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AASHTO MEPDG REGIONAL PEER EXCHANGE MEETINGS

FINAL TECHNICAL REPORT

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16. Abstract

Based on the demonstrated success of the Wisconsin peer exchange and the continued advancement of SHA implementation of the MEPDG, the Federal Highway Administration (FHWA) hosted four additional regional peer exchange meetings to foster the sharing of SHA experiences and assist in the overall implementation effort. The four regional peer exchange meetings included:

- Southeast AASHTO Region 2, Atlanta, Georgia, November 5-6, 2014
- Southwest AASHTO Region 4, Phoenix, Arizona, January, 20-22, 2015.
- Northwest AASHTO Region 4, Portland, Oregon, April 14-15, 2015
- Northeast AASHTO Region 1, Albany, New York, May 13-14, 2015.

This report summarizes the discussions of all four peer exchange meetings.

The overarching goals of the four MEPDG regional peer exchange meetings included:

- Provide an opportunity for peers to discuss issues related to the MEPDG and the accompanying AASHTOWare software
- Provide a forum for the exchange of information for the participating SHAs
- Prepare peer exchange meeting reports that provide a way of documenting the significant findings so that they may be effectively used by SHAs and others pursuing the implementation of the MEPDG and AASHTOWare Pavement ME Design™.

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Introduction and Background

In 2008, the American Association of State Highway and Transportation Officials (AASHTO) released the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (MEPDG). The MEPDG is the first mechanistic-empirical pavement design procedure to be based on nationally calibrated pavement performance prediction models (AASHTO 2008). The accompanying software, AASHTOWare Pavement ME Design™, was released in 2011.

In September 2013, the Wisconsin Department of Transportation (WisDOT) initiated an outreach program to conduct an MEPDG implementation peer exchange meeting with state highway agencies (SHA) in AASHTO Region 3 (which includes Illinois, Indiana, Iowa, Kansas, Kentucky, Michigan, Minnesota, Missouri, Ohio, and Wisconsin). The intent of this peer exchange was to share experiences with five key aspects of MEPDG implementation: calibration, materials testing, traffic data, design acceptance, and deployment (WisDOT 2013). The Wisconsin peer exchange meeting identified a number of key findings that could aid SHAs in MEPDG implementation, including (WisDOT 2013):

- SHAs are generally moving forward with implementing the MEPDG and most have plans for full adoption by 2015.
- Local calibration is essential for establishing accuracy, knowledge, and acceptance of the MEPDG.
- More information is needed on what SHAs are adopting for default versus calibrated inputs, and calibration guidance following software updates.
- Concerns with the timing between establishing the pavement design and initiating construction (i.e., difficult to quantify in situ material properties during the design stage).
- SHAs are just beginning to evaluate concrete thermal expansion in accordance with AASHTO T-336.
- Concerns that the MEPDG traffic data needs exceed the suitability of available traffic data, as well as concerns with growth rates, seasonal changes, and data verification.
- Uncertainty with the design acceptance process for design-build, public-private partnerships, and consultant designs.
- Training for the overall MEPDG concept and software is needed.
- SHAs should carefully set policies for inputs, level of design, and other variables.
- More information is needed on SHA deployment issues and how future software upgrades will affect usage.

Overall, the Wisconsin peer exchange meeting proved to be successful in providing SHAs with a platform for exchanging and sharing ideas, experiences, tips, and concerns in relation to implementing the MEPDG. Additionally, participants concluded that more state-by-state information could prove useful to individual SHAs for assessing the implementation process and for customizing the MEPDG to agency conditions (WisDOT 2013).

FHWA Peer Exchange Meetings

Based on the demonstrated success of the Wisconsin peer exchange and the continued advancement of SHA implementation of the MEPDG, the Federal Highway Administration (FHWA) hosted four additional regional peer exchange meetings to foster the sharing of SHA experiences and assist in the overall implementation effort. The four regional peer exchange meetings included:

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This report summarizes the discussions of all four peer exchange meetings.

Meeting Goals

The overarching goals of the four MEPDG regional peer exchange meetings included:

- Provide an opportunity for peers to discuss issues related to the MEPDG and the accompanying AASHTOWare software.
- Provide a forum for the exchange of information for the participating SHAs.
- Prepare peer exchange meeting reports that provide a way of documenting the significant findings so that they may be effectively used by SHAs and others pursuing the implementation of the MEPDG and AASHTOWare Pavement ME Design™.

Participants

A total of thirty-four state and provincial highway agencies (including two Canadian provinces) participated in the MEPDG peer exchange meetings. In addition, participants representing six universities, AASHTO, consultants, and the concrete and asphalt industries also attended. Figure 1 illustrates the highway agencies that attended the MEPDG peer exchange meetings. The meeting participants are listed in Appendix A.

Agenda

The typical agenda used for the MEPDG peer exchange meetings is provided in Appendix B.

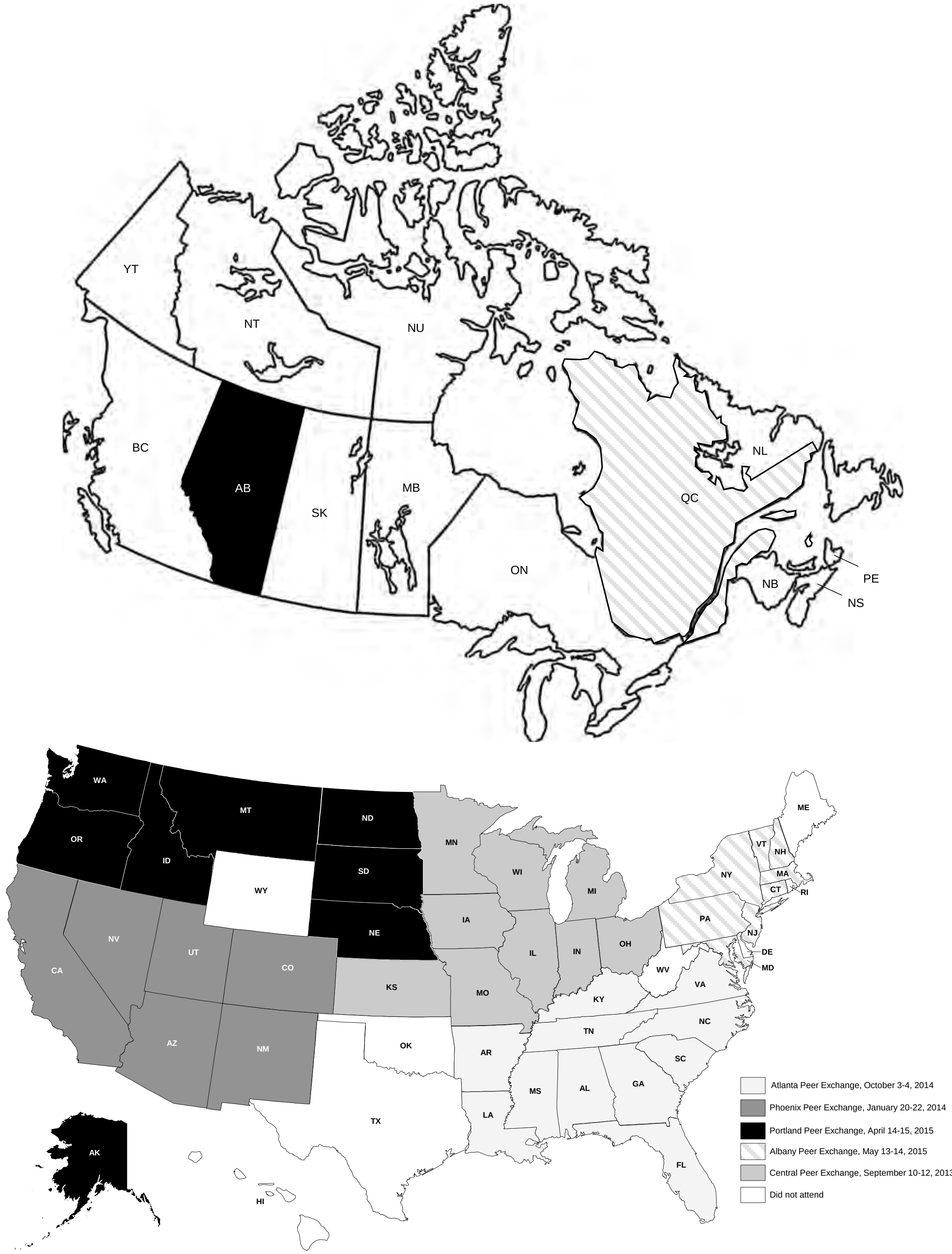


Figure 1. Photo. MEPDG peer exchange meeting attendees.

AASHTOWare Pavement ME Design™ Update

The following provides a summary of the current software release, the upcoming software release, and the proposed future software enhancements.

Current Version

- Released summer 2014, new features included:
  - Backcalculation summary reports – includes option to generate a backcalculation summary report that includes specific distress data per station, and a unique chart showing the average, standard deviation, and percent passing for each distress type. Also includes the option to run backcalculation with layer thickness optimization.
  - Automatic software updater – allows the user the option of automatically checking, downloading, and installing software updates.
  - Subgrade modulus sensitivity analysis – allows the user to conduct a subgrade layer moduli sensitivity analysis.
  - Context sensitive help – allows the user to point and click on terms and be directed to the appropriate location in the software help document.

  Note: additional details included in the software release notes for v 2.1 may be found at http://me-design.com/MEDesign/data/AASHTOWare%20Pavement%20ME%25%2020Design%20Build%202.1.x%20Release%20Notes.pdf.
- Special traffic loading feature for flexible pavements.
- Stand-alone version of the *Drainage Requirement in Pavements (DRIP)* and user guide (available for download at http://me-design.com/MEDesign/DRIP.html).
- Current software licenses: 48 educational, 60 stand-alone, and 69 consultant licenses.

2015 Release

- Release date – July 2015.
- Correct error in freezing index calculation (primarily an issue with rigid pavements).
- Correct issue with automatic updater (patch has already been released).
- Incorporate the reflection cracking model for asphalt pavements developed under NCHRP 1-41 project, *Models for Predicting Reflection Cracking of Hot-Mix Asphalt Overlays* (Lytton et al. 2010).
- Include the FHWA Long-Term Pavement Performance (LTPP) high quality traffic data (generation of new traffic XML files) and additional climate data (2006 to present).
- Develop MapME to provide GIS data links for climate, traffic, and soils data. MapME will be released separately from AASHTOWare Pavement ME Design™ release.
- Develop application programming interface (API) for the integrated climatic model (ICM), JULEA, and project file.

