the laboratory for the instrumented projects can be used to provide as a check on this default value. Based on laboratory test results, the samples from Projects 1 and 2 reached 50 percent of their ultimate drying shrinkage value in about 40 days. This age, however, for Projects 3 and 4 was only about 15 days. The default value of 35 days should be used for the design when using the MEPDG until more test results are available to further investigate this parameter.

Curing Method

Two different curing methods can be defined when using the MEPDG: wet curing and curing compound. Curing compound versus wet curing, increases the drying shrinkage estimated by the MEPDG, by 20 percent. However, multiple runs of the MPEDG showed that the effect of this change on the predicted distresses is not significant and does not typically affect the design thickness. To investigate this further, previous sensitivity studies were reviewed. Two studies included the curing method as a variable in their sensitivity analysis. 1-The study performed at the University of Arkansas (Hall and Beam 2005): based on this study both faulting and cracking predicted by the MEPDG are “Insensitive” to the curing method. 2- Iowa State University study (Coree, Ceylan et al. 2005; Coree, Ceylan et al. 2005): this study also reports the curing method as “Insensitive/Low sensitive” to faulting and as “Insensitive” to cracking.

Based on the PennDOT Publication 408, Section 711.1, allowable curing methods include Polyethylene sheeting and liquid membrane-forming curing compound. Curing compound appears to be the most commonly-used method in the state. Therefore, when using the MEPDG, between the wet curing and curing compound, the latter method should be used for design.

Strength

The easiest way to define the strength in the MEPDG is at input Level 3, when the minimum amount of data is available on the PCC material properties. At this input level, either the 28-day flexural strength or the 28-day compressive strength is required. This input data is then used in a regression model presented in Equation 9 to produce the flexural strength-gain curve. It should be noted that when 28-day compressive strength is entered instead of 28-day flexural strength the relation provided in Equation 10 will be used to convert the compressive strength to flexural strength.

$$MR(t) = (1 + 0.12*\log_{10}(t/0.0767) - 0.01566*\log_{10}(t/0.0767)^{2})* MR_{28\text{-day}} \qquad \text{Equation 8}$$

where,

t = Specimen age in years

$MR_{28\text{-day}}$= PCC flexural strength at 28 days.

Once the flexural strength gain curve is established, the modulus of rupture (MR) can be obtained at 3, 7, 14 and 90 days. The MR at each day is then converted to f_c using the relation provided in Equation 9.

Equation 9

where, k = 9.5 in the MEPDG

PCC stiffness can then be obtained at any age can be estimated based on the *f'c* at that age using the empirical relation provided in Equation 10.

Equation 10

To verify the accuracy of the three models used in the MEPDG as presented above, the MR was established in the laboratory at 7-, 14-, 28- and 90 days for the PCC mixture used for each of the instrumented projects.

The average 28-day MR for all projects, which was 812 psi, was used in Equation 9 to establish the flexural strength-gain curve. This was then compared to the curve established based on actual laboratory test results based on an average of all projects. This is presented in *Figure 10*. Error bars in *Figure 10* represent the standard deviation of the MR for the four projects. The curve from the MEPDG appears to be overestimating MR at early ages but shows very good agreements with the 90 day results. The reason behind this could due to the fact that the regression model employed by the MEPDG does not include the effects of SCMs, such as fly ash and slag, in the PCC mixture. These SCMs result in lower early-age (less than 28-days) strengths and higher later-age (greater than 28-days) strengths.

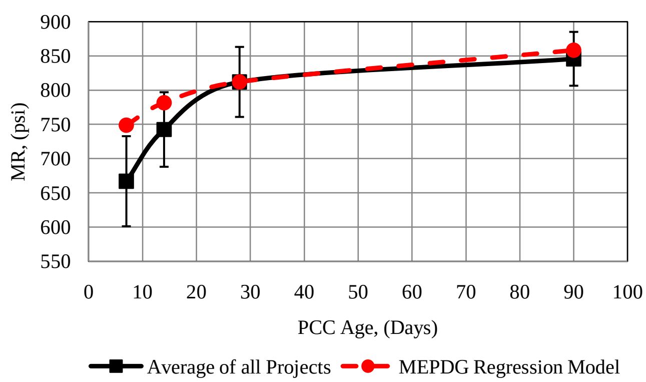

Figure 10 MR at different ages, established in the lab for four projects versus predicted by the MEPDG.

To evaluate the accuracy of Equation 10, the MR established at different ages using the MEPDG regression model were converted into *f'c* at different ages using Equation 10. The results are presented in *Figure 13* together with the average *f'c* established in the laboratory at 3-, 7-, 14-, 28- and 90 days for all projects. These two curves are presented in *Figure 11*. As seen in the figure, the average *f'c* established in the laboratory is significantly lower than the values established through the empirical relation used in the MEPDG at all ages.

One source of error in this equation could be sought in factor k. This factor is defined as 9.5 in the MEPDG. It is interesting to note that this factor is recommended by the PennDOT in the Pavement Policy Manual to be defined as 9.0 when using Equation 10. The influence of the k factor in Equation 10 was studied by changing this factor to different values and comparing the results to the measured values. This is presented in *Figure 12*. As seen in *Figure 12*, both the k factors recommended in the MPEDG and by the PennDOT overestimate *f'c* at all ages. Furthermore, based on *Figure 12*, when the k factor is assumed as 11.0 the most accurate predictions are obtained.

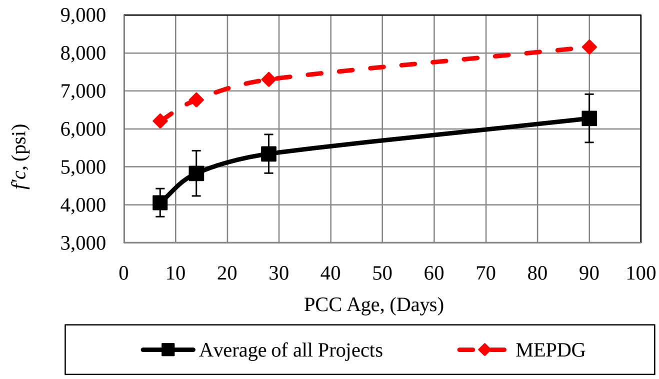

Figure 11 f'c at different ages, measured versus predicted by the MEPDG.

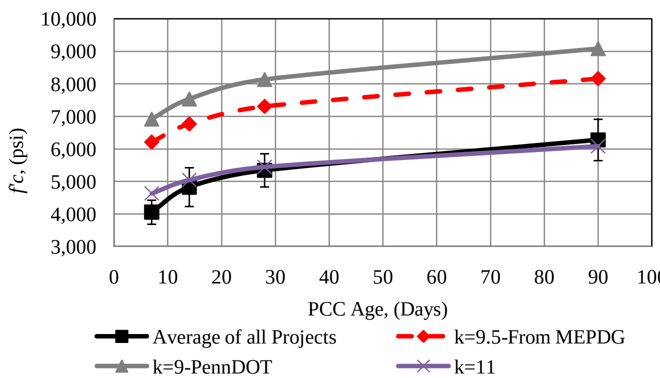

Figure 12 Effect of k factor on the predicted f'c.

The *f'c* of the PCC is important in the sense that it is used to establish the elastic modulus, Ec. Ec is directly used in the design by the MEPDG. The relation provided in Equation 11 is used in the MEPDG to establish Ec based on *f'c*. The accuracy of this relation is evaluated by using the *f'c* established through Equation 10 with the k factor set equal to different values (9.5, 9.0 and 11.0). Using these values for *f'c* and Equation 11, the elastic modulus can be established. These results are presented in *Figure 13* together with the average Ec measured in the laboratory at 1-, 3-, 7-, 14-, 28- and 90 days for all four projects. Error bars in this figure present the standard deviation for all projects. Based on *Figure 13,* Ec established using k equal to 9.5, recommended by the MEPDG, shows the best agreement with the measured values at 90 days. Even at the early ages the values established using k equal to 9.5 falls within the standard deviation for the laboratory-established values.

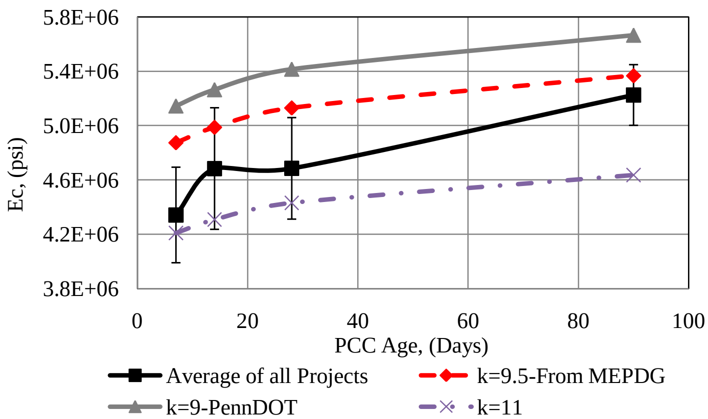

Figure 13 Ec at different ages, measured versus predicted by the MEPDG.

It should also be noted that a different empirical relation is recommended by the PennDOT for establishing Ec based on *f'c* of PCC. This relation is presented in Equation 11.

Equation 11

This equation was also used to convert the average *f'c* for all projects to average Ec at different ages. The results were then compared to the average Ec measured in the laboratory for all projects. This is presented in *Figure 14*. Based on *Figure 14*, the relation recommended by the PennDOT to establish Ec based on *fc*, underestimates Ec.

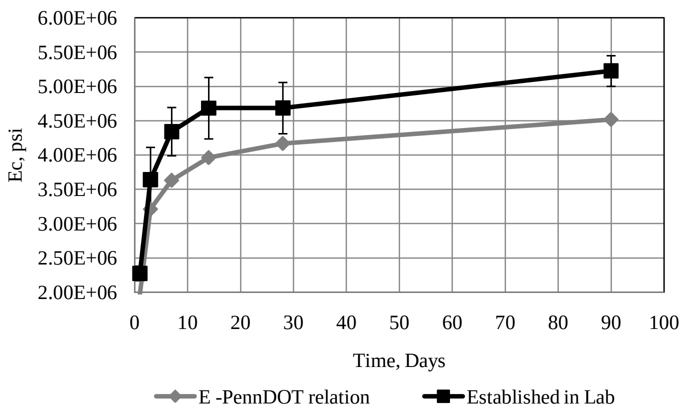

Figure 14 Ec established by using the PennDOT relation.

Overall, since the relation used in the MEPDG provided the best agreements with laboratory measured values for 90-day Ec as seen in *Figure 13,* it appears to be safe to rely on Ec predictions by the MEPDG based on 28-day *f'c*. Further work is suggested to better define these relationships for the PennDOT mixture designs.

A proper 28-day *f'c* needs to be used in the design when using the MEPDG. It is reported in the PennDOT Pavement Policy Manual, Section 8.4, Page 8-2, that the average 28-day *f'c* for the PCC Class AA in Pennsylvania is 4,925 psi with a standard deviation of 432 psi. It could not be determined what data source was used to establish this value and therefore how representative it is of the mixtures currently used by the PennDOT. In addition to this data, the University of Pittsburgh research team was supplied with the strength data from year 1993 to year 2008 from the Construction and Materials Management System (CAMMS) database. This data was used in Task 1 for WO#13 (Gatti, Nassiri et al. 2009) to establish the mean 28-day strength for PCC mixtures in Pennsylvania. This value was established at 5,093 psi with a standard deviation of 746 psi. This result was established based on a strength database for Class AA paving mixes. However, it did not differentiate between slip-formed and fixed-form paving and also did not include the mixture design properties such as cementitous content or w/c ratios. It is also noteworthy that the results established based on CAMMS data falls within one standard deviation reported by the PennDOT for Pennsylvania mixture.

Future options should include better defining the PCC strength and stiffness at different ages for different typical PCC mixtures across the state. Options to consider should also include establishing the CTE for the aggregate sources used within Pennsylvania since this parameter has a significant influence on the thickness design.

5.3.2. Base Material

5.3.2.1. Treated Permeable Base Layer

ATPB

The inputs required to define an ATPB layer in the MEDPG are categorized into three groups: “asphalt mix,” “asphalt binder” and “asphalt general.”

To define the asphalt mixture, the gradation of the aggregate used in the mix is required. According to the PennDOT Publication 408, Section 320, aggregate Type C, No. 2A can be used in the ATPB mix. The gradation for this type of aggregate is presented in *Table 23*. The next asphalt input parameter that needs to be defined is the asphalt binder. The MEPDG allows for the binder to be defined in any of the following three different formats: “Superpave binder grading,” “conventional viscosity grade” and “conventional penetration grade.” Based on the PennDOT recommendations in Publication 408, Section 320, in the “conventional viscosity grade” category, Class AC-20 or Class AC-10 should be selected for the binder.

Table 23 Gradation for No. 2A aggregate.

Sieve Size	Percent Passing
2"	100
3/4"	52-100
3/8"	36-70
#4	24-50
#16	10-30
#200	$\leq$ 10

The last group of inputs that need to be defined are the asphalt general properties. This includes the reference temperature, effective binder content, air voids, unit weight and thermal properties of asphalt. The values for effective binder content and air voids recommended by the PennDOT are 3.5 and 8.5, respectively. Therefore, these values should be used in the MEPDG design.

Other parameters in this category include asphalt unit weight, Poisson's ratio and thermal properties. The MEPDG is not sensitive to these parameters so it is suggested to use the typical values recommended in the MEPDG. This means values of 0.35-, 0.67- and 0.23 BTU/hr-lb-°F should be used for the Poisson's ratio, thermal conductivity and heat capacity of asphalt, respectively.

CTPB

The inputs required to define a CTPB layer in the MEPDG includes the unit weight, Poisson's ratio, elastic modulus and thermal properties.

Details on construction of a cement treated aggregate base course are discussed in the PennDOT Publication 408, Section 321. (It is referred to as a cement treated permeable base but this same material would be referred to as a lean concrete base in the MEPDG.) According to this section, the compressive strength of this mix after being cured in 100 percent relative humidity for a duration of 7 days should not be less than 650 psi. It is suggested that a typical value of 2.0E+6 psi, as recommended by the MEPDG, be used for the elastic modulus of this material.