- Incorporate the results of NCHRP 20-07/Task 327, *Developing Recalibrated Concrete Pavement Performance Models for the Mechanistic-Empirical Pavement Design*. This NCHRP project recalibrated the concrete pavement performance prediction models using the coefficient of thermal expansion values obtained from laboratory testing.

2016 Enhancements

- Code clean-up, include U.S. Customary and SI units, and technical audit of code for engineering errors (e.g., removing code that is not used by the software, adding code comments, correcting hard-coded constant numeric values, and providing consistent logic levels). This enhancement will also correct an issue with the thermal cracking model (the tensile strength calculation is not temperature dependent and will require recalibration). This is the task force's top priority.
- Process for evaluating thin bonded concrete overlays. Additional information on bonded concrete overlays of asphalt pavements mechanistic-empirical design procedures can be found at http://www.engineering.pitt.edu/Vandenbossche/BCOA-ME/.
- Backcalculation (Part 1) pre-processing tool. Parts 2 and 3 will include incorporation of other backcalculation software programs (MODCOMP, MODULUS, and Evercalc), and is dependent on backcalculation programs source code availability.
- Training on mechanistic-empirical pavement design principles, MEPDG, and software.
- Review upcoming research results for potential incorporation into the software. Research results require approval from both the AASHTO Joint Technical Committee on Pavements and the AASHTOWare Pavement ME Design™ Task Force prior to being included in the software.

On-going Efforts

- Incorporate enhanced climate data from the Modern-Era Retrospective Analysis for Research and Applications (MERRA) database (see Appendix C for additional details).
- Develop AASHTOWare Pavement ME Design™ as a web-based application (estimated cost $1 to $1.5 million, anticipated to begin no sooner than 2017).

Agency MEPDG Implementation Status

The following provides a brief summary of the participating agencies' MEPDG implementation status. Agency presentations are provided in Appendix D.

- **Alabama Department of Transportation (DOT)**. Through the Auburn University, Alabama DOT is providing MEPDG training, conducting a study to automatically generate an axle load spectra file, and developing a materials library containing California Bearing Ratio (CBR) values for soils and hot mix asphalt (HMA) dynamic modulus.
- **Alaska Department of Transportation and Public Facilities (DOT&PF)**. The Alaska DOT&PF has developed its own mechanistic–empirical asphalt pavement design procedure and software, Alaska Flexible Pavement Design (AKFPD) (McHattie 2004). In cooperation with the University of Alaska Fairbanks, a life-cycle analysis module is being added to the AKFPD process (Lee, McHattie, and Liu 2012). Since an Alaska-

specific ME analysis tool has been developed, the implementation of MEPDG is not presently the highest priority pavement effort for the Alaska DOT&PF.

- **Alberta Transportation**. At this time, no Canadian Province has fully implemented the MEPDG; however, Ontario is probably the farthest along in the evaluation. In addition, the Canadian Provinces have initiated a MEPDG User Group and have developed a *Canadian Guide: Default Parameters for AASHTOWare* Pavement ME Design™ (see Appendix E).
- **Arizona DOT**. Arizona DOT has recently completed an MEPDG calibration and implementation study and are conducting parallel designs using DARWin. In addition, they are in the process of finalizing a MEPDG user manual. They indicated that they are looking into how to transition from local calibration to implementation, what issues need to be resolved, and what are the necessary steps.
- **Arkansas State Highway and Transportation Department (SHTD)**. The Arkansas SHTD is in the process of conducting concurrent designs (less than five conducted to date), developing a materials library and design input catalogs, and calibrating the HMA pavement performance prediction models. The majority of the Arkansas SHTD MEPDG implementation effort is being conducted by Kevin Hall at the University of Arkansas.
- **California DOT (Caltrans)**. Caltrans has implemented the rigid pavement design portion of the MEPDG. A pavement design catalog has been developed for use by Caltrans Design Engineers. Pavement ME Design™ is currently licensed by the Central Office for research, forensic, and investigation purposes.
- **California DOT (Caltrans)**. Caltrans has implemented the rigid pavement design portion of the MEPDG. A pavement design catalog has been developed for use by Caltrans Design Engineers. AASHTOWare Pavement ME Design™ is currently licensed by the Central Office for research, forensic, and investigation purposes.
- **Colorado DOT**. Colorado DOT has implemented the MEPDG for use on all pavement designs. Colorado is also looking to identify what's been done by other agencies, what still needs to be completed, and what are the training needs.
- **Connecticut DOT**. The University of Connecticut conducted a study to develop MEPDG design inputs specific to Connecticut (Yut, Mahoney, and Zinke 2014). The Phase II study is anticipated to conduct calibration/validation and develop a user guide.
- **Florida DOT**. The Florida DOT has implemented the jointed plain concrete pavement (JPCP) portion of the MEPDG. Currently, they are evaluating the new software release to determine the impacts and changes and whether or not they will need to recalibrate the JPCP performance prediction models. The Florida DOT is also in the process of constructing a concrete test road for further evaluation of the JPCP designs.
- **Georgia DOT**. The Georgia DOT is in the final stages of an MEPDG implementation study and the consultant is conducting the initial MEPDG training course. The Georgia DOT is also conducting a local calibration study that is expected to be completed by January 2015. The Georgia DOT MEPDG user guide is being finalized and concurrent pavement designs using the MEPDG will be conducted starting in 2015. The state currently uses the AASHTO 1972 pavement design procedure.

- **Idaho Transportation Department (TD)**. Districts are currently using the AASHTOWare Pavement ME Design™ as a design check for the current pavement design procedure. They anticipate full implementation within the next couple of years.
- **Kentucky Transportation Cabinet (TC)**. The Kentucky TC has been conducting mechanistic-empirical-based designs since the 1970s. They are currently in the first phase of the MEPDG validation and calibration process.
- **Louisiana Department of Transportation and Development (DOTD)**. The Louisiana DOTD has completed materials characterization and traffic evaluation using PrepME (developed under pooled fund study TPF-5(242) to assist agencies in preparing and managing the input data for the AASHTOWare Pavement ME Design™) and are conducting a study on local calibration through the Louisiana Transportation Research Center (LTRC). The Louisiana DOTD will be conducting concurrent designs and comparing the MEPDG results with the results from DARWin. They are also in the process of constructing additional weigh-in-motion WIM sites and determining distress threshold criteria.
- **Maryland State Highway Administration (SHA)**. The Maryland SHA has included a chapter in the *Pavement & Geotechnical Design Guide* (MDSHA 2015) for use of the MEPDG for Maryland SHA new construction projects. The AASHTO *1993 Pavement Design Guide* is used as a design check.
- **Massachusetts DOT**. The Massachusetts DOT has tried to calibrate the MEPDG pavement performance models, but has not completed this effort due to very few new construction pavement designs.
- **Mississippi DOT**. The Mississippi DOT has completed traffic characterization, a climate evaluation study through the National Center for Asphalt Technology (NCAT), and HMA dynamic modulus testing, and are in the process of characterizing concrete materials. An MEPDG implementation plan has been developed (State Study 163) and field work for collecting falling weight deflectometer (FWD) data to characterize in situ materials for local calibration will begin in February 2015.
- **Mississippi DOT**. The Mississippi DOT has completed traffic characterization, a climate evaluation study through the National Center for Asphalt Technology (NCAT), and HMA dynamic modulus testing, and are in the process of characterizing concrete materials. An MEPDG implementation plan has been developed (State Study 163) and field work for collecting falling weight deflectometer (FWD) data to characterize in situ materials for local calibration will begin in 2015.
- **Montana DOT**. The Montana DOT has conducted a MEPDG performance prediction model calibration study (Von Quintus and Moulthrop 2007). However, they are unsure of the impact of model changes that have occurred between the current version of the AASHTOWare Pavement ME Design™ and the NCHRP 1-37A software version.
- **Nebraska Department of Roads (NOR)**. A study was conducting in 2009 (Ala, Stanigzai, and Azizinamini 2009) that evaluated the development of needed field data for MEPDG implementation, but not much work has been conducted with implementation since that time.

- **Nevada DOT**. Nevada DOT conducts pavement designs using modified AASHTO 93 guide as the final design for construction. These designs are redone using the MEPDG for comparison and evaluation. Nevada DOT has completed calibration of the concrete pavement performance models (study conducted by the University of Nevada – Las Vegas) and are working on calibration of the asphalt pavement performance models (study being conducted by the University of Nevada – Reno). Nevada DOT has implemented the rigid pavement design portion of the MEPDG and plans implementation of the asphalt portion by July 2015.
- **New Hampshire DOT**. The New Hampshire DOT is currently using the AASHTO 1972 design procedure. They have had some activity in the evaluation of the MEPDG, but have yet to decide on whether or not they will implement the MEPDG.
- **New Jersey DOT**. The New Jersey DOT has developed an MEPDG materials database. Pavement designs are currently conducted using the AASHTO *1993 Pavement Design Guide* and DARWin v3.1 software.
- **New Mexico DOT**. New Mexico DOT currently conducts all pavement designs using a hybrid-version of the AASHTO 1993 Pavement Design Guide and compares the results to the MEPDG. New Mexico DOT is currently calibrating the asphalt pavement performance models. They are also interested in being able to design thin bonded concrete overlays and evaluate the use of mechanically stabilized materials in the MEPDG. In 2012, the New Mexico DOT instrumented and asphalt pavement on Interstate 40 west of Albuquerque, New Mexico. The instrumented pavement was designed using the AASHTOWare Pavement ME Design™ software and material testing was conducted to validate the pavement design. They are also conducting falling weight deflectometer (FWD) testing and possibly trench studies for validation of in-place layer moduli and distress.
- **New York State DOT**. New York State DOT has participated in a number of pool-fund studies in relation to the MEPDG. Current pavement design tables are based on the AASHTO *1993 Pavement Design Guide* and DARWin v3.1. However, they are in the process of revising the design tables using results from the MEPDG.
- **North Carolina DOT**. The North Carolina DOT has been conducting pavement design/analysis using the MEPDG since 2011. At least twenty-four pavement designs have been conducted to date (mostly new construction). Studies for materials and traffic characterization and local calibration have been completed; however, the DOT is evaluating whether or not the models need to be recalibrated. The North Carolina DOT is conducting two studies, one to evaluate the cost competitiveness of aggregate base course designs compared to full-depth asphalt pavements and another to determine the impacts of subgrade resilient modulus on the resulting layer thicknesses. The North Carolina DOT is conducting pavement designs using level 2 inputs. A pooled-fund study, *Pavement Subgrade Performance Study*, SPR-2(208), is being conducted to improve the mechanistic subgrade failure criteria and evaluate the effect of the environment on the subgrade resilient modulus. The SPR-2(208) pooled-fund study is expected to be completed by the end of 2014.
- **North Dakota DOT**. The North Dakota DOT has locally calibrated the performance prediction models for JPCP.

- **Oregon DOT**. The Oregon DOT used a low-budget approach (e.g., minimal materials testing, model calibration/validation) for calibrating the pavement performance prediction models. At this time, the AASHTOWare Pavement ME Design™ software is used to evaluate concrete pavement designs, JPCP and continuously reinforced concrete pavement (CRCP), and asphalt pavements subjected to high volume traffic loads. Oregon DOT also stated that they are uncomfortable with the analysis results that suggests thinner asphalt pavement sections are appropriate.
- **Pennsylvania DOT**. The University of Pittsburgh and the Pennsylvania State University have conducted research for the Pennsylvania DOT. Pennsylvania DOT is currently developing a MEPDG user guide specific to Pennsylvania conditions. Full implementation of the MEPDG is anticipated within 18 months.
- **Quebec Ministry of Transportation (MOT)**. The Quebec Ministry of Transportation has developed material, traffic, climatic, and calibration databases. They are beginning to look at the calibration process. The Ontario MOT is probably the farthest along of Canadian Provinces in the evaluation of the MEPDG. The Canadian Provinces have initiated a MEPDG user group and have developed a *Canadian Guide: Default Parameters for AASHTOWare Pavement ME Design*™ (see Appendix E).
- **South Carolina DOT**. The South Carolina DOT has issued a Request for Proposal (RFP) for HMA dynamic modulus and portland cement concrete (PCC) coefficient of thermal expansion (CTE) testing. The South Carolina DOT is also in the early stages of a calibration study to determine sources of available data, and develop a test site implementation plan. The DOT plans on conducting concurrent designs using level 3 design inputs.
- **Tennessee DOT**. The Tennessee DOT is conducting research efforts to develop an HMA materials library, to perform a sensitivity analysis, and to calibrate the MEPDG pavement performance prediction models. Implementation of the MEPDG is expected to occur by 2016.
- **Utah DOT**. Utah DOT began conducting pavement designs using the MEPDG in 2004. They have been conducting side-by-side comparisons with the DARWin since 2010. As of 2011 they have been using the MEPDG on all pavement design projects except for Federal Aid – Local projects. The Federal Aid – Local projects will be required to use the MEPDG for all pavement designs starting in 2015. They are currently in the process of providing training to local agencies through UDOT regional personnel. The MEPDG has shown to work well with typical pavement designs in Utah; however, it is difficult (at least not as intuitive) to use with other rehabilitation designs, such as, hot in-place recycling, cold in-place recycling, and thin concrete overlays.
- **Vermont Agency of Transportation (AOT)**. The Vermont AOT is currently in the calibration phase of the MEPDG implementation and are deciding whether or not to change the default performance prediction equation calibration coefficients. Pavement designs are currently conducted using the AASHTO *1993 Pavement Design Guide.*
- **Virginia DOT**. The Virginia DOT is developing a materials library, conducting traffic analysis and subgrade classification studies. The DOT is in the process of conducting district training and began conducting concurrent designs in 2014.

- **Washington State DOT**. The Washington State DOT calibrated both the asphalt and JPCP models using the NCHRP 1-37A software (MEPDG v 0.9). The calibrated models have yet to be validated. The primary asphalt pavement distress type in Washington State is top-down cracking. Since the top-down cracking model does not accurately reflection local conditions, the DOT has yet to fully implement the design procedure.

Climate

The following provides a brief summary of agency activities related to climate characterization.

- **Colorado DOT**. Colorado DOT has developed a white paper that describes the process for including additional weather stations into the AASHTOWare Pavement ME Design™ software (see Appendix E). Colorado DOT mentioned that the majority of larger airports include a Class 1 weather station.
- **Florida DOT**. The Florida DOT is finding significant differences in concrete layer thicknesses by changing only the weather station location; all other inputs being held constant. The Kentucky TC noted that in their evaluation they did not find large differences in results by varying only the weather station.
- **Louisiana DOTD**. The Louisiana DOTD is in the process of developing climate files for each parish.
- **Maryland SHA**. Currently, there are only four weather stations in the state of Maryland; however, two of the weather stations have missing information. Data for adjacent states are being used to develop virtual weather stations. Maryland SHA will be evaluating use of the MERRA data when it becomes readily available.
- **Mississippi DOT**. The Mississippi DOT conducted a research study to develop more accurate 40-year historic climate data input files (Truax, Heitzman, and Takle 2011). A sensitivity analysis showed that repeating limited climate data in the MEPDG results in significantly higher predicted distress (in some cases).
- **Montana DOT**. Montana is a very large state with many microclimates. Due to the cold climate, transverse cracking is a significant issue. A Montana DOT research project (Von Quintus and Moulthrop 2007) determined an issue with the climate data, resulting in potential issues with the transverse cracking model.
- **Nevada DOT**. Nevada DOT noted that there are only seven weather stations included in the MEPDG for the state of Nevada.
- **New Mexico DOT**. New Mexico DOT is concerned that the MEPDG will not accurately capture climatic effects in New Mexico.
- **North Carolina DOT**. The North Carolina DOT is adding 20 years of climate data (to be completed soon).
- **South Dakota DOT**. The AASHTOWare Pavement ME Design™ software currently includes data for only eleven weather stations in South Dakota. The South Dakota DOT initiated a research study to determine the availability and quality of climate and groundwater data from other existing data sources. Through this research, data from 1,572 additional ground water monitoring wells and 176 weather stations were identified. The additional weather stations include ground-based weather stations, environmental

sensing stations, and MERRA weather stations. This project will be completed August 2015 and will develop a climate database that incorporates the MERRA weather stations and groundwater tables for project-specific locations.

- **Transportation Association of Canada (TAC) MEPDG User Group**. The User Group has developed a climatic database that has been included in AASHTOWare Pavement ME Design™.
- The Modern Era Retrospective-Analysis for Research and Applications (MERRA) provides climate and weather data through modeling and data assimilation of satellite observations (http://gmao.gsfc.nasa.gov/merra/). MERRA contains all of the MEPDG needed climate information for more than 3,000 uniformly distributed grid points in the contiguous U. S., with more stations abroad. The MERRA data provides better continuous data (no data gaps from 1979 to present), higher quality data (NASA data checks), and provides planned improvements over time (spatial resolution on the order of 10 meters) (Schwartz, Forman, and Leininger 2015).
- It has been noted that changing weather stations can impact concrete pavement design results (all other inputs held constant). It is highly recommended that the climate data be reviewed to check for and remove any data anomalies. It is also recommended that a virtual weather station be created to minimize potential data issues.
- It was also noted that many of the enhanced integrated climatic model (EICM) default values should not be changed unless recalibration is conducted.

Traffic

The following provides a brief summary of agency activities related to traffic characterization.

- **Alabama DOT**. The Alabama DOT is developing a process to automatically generate axle load spectra files from WIM site data based on project location. Traffic analysis indicates that the actual truck loads are drastically different than the national (default) values included in the AASHTOWare Pavement ME Design™ software. The Alabama DOT is also determining whether or not additional WIM sites are needed.
- **Arizona DOT**. Arizona DOT, under project SPR-672, has characterized traffic loadings, vehicle distribution, lane distribution, and other traffic inputs. SPR-672 project objectives included identifying MEPDG traffic data input needs (level 2), evaluating Arizona DOTs traffic data collection, storage, and analysis practices, conducting data quality checks, and developing an action plan for obtaining needed traffic data. The traffic data analysis project was conducted using the following steps:
  - Identify traffic data sources and compare data collection, accuracy, and storage practices.
  - Conduct data processing and review, identify anomalies and errors, and conduct data cleansing.
  - Conduct statistical analysis for generating traffic data clusters.
  - Determine optimum number of clusters by traffic data type.
  - Conduct sensitivity analysis and interpret results.

   - Develop default statewide MEPDG level 2 traffic inputs.

  Arizona DOT noted that they have fairly decent traffic data and plan on using six clusters for characterizing traffic across the state. In 2015 they plan on adding fifteen additional weigh-in-motion (WIM) sites and are conducting a feasibility study for an additional thirty WIM sites in 2016. WIM sites are primarily being added for enforcement purposes.

- **Arkansas DOT**. The Arkansas DOT is in the process of adding more WIM sites. In addition, portable WIM sites are being added on the secondary roads primarily due to pavement failure due to heavy truck haul.
- **Georgia DOT**. The Georgia DOT currently maintains thirty WIM sites; however, these are primarily used for safety and enforcement. Data collection at the WIM sites is outsourced and traffic files are provided to the DOT in AASHTOWare Pavement ME Design™ format.
- **Idaho TD**. The Idaho TD has a total of twenty-seven weigh-in-motion (WIM) sites, most of which are located along the I-84 corridor. The TrafLoad software was used to process the WIM data; however, two of the sites could not be analyzed and only twenty-one of the sites contain continuous classification data. In addition, FHWA quality data checks were conducted on the Class 9 truck weights and it was determined that only fourteen of the WIM sites complied with the data quality requirements (Bayomy, El-Badawy, and Awed 2012). A traffic input database was developed for Idaho conditions; AADTT can be modified and the database tool will generate the needed MEPDG data inputs based on WIM site data. A presentation on evaluating mega loads in Idaho using the MEPDG has been provided (Von Quintus 2011).
- **Kentucky TC**. The Kentucky TC is collecting additional WIM data and using PrepME for data quality control. WIM data are being grouped according to roadway functional class. The use of the initial count and percent growth rate without traffic forecasting is a mindset shift for the traffic division. Default values are being used until more WIM data can be collected. The Kentucky TC has good traffic characterization data.
- **Louisiana DOTD**. The Louisiana DOTD is adding twenty-seven additional WIM sites.
- **Maryland SHA**. The Maryland SHA has completed a study on traffic implementation and determined that there is an insufficient number of WIM sites across the state. Maryland SHA is looking to partner with the Motor Carrier Division to develop joint WIM sites that will serve mutual needs, as well as potentially upgrading qualified automatic traffic recorder sites to WIM sites. When more WIM data is available, the primary data processing tool is envisioned to be PrepME.
- **Montana DOT**. Montana DOT maintains a total of sixteen WIM sites across the state.
- **New Mexico DOT**. New Mexico DOT is working on developing their traffic database; however, are having some challenges in figuring out a method for importing the traffic database into the MEPDG. At this time, the DOT has five WIM sites, three of which are on NM-550.
- **New York State DOT**. As part of the MEPDG flexible pavement design table project, it was determined that cluster averages did not significantly affect predicted pavement performance. Based on this analysis it was recommended that single statewide average

values be used for vehicle class distribution (VCD), monthly distribution factor (MDF), axle group per vehicle (AGPV), and axle load spectra be used to characterize traffic conditions in New York State (Romanoschi, Abdullah, and Bendana 2014).