Typical values suggested by the MEPDG should be used for other parameters such as the Poisson's ratio and the unit weight; these values for a CTPB are 0.2 and 150 lb/ft^3, respectively.

Thermal properties, including the conductivity and specific heat, can be defined the same as the ones used to define the PCC slab properties in Section 5.3.1. The CTE of the CTPB can, however, be different from that of the PCC mixture. This is because there is less paste and more coarse aggregate and voids present in the CTPB mixture. A typical PennDOT CTPB mix includes approximately 14 ft^3 coarse aggregate in 27 ft^3 of the mix. This means that the CTPB mix is comprised of approximately 52 percent coarse aggregate. Assuming a value of 10 percent for the voids present in the mix, the percentage of the paste is 38 percent. Based on the recommended values in the reference (ARA Inc. ERES Consultants Division 2004), a typical value for the CTE of coarse aggregate made of limestone and a concrete paste with a w/cm ratio of between 0.4 and 0.6 is 2 με/°F and 10 με/°F, respectively. Using the empirical relation available for estimating the CTE of the mix, provided below, the CTE of the CTPB mix is estimated as 4.8 με/°F.

$$\alpha_{mix} = \alpha_{agg} V_{agg} + \alpha_{paste} V_{paste} \qquad \text{Equation 12}$$

The same procedure can be followed to establish the CTE for the CTPB mix made of coarse aggregate of other rock types. It is suggested that one follows this procedure, unless more accurate data is available for the CTE of the CTPB mix.

Typical values suggested by the MEPDG should be used for other parameters such as the Poisson's ratio and the unit weight; these values for a CTPB are 0.2 and 150 lb/ft^3, respectively.

Thermal properties can be defined the same as the ones used to define the PCC slab properties in Section 5.3.1.

Comment [JMV1]: Not really…

5.3.3. Subbase

A crushed gravel layer is typically used in the pavement structure as a separator layer between the stabilized base and the subgrade. According to the PennDOT Publication 408, Section 350, this material is typically a No. 2A aggregate. The criterion for the gradation of this type of aggregate was presented in *Table 23*.

Other input parameters that are required to define this layer when using the MEPDG, include Poisson's ratio, coefficient of lateral pressure (k_0) and the resilient modulus. Typical values of 0.35; 0.5; and 25,000 psi are recommended in the MEPDG to be used for these parameters, respectively. Since this layer does not have any influence on the design and is used solely to prevent migration of fines between the layers, further characterization of the above mentioned parameters is not necessary. Therefore, the values recommended in the MEPDG should be used for the design unless more representative data is available

5.3.4. Subgrade

Subgrade material can be defined in the MEPDG using either AASHTO or unified soil classification (USC) definitions. Once the soil classification is defined, three groups of inputs regarding the subgrade material are needed to be defined. This includes: "pavement response model material inputs," "EICM material inputs" and "other material properties."

Pavement response model materials input required are resilient modulus (M_r) and Poisson's ratio. Material parameters associated with EICM are those parameters that are required and used by the EICM models to predict the temperature and moisture conditions within a pavement system. Key inputs include gradation, Atterberg limits, and hydraulic conductivity. The "other" category of materials properties constitutes those associated with special properties required for the design. An example of this category is the coefficient of lateral pressure.

When using the MEPDG for design in Pennsylvania, it is suggested that the subgrade material be defined based on the soil classifications provided in the geotechnical report for that the section of roadway being designed. Other input parameters regarding the subgrade characterization can be kept as the typical values used in the MEPDG for each soil classification unless more accurate data is available regarding each input. It is also noteworthy that these inputs are not expected to affect the design performed with the MEPDG. The sensitivity analysis of the performance models within the MPEDG, which was performed as part of Task 2 for WO#13, revealed that predicted distress in JPCP is not sensitive to subgrade type (Nassiri, Vandenbossche et al. 2009).

6. Performing the Design Analysis

Once all the required inputs are defined properly, a trial run of the MEPDG should be performed. Once the run is completed over the design life, the predicted distress at the end of the design life is compared against the desired design criteria. If the pavement fails to meet the criteria, the design thickness should be increased and the design run should be performed again. This iterative design procedure should be followed until the desired design thickness is achieved for the section.

7. Permanent Curl/Warp Effective Temperature Difference

The remainder of this report will focus on establishing the design parameter, the permanent curl/warp effective temperature difference in the MEPDG. This parameter falls under the category of "design features" when defining the inputs in the MEPDG. Due to its significance and complexity, this parameter is discussed here and separate from all other parameters. This parameter is also referred to as the built-in gradient. The following section provides a brief description of this parameter.

The permanent curl/warp effective temperature difference or the built-in gradient in concrete slabs is a significant factor that together with the transient temperature and moisture gradients present in the slab defines the shape of the slab. The built-in gradient consists of four different components including temperature and moisture gradients present in the slab at zero-stress time; permanent irreversible deferential drying shrinkage and also the portion of these gradients recovered through creep.

The built-in temperature gradient is defined as the temperature gradient that locks into the PCC slab at the time when the concrete sets, referred to as time-zero (Tz). The built-in temperature gradient is significant because it influences the future shape of the slab. As a result of the built-in temperature gradient in the slab, the slab does not remain flat when temperature and moisture gradients are not present.

The second component of the built-in gradient, as mentioned above, is the built-in moisture gradient, present in the PCC slab at Tz. This moisture gradient at Tz is defined as the built-in moisture gradient and is part of the built-in gradient. However, moisture measurements in the field in a previous study (Asbahan 2009) revealed that the slabs are fully saturated during the period of hardening. Since the slabs are fully saturated and the moisture condition is the same throughout the slab, it is safe to assume that the effects of the built-in moisture gradient are negligible.

The third component of the built-in gradient is the irreversible drying shrinkage gradient. The magnitude of the drying shrinkage that occurs throughout the PCC slab depends on various factors such as PCC mixture, ambient temperature and relative humidity, geometry and age of the slab and also time of the year. Field measurements have shown that daily fluctuations in drying shrinkage are minor with respect to seasonal variations. A portion of the drying shrinkage experienced in the drier seasons can be reversed upon rewetting in seasons with more precipitation. Furthermore, the irreversible drying shrinkage is a PCC property that stabilizes over years. It takes at least 5 years for the slab to reach its maximum irreversible drying shrinkage (Burnham and Koubaa 2001). The irreversible drying shrinkage gradient established in the slab after at least 5 years is the third component of the built-in gradient.

The built-in gradient is a design input in the MEPDG required when performing a rigid pavement design. The sensitivity study performed under Task 2 for WO#13 revealed that this design input is sensitive to the distresses predicted by the MEPDG. This input is referred to as the "permanent curl/warp effective temperature difference" in the design software. This term refers to combined effects of all four components of the built-in gradient discussed above and was established as -10 ºF. This value was established through the calibration process of JPCP cracking model based on data from the LTPP database. This same value is recommended in the MEPDG to be used for all concrete pavements.

However, many factors, including the pavement design features, climatic conditions present at the time of paving and the PCC mixture design, can affect this parameter. Therefore, assuming a constant value for this sensitive parameter can result in over- or underestimation of the design thickness.

Establishing the proper value for this parameter will be the focus of this study**.** **<u>It should be noted that the values established in this study for the built-in gradient cannot yet be incorporated into the MEPDG when performing a design and therefore, the default temperature difference of -10 ºF should be used to define the permanent curl/warp temperature difference when performing a design. This is because the calibration of the cracking model, which is currently incorporated into the MEPDG, was optimized by using the value of -10 ºF for the permanent curl/warp temperature difference. In order to be</u>**

able to employ the values established in this study for the permanent curl/warp temperature difference in the MEPDG, the cracking and faulting model need to be recalibrated based on distress data from sections in Pennsylvania while using the predicted built-in temperature differences. This is highly recommended and could be the focus of future research studies in this field.

In order to establish the built-in temperature gradient in PCC slabs, two computer-based numerical models were implemented. These models predict the temperature and relative humidity variation in throughout the depth of the slab based on inputs such as ambient weather conditions, PCC mixture properties and cement composition as inputs. Just like any numerical model, these models need to be validated with field measurements. The temperature data from the four pavement sections instrumented under Task 4 for WO#13 are used in the validation of the temperature model. The moisture data from moisture sensors from another instrumented section in Murrysville, PA constructed in August 2004 will be used to validate the relative humidity model. This was performed as part of a project funded also by PennDOT.

The predictions from the two models will be used to establish the built-in temperature and the irreversible drying shrinkage gradients as discussed above. The details regarding the temperature model, including the theory, boundary conditions and validation, will be discussed comprehensively first. This will be followed by the theory and validation of the relative humidity model. Proper values will be suggested at the end for the built-in gradient for pavement structures constructed in Pennsylvania.

7.1. Numerical Model to Predict Temperature in PCC Slab

Many researchers in the past have developed heat transfer models to predict the temperature variations across the slab depth (Thompson, Dempsey et al. 1987; Jeong, Wang et al. 2001; Ruiz, Kim et al. 2001). The one-dimensional equation of the heat transfer with respect to distance, x, and time, t, as presented in Equation 13 has been used commonly for this purpose.

Equation 13

where, T = Temperature, °C

t = Time, hours

ρ = Density, kg/m^3

c_p = Specific heat capacity, J/kg/°C

Q_H = Generated heat per unit time and volume, W/m^3

k = Thermal conductivity, W/m/°C

The finite difference method (FDM) is a numerical technique that is commonly used to solve deferential equations, such as the one presented above. With this approach, the temperature across the pavement structure can be predicted at discrete times after placement. Boundary conditions need to be chosen properly to satisfy compatibility with the field conditions (Ruiz, Kim et al. 2001). Furthermore, Q_H, the heat of hydration of the cement, in this equation needs to be defined at each increment of time. Therefore, another model needs to be incorporated into the major temperature model to estimate the heat of hydration that is produced in the slab at each increment of time. This model, together with the corresponding boundary conditions, will be discussed briefly in the following section.

It should also be noted that besides the hydration model, several minor models also work together with the temperature model in order to define the PCC material properties as the hydration process progresses. These properties include thermal material properties such as PCC specific heat, Cp, and thermal conductivity, k. These models consider the changes in the thermal properties with changes in the degree of hydration. Besides the degree of hydration of the cement, the Cp model considers the proportional weight of the PCC constituents, water, cement and aggregate, and uses typical recommended specific heat values for each. The thermal conductivity model considers the degree of hydration of the cement and uses typical values for PCC based on the aggregate type.

7.1.1. Heat of Hydration Model

The hydration reaction of Portland cement is an exothermic process. Therefore, when using the general heat transfer equation, presented in Equation 12, to predict the temperature variations across the pavement structure, the rate of heat liberation, Q_H, needs to be considered at each increment of time. A regression model has been developed (Schindler and Folliard 2005) to predict the heat of hydration. This model as presented in Equation 13, considers the effect of cement chemical composition by evaluating the rate of hydration. This model is also modified to account for SCMs in the PCC mixture.

Equation 14

where, Q_H = Rate of heat liberation, W/m^3

C_c = Cementitious materials content, kg/m^3

R = Universal gas constant, 8.3144 J/(mol K)

T_r = Reference temperature, 23 °C is normally used for US

Tc = Nodal slab temperature, ºC

H_u = Total heat of hydration of cementitious materials at 100 percent hydration, J/kg, obtained from a relation provided in the reference.

E = Activation energy, J/mol, obtained from a relation provided in the reference.

= Hydration shape parameter, obtained from a relation provided in the reference.

= Hydration time parameter, obtained from a relation provided in the reference.

Equation 13 consists of several constituents and can be defined with respect to the composition of the cement, Blaine and amount of SCMs used in the PCC mixture. One of the inputs, for estimating the Q_H, as seen in Equation 18, is the slab nodal temperature. Numerical solutions, such as FDM, which solve equations through iterations, make it possible to use the data from the previous iteration in the current iteration. This way Q_H can be estimated at every desired increment of time.

7.1.2. Boundary Conditions

When the PCC slab is placed in the field, heat will be transferred to and from the surrounding conditions. The temperature development in the PCC is determined by the balance between the heat generation in the PCC and the heat exchange with the environment. Heat transfer to the surroundings occurs in four ways: conduction, convection (q_c), irradiation (q_r), and solar absorption (q_s). This is presented schematically in Figure 15. The models used to estimate each of the 4 boundary conditions are summarized in *Table 24*.

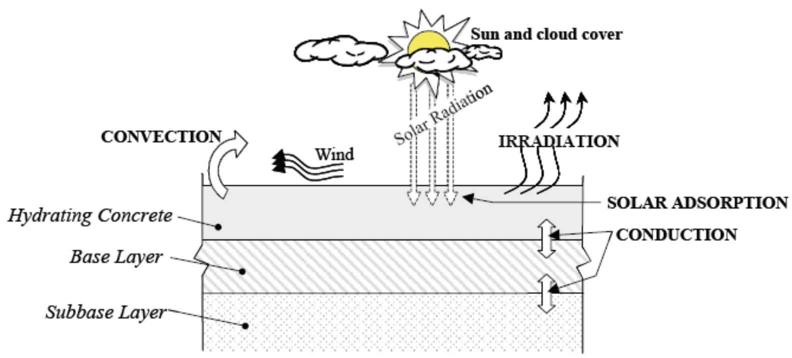

Figure 15 Heat transfer mechanisms between pavement and its surroundings (Ruiz, Rasmussen et al. 2006).

The essential boundary conditions at the top and bottom of the pavement structure are provided in Equations 14 and 15.

on pavement surface	Equation 15
on pavement bottom	Equation 16

In order to be able to define the boundary conditions at the bottom of the pavement structure, a constant temperature of 12 ºC was assumed at the depth of 100 inches. This was based on results from previous studies which found the temperature of the earth maintains an almost constant temperature at this depth.

Table 24 Boundary conditions used for the heat transfer model.

Type of Heat Transfer	Corresponding Model	Definition of Parameters
Conduction		q = Heat flux (W/m^2), k_0 = Thermal conductivity (W/m^2/°C), Tc= Concrete surface temperature (°C), and Ta = Air temperature (°C).
Convection	If $w \leq 5$ m/s: $hc = 20 + 14w$ else: $hc = 25.6*0.78w$	= Convection heat flux (W/m^2), h_c = Surface convection coefficient (kJ/m^2/h/°C), w = Windspeed (m/s).
Solar Absorption		q_s = Solar absorption heat flux (W/m^2), βε = Solar absorptivity, I_f = Intensity factor to account for angle of sun during a 24-hour day, q_{solar} = Instantaneous solar radiation, (W/m^2).
Irradiation		q_r = Heat flux of heat emission from the surface (W/m^2), σ = Stefan-Boltzmann radiation constant (5.67×10^{-8} W/m^2/°C^4), ε = Surface emissivity of concrete, 0.88 to be used for concrete. Tc = Concrete surface temperature, (°C), and T_∞ = Surrounding air temperature, (°C).

7.2. Temperature Model Validation

The validity and accuracy of any numerical model needs to be investigated thoroughly to make sure the solutions are reliable and realistic. In order to validate the implemented temperature model in this study, the predictions from the model were compared against the actual temperature measurements in the field for the four instrumented projects. Details on the instrumentation of the four projects were presented thoroughly in Task 4 report for WO#13 and will not be provided here to avoid repetition.

The predicted and measured temperatures in the slabs will be presented against each other for three different cells in each of the projects and the accuracy of the model predictions will be investigated separately.

7.2.1. Project 1-Westmoreland County, PA, SR-22, Section B09

Section B09 of SR-22 was the first of four test sections to be instrumented under Task 4. The date of paving for this section was Sept. 10th of 2009. The new pavement structure is a JPCP with 15-ft transverse joint spacing and 12-ft wide lanes. A description of the different layers in the pavement structure is provided in Figure 16. The PCC mixture design and also the cement composition used to pave this section are provided in *Table 25* and *Table 26*, respectively. More details on the construction of this project can be found elsewhere (Nassiri and Vandenbossche 2010).

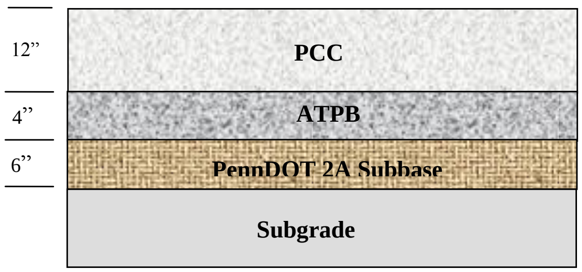

Figure 16 Pavement design layer thicknesses, Project 1.

Table 25 PCC mixture for the pavement test section, Project 1.

Material	Batch Weight (per yd^3)		
	Cell 1	Cell 2	Cell 3
Type I Cement (Armstrong)	500 lbs	500 lbs	500 lbs
Fly Ash-Class C (Essroc)	88 lbs	88 lbs	88 lbs
Fine Aggregate (Hanson, PennDOT Spec.)	1334 lbs	1322 lbs	1302 lbs
Coarse Aggregate (Hanson, AASHTO No. 57)	1857 lbs	1853 lbs	1857 lbs
Water Content (City Water)	19.21 gals	21.05 gals	20.70 gals

Table 26 Composition of the cement used in the PCC mix for Project 1.

Component	Value (%)
SiO_2 (%)	20.2
Al_2O_3 (%)	5.3
Fe_2O_3 (%)	4.3
CaO (%)	64.4
MgO (%)	1.0
SO_3 (%)	2.95
C_3S (%)	59
Blaine (cm^2/g)	6.7
SiO_2 (%)	3800

The PCC mixture design properties and the cement composition used to pave Project 1 were incorporated into the implemented numerical temperature model discussed earlier. The results were validated based on the measured data. This is presented in *Figure 17* to *Figure 24*. The first three graphs, *Figure 17*, *Figure 18* and *Figure 19*, show both the predicted and measured temperature variations over time at the surface of the slabs in Cells 1, 2 and 3, respectively. The same order is followed in *Figure 20*, *Figure 21* and *Figure 22* for temperature variations at the upper portion of the slabs and in *Figure 23*, *Figure 24* and *Figure 25* for the temperature at the bottom of the slabs. As seen in these figure, the numerical model is able to predict temperature variations in the slabs relatively well in all three cells.

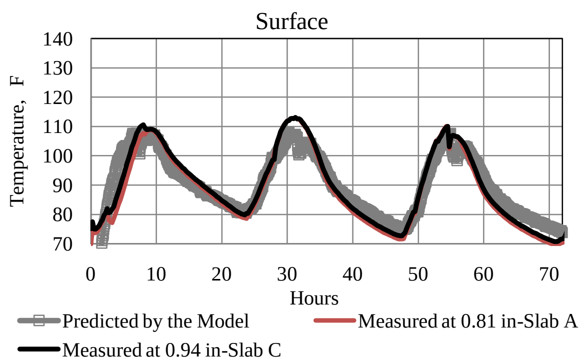

Figure 17 Predicted temperature vs. measured at the surface of the slab, Project 1, Cell 1, Paved at 8:00 AM.

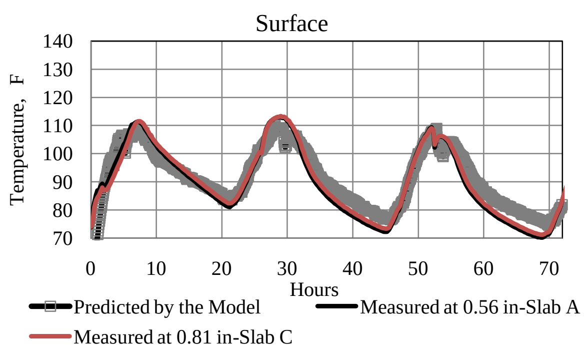

Figure 18 Predicted temperature vs. measured at the surface of the slab, Project 1, Cell 2, Paved at 10:30 AM.

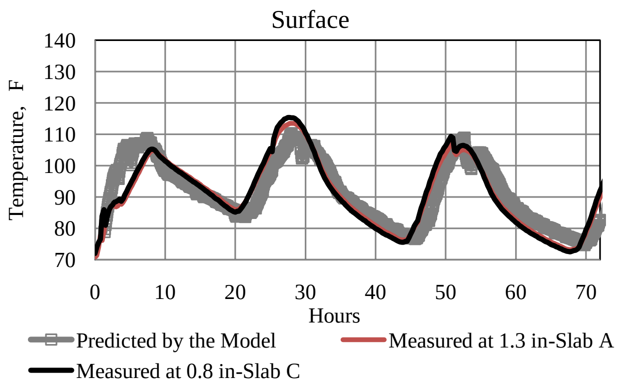

Figure 19 Predicted temperature vs. measured at the surface of the slab, Project 1, Cell 3, Paved at 11:30 AM.

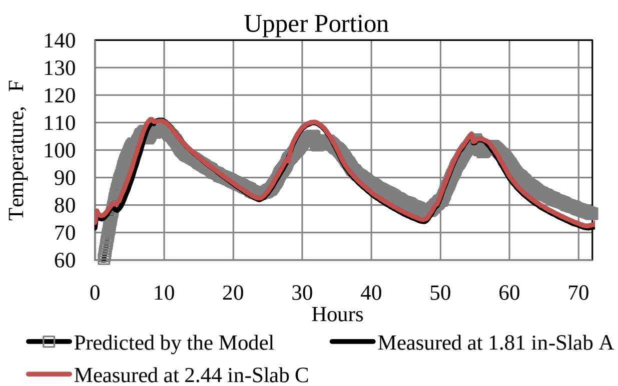

Figure 20 Predicted temperature vs. measured at the upper part of the slab, Project 1, Cell 1, Paved at 8:00 AM.

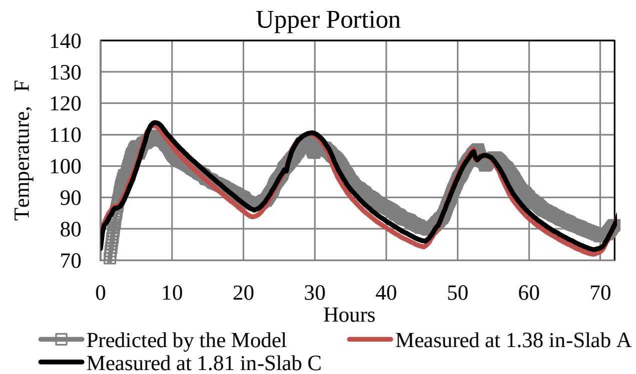

Figure 21 Predicted temperature vs. measured at the upper part of the slab, Project 1, Cell 2, Paved at 10:30 AM.

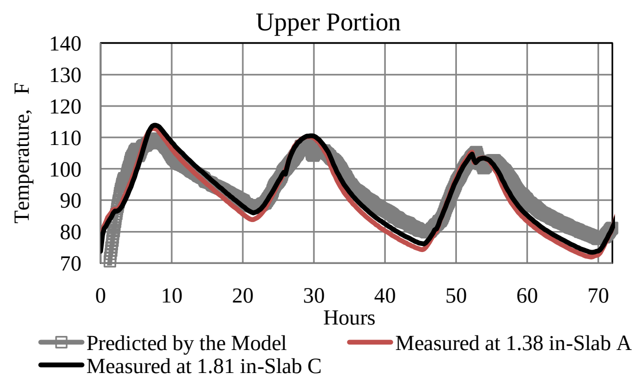

Figure 22 Predicted temperature vs. measured at the upper part of the slab, Project 1, Cell 3, Paved at 11:30 AM.

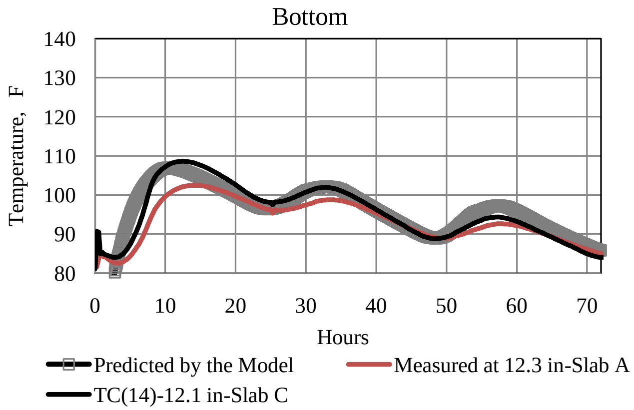

Figure 23 Predicted temperature vs. measured at the bottom of the slab, Project 1, Cell 1, Paved at 8:00 AM.

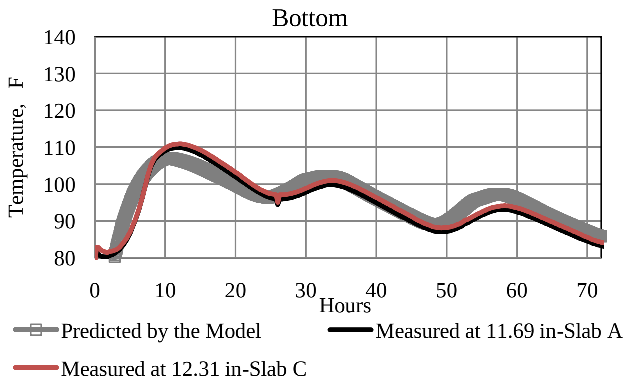

Figure 24 Predicted temperature vs. measured at the bottom of the slab, Project 1, Cell 3, Paved at 11:30 A

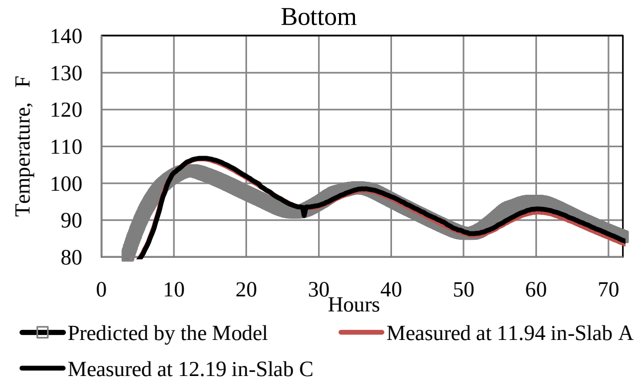

Figure 25 Predicted temperature vs. measured at the bottom of the slab, Project 1, Cell 2, Paved at 10:30 AM.

7.2.2. Project 2-Westmoreland County, PA, SR-22, Section B10

The second project instrumented as part of Task 4 for WO#13 included a total number of 9 slabs on SR 22 in Westmoreland County. Temperature measurements from this project will not be used for the temperature model validation because the slabs in all three cells were covered with plastic sheets upon placement due to cold weather and rain. The application of plastic sheets on the newly constructed slabs prevented heat exchange between the slabs and the environment, influencing temperature variation in the slabs.

7.2.3. Project 3-Indiana County, PA, US-22

Project 25833 in Clyde, Pennsylvania was instrumented as the third project in Task 4 for WO#13. The paving of this section of Route 22 started at 8:00 AM, on April 29th of 2010. Paving on the 29th continued until about 6:30 PM and was started again the next morning at 8:00 AM. Cell 1 and Cell 2 were paved during the first day of paving and in the afternoon, while Cell 3 was constructed in the morning of the second day. The new pavement structure is a JPCP with transverse joint spacing varying from 13 feet at the start to 15 feet at the end. The paving width is 24-ft to accommodate two adjacent 12-ft wide lanes. The design thicknesses for each of the layers in the pavement structure are provided in *Figure 26*. The PCC mixture design and the composition of the cement used for this project are provided in *Table 27* and *Table 28*.

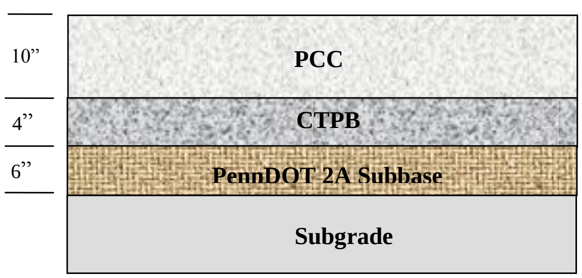

Figure 26 Pavement design layer thicknesses, Project 3.