- **North Carolina DOT**. The North Carolina DOT is using a clustering approach for analyzing the forty-two WIM sites across the state. Traffic data on secondary roads are limited. Significant cleansing of the traffic data file is needed prior to use. North Carolina DOT has developed nine MEPDG traffic data files based on roadway functional class.
- **Oregon DOT**. The Oregon DOT has established the required MEPDG traffic inputs. This effort used a “virtual” truck such that no class-specific weight distribution data would be needed. Average values were used for the number of axles per truck and the axle spacing along with the hourly truck volume distribution data.
- **Quebec MOT**. The Quebec MOT has developed axle load spectra from their WIM sites.
- **South Carolina DOT**. The South Carolina DOT is conducting a study to determine what traffic data needs to be collected and whether or not they can use portable WIM sites to collect the needed data. Their evaluation of one WIM site showed that 8.3 percent of total truck observations were either overweight per axle or gross weight. Since the percent of trucks is expected to increase over the next 20 years, the impact of this needs to be evaluated
- **Tennessee DOT**. The Tennessee DOT currently only has one WIM site, but is looking to add additional sites, possibly portable WIM stations.
- **Utah DOT**. Utah DOT stated that they have sufficient WIM and automated traffic counter (ATC) sites to generate all needed level 1 traffic inputs. Their biggest challenge was converting ten traffic data files from the original MEPDG software into two traffic files for use with the AASHTOWare Pavement ME Design™ software. ARA modified the traffic converter software that was originally developed for the Mississippi DOT for use with Utah DOT traffic data.
- **Virginia DOT**. The Virginia DOT uses one statewide traffic load distribution for all designs. They also noted that truckers may avoid portable WIM sites since they will think it is being used for enforcement. If truckers are avoiding the portable sites, then the number and type of trucks in the traffic stream may be biased.
- The PrepME tool was developed to assist agencies in data preparation and to improve the management and workflow of AASHTOWare Pavement ME Design™ data inputs (Wang, Li, and Chen 2015). PrepME software for traffic and data preparation for AASHTO MEPDG analysis and design is available to state highway agencies by contacting Dr. Doc Zhang at the Louisiana Transportation Research Center (doc.zhang@la.gov or 225-767-9162). Additional details for PrepME are provided in Appendix G.
- Although the traffic growth rate is typically based on the overall traffic growth rate (i.e., cars and trucks), having individual truck growth rates for each truck vehicle classification would be ideal; however, since Class 9 vehicles are the most predominant truck type, having the growth rate for this vehicle class would be acceptable.

- FHWA has developed guidelines and software for assisting agencies in selecting axle load defaults for use with the AASHTOWare Pavement ME Design™ software. The LTPP Pavement Loading User Guide (LTPP-PLUG) provides guidelines on selecting default axle loads as well as the process for generating additional MEPDG traffic loading defaults based on agency WIM data.

Materials

The following provides a brief summary of agency activities related to materials characterization.

- **Alaska DOT&PF**. The primary pavement type in Alaska is hot mix asphalt (HMA) over granular and/or asphalt treated base. Master curves and corresponding coefficients were determined for each of the primary HMA mixtures used by the Alaska DOT&PF. Granular base course testing was conducted and included resilient modulus (repeated load triaxial) testing and determining k_1, k_2, and k_3 coefficients based on the percent of fine material and moisture content. Characterization of the asphalt treated base includes resilient modulus based on asphalt content. A materials database has been developed and includes test results for HMA, granular base, and asphalt-treated base materials.
- **Colorado DOT**. Colorado DOTs current method for quantifying subgrade soils may underestimate the resilient modulus at low R-values and overestimate at high R-values. Colorado DOT uses a modified version of AASHTO T 307, *Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials*, which requires trimming the sample prior to testing. Performance models have been calibrated based on the results of the AASHTO T 307 test results. FWD and backcalculation results can be used; however, moisture content at time of FWD testing needs to be collected. Colorado DOT noted that modeling of expansive soils and frost susceptibility is currently not include in the MEPDG or accompanying software. Colorado DOT stated that the new CTE values for Colorado where relatively close to each other and is considering collecting more CTE values.
- **Idaho TD**. Idaho TD has developed a flexible pavement database that includes asphalt material characterization for binders (G* and delta) and mixtures (E* and volumetric properties). In addition, a gyratory stability-based model has been developed to determine E* for typical Idaho TD asphalt mixtures (see Abdo et al. 2009 for additional details). The unbound materials and subgrade soils characterization includes R-value, resilient modulus (M_r) using a correlation with the R-value, liquid limit, and plasticity index. An interactive Microsoft Excel workbook has been developed for accessing the Idaho TD materials, traffic, and climate database (see screenshot shown in figure 2).
- **Maryland SHA**. The University of Maryland MEPDG asphalt pavement sensitivity study determine that binder grade alone does not result in a significant change in asphalt pavement performance prediction (Schwartz et al. 2011). This same study also determined that the difference between level 1, 2, 3 inputs did make a difference in performance prediction. During the local calibration process, Maryland SHA plans to investigate the influence of the dynamic modulus on pavement performance prediction. Maryland SHA routinely collects all physical concrete mixture data (e.g., water-cement ratio, cement type) during construction and plans to conduct 28-day strength testing on future concrete paving projects. For unbound materials and subgrade soils, the MEPDG

assumes that the resilient modulus value is at optimum moisture content. If the moisture content is unknown, it is better to use the MEPDG default values or the user can disconnect the Enhanced Integrated Climatic Model (EICM) and input resilient modulus for each month of the year. Von Quintus and Killingsworth (1997) provide suggested procedures for determining the design resilient modulus for subgrade soils. Maryland SHA has a reasonable amount of resilient modulus data available for A-2-4, A-4, and A-6 materials, but has gaps in the data for A-1-a, A-1-b, A-7-5, and A-7-6 materials.

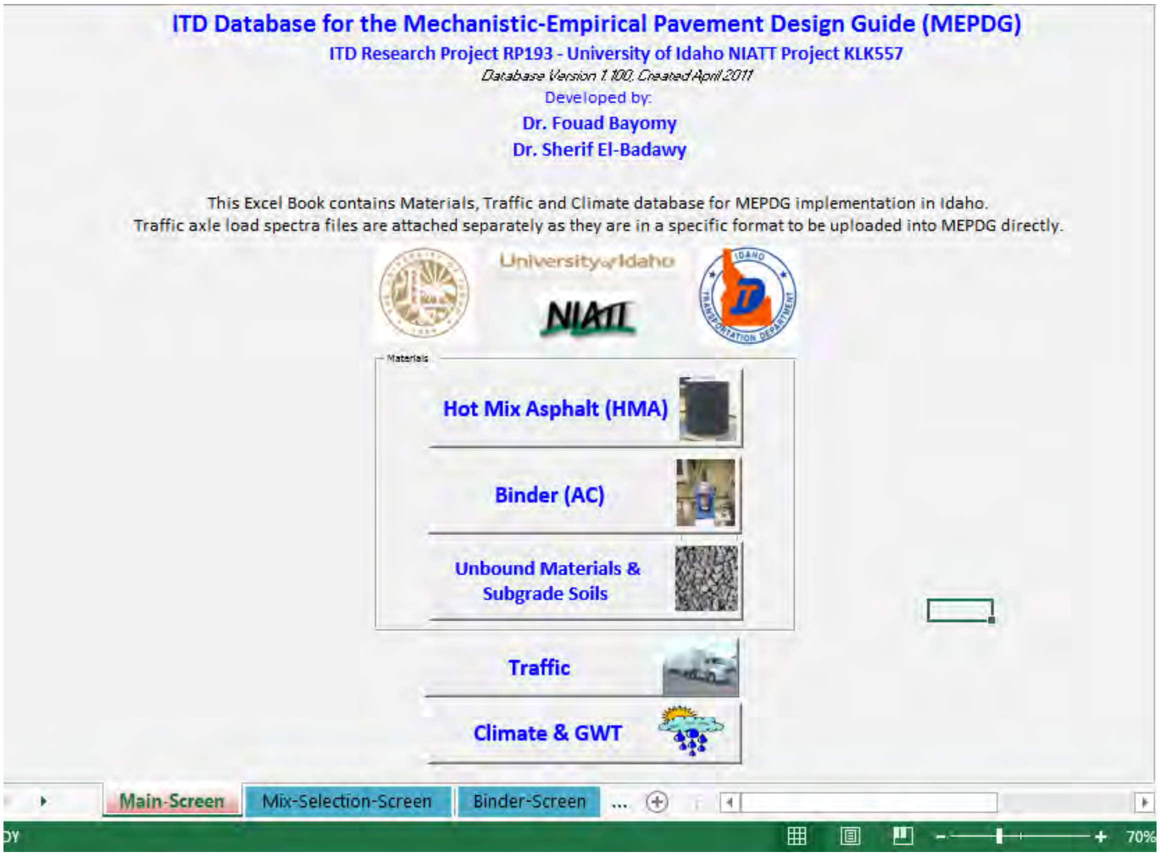


Figure 2. Photo. Screenshot of Idaho TD MEPDG database access.

- **Mississippi DOT**. When possible, the Mississippi DOT characterizes materials using FWD deflection data, backcalculated layer moduli, and in situ moisture content.
- **Montana DOT**. The Montana DOT has performed materials characterization including collecting material samples on previously constructed pavement sections and determining layer thickness, water table or rigid layer depth, and conducting falling weight deflectometer (FWD) testing. On newly constructed sections, material samples were obtained for asphalt binder, plant mix crushed aggregate, plant mix (sampled from the windrow during laydown), base course crushed aggregate, and subgrade soil. In addition, FWD testing was conducted on newly constructed pavement sections. Asphalt mixture material testing included aggregate gradation, asphalt content, maximum theoretical density, bulk density, asphalt binder penetration and viscosity, indirect tensile, and creep compliance. Unbound base, subbase, and subgrade soil testing included resilient modulus, and moisture-density (modified Proctor), whereas elastic modulus and

compressive strength testing was performed for cement-treated bases. Montana DOT is also in the process of developing a GIS map for accessing asphalt mixture properties (e.g., binder type, asphalt content, aggregate size, mix design properties) on all Montana DOT asphalt pavement projects since 2000.

- **New Jersey DOT**. The New Jersey DOT conducted a study to evaluate the precision of the dynamic modulus test, to develop a database of dynamic modulus for asphalt materials, and to compare the dynamic modulus prediction equation to the measured results (Bennert 2009).
- **New York State DOT**. The New York State DOT is developing design tables for both flexible and rigid pavements. The flexible pavement design tables are based on materials testing to characterize asphalt material properties. The test results indicate a very good fit between measured and estimated dynamic modulus using the Witczak model (Romanoschi, Abdullah, and Bendana 2014).
- **Oregon DOT**. Dynamic modulus master curves were generated based on the results of the Asphalt Mixture Performance Tester (AMPT). A research study, conducted by the Oregon State University, generated an initial database that has continued to be populated with additional mixture testing results, including 50 percent recycled asphalt pavement (RAP) blends and polymer modified asphalt mixtures (Lundy et al. 2005). The National Center for Asphalt Technology (NCAT) is currently conducting a data review of Oregon DOT instrumented pavement segments.
- **Pennsylvania DOT**. Through the University of Pittsburgh, the Pennsylvania DOT conducted a study on establishing inputs for rigid pavement design (Nassiri and Vandenbossche 2011). The Pennsylvania DOT materials lab has obtained and is conducting testing using the thermal expansion (CTE) and asphalt mixture performance tester (AMPT) equipment, has plans for evaluating the resilient modulus testing equipment, and will be developing a materials database.
- **South Dakota DOT**. The South Dakota DOT has conducted testing and developed a database for characterizing typical base materials and subgrade soils in South Dakota (Bennett nd). Subgrade soil testing included particle size, hydrometer, Atterberg Limits, moisture and density relationships, California Bearing Ratio (CBR), resilient modulus, and k_1, k_2, and k_3 values. Base material testing included particle size, Atterberg Limits, moisture and density relationships, and resilient modulus. Asphalt mixture testing was also conducted and included dynamic modulus, repeated triaxial load testing, and determination of the master curve for several asphalt mixtures using the AMPT. Asphalt mixture testing was conducted by the South Dakota School of Mines and Technology (SDSMT). It was expected that significant difference would be seen in the master curve results, but little difference was noted. A research project was initiated in 2014 to conduct further evaluation of asphalt mixtures using the Simple Performance Tester (SPT). Coefficient of thermal expansion (CTE) testing has been conducted on typical concrete mixtures.
- **Utah DOT**. Utah DOT conducted a study to measure the resilient modulus of unbound aggregate materials obtained from several sources across the state. Resilient modulus testing indicated a modulus range of 18,000 to 32,000 psi, with an average of 25,000 psi. During the original performance model calibration, Utah DOT used resilient modulus values ranging from 25,000 to 40,000 psi. Due to the impact of base stiffness on asphalt

layer thickness determination, recalibration of the asphalt pavement models is warranted (but has not yet been conducted). Utah DOT also conducted a study to determine the coefficient of thermal expansion (CTE) for nineteen aggregate sources across the state. CTE values ranged from 4.27 (volcanic) to 6.16 (quartzite). They noted that CTE values should be checked during mix design and/or construction.

- **Quebec MOT**. The Quebec MOT has developed extensive databases for complex modulus of asphalt mixes and resilient modulus of granular materials.
- Texas DOT is requiring material source certification to include CTE testing. It was also noted that in certain environments as the k-value increases, the thickness of the concrete slab increases.
- As a rule of thumb, for asphalt pavements designed over weak soils, it was noted that the resilient modulus of the base layer should be no more than two to three times the resilient modulus of the subgrade soil.
- Important tests for quantifying concrete materials include CTE, resilient modulus, and strength.

Thresholds/Reliability/Hierarchical Levels

Tables 1 through 6 provide a summary of agency hierarchical levels, reliability values, and performance criteria limits used by the participating highway agencies. (Note: Tables 4 and 6 represent the SI Unit version of tables 3 and 5).

- The North Carolina DOT suggested that the pavement management system be queried to determine expected (typical) threshold limits. They are currently using the reliability levels and calibration coefficients recommended in the AASHTO *MEPDG Manual of Practice*/AASHTOWare Pavement ME Design™ software. North Carolina DOT's evaluation of performance data indicates that the International Roughness Index (IRI) does not change much from year to year, and that top-down or bottom-up fatigue cracking and cracking due to oxidation are the primary distress types.
- The threshold, reliability, and hierarchical levels are a policy decision for the Kentucky TC. They noted that fatigue cracking is not a typical distress unless there is base failure, which is minimal on the Kentucky highway network.

Table 1. Agency MEPDG input hierarchical levels.

AASHTO Region	Agency	Asphalt	Concrete	Unbound Agg. & Soils	Traffic	Rehab
1	Connecticut DOT	3	3	3	3	3
1	Maryland SHA	[1]	[1]	[1]	[1]	[1]
1	New Hampshire DOT[2]	1	[3]	1	1	[3]
1	New Jersey DOT	1	3	1	2	2
1	New York DOT	3	2	3	2	—
1	Pennsylvania DOT	[3]	[3]	[3]	[3]	[3]
1	South Carolina DOT	[3]	[3]	[3]	[3]	[3]
1	Vermont AOT	2	—	1	2	—
2	Arkansas DOT	3	3	3	1	3
2	Florida DOT	—	2	2	1	1
2	Georgia DOT	2	2	2, 3	2	2, 3
2	Kentucky TC	2, 3	[3]	1-3	1-3	[3]
2	Louisiana DOTD[4]	2, 3	3	2, 3	2, 3	2, 3
2	Mississippi DOT[4]	2	2	2	2	1, 2
2	North Carolina DOT	2	3	3	2	3
2	Tennessee DOT	2	3[5]	1[6]	2[7]	3
2	Virginia DOT	2[8]	2	2	2	3
3	Indiana DOT	1, 2	2	2	2	2, 3
3	Michigan DOT	1, 2	2	2, 3	1-3	[3]
3	Missouri DOT	2, 3	2, 3	2, 3	1 - 3	[3]
3	Ohio DOT	1-3	1-3	1-3	1-3	[3]
4	Alaska DOT&PF	1, 2	—	1, 2	3	[3]
4	Arizona DOT	2	2	2	2	2
4	California DOT	—	2, 3	3	2	2, 3
4	Colorado DOT	1, 2	1, 2	2, 3[9]	2	2
4	Idaho TD	1, 3	3	2, 3	2	2, 3
4	Montana DOT	2, 3	3	3	1	3
4	Nebraska DOR	[3]	[3]	[3]	[3]	[3]
4	Nevada DOT[4]	2	2	1[10], 2[11]	1[12], 2[13]	[3]
4	New Mexico DOT	2	2, 3	2	2	3
4	North Dakota DOT	[3]	[3]	[3]	[3]	[3]
4	Oregon DOT	3	3	3	1	3
4	South Dakota DOT	[3]	[3]	[3]	[3]	[3]
4	Utah DOT	1[14], 2, 3	1[14]	3	1[15]	1[9], 2, 3
4	Washington State DOT	3	3	3	1, 2, 3	3
N/A	Alberta Transportation	1	1, 2	3	1, 2, 3	2, 3
N/A	Manitoba	see Appendix E				
N/A	Ontario MOT	see Appendix E				
N/A	Quebec MOT	see Appendix E				

1 See Chapter 4, Maryland SHA Pavement Design Guide (http://www.marylandroads.com/OMT/MDSHA-Pavement-Design-Guide.pdf).
2 Based on one comparative design conducted in 2007.
3 To be determined.
4 Under review
5 Level 2 for CTE.
6 Laboratory testing to determine k1, k2, and k3 values has been completed.
7 Tennessee DOT developed equation to calculate ESALs from AADT.
8 Asphalt mix properties from statewide average test data are entered as Level 1 inputs.
9 FWD testing and backcalculation of layer moduli.
10 Aggregate base.
11 Subgrade.
12 Interstate and major US highways.
13 All others.
14 Level 1 for major projects or unusual materials; Level 2-3 for all others.
15 Level 2-3 on remote highways.

Table 2. Agency MEPDG reliability criteria.

AASHTO Region	Agency	Interstate	Principal Arterials	Major Collectors	Local
N/A	MEPDG default	95	90/85	80/75	75/70
1	Connecticut DOT	95	90/85	80/75	75/70
1	Maryland SHA	[1]	[1]	[1]	[1]
1	New Hampshire DOT	90	[2]	[2]	[2]
1	New Jersey DOT	95	90/85	80/75	75/70
1	New York DOT	90	85	[2]	[2]
1	Pennsylvania DOT	95	90/85	80/75	75/70
1	South Carolina DOT	[2]	[2]	[2]	[2]
1	Vermont AOT	95	90/85	80/75	75/70
2	Arkansas DOT	95/90	90/85	80/75	80
2	Florida DOT	95	90	90	75
2	Georgia DOT	95	90	90	75[3]
2	Kentucky TC	90	80	70	70
2	Louisiana DOTD[4]	95	90	80	80
2	Mississippi DOT	95	90	90	75
2	North Carolina DOT	90	90	80	80
2	Tennessee DOT	95	90	80/75	75/70
2	Virginia DOT	[4]	[4]	[4]	[4]
3	Indiana DOT	90	85	80	70
3	Michigan DOT	95	95	95	95
3	Missouri DOT	50	50	50	50
3	Ohio DOT	95	90/85	[2]	[2]
4	Alaska DOT&PF	95	90/85	80/75	75/70
4	Arizona DOT	97	[5]	[5]	[5]
4	California DOT	90	90	90	90
4	Colorado DOT	80-95	75-95	75-95	50-80
4	Idaho TD	95	90/85	80/75	75/70
4	Montana DOT	90-95	85	75-95	75-95
4	Nebraska DOR	90	85	80	80
4	Nevada DOT	95	90/85	80/75	75/70
4	New Mexico DOT	90	85	80	75/70
4	North Dakota DOT	[2]	[2]	[2]	[2]
4	Oregon DOT	95	90/85	85/80	75/70
4	South Dakota DOT	95	90	90	90
4	Utah DOT	95	90	90	90
4	Washington State DOT	95	85	75	75
N/A	Alberta Transportation[6]	85-95	50-90	50-85	50-85
N/A	Manitoba	90/90	85/90	80/80	—
N/A	Ontario MOT	95/95	90/85	80/75	75/75
N/A	Quebec MOT[7]	90-95	80-90	70-80	66-70

[1] New pavement (ride only) = 50; new pavement (all other distresses) = 90; and existing pavement = 50.
[2] To be determined.
[3] < 500 trucks/day
[4] Under review.
[5] > 10,000 ADT – 95 percent; 2,001 to 10,000 ADT – 90 percent; 501 to 2,000 ADT – 80 percent; and < 500 ADT – 75 percent.
[6] Based on 20-year design ESALs and type of construction (see Appendix E).
[7] Depends on functional classification and AADT