Table 27 PCC mixture design for Smart Pavement test section, Project 3.

Material	Batch Weight (per yd^3)		
	Cell 1	Cell 2	Cell 3
Type I/II Cement	500 lbs	500 lbs	500 lbs
Fly Ash-Class F	88 lbs	88 lbs	88 lbs
Fine Aggregate (Hanson)	1166 lbs	1166lbs	1166lbs
Coarse Aggregate (Hanson, AASHTO No. 57)	1840 lbs	1840lbs	1840lbs
Water Content (City Water)	21.6gals	23.28gals	22.44gals
Unit Weight	142.6 lb/ft3	142.6 lb/ft3	142.6 lb/ft3.

Table 28 Cement composition for Project 3.

Component	Value (%)
SiO_2 (%)	20.7
Al_2O_3 (%)	4.5
Fe_2O_3 (%)	3.2
CaO (%)	63.1
MgO (%)	2.8
SO_3 (%)	2.7
C_3S (%)	59
Blaine (cm^2/g)	12
SiO_2 (%)	6
Al_2O_3 (%)	3909

The data from the above tables were incorporated into the temperature model to predict temperature variation at different depths of the slabs. The results were validated based on field data. This is presented in *Figure 27* to *Figure 35*.

According to the figures, the model is able to predict temperature variation in the slabs and the predictions agree relatively well with the measurements in the field. Some variation between the predicted and measured values is seen in Cell 1 and 2 at very early ages. The heat of hydration is predicted too early during the afternoon of the first day of paving for Cell 1. The heat of hydration is also overestimated in Cell 2. This could be because the hydration time and shape factors were established by employing regression models. For more accurate results at early ages, these factors need to be established based on the heat signature graphs established using calorimeter tests for the PCC mixture.

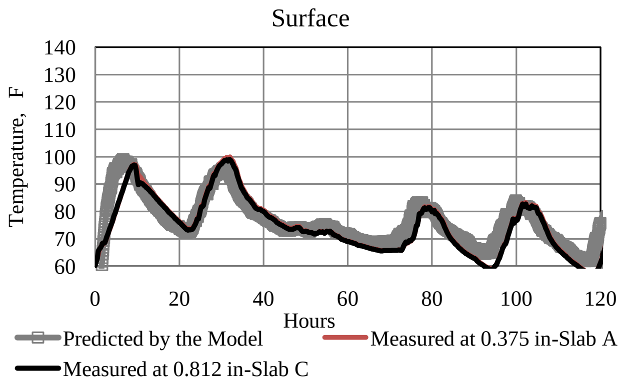

Figure 27 Predicted temperature vs. measured at the surface of the slab, Project 3, Cell 3, Paved at 8:00 AM.

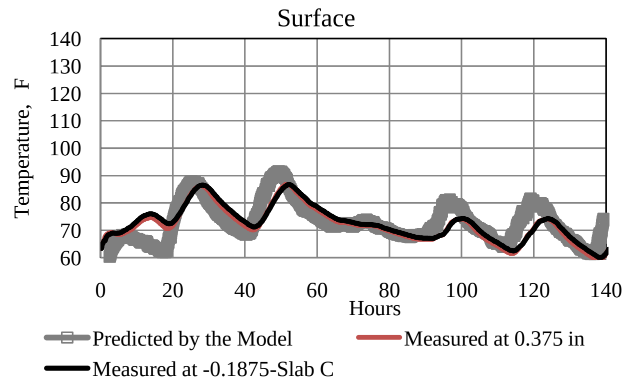

Figure 29 Predicted temperature vs. measured at the surface of the slab, Project 3, Cell 1, Paved at 2:30 PM.

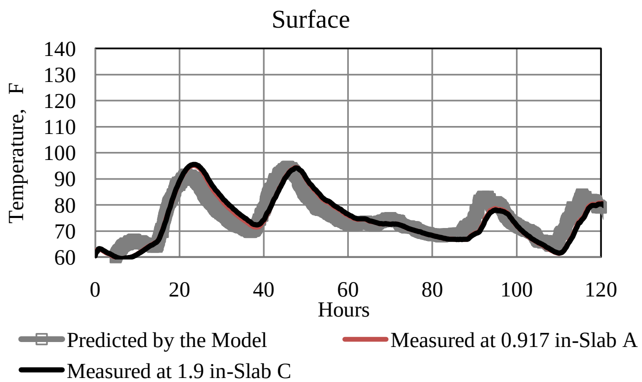

Figure 28 Predicted temperature vs. measured at the surface of the slab, Project 3, Cell 2, Paved at 5:30 PM.

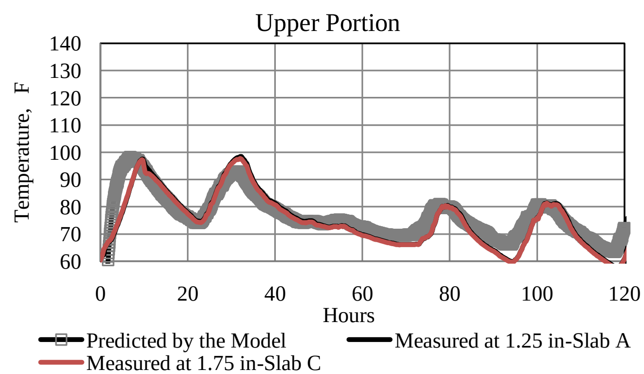

Figure 30 Predicted temperature vs. measured at the upper portion of the slab, Project 3, Cell 3, Paved at 8:00 AM.

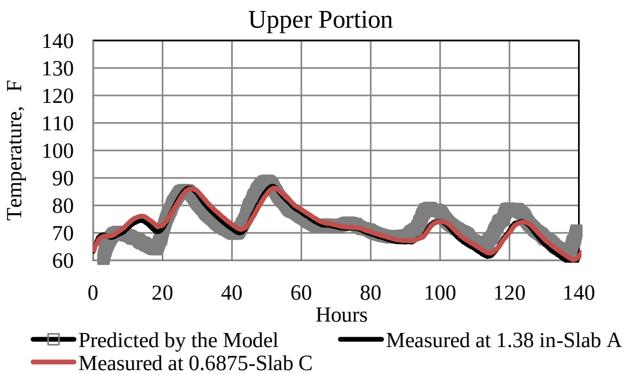

Figure 31 Predicted temperature vs. measured at the upper portion of the slab, Project 3, Cell 1, Paved at 2:30 PM.

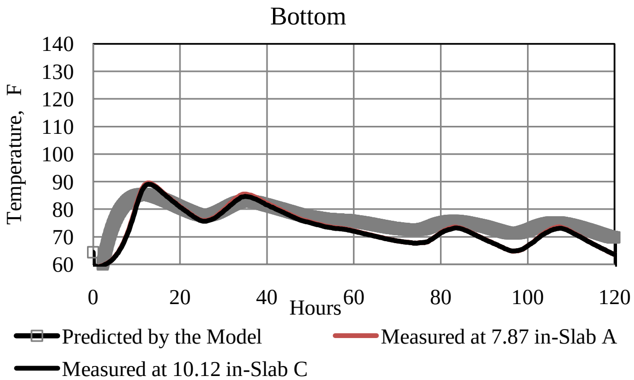

Figure 32 Predicted temperature vs. measured at the bottom of the slab, Project 3, Cell 3, Paved at 8:00 AM.

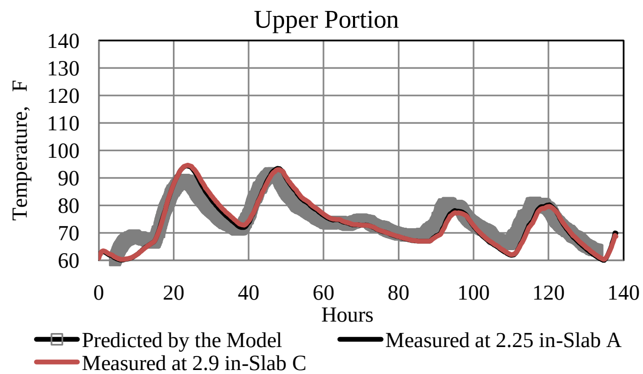

Figure 33 Predicted temperature vs. measured at the upper portion of the slab, Project 3, Cell 2, Paved at 5:30 PM.

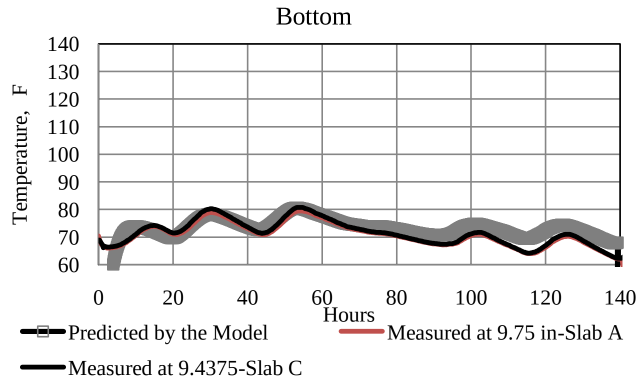

Figure 34 Predicted temperature vs. measured at the bottom of the slab, Project 3, Cell 1, Paved at 2:30 PM.

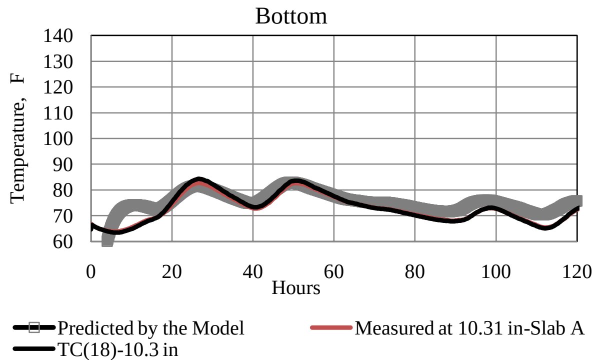

Figure 35 Predicted temperature vs. measured at the bottom of the slab, Project 3, Cell 2, Paved at 5:30 PM.

7.2.4. Project 4-Westmoreland County, PA, SR-22

Project 31477 in Westmoreland County, Pennsylvania was the last instrumented project in Task 4 for WO#13. The paving of this section of SR 22 started at 6:00 AM on May 10th of 2010. Paving continued until about 6:00 PM on the paving day. The new pavement structure is a JPCP with transverse joint spacing of approximately 15 ft. Some instrumented slabs at the location of Cell 2 had shorter joint spacings ranging between 11 and 12 ft. The design PCC slab, base, and subbase layer thicknesses are provided in *Figure 36*.

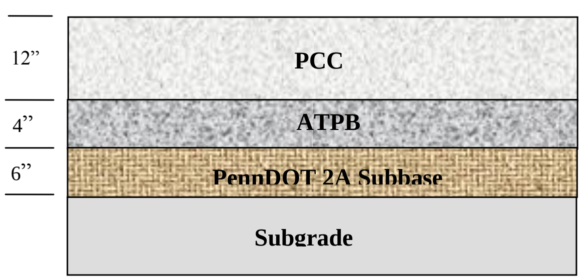

Figure 36 Pavement design layer thicknesses, Project 4.

Table 29 PCC mixture properties used to pave Project 4.

Material	Batch Weight (per yd^3)		
	Cell 1	Cell 2	Cell 3
Type I Cement (Armstrong)	500 lbs	500 lbs	500 lbs
Fly Ash-Class C (Essroc)	88 lbs	88 lbs	88 lbs
Fine Aggregate (Hanson, PennDOT Spec.)	1316 lbs	1285 lbs	1302 lbs
Coarse Aggregate (Hanson, AASHTO No. 57)	1860 lbs	1860 lbs	1853 lbs
Water Content (City Water)	18.00gals	19.55gals	20.55gals

Table 30 Composition of cement used in the PCC mixture for Project 4.

Component	Value
SiO_2 (%)	20.1
Al_2O_3 (%)	5.35
Fe_2O_3 (%)	4.4
CaO (%)	63.4
MgO (%)	1.0
SO_3 (%)	2.85
C_3S (%)	54.9
Blaine (cm^2/g)	3580

The data from the above tables was incorporated into the numerical temperature model to predict temperature variation in the slabs. The results were validated based on the field data. This is presented in *Figure 37* to *Figure 43*.

Based on the figures, the numerical model is able to predict temperature variation trends in the slabs. The predictions are in closer agreement with the measurements for Cell 1 and Cell 3.

Overall, it can be concluded that the implemented numerical temperature model is able to predict temperature variation in the slabs at around Tz, which is the focus of this study, relatively accurately. The accuracy of the predictions is usually low during the first hours after the slabs were initially placed. This is because regression hydration models were used to predict the hydration process of the cement. In order to increase the precision of the predictions, the activation energy and also the heat signature graphs need to be established for the cement used in the PCC mixture using calorimeter tests. However, this test equipment is relatively costly and this work was not in the scope of WO#13.

The validated temperature model can now be employed to establish the built-in temperature gradient for different regions in the state. To achieve this goal, two major steps need to be taken. First, the state is divided into several regions with relatively common ambient conditions, and second, Tz is established in the slabs. The latter task was accomplished under Task 5 for WO#13 and will be briefly reviewed here.

The following section of the report will focus on establishing climatic regions in the state.

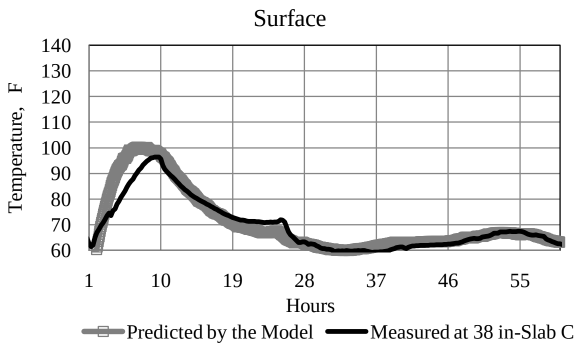

Figure 37 Predicted temperature vs. measured at the surface of the slab, Project 4, Cell 1, Paved at 8:00 AM.

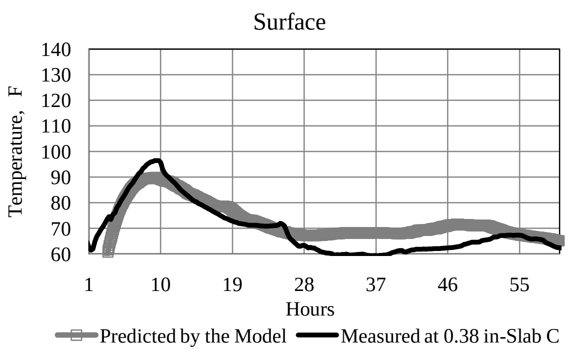

Figure 38 Predicted temperature vs. measured at the surface of the slab, Project 4, Cell 2, Paved at 3:30 PM.

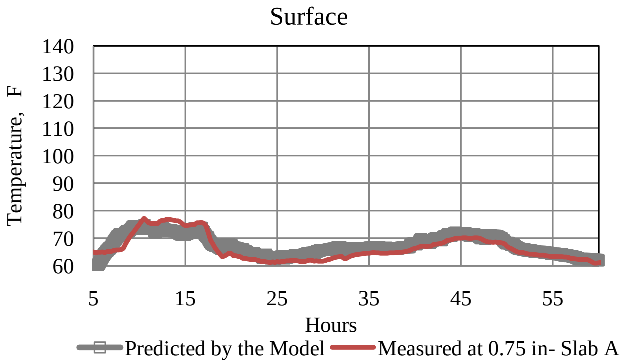

Figure 39 Predicted temperature vs. measured at the surface of the slab, Project 4, Cell 2, Paved at 4:30 PM.

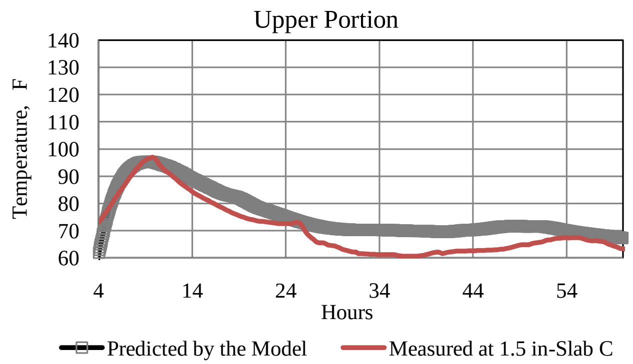

Figure 41 Predicted temperature vs. measured at the upper portion of the slab, Project 4, Cell 2, Paved at 3:30 AM.

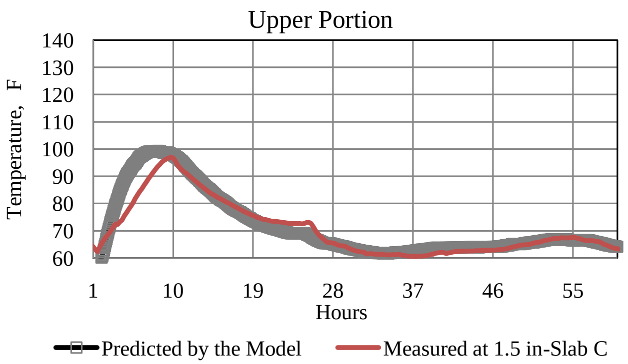

Figure 40 Predicted temperature vs. measured at the upper portion of the slab, Project 4, Cell 1, Paved at 8:00 AM.

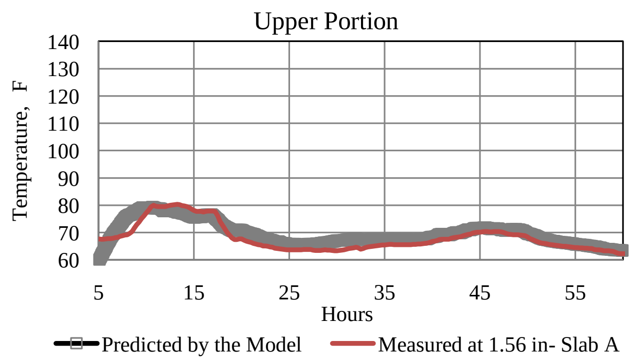

Figure 42 Predicted temperature vs. measured at the upper portion of the slab, Project 4, Cell 3, Paved at 8:00 AM.

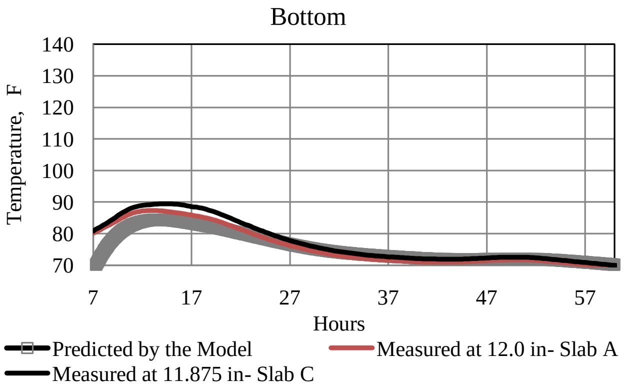

Figure 43 Predicted temperature vs. measured at the bottom of the slab, Project 4, Cell 3, Paved at 4:30 PM

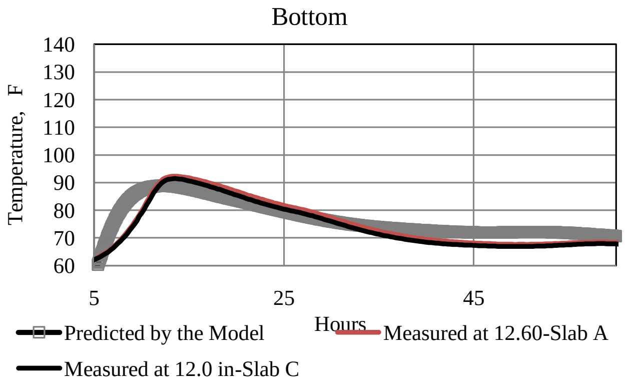

Figure 44 Predicted temperature vs. measured at the bottom of the slab, Project 4, Cell 2, Paved at 3:30 PM.

7.3. Climate Regions in Pennsylvania

In an effort to establish the built-in temperature gradient for pavement sections constructed in different regions of Pennsylvania, the state was divided into different climatic regions. This was necessary because the climatic databases available in the MEPDG design software do not cover the entire state. Due to this lack of climatic data for several counties and regions in the state, some approximation for the regions without available climatic data is necessary.

The climatic regions were defined mainly based on the variation in elevation, freezing index (FI) and annual ambient air temperature. The climatic databases available for Pennsylvania in the MEPDG design software were used to establish the major climatic indices, such as annual air temperature, rainfall, wind speed, relative humidity and FI for each database.

The FI for each climatic station together with their elevations are presented in *Figure 45*. The mean annual air temperature for each climatic station together with the elevation is presented in *Figure 46*. The stations with similar elevations, FI and mean annual air temperature were grouped together. Five different climatic regions were established for the state. These different climatic regions are shown with the red circles in *Figure 45* and *Figure 46*. The five different regions were also presented on a county map of the state in *Figure 47*. It is noteworthy that Pocono Mountains, marked as Region VI in *Figure 47*, is not included in the study due to a lack of climatic data. Therefore the same climatic properties available fordd Region II, Allegheny Mountains, can be assumed for Region VI until further studies provide more climatic data for this region.

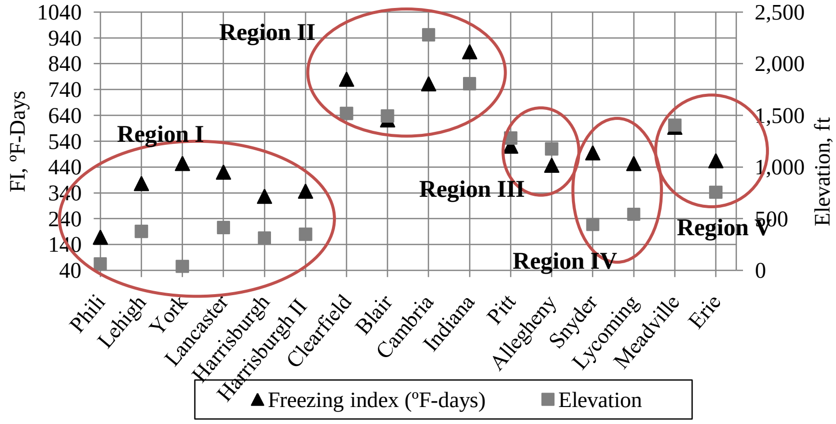

Figure 45 Five climatic regions based on the FI and elevation of the climatic stations available in the MEPDG.

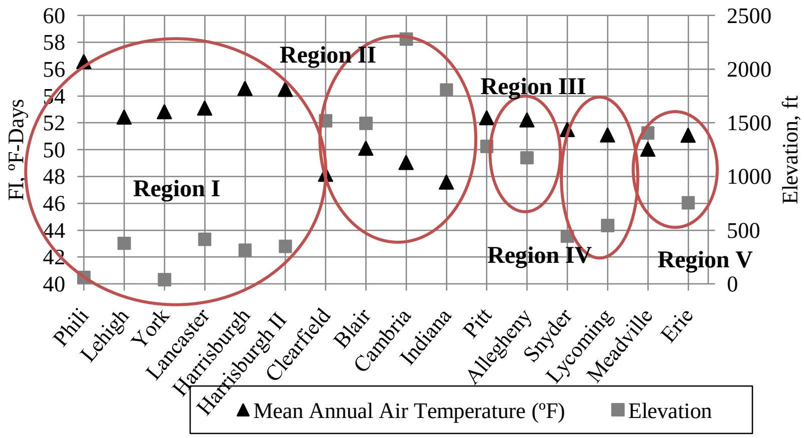

Figure 46 Climatic regions based on the mean annual air temperature and elevation of the climatic stations available in the MEPDG.

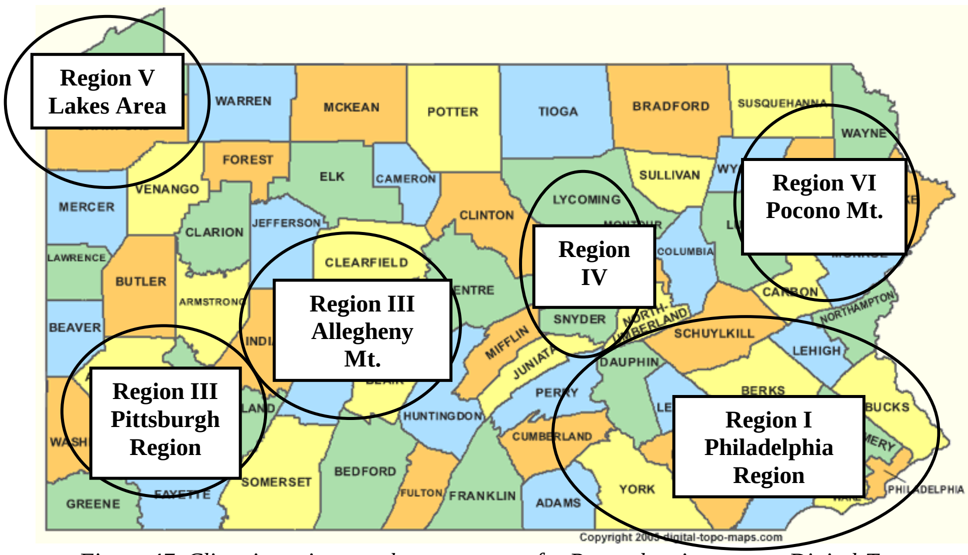

Figure 47 Climatic regions on the county map for Pennsylvania, source: Digital-Topo-Maps.com.

The range for the FI, mean annual air temperature and elevation in each climatic region is summarized in *Table 31*.

Table 31 Ranges for FI, mean annual air temperature and elevation for each climatic region.

Region/Range	Region I		Region II		Region III		Region IV		Region V	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Mean Annual Air Temperature (ºF)	52	57	48	50	50	52	50	52	50	51
Freezing Index (ºF-days)	150	500	600	1000	400	550	400	500	400	600
Elevation (ft)	0	500	1500	2500	1100	1300	500	500	700	1500

It is noteworthy that the five climatic regions established in this section do not include all the counties in the state, as seen in *Figure 47*. The proper climatic region needs to be assigned to each county based on the ranges provided in *Table 31* for each region and by using the county's FI, mean annual temperature and elevation. To do so, the FI for each county can be pulled from the PennDOT Pavement Policy Manual, Publication 242, Appendix D. The mean annual temperature for each county can also be extracted from the National Oceanic and Atmospheric Administration (NOAA) website.

A climatic database was established for each of the five climatic regions discussed above. This was performed by taking the average of hourly climatic data for the stations in each region for every year. This data was then averaged over the available years (at least 5 years). The results for each region are presented in the form of average monthly temperature, relative humidity, percent sunshine, wind speed and rainfall in *Figure 48* through *Figure 51*.

The established climatic database for each region can then be incorporated into the validated numerical temperature model discussed earlier to find the temperature gradient at Tz in the slabs constructed in different locations in Pennsylvania. To do so, Tz needs to be established in the slabs first. The process used to establish Tz will be disused in the next section of this report.

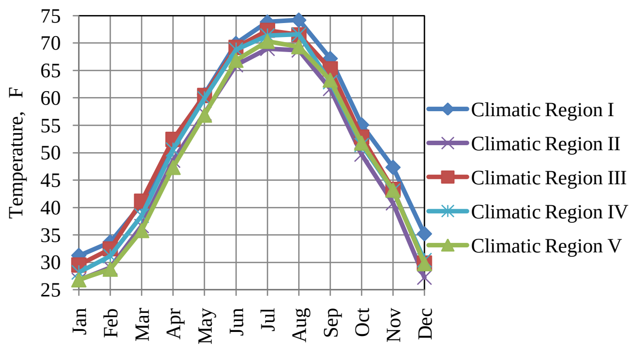

Figure 48 Average monthly temperature for each climatic region in the state.