Table 3. Agency MEPDG performance criteria limits—asphalt pavements (US Customary).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (ft/mi)	Total Rut Depth (in)	Asphalt Rut Depth (in)	Transverse Cracking (ft/mi)	IRI (in/mi)
N/A	MEPDG default	< 10 (I)[1] < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65[2]	Not specified	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Connecticut DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Maryland SHA[3]	Based on RSL	Based on RSL	Based on RSL	Not used for design	Based on RSL	Based on RSL
1	New Hampshire DOT	< 25 (I)	Not used for design	< 0.75 (I)	< 0.40 (I)	< 1,000 (I)	< 200 (I)
1	New Jersey DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 170
1	New York DOT	[4]	[4]	[4]	[4]	[4]	< 225
1	Pennsylvania DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Vermont AOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65	< 0.75	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Virginia DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Arkansas DOT[5]	< 25	Not used for design	< 0.75	< 0.50	Not used for design	< 172
2	Florida DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Georgia DOT	< 10 (I) < 20 (P) < 25[6]	Not used for design	< 0.35 (I) < 0.40 (P) < 0.40[6]	Not used for design	< 1,000 (I) < 1,500 (P) < 1,500[6]	< 175 (I) < 175 (P) < 220[6]
2	Kentucky TC[5]	< 25	< 2,000	< 0.75	< 0.25	< 1,000	< 172
2	Louisiana DOTD[5]	< 15 (I) < 25 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65 (S)	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
2	Mississippi DOT[5]	< 15 (I) < 15 (P) < 25 (S)	Not used for design	Not used for design	< 0.35 (I) < 0.35 (P) < 0.40 (S)	Not used for design	< 175 (I) < 210 (P) < 230 (S)
2	North Carolina DOT	< 10	< 1,000	< 0.75	< 0.50	Not used for design	< 185 (I) < 185 (P) < 200 (S)
2	South Carolina DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Tennessee DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P)	< 0.15 (I)	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)

Table 3. Agency MEPDG performance criteria limits—asphalt pavements (continued).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (ft/mi)	Total Rut Depth (in)	Asphalt Rut Depth (in)	Transverse Cracking (ft/mi)	IRI (in/mi)
3	Indiana DOT	< 10 (I) < 25 (P) < 35 (S)	Not used for design	Not used for design	< 0.40	< 500	< 160 (I) < 190 (P) < 200 (S)
3	Michigan DOT	< 20	Not used for design	< 0.50	Not used for design	< 1,000	< 172
3	Missouri DOT	< 2	Not used for design	Not used for design	< 0.25	Not used for design	Not used for design
3	Ohio DOT	< 10 (I) < 20 (P)	Not used for design	< 0.40 (I) < 0.50 (P)	< 0.40 (I) < 0.50 (P)	< 500 (I) < 700 (P)	< 160 (I) < 200 (P)
4	Alaska DOT&PF	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.50 (I) < 0.75 (P) < 0.75 (S)	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 170 (I) < 220 (P) < 220 (S)
4	Arizona DOT	< 10 (I) < 15 (P) < 20 (S)	Not used for design	< 0.50	Not used for design	< 1,000 (I) < 1,500 (P) < 1,500 (S)	< 150
4	California DOT	MEPDG is not used for asphalt pavement design					
4	Colorado DOT	< 10 (I) < 25 (P) < 35 (S)	< 2,000 (I) < 2,500 (P)	< 0.55 (I) < 0.65 (P) < 0.80 (S)	< 0.40 (I) < 0.50 (P) < 0.65 (S)	< 1,500	< 160 (I) < 200 (P) < 200 (S)
4	Idaho TD[5]	< 10 (I) < 25 (P) < 20 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65 (S)	Not used for design	< 1,000 (I) < 1,500 (P) < 1,500 (S)	< 160 (I) < 175 (P) < 200 (S)
4	Montana DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65	Not used for design	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 200 (P) < 200 (S)
4	Nebraska DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Nevada DOT	[5]	[5]	[5]	[5]	[5]	[5]
4	New Mexico DOT	< 10 (I) < 25 (P) < 35 (S)	Not used for design	< 0.40 (I) < 0.50 (P) < 0.65 (S)	< 0.25 (I) < 0.35 (P) < 0.50 (S)	< 1,500	< 160 (I) < 200 (P) < 200 (S)
4	North Dakota DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Oregon DOT	< 10 (I) < 20 (P) < 35 (S)	< 1,600 (I) < 2,000 (P) < 2,000 (S)	< 0.90 (I) < 1.00 (P) < 1.00 (S)	< 0.40 (I) < 0.50 (P) < 0.65[7]	< 500 (I) < 700 (P) < 700 (S)	< 160 (I) < 180 (P) < 180 (S)
4	South Dakota DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Utah DOT	< 10 (I) < 15 (P) < 25 (S)	Not used for design	< 0.75	< 0.75	< 905 (I) < 1,267 (P) < 1,267 (S)	< 170
4	Washington State DOT	20 - 50[8]	5,280 - 13,200[8]	Not used for design	< 0.5	7,920 - 19,800[8]	< 222

Table 3. Agency MEPDG performance criteria limits—asphalt pavements (continued).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (ft/mi)	Total Rut Depth (in)	Asphalt Rut Depth (in)	Transverse Cracking (ft/mi)	IRI (in/mi)
N/A	Alberta Transportation	< 15 (a)[9] < 15 (b) < 20 (c) < 25 (d) < 30 (e)	< 2,006[10]	< 0.59	< 0.59	< 158 (New) < 1,158 (Rehab)	< 120 (a) < 133 (b) < 146 (c) < 165 (d) < 190 (e)
N/A	Manitoba	< 15 (E)[11] < 20 (P&S) < 25 (C)	Not used for design	< 0.75	< 0.47	< 1,056	< 158 (E) < 158 (P) < 171 (S) < 190 (C)
N/A	Ontario MOT	< 10 (F)[12] < 20 (A) < 35 (C)	< 2,006[10]	< 0.75	< 0.24	< 1,003[10]	< 120 (F) < 146 (A) < 171(C) < 209 (L)
N/A	Quebec MOT	< 10 (H)[13,14] < 15 (N) < 20 (R) < 25 (C) < 30 (O)	< 2,006[10]	< 0.47	< 0.47	< 1,056	< 139 (I) < 158 (N) < 190 (R) < 222 (C) < 285 (O)

1 I – interstate; P – primary; and S – secondary routes.
2 Other roadways (< 45 mph).
3 Individual distress conditions are converted to RSL; the lowest RSL is converted back and applied to all distresses to get targets. RSL varies based on functional class. Essentially, performance criteria limits match the existing condition. With new pavement or major rehabilitation, based on RSL = 20. See chapter 6.01 of the SHA Pavement Design Guide.
4 To be determined.
5 Under review.
6 Two-lane state routes.
7 Fatigue cracking as percent of total area, not just wheel paths. Speed <45 mph; Speed ≥ 45 mph: 0.50 inch.
8 Depends on severity level.
9 a - > 8,000; b - 6,000 - 8,000; c - 1,500 - 6,000; d - 400 - 1,500, e < 400.
10 For information only, not used for acceptance or rejection of design.
11 E - expressway; P - primary arterial; S - secondary arterial; and C - collector.
12 F - freeway; A - arterial; C - collector, and L - local.
13 H - highway; N - national; R - regional, C - collector; and O - other.
14 Needs additional calibration to local conditions.

Table 4. Agency MEPDG performance criteria limits—asphalt pavements (SI).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (m/km)	Total Rut Depth (mm)	Asphalt Rut Depth (mm)	Transverse Cracking (m/km)	IRI (m/km)
N/A	MEPDG default	< 10 (I)[1] < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17[2]	Not specified	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Connecticut DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Maryland SHA[3]	Based on RSL	Based on RSL	Based on RSL	Not used for design	Based on RSL	Based on RSL
1	New Hampshire DOT	< 25 (I)	Not used for design	< 19 (I)	< 10 (I)	< 189 (I)	< 3.2 (I)
1	New Jersey DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.7
1	New York DOT	[4]	[4]	[4]	[4]	[4]	< 3.6
1	Pennsylvania DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Vermont AOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	< 19	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Virginia DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Arkansas DOT[5]	< 25	Not used for design	< 19	< 13	Not used for design	< 2.7
2	Florida DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Georgia DOT	< 10 (I) < 20 (P) < 25[6]	Not used for design	< 9 (I) < 10 (P) < 10[6]	Not used for design	< 189 (I) < 284 (P) < 284[6]	< 2.8 (I) < 2.8 (P) < 3.6[6]
2	Kentucky TC[5]	< 25	< 32	< 19	< 6	< 189	< 2.7
2	Louisiana DOTD[5]	< 15 (I) < 25 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
2	Mississippi DOT[5]	< 15 (I) < 15 (P) < 25 (S)	Not used for design	Not used for design	< 9 (I) < 9 (P) < 10 (S)	Not used for design	< 2.8 (I) < 3.3 (P) < 3.6 (S)
2	North Carolina DOT	< 10	< 16	< 19	< 13	Not used for design	< 2.9 (I) < 2.9 (P) < 3.2 (S)
2	South Carolina DOT	[4]	[4]	[4]	[4]	[4]	[4]
2	Tennessee DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P)	< 4 (I)	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)