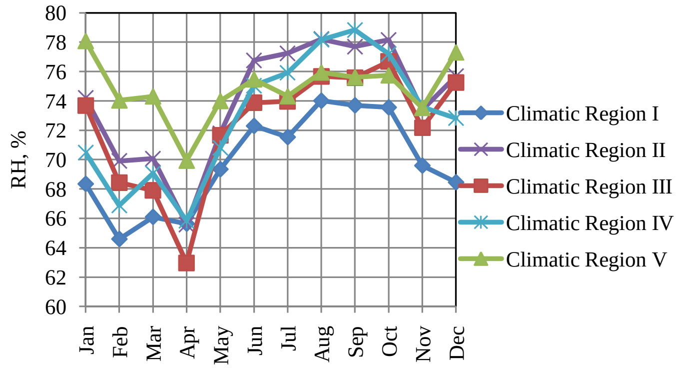

Figure 49 Average monthly relative humidity for each climatic region in the state.

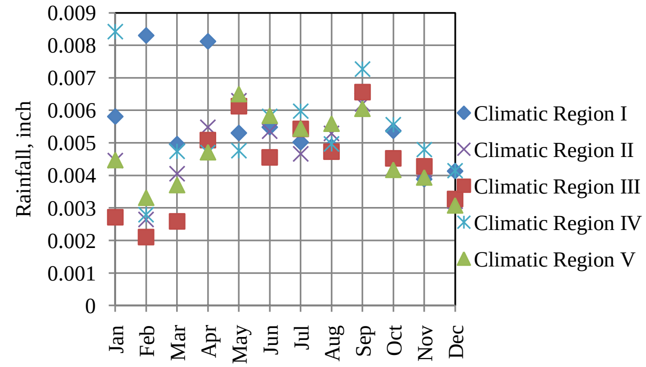

Figure 51 Average monthly rainfall for each climatic region in the state.

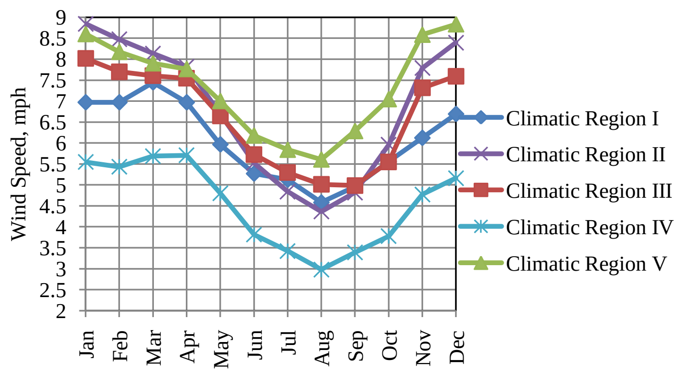

Figure 50 Average monthly wind speed for each climatic region in the state.

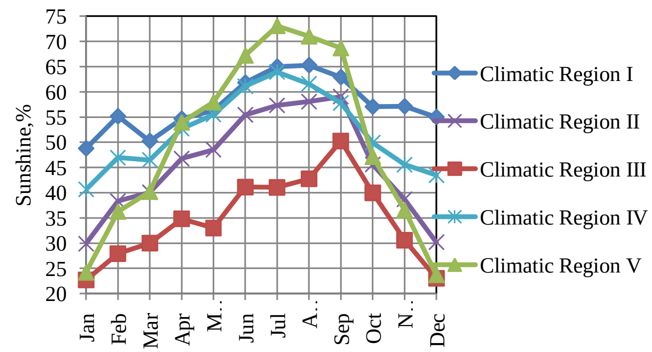

Figure 52 Average monthly percent sunshine for each climatic region in the state.

8. Establishing the Built-in Temperature Gradient in PCC Slabs

The final goal of this section of the report is to establish the built-in temperature gradient for any pavement structure that consists of either an ATPB or CTPB with a 10- to 12-in. slab. The slab should be paved with typical PennDOT Class AA 500 concrete mixture with 15 percent fly ash. This pavement structure could also be constructed in any of the five climatic regions defined in Section 7.3 and at any time of the day.

Comment [JMV2]: Based on the Mn/ROAD data can we make any assumptions regarding the appropriatness of slab thickness outside of this range...Are these values appropriate for a slab 9 to 13in thick? This would cover most of the state roads.

As discussed earlier, the built-in temperature gradient is the nonlinear temperature gradient that exists in the slab at the time of zero-stress or Tz. Therefore, to establish the built-in temperature gradient in the slab, Tz needs to be established in first. Establishing Tz was the main focus of Task 5 for WO#13. In this task, Tz was established for each instrumented project based on the strain and temperature changes in the PCC slabs during hardening. A very brief summary of the findings from this study will be provided here.

8.1. Establishing the built-in Temperature Gradient in Pavements with ATPB

Out of four, two of the instrumented sections consisted of ATPB and the other two consisted of CTPB. Since base/slab friction is one of the major factors that influence Tz, the sections with the same base layer should be analyzed together.

Both Project 1 and Project 4 included an ATPB layer. Tz was established for each project based on the strain and temperature changes recorded by the VW strain gages installed at the top and bottom of each slab. The degree of hydration the cement reached at Tz was also established for each slab. The average degree of hydration at Tz for the three consecutive slabs in each cell was also established in each project for both the top and bottom of the slabs. Between the two degrees of hydration (one at the top of the slab and one at the bottom), the lower value one was selected as the deterministic degree of hydration in the slab, as was described in the Task 5 report. This is presented in *Figure 53* for both Projects 1 and 4.

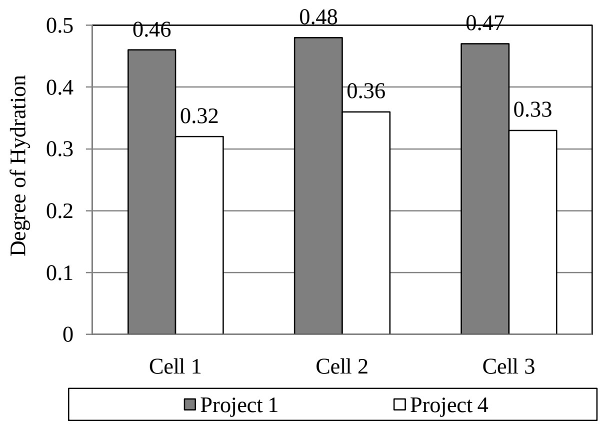

Figure 53 Average degree of hydration in each cell for Projects 1 &4.

The average degree of hydration reached at Tz in the slabs for Project 1 is 0.47 while this value is established at 0.34 for Project 4. The variables that were likely to be responsible for this difference in the degree of hydration are discussed in detail in the Task 5 report and will not be repeated here.

To be able to extend these results to slabs beyond the four instrumented projects and establish one level for degree of hydration (referred to as $\alpha_{critical}$ hereafter) for all pavement structures with an ATPB layer and the same PCC mixture, the built-in temperature gradient will be established using both values of 0.47 and 0.34. The final value for $\alpha_{critical}$ will be established based on the sensitivity of the level of degree of hydration to the established built-in temperature gradient. To perform this study, the climatic database for Region 1 was incorporated into the validated numerical temperature model discussed earlier. Using the predictions from the temperature model, the built-in temperature gradient was established both when the slabs reached a degree of hydration of 0.47 (based on Project 1 results) and at the time the slabs reached a degree of hydration of 0.34 (based on the results from Project 4). This was performed assuming the slabs were constructed at 8:00 AM and on the first day of the month and is presented in *Figure 54*. Based on this figure, the range of variation in the predicted built-in temperature gradient for the two different degrees of hydration is between 0.01 and 0.35 °F/in. This level of variation in the built-in temperature gradient is not expected to affect the design thickness established by the MEPDG. Therefore, the average value of 0.405 will be used for $\alpha_{critical}$ in slabs constructed on an ATPB layer.

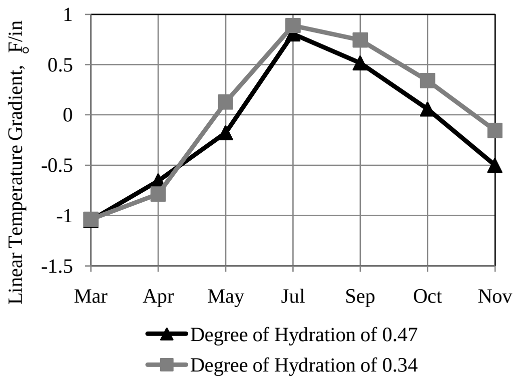

Figure 54 Sensitivity of $\alpha_{critical}$ *to the predicted built-in temperature gradient.*

In order to establish the sensitivity of the month of construction to the predicted built-in temperature gradient, the built-in gradient was established for all 12 months in a year using the Region 1 climatic database. The results are presented in *Figure 55*. Since the construction season does not start until the warm days in March, the months in winter was not included in the study. It is also noteworthy that spring includes months of March, April and June. Based on *Figure 55*, the highest variability is seen in the spring and the fall seasons as expected, while very low variability is seen over the summer. Therefore, the built-in temperature gradient will be established for only one month during the seasons with low variability. This parameter will be established for all the three months in the seasons with high variability (spring and fall).

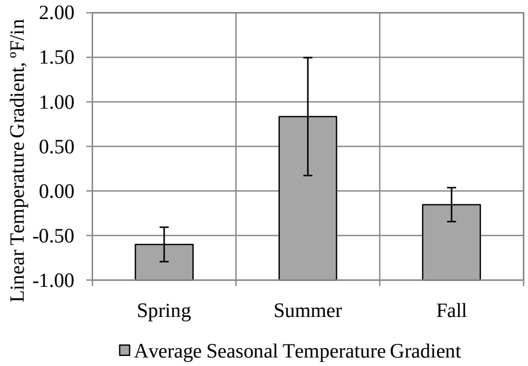

Figure 55 Average seasonal built-in temperature gradient for different seasons.

Sensitivity of the time of paving to the built-in temperature gradient is another issue that needs to be accounted for. To address this issue, three different times of paving during the day were selected; 8:00 AM, 2:00 PM and 5:00 PM. These times were selected based on changes in weather conditions during the day. The built-in temperature gradient was established for slabs paved at the three times of day mentioned above and on the first day of each month. Using the climatic database for Region 1 the results are presented in *Figure 56*. Based on the figure, the two paving times of 2:00 PM and 5:00 PM show very similar results, however, variability is observed between morning and afternoon paving times during the fall and spring seasons. This variability is minimal during the summer. To further investigate the effects of time of paving on the established built-in temperature gradient, another small study was performed.

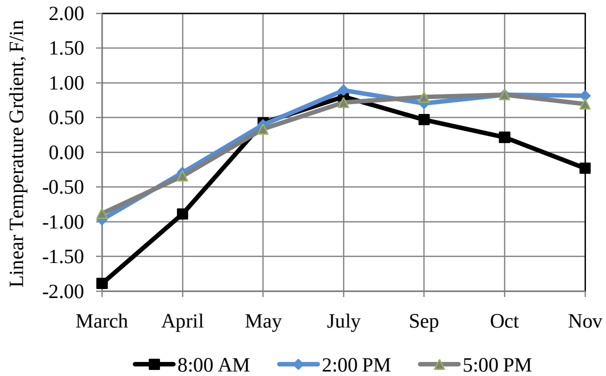

Figure 56 Effects of time of paving on the built-in temperature gradient.

In this study, the two months of March and April that appear to show high variability for different times of paving in *Figure 56* were selected for further investigations. The built-in temperature gradient was established for 11 different times of paving during the first day in April and the third day in March (since the first day included some hours with freezing temperatures). The results are presented in *Figure 57*.

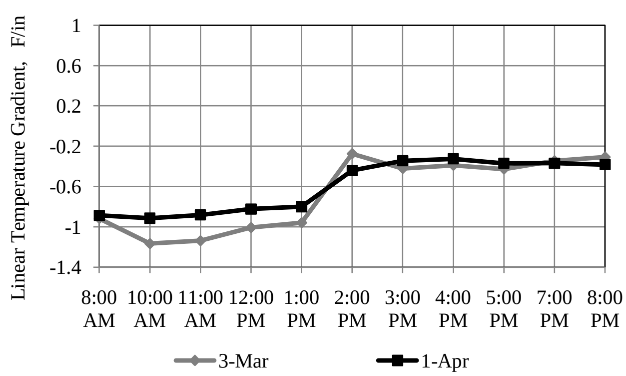

Figure 57 Effects of time of paving on the built-in temperature gradient for March and April on hourly basis.

From *Figure 61*, the built-in temperature gradient is relatively constant from 8:00 AM to 1:00 PM and also from 2:00 PM to 8:00 PM. Based on these results and the results from the previous section, the following plan will be employed to establish the built-in temperature gradient for different regions in the state. The built-in temperature gradient will be established for the selected months of the year for the two paving times of 8:00 AM and 5:00 PM. The average of these two values is suggested to be used in the design. Following this plan for each of the climatic regions in Pennsylvania gives results for morning and afternoon paving that are presented in *Figure 58* and *Figure 59*, respectively. The average of the two graphs in *Figure 58* and *Figure 59* is also presented in *Figure 60*. Recommended values to be used for design in Pennsylvania are also listed in *Table 32*.

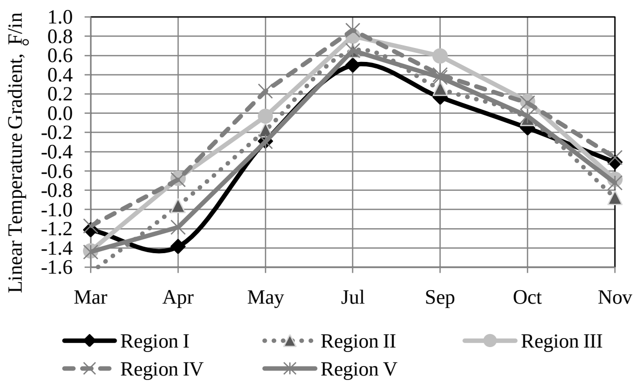

Figure 58 Built-in temperature gradient for five different regions in PA, for different construction month and time of paving of 8:00 AM.

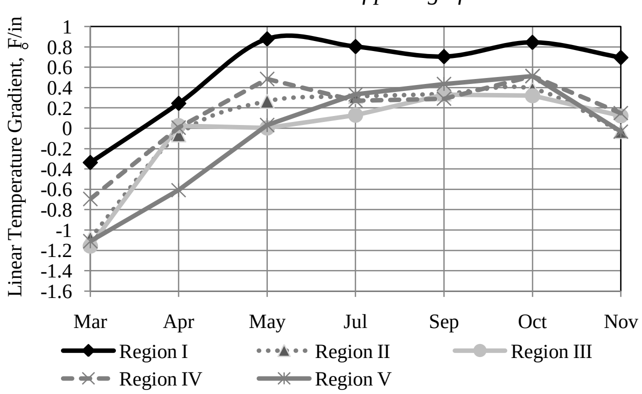

Figure 59 Built-in temperature gradient for five different regions in PA, for different construction month and time of paving of 5:00 PM.

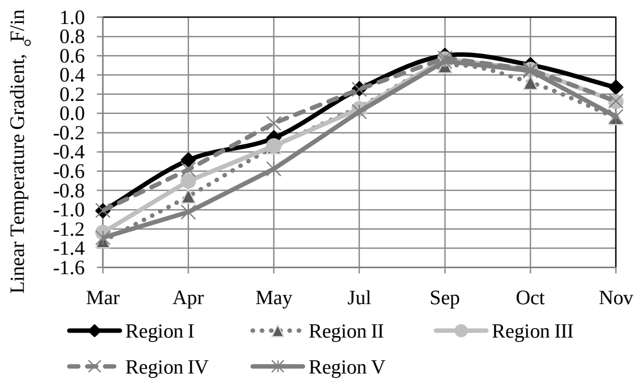

Figure 60 Average of the values of morning and afternoon paving

Based on the data from *Table 32*, the built-in temperature gradients for Region I in all months are very close to the ones established for Region IV. These two regions are two neighboring regions on the county map in *Figure 47* and can be combined into one climatic region, namely Climatic Region A. Furthermore, again based on data from *Table 32*, Regions II and III show close values for the built-in temperature gradient in almost all months. These two regions are again neighbors on the county map presented in *Figure 47* and can be combined into one climatic region, namely Climatic Region B. Region V or the lakes region will be referred to as Climatic Region C.

The final average built-in temperature gradients are summarized in *Table 33* for Climatic Regions A, B and C.

Table 32 Built-in temperature gradient for five climatic regions in the state.

Month of Construction/ Climatic Region	Mar.	Apr.	May	July	Sep.	Oct.	Nov.
Region I	-1.0	-0.5	-0.3	0.3	0.6	0.5	0.3
Region II	-1.3	-0.9	-0.3	0.1	0.5	0.3	0.0
Region III	-1.2	-0.7	-0.3	0.0	0.6	0.5	0.1
Region IV	-1.0	-0.6	-0.1	0.3	0.6	0.5	0.1
Region V	-1.3	-1.0	-0.6	0.0	0.5	0.4	0.0

Table 33 Built-in temperature gradient for three final climatic regions in the state.

Month of Construction/ Climatic Region	Mar.	Apr.	May	July	Sep.	Oct.	Nov.
Region A	-1.0	-0.5	-0.2	0.3	0.6	0.5	0.2
Region B	-1.3	-0.8	-0.3	0.1	0.5	0.4	0.0
Region C	-1.3	-1.0	-0.6	0.0	0.5	0.4	0.0

8.2. Establishing built-in Temperature Gradient for Pavements with CTPB

Tz was established for Projects 2 and 3 following the same method discussed in the previous section for the pavement sections with ATPB. This was the focus of Task 5 for WO#13. Tz established for Project 3 under Task 5 study was revisited in this study and were established at different times in Cell 1 and Cell 2. The correct Tz is presented on the strain-temperature graphs for Slabs A and B in Cell 1 in *Figure 61*and *Figure 62* as samples. The degree of hydration at the new Tz is also presented in Table 9. As before, the lower value between the degrees of hydration reached at Tz at the top and bottom of the slab was selected in each cell.

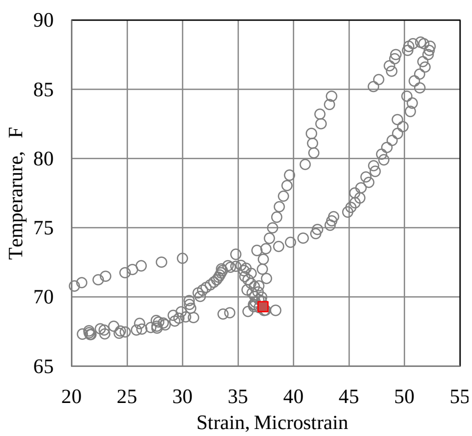

Figure 61 New Tz at the top of Slab B, Cell 1, Project 3.

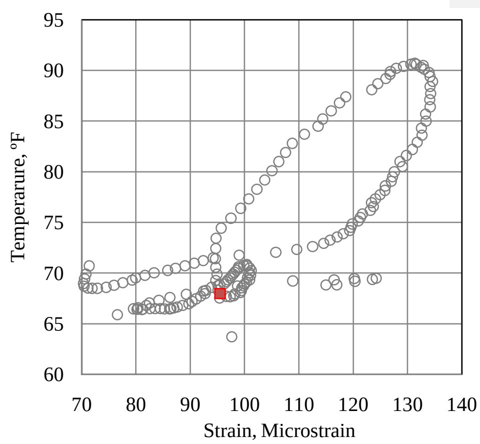

Figure 62 New Tz at the top of Slab C, Cell 1, Project 3.

Table 34 Degree of hydration at Tz, Project 3.

	Critical Degree of Hydration								
	Cell 1			Cell 2			Cell 3		
	Slab A	Slab B	Slab C	Slab A	Slab B	Slab C	Slab A	Slab B	Slab C
Top	0.43	0.45	0.44	0.52	0.52	0.52	0.45	0.46	0.45
Bottom	0.46	0.45	0.46	0.52	0.52	0.52	0.41	0.42	0.42

Project 2 was also a pavement structure with a CTPB layer. However, the Tz established for the slabs of this project cannot be compared directly to that of Project 3. This is because the slab thicknesses in the two projects are significantly different. The design slab thickness for Project 2 was 14 in. but surveying prior and after the paving revealed that the as-built thickness varied significantly along the pavement section. The design slab thickness for Project 3, on the other hand, was only 10 in. The as-built average slab thickness at each cell for both Project 2 and 3 are presented in *Figure 63*.

The degree of hydration at Tz established for Project 4 was established at about 0.51 for all three cells. More details on this can be found in the Task 5 report. The value of 0.51 is higher than that established for the cells in Project 3. This agrees with logic, since the larger slab thickness for Project 4 will result in heavier slabs and eventually more friction between the base and the slab. Higher friction at the base/slab interface can affect the Tz established in the slabs.

Comment [JMV3]: Would this really effect the built in gradients substantially? Would be a way for us to either establish a relationship with thickenss or prove it is insignificant.

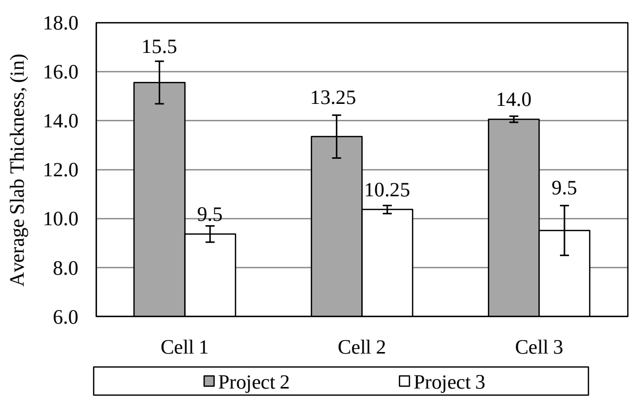

Figure 63 As-built PCC slab thickness in Projects 2 and 3.

Since a 10 in PCC slab appears to be more common in practice than 14 in. PCC slabs, the average degree of hydration of 0.46 established for Project 3 will be used to establish the built-in temperature gradient for section with a CTPB layer constructed in different regions in the state. The average value for degree of hydration of 0.46, established for the pavement sections with a CTBP layer, is very close to the average value of 0.41 that was used for sections with an ATPB layer and is not expected to result in any significant variation in the built-in temperature gradient. Therefore the results from *Table 34* should be used for both pavement sections with ATPB and CTPB layers.

9. Establishing Irreversible Drying Shrinkage Gradient in PCC Slabs

The temperature gradient present in the slab at Tz, however, is not the only factor that defines the built-in gradient that exists in the slab over its performance life. The third component of the built-in gradient, as discussed earlier, is the gradient due to irreversible deferential drying shrinkage. The level of irreversible drying shrinkage in the slab stabilizes over years as the slab ages. It is believed that drying shrinkage requires at least 5 years to reach its maximum value (Burnham and Koubaa 2001). The gradient caused by the irreversible non-uniform drying shrinkage across the slab needs to be transformed into an equivalent linear temperature gradient and added to the other components to produce the built-in gradient. This goal will be achieved through implementing a numerical model to predict the relative humidity variations in PCC slabs as a function of time and depth.

9.1. Theory and Equation

Moisture flux in a porous material, such as cement paste, can be expressed in the form of the relation presented in Equation 16. This equation describes the moisture transport in a material due to drying and wetting. It has been employed by several researchers in the past (Oh and Cha. 2003; Jeong and Zollinger 2006; Xu, Ruiz et al. 2009)

— — — Equation 17

where, t = Time, hours,

H =Internal (pore) relative humidity, 0 ,

W = Total water content for unit volume of material,

C(H) = PCC moisture diffusivity, .

Boundary Conditions

At the surface, the relative humidity of the slab needs to be in balance with the ambient relative humidity. This essential boundary condition at the surface is presented in Equation 17. Factor *f* in Equation 17 is the surface coefficient (Akita, Fujiwara et al. 1997) modified for curing conditions (Xu, Ruiz et al. 2009).

— Equation 18

At the bottom, the slab is considered fully saturated. Numerical solutions were developed for the relation in Equation 5 using the finite difference method.

9.2. Model Validation

To validate the relative humidity model, the relative humidity measurements from a 6-year old instrumented test section on SR 22 located in Murrysville, Pennsylvania, namely SR 22 smart pavement, was used. This test section was paved in August of 2004 and was instrumented with SHT75 relative humidity sensors and VW static strain gages at different locations and depths prior to paving. *Figure 64* presents a schematic layout of the instrumented slabs. These six slabs were divided into two different cells. As seen in *Figure 64*, Cell 3 consisted of 3 unrestrained (no dowel or tie bars) PCC slabs, while Cell 4 included 3 restrained PCC slabs. Slabs in Cell 3 were instrumented with VW strain gages, while slabs in Cell 4 were instrumented with thermocouples and relative humidity sensors as well as VW strain gages.

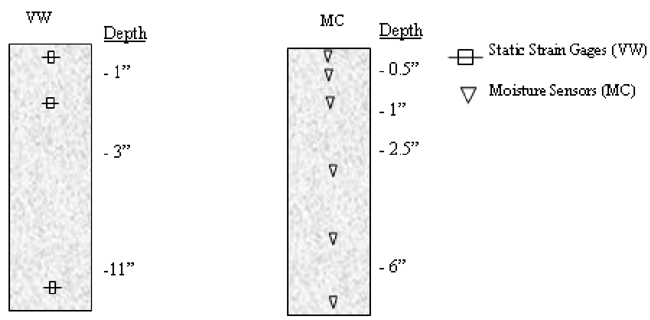

Figure 64 Location and depth of VW, moisture and temperature sensors in the test section.

Once the variation in relative humidity in the slab over time is established, the linear temperature gradient equivalent to the existing moisture gradient can be obtained by using the method developed by Mohamed and Hansen. In this method a third-degree polynomial is fit through the relative humidity difference coefficient throughout the slab depth. Based on this, an equivalent linear humidity difference coefficient is estimated for every increment of time. Equations 18 and 19 show the relations used in this methodology.

$$\Delta[1-(RH/100)^3] = A + Bz + Cz^2 + Dz^3$$ Equation 19

$$\Delta[1-(RH/100)^3]eqv. = -12\left(\frac{Bh}{12}+\frac{Dh^3}{80}\right)$$ Equation 20

where, z = Coordinate defined as zero at the mid-depth of the slab, where upward is negative and downward is positive

A, B, C, D = Regression coefficients, and

h = Thickness of the concrete slab.

The equivalent linear temperature gradient to the nonlinear moisture gradient can then be estimated based on the humidity difference coefficient and the ultimate drying shrinkage of the PCC. The relation is provided in Equations 20 (Jeong and Zollinger 2005).

$$\Delta T_{eq.} = {-\varepsilon_\infty \Delta\left[1-\left(\frac{RH}{100}\right)^3\right]_{eq}} \Big/ {\alpha_c}$$ Equation 21

where: ε_{ω} = Ultimate drying shrinkage strain

ΔT_{eq} = Equivalent linear temperature difference.

The ultimate drying shrinkage required in Equation 20, was established for PennDOT Class AA 500 slip form concrete, based on laboratory tests performed on samples from the concrete for Projects 1 and 2. The ultimate drying shrinkage for this mix can be assumed as approximately 600 με, as reported in the Task 5 report. The CTE of PCC is another factor that is required in Equation 20. This property of concrete was also established for samples from the four instrumented projects. The average value of 5.2 με/°F was established for PCC mixtures with coarse aggregate made of limestone.

The product of the $\Delta[1-(RH/100)^3]eq$ predicted by the humidity model for each climatic region and the ultimate drying shrinkage for the mix is the drying shrinkage strain in the slabs. The mean monthly data corresponding to only one month in every year was selected for each case to eliminate the effects of seasonal fluctuations on the drying shrinkage. The result is presented in *Figure 65* for all the five climatic regions in Pennsylvania.