Table 4. Agency MEPDG performance criteria limits—asphalt pavements (continued).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (m/km)	Total Rut Depth (mm)	Asphalt Rut Depth (mm)	Transverse Cracking (m/km)	IRI (m/km)
3	Indiana DOT	< 10 (I) < 25 (P) < 35 (S)	Not used for design	Not used for design	< 10	< 95	< 2.5 (I) < 3.0 (P) < 3.2 (S)
3	Michigan DOT	< 20	Not used for design	< 13	Not used for design	< 189	< 2.7
3	Missouri DOT	< 2	Not used for design	Not used for design	< 6	Not used for design	Not used for design
3	Ohio DOT	< 10 (I) < 20 (P)	Not used for design	< 10 (I) < 13 (P)	< 10 (I) < 13 (P)	< 95 (I) < 133 (P)	< 2.5 (I) < 3.2 (P)
4	Alaska DOT&PF	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 13 (I) < 19 (P) < 19 (S)	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.7 (I) < 3.5 (P) < 3.5 (S)
4	Arizona DOT	< 10 (I) < 15 (P) < 20 (S)	Not used for design	< 13	Not used for design	< 189 (I) < 284 (P) < 284 (S)	< 2.4
4	California DOT	MEPDG is not used for asphalt pavement design					
4	Colorado DOT	< 10 (I) < 25 (P) < 35 (S)	< 32 (I) < 39 (P) < 47 (S)	< 14 (I) < 17 (P) < 20 (S)	< 10 (I) < 13 (P) < 17 (S)	< 284	< 2.5 (I) < 3.2 (P) < 3.2 (S)
4	Idaho TD[5]	< 10 (I) < 25 (P) < 20 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 189 (I) < 284 (P) < 284 (S)	< 2.5 (I) < 2.8 (P) < 3.2 (S)
4	Montana DOT	< 10 (I) < 20 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	Not used for design	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
4	Nebraska DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Nevada DOT	[5]	[5]	[5]	[5]	[5]	[5]
4	New Mexico DOT	< 10 (I) < 25 (P) < 35 (S)	Not used for design	< 10 (I) < 13 (P) < 17	< 6 (I) < 9 (P) < 13	< 284	< 2.5 (I) < 3.2 (P) < 3.2 (S)
4	North Dakota DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Oregon DOT	< 10 (I) < 20 (P) < 35 (S)	< 25 (I) < 32 (P) < 32 (S)	< 23 (I) < 25 (P) < 25 (S)	< 0.10 (I) < 0.13 (P) < 0.17[7]	< 95 (I) < 133 (P) < 133 (S)	< 2.5 (I) < 2.8 (P) < 2.8 (S)
4	South Dakota DOT	[4]	[4]	[4]	[4]	[4]	[4]
4	Utah DOT	< 10 (I) < 15 (P) < 25 (S)	Not used for design	< 19	< 19	< 171 (I) < 240 (P) < 240 (S)	< 2.7
4	Washington State DOT	20 - 50[8]	83 - 208[8]	Not used for design	< 13	1,500 - 3,750[8]	< 3.5

Table 4. Agency MEPDG performance criteria limits—asphalt pavements (continued).

AASHTO Region	Agency	Bottom-Up Cracking (percent)	Top-Down Cracking (m/km)	Total Rut Depth (mm)	Asphalt Rut Depth (mm)	Transverse Cracking (m/km)	IRI (m/km)
N/A	Alberta Transportation	< 15 (a)[9] < 15 (b) < 20 (c) < 25 (d) < 30 (e)	< 380[10]	< 15	< 15	< 30 (New) < 225 (Rehab)	< 1.9 (a) < 2.1 (b) < 2.3(c) < 2.6 (d) < 3.0 (e)
N/A	Manitoba	< 15 (E)[11] < 20 (P&S) < 25 (C)	Not used for design	< 19	< 12	< 200	< 2.5 (E) < 2.5 (P) < 2.7 (S) < 3.0 (C)
N/A	Ontario MOT	< 10 (F)[12] < 20 (A) < 35 (C)	< 380[10]	< 19	< 6	< 190[10]	< 1.9 (F) < 2.3 (A) < 2.7(C) < 3.3 (L)
N/A	Quebec MOT	< 10 (H)[13,14] < 15 (N) < 20 (R) < 25 (C) < 30 (O)	< 380[10]	< 12	< 12	< 200	< 2.2 (I) < 2.5 (N) < 3.0 (R) < 3.5 (C) < 3.5 (O)

1 I – interstate; P – primary; and S – secondary routes.
2 Other roadways (< 72 kph).
3 Individual distress conditions are converted to RSL; the lowest RSL is converted back and applied to all distresses to get targets. RSL varies based on functional class. Essentially, performance criteria limits match the existing condition. With new pavement or major rehabilitation, based on RSL = 20. See chapter 6.01 of the SHA Pavement Design Guide.
4 To be determined.
5 Under review.
6 Two-lane state routes.
7 Fatigue cracking as percent of total area, not just wheel paths. Speed <45 mph; Speed ≥ 45 mph: 0.50 inch.
8 Depends on severity level.
9 a - > 8,000; b - 6,000 - 8,000; c - 1,500 - 6,000; d - 400 - 1,500, e < 400.
10 For information only, not used for acceptance or rejection of design.
11 E - expressway; P - primary arterial; S - secondary arterial; and C - collector.
12 F - freeway; A - arterial; C - collector, and L - local.
13 H - highway; N - national; R - regional, C - collector; and O - other.
14 Needs additional calibration to local conditions.

Table 5. Agency MEPDG performance criteria limits—JPCP (US Customary).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
N/A	MEPDG default	< 0.15 (I)[1] < 0.20 (P) < 0.25 (S)[2]	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Connecticut DOT	Do not construct concrete pavements		
1	Maryland SHA[3]	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	Based on RSL
1	New Hampshire DOT	Do not construct concrete pavements		
1	New Jersey DOT	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)
1	New York DOT	< 0.15 (I)	< 10 (I)	< 200 (I)
1	Pennsylvania DOT	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)
1	Vermont AOT	Do not construct concrete pavements		
1	Virginia DOT	[4]	[4]	[4]
2	Arkansas DOT	[4]	[4]	[4]
2	Florida DOT	< 0.12	< 10	< 180
2	Georgia DOT	< 0.125 (I) < 0.20 (P) < 0.20[5]	< 10	< 175 (I) < 175 (P) < 220[5]
2	Kentucky TC	[4]	[4]	[4]
2	Louisiana DOTD	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)
2	Mississippi DOT[6]	< 0.19 (I) < 0.19 (P) < 0.25 (S)	< 4 (I) < 4 (P) < 4 (S)	< 250 (I) < 270 (P) < 300 (S)
2	North Carolina DOT	< 0.15	< 10	< 185 (I) < 200 (P) < 200 (S)
2	South Carolina DOT	[4]	[4]	[4]
2	Tennessee DOT	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)

Table 5. Agency MEPDG performance criteria limits—JPCP (continued).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
3	Indiana DOT	< 0.15 (I) < 0.22 (P) < 0.25 (S)	< 10	< 160 (I) < 190 (P) < 200 (S)
3	Michigan DOT	< 0.125	< 15	< 172
3	Missouri DOT	< 0.15	< 1.5	Not used for design
3	Ohio DOT	< 0.15 (I) < 0.20 (P)	< 10 (I) < 15 (P)	< 160 (I) < 200 (P)
4	Alaska DOT&PF	Do not construct concrete pavements		
4	Arizona DOT	< 0.12	< 10 (I) < 15 (P) < 25 (S)	< 150
4	California DOT	< 0.10	< 10	< 160
4	Colorado DOT	< 0.12 (I) < 0.14 (P) < 0.20 (S)	< 7[6]	< 160 (I) < 200 (P) < 200 (S)
4	Idaho TD[6]	< 0.12 (I) < 0.15 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 175 (P) < 200 (S)
4	Montana DOT	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 200 (P) < 200 (S)
4	Nebraska DOT	[4]	[4]	[4]
4	Nevada DOT	[6]	[6]	[6]
4	New Mexico DOT	< 0.12 (I) < 0.14 (P) < 0.20 (S)	< 7	< 160 (I) < 200 (P) < 200 (S)
4	North Dakota DOT	[4]	[4]	[4]
4	Oregon DOT	< 0.15 (I) < 0.20 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 160 (I) < 180 (P) < 180 (S)
4	South Dakota DOT	[4]	[4]	[4]
4	Utah DOT	< 0.15 (I) < 0.25 (P) < 0.25 (S)	< 10 (I) < 15 (P) < 20 (S)	< 170
4	Washington State DOT	< 0.236	< 15 (multi-cracked slabs)	< 222

Table 5. Agency MEPDG performance criteria limits—JPCP (continued).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
N/A	Alberta Transportation	[4]	[4]	[4]
N/A	Manitoba[7]	< 0.12	< 10 (E) < 15 (P)	< 158 (E) < 158 (P) < 171 (S)
N/A	Ontario MOT[8]	< 0.12	< 10 (F) < 15 (A) < 20 (C)	< 152 (F) < 171 (A) < 171(C)
N/A	Quebec MOT[9]	< 0.12	< 8	< 139 (I) < 158 (N) < 190 (R) < 222 (C) < 285 (O)

1 I – interstate; P – primary; and S – secondary routes.
2 Other roadways (< 45 mph)
3 Individual distress conditions are converted to RSL; the lowest RSL is converted back and applied to all distresses to get targets. RSL varies based on functional class. Essentially, performance criteria limits match the existing condition. With new pavement or major rehabilitation, based on RSL = 20. See chapter 6.01 of the SHA Pavement Design Guide.
4 To be determined.
5 Two-lane state routes.
6 Under review.
7 E - expressway; P - primary arterial; S - secondary arterial; and C - collector.
8 F - freeway; A - arterial; C - collector, and L - local.
9 H - highway; N - national; R - regional, C - collector; and O - other.

Table 6. Agency MEPDG performance criteria limits—JPCP (SI).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
N/A	MEPDG default	< 4 (I)[1] < 5 (P) < 6 (S)[2]	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Connecticut DOT	Do not construct concrete pavements		
1	Maryland SHA[3]	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	Based on RSL
1	New Hampshire DOT	Do not construct concrete pavements		
1	New Jersey DOT	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	New York DOT	< 4 (I)	< 10 (I)	< 3.2 (I)
1	Pennsylvania DOT	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
1	Vermont AOT	Do not construct concrete pavements		
1	Virginia DOT	[4]	[4]	[4]
2	Arkansas DOT	[4]	[4]	[4]
2	Florida DOT	< 3	< 10	< 2.8
2	Georgia DOT	< 3 (I) < 5 (P) < 5[5]	< 10	< 2.8 (I) < 2.8 (P) < 3.6[5]
2	Kentucky TC	[4]	[4]	[4]
2	Louisiana DOTD	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)
2	Mississippi DOT[6]	< 5 (I) < 5 (P) < 6 (S)	< 4 (I) < 4 (P) < 4 (S)	< 3.9 (I) < 4.3 (P) < 4.3 (S)
2	North Carolina DOT	< 4	< 10	< 2.9 (I) < 3.2 (P) < 3.2 (S)
2	South Carolina DOT	[4]	[4]	[4]
2	Tennessee DOT	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.2 (P) < 3.2 (S)

Table 6. Agency MEPDG performance criteria limits—JPCP (continued).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
3	Indiana DOT	< 4 (I) < 6 (P) < 6 (S)	< 10	< 2.5 (I) < 3.0 (P) < 3.2 (S)
3	Michigan DOT	< 3	< 15	< 2.7
3	Missouri DOT	< 4	< 1.5	Not used for design
3	Ohio DOT	< 4 (I) < 5 (P)	< 10 (I) < 15 (P)	< 2.5 (I) < 3.2 (P)
4	Alaska DOT&PF	Do not construct concrete pavements		
4	Arizona DOT	< 3	< 10 (I) < 15 (P) < 25 (S)	< 2.4
4	California DOT	< 3	< 10	< 2.5
4	Colorado DOT	< 3 (I) < 4 (P) < 5 (S)	< 7[6]	< 2.5 (I) < 3.2 (P) < 3.2 (S)
4	Idaho TD[6]	< 3 (I) < 4 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 2.8 (P) < 3.2 (S)
4	Montana DOT	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 3.21 (P) < 3.2 (S)
4	Nebraska DOT	[4]	[4]	[4]
4	Nevada DOT	[6]	[6]	[6]
4	New Mexico DOT	< 3 (I) < 4 (P) < 5 (S)	< 7	< 2.5 (I) < 3.2 (P) < 3.2 (S)
4	North Dakota DOT	[4]	[4]	[4]
4	Oregon DOT	< 4 (I) < 5 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.5 (I) < 2.8 (P) < 2.8 (S)
4	South Dakota DOT	[4]	[4]	[4]
4	Utah DOT	< 4 (I) < 6 (P) < 6 (S)	< 10 (I) < 15 (P) < 20 (S)	< 2.7
4	Washington State DOT	< 6	< 15 (multi-cracked slabs)	< 3.5

Table 6. Agency MEPDG performance criteria limits—JPCP (continued).