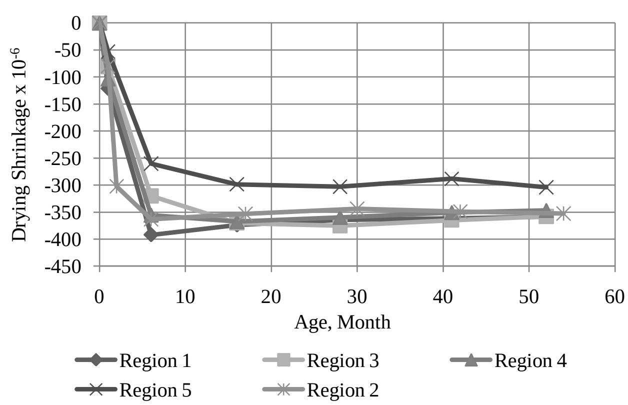

Figure 65 Predicted drying shrinkage for slabs in the 5 climatic regions in Pennsylvania.

The predicted drying shrinkage should be corrected for the effects of the restraining factors in the field, such as the base friction and dowel and tie bars, and also the negating effects of long-term creep. This was done by using the long-term strain data from the VW static strain gages installed in the 6-year old SR 22 smart pavement project located in Murrysville, Pennsylvania. This data was used to establish the strain due to drying shrinkage in the field. It is noteworthy that the corresponding results include the effects of restraints provided by the base, dowel and tie bars and also the relaxing effects of creep. These results are presented in *Figure 66* for doweled and undoweled slabs.

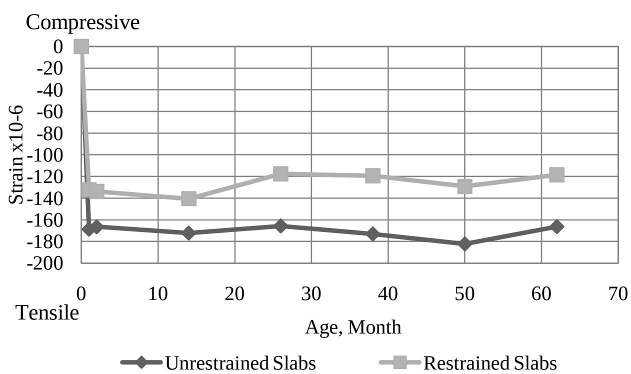

Figure 66 Measured drying shrinkage strain for slabs in SR 22 project.

The predicted drying shrinkage established based on the humidity model predictions together with the ultimate drying shrinkage for the PennDOT concrete mix can be compared the measured drying shrinkage. The difference is a field factor that accounts for the possible effects of relaxation due to creep and the restraints provided by the base and the dowels/tie bars on the development of drying shrinkage over time.

The predicted drying shrinkage strains for the five regions in the state, provided in *Figure 65*, was corrected for the restraining factors in the field based on the measured drying shrinkage provided in *Figure 66*.

The long-term corrected $\Delta[1-(RH/100)^3]eq$ together with the CTE and slab thickness were incorporated into Equation 20 to establish the warping gradient in the slabs. The result for each region is provided in *Table 35*.

Table 35 Magnitude of the 5-year permanent warping for the 5 climatic regions in Pennsylvania.

	Equivalent Linear Temperature Gradient (°F/in)				
	Region 1	Region 2	Region 3	Region 4	Region 5
Doweled	1.7	1.9	2.1	2.1	1.7
Undoweled	2.7	2.9	3.0	2.9	2.5

The results in *Table 35* are similar for all the 5 regions and therefore an average value of 1.9 °F/in for doweled slabs and average value of 2.8 °F/in can be used for permanent warping in 12-inch slabs. The permanent curling temperature gradient established previously, should be added to the permanent warping gradient. This is performed to established the permanent effective curl/warp temperature gradient for Regions A, B and C. The final result for the doweled and undoweled pavement sections constructed in different months of the year in Pennsylvania is provided in *Table 36*. It should be noted that the values from *Table 36*, need to be multiplied by the slab thickness and also a negative sign when used in the MEPDG.

Table 36 Magnitude of the permanent effective curl/warp temperature gradient for the 5 climatic regions in Pennsylvania.

	Month of Construction/ Climatic Region	March	Apr.	May	July	Sep.	Oct.	Nov.
Doweled	Region A	0.9	1.4	1.7	2.2	2.5	2.4	2.1
	Region B	0.6	1.1	1.6	2.0	2.4	2.3	1.9
	Region C	0.6	0.9	1.3	1.9	2.4	2.3	1.9
Undoweled	Region A	1.8	2.3	2.6	3.1	3.4	3.3	3.0
	Region B	1.5	2	2.5	2.9	3.3	3.2	2.8
	Region C	1.5	1.8	2.2	2.8	3.3	3.2	2.8

10. Options to Consider

10.1. Characterize Industry Specific Traffic Streams

In an effort to use the traffic data collected by PennDOT to define the traffic requirements in the MPEDG, the University of Pittsburgh research team was supplied with the traffic data from a total of 10 traffic stations for a period of one week in November of 2008. This traffic data was analyzed under Task 1 to establish values for each traffic input parameter in the MEPDG wherever possible. Based on the preliminary study performed under Task 1, the nationally established default vehicle distributions and load spectra based on the Long Term Performance Data can be used. Traffic data for roadways servicing special industry, such as mining, farming, etc., must be collected and analyzed so that factors such as vehicle class distributions and axle load spectra can be established based on the industry the roadway services.

10.2. Establish Built-in Gradient for a Larger Range of Pavement Structures

As part of WO#13, the built-in temperature gradient was established for 4 different pavement projects in Pennsylvania through instrumentation of the slabs with VW static strain gages. This was achieved for two pavements with 12-inch slabs and a ATPB; a 10-inch slab and a 14-inch slab both with a CTPB. This work needs to be expanded to include a larger variety of pavement structures representing the range of pavement structures commonly constructed in the state.

10.3. Validate Climatic Data

Environmental conditions have a significant impact on the performance of rigid pavements. Factors such as ambient temperature, relative humidity, precipitation, freeze-thaw cycles and depth to the water-table can influence the performance of the pavement at different time of the year. The change in temperature and moisture profiles in the pavement structure is considered in the MEPDG through a comprehensive climatic modeling tool named the Enhanced Integrated Climatic Model (EICM). The EICM uses climatic databases containing hourly data for sunshine, rainfall, wind speed, air temperature, and relative humidity. The data from more than 800 climatic stations in the United States is already incorporated into the MEPDG to be selected based on the project location. A total number of 23 climatic stations are available in the MEPDG for the state of Pennsylvania.

One issue that needs to be considered in regards to these climatic databases is the duration of the available data. A study of each climatic station in WO#13 revealed that all the Pennsylvania stations included at least five years of climatic data. Another significant concern with the climatic databases used for the design is the quality of the data. Data slips or possible errors in storing the hourly climatic data with respect to time can result in changes in the design thickness of the slab established by the MEPDG. To investigate the quality of the climatic data in each of the 23 climatic stations available for Pennsylvania, a sensitivity analysis can be performed. The 23 MEPDG climate stations available for the state will be used to determine the effect of climate on the predicted pavement performance for pavements constructed in Pennsylvania. Comparisons of the performance predicted by the MEPDG for different locations can be used to investigate the quality of the data used in the climatic databases.

10.4. Establish Coefficient of Thermal Expansion (CTE)

Results from the sensitivity study performed under Task 2 for WO#13 revealed that the predicted joint faulting and transverse cracking are sensitive to the CTE of the PCC and therefore, this parameter has a significant effect on the thickness design. CTE was established in WO#13 for only four different PCC mixtures used for the construction of four instrumented pavement sections in Western Pennsylvania. It is strongly encouraged to establish the CTE values for PCC made of aggregate from different quarries/sources across the state.

10.5. PCC Strength and Stiffness

A comprehensive study needs to be performed to establish the PCC strength and stiffness at different ages for different typical PCC mixtures across the state. The results of this study will be invaluable in two ways. First, it can be used to define the corresponding design inputs more accurately when using the MEPDG. Secondly, the empirical relations used to convert compressive strength to flexural strength and elastic modulus can be developed based on this data for Pennsylvania conditions.

10.6. Calibration of Cracking and Faulting Models

The performance models in the MEPDG, including the cracking and the joint faulting models, were calibrated based on the performance of about 248 field sections located in 22 States across the nation (ARA 2004). It must be pointed out that no sections in Pennsylvania were included in the calibration of the models in the MEPDG. Local calibration of the performance models is highly recommended and essential. For this purpose, the following type of data needs to be collected for different roadways in the state:

1. Pavement location, date and time of construction, design features and layers properties
2. Traffic data
3. Material characterization, including laboratory and field testing by performing Falling Weight Deflectometer (FWD)
4. Current distress surveys including percent slabs cracked and their severity and transverse joint faulting to validate the automated data
5. History of rehabilitation and maintenance

This work can be very challenging since the data for each road section can be scattered in different sources or broken into different sections. The data needs to be collected from online databases, PennDOT personnel, contractors, publications etc. and put together to complete a database for every section. Once the database is complete for a sufficient number of road sections, the calibration of the performance models can be performed.

References

AASHTO (1993). "Guide for Design of Pavement Structures." American Association of State Highway and Transportation Officials, Washington, DC.

Akita, H., T. Fujiwara and Y. Ozaka (1997). "A Practical Procedure for the analysis of Moisture Transfer in Concrete Due to Drying." Magezine of Concrete Research **49**(179): 129-137.

ARA, I., ERES Consultants Division. (2004). Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. National Cooperative Highway Research Program,. Transportation Research Board. Champaign, IL.

Asbahan, R. (2009). Effects of the Built-in Construction Gradient and Environmental Conditions on Jointed Plain Concrete Pavements. Civil Engineering. Pittsburgh, University of Pittsburgh. **PhD:** 534.

Burnham, T. and A. Koubaa (2001). A New Approach to Estimate the In-situ Thermal Coefecient and Drying Shrinkage for Jointed Concrete Pavements. 7th International Conference on Concrete Pavements, Orlando, FL.

Coree, B., H. Ceylan and D. Harrington (2005). Implementing the Mechanistic-Empirical Pavement Design Guide: Implementation Plan. Report No. IHRB Project TR-509. Ames, IA.

Coree, B., H. Ceylan and D. Harrington (2005). Implementing the Mechanistic-Empirical Pavement Design Guide: Technical Review. Report No. 03-166. Ames, IA.

Crawford, G. L., J. M., Gudimettla, J., Tanesi (2010). Inter-laboratory Study on Measuring Coefficient of Thermal Expansion of Concrete. Transportation Research Board 89th Annual Meeting. Washington, DC.

Gatti, K. A., S. Nassiri and J. M. Vandenbossche (2009). Establishing Appropriate Inputs When Using the Mechanistic-Empirical Pavement Design Guide (MEPDG) To Design Rigid Pavements in Pennsylvania-Task 1. Draft Report Prepared for Pennsylvania Department of Transportation. Pittsburgh, PA, University of Pittsburgh.

Hall, K. D. and S. Beam (2005). "Estimating the Sensitivity of Design Input Variables for Rigid Pavement Analysis with a Mechanistic-Empirical Design Guide." Transportation Research Board **1919**: 65-73

Jeong, J. H., L. Wang and D. G. Zollinger (2001). A Temperature and Moisture for Hydration Portland Cement Concrete Pavements. International Conference on Concrete Pavements, Orlando, FL.

Jeong, J. H. and D. Zollinger (2006). "Finite-Element Modeling and Calibration of Temperature Prediction of Hydrating Portland Cement Concrete Pavements." Journal of Materials in Civil Engineering © ASCE **18**(3): 317-324.

Jeong, J. H. and D. G. Zollinger (2005). "Environmental Effects on the Behavior of Jointed Plain Concrete Pavements." Journal of Transportation Engineerg., ASCE **131**(2): 140-148.

Nassiri, S., L. C. Ramirez and J. M. Vandenbossche (2009). Establishing Appropriate Inputs When Using the Mechanistic-Empirical Pavement Design Guide (MEPDG) To Design Rigid Pavements in Pennsylvania-Task 3. Draft Report Prepared for Pennsylvania Department of Transportation. Pittsburgh, PA, University of Pittsburgh.

Nassiri, S. and J. M. Vandenbossche (2010). Establishing Appropriate Inputs When Using the Mechanistic-Empirical Pavement Design Guide (MEPDG) To Design Rigid Pavements in Pennsylvania-Task 4. Draft Report Prepared for Pennsylvania Department of Transportation. Pittsburgh,PA, University of Pittsburgh.

Nassiri, S., J. M. Vandenbossche and K. A. Gatti (2009). Establishing Appropriate Inputs When Using the Mechanistic-Empirical Pavement Design Guide (MEPDG) To Design Rigid Pavements in Pennsylvania-Task 2. Draft Report Prepared for Pennsylvania Department of Transportation. Pittsburgh, PA, University of Pittsburgh.

Oh, H. B. and S. W. Cha. (2003). "Nonlinear Analysis of Temperature and Moisture Distributions in Early-Age Concrete Structures Based on Degree of Hydration." ACI Materials Journal **100**: 361-369.

Ruiz, J., M., P. J. Kim and A. K. Schindler (2001). "Validation of HIPERPAV for Prediction of Early-Aage Jointed Concrete Pavement Behavior." Transportation Research Board **1778**: 17-25.

Ruiz, J. M., R. O. Rasmussen, G. K. Chang, J. C. Dick, P. K. Nelson, A. K. Schindler, D. J. Turner and W. J. Wilde (2006). Computer-Based Guidelines for Concrete Pavements, Volume III: Technical Appendices, FHWA-HRT-04-127.

Schindler, A. K. and K. J. Folliard (2005). "Heat of Hydration Models for Cementitious Materials." ACI Materials Journal **102**(1): 24-33.

Thompson, M. R., B. J. Dempsey, H. Hill and J. Vogel (1987). "Characterizing Temperature Effects for Pavement Analysis and Design." Transportation Research Board **1121**: 14-22.

Vandenbossche, J. M. and F. Mu. (2008). Evaluation of the NCHRP 1-37A Mechanistic-Empirical Rigid Pavement Design Guide Procedure, Task 3. FHWA Contract Number: DTFH61-05-P-00094.

Xu, Q., M. Ruiz, G. K. Change, R. O. Rasmussen and D. K. Rozycki (2009). "A Moisture Transport Model for Enhancing the FHWA HIPERPAV Predictions." Transportation Research Board.