AASHTO Region	Agency	Mean Joint Faulting (in)	Transverse Cracking (percent)	IRI (in/mi)
N/A	Alberta Transportation	[4]	[4]	[4]
N/A	Manitoba[7]	< 3	< 10 (E) < 15 (P)	< 2.5 (E) < 2.5 (P) < 2.7 (S)
N/A	Ontario Ministry of Transportation[8]	< 3	< 10 (F) < 15 (A) < 20 (C)	< 2.4 (F) < 2.7 (A) < 2.7(C)
N/A	Quebec Ministry of Transport[9]	< 3	< 8	< 2.2 (I) < 2.5 (N) < 3.0 (R) < 3.5 (C) < 4.5 (O)

1 I – interstate; P – primary; and S – secondary routes.
2 Other roadways (< 72 kph)
3 Individual distress conditions are converted to RSL; the lowest RSL is converted back and applied to all distresses to get targets. RSL varies based on functional class. Essentially, performance criteria limits match the existing condition. With new pavement or major rehabilitation, based on RSL = 20. See chapter 6.01 of the SHA Pavement Design Guide.
4 To be determined.
5 Two-lane state routes.
6 Under review.
7 E - expressway; P - primary arterial; S - secondary arterial; and C - collector.
8 F - freeway; A - arterial; C - collector, and L - local.
9 H - highway; N - national; R - regional, C - collector; and O - other.

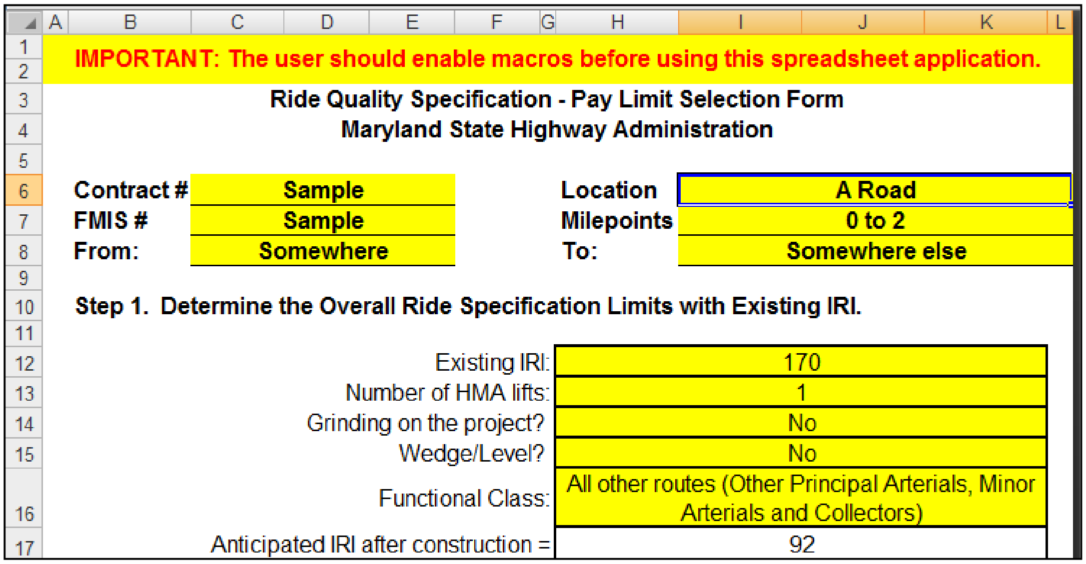
Maryland SHA conducts a project-by-project analysis to determine performance criteria limits based on current pavement condition. This process includes the following steps:

1. Row 9: The engineer fills in the existing condition data (see figure 3).
2. Row 10: Each piece of existing condition data is converted to remaining service life (RSL). The overall RSL is the lowest of the five individual RSLs. The example shown in figure 3 indicates that International Roughness Index (IRI) has the lowest RSL value.
3. Row 12: The engineer fills in the post-treatment predicted condition.
4. Row 14: The terminal performance targets are generated based on the existing overall RSL. For the example shown in figure 3, the existing overall RSL = 16, then the terminal targets for IRI, structural cracking, functional cracking, rutting, and friction are all converted from RSL = 16. The RSL conversion varies depending on roadway functional class.
5. Row 15: The crack indices are converted to density of cracking (MEPDG requirement).
6. Row 16: The engineer inputs the percentage of the structural cracking index that will result from bottom-up cracking and from top-down cracking. Adjusting this input value alters the allowable amount of bottom-up and top-down cracking for MEPDG targets.

   Cells C14, D17, D18, E17, and F14 are now all of the MEPDG targets.

7. Row 19: The engineer determines how many years until the performance targets are reached, using, among other tools, the MEPDG. The overall life extension is the shortest among the individual life extensions. In the figure 3 example, it is functional cracking, even though ride quality was initially the worst.

   The engineer fills in lane-miles and cost to determine lane-mile-year (LMY) benefit and cost/LMY, with the goal of finding the treatment that minimizes the $/LMY.


IMPORTANT: The user should enable macros before using this spreadsheet application.

Ride Quality Specification - Pay Limit Selection Form
Maryland State Highway Administration

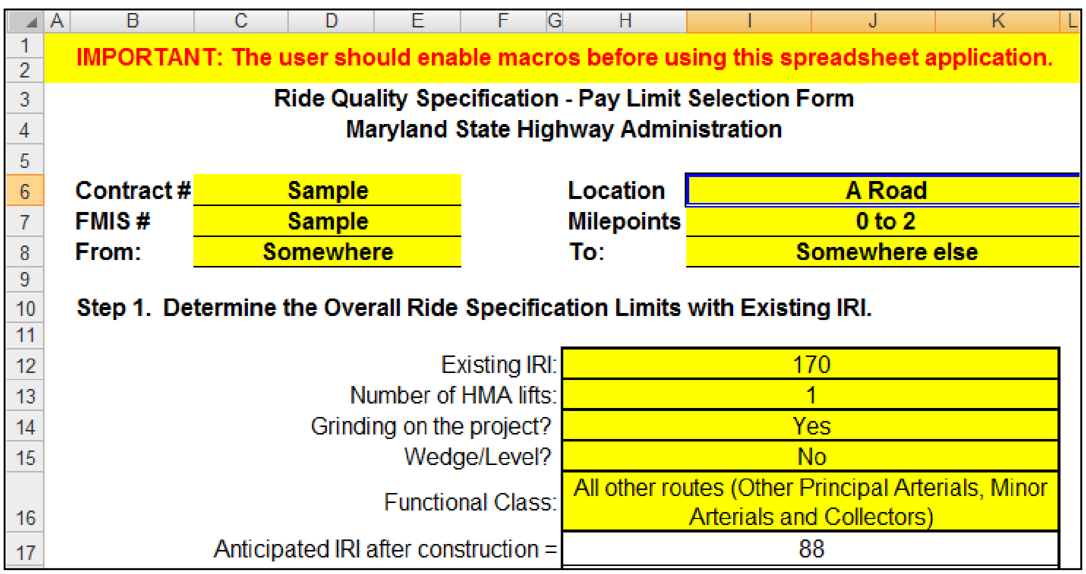
Contract #	Sample	Location	A Road
FMIS #	Sample	Milepoints	0 to 2
From:	Somewhere	To:	Somewhere else

Step 1. Determine the Overall Ride Specification Limits with Existing IRI.

Existing IRI:	170
Number of HMA lifts:	1
Grinding on the project?	No
Wedge/Level?	No
Functional Class:	All other routes (Other Principal Arterials, Minor Arterials and Collectors)
Anticipated IRI after construction =	92

Figure 3. Photo. IRI prediction – single asphalt lift, no milling/grinding, no wedge/level.

Figures 4 and 5 illustrate the spreadsheet application for additional treatment options, while figure 6 illustrates the spreadsheet application for determining RSL based on pavement condition.


IMPORTANT: The user should enable macros before using this spreadsheet application.

Ride Quality Specification - Pay Limit Selection Form
Maryland State Highway Administration

Contract #	Sample	Location	A Road
FMIS #	Sample	Milepoints	0 to 2
From:	Somewhere	To:	Somewhere else

Step 1. Determine the Overall Ride Specification Limits with Existing IRI.

Existing IRI:	170
Number of HMA lifts:	1
Grinding on the project?	Yes
Wedge/Level?	No
Functional Class:	All other routes (Other Principal Arterials, Minor Arterials and Collectors)
Anticipated IRI after construction =	88

Figure 4. Photo. IRI prediction – single asphalt lift, milling/grinding, no wedge/level.

IMPORTANT: The user should enable macros before using this spreadsheet application.

Ride Quality Specification - Pay Limit Selection Form
Maryland State Highway Administration

Contract #	**Sample**	**Location**	**A Road**
FMIS #	**Sample**	**Milepoints**	**0 to 2**
From:	**Somewhere**	**To:**	**Somewhere else**

Step 1. Determine the Overall Ride Specification Limits with Existing IRI.

Existing IRI:	170
Number of HMA lifts:	1
Grinding on the project?	Yes
Wedge/Level?	Yes
Functional Class:	All other routes (Other Principal Arterials, Minor Arterials and Collectors)
Anticipated IRI after construction =	75

Figure 5. Photo. IRI prediction – single asphalt lift, milling/grinding, wedge/level.

Project Description

Somewhere that probably has a lot of traffic

Treatment Option	**Functional Class**
Something that will probably look better	14

		IRI	CI_{struc}	CI_{func}	Rut	FN	Overall
RSL is in years							
LM = Lane-Miles	Existing Performance Value	170	80	70	0.2	40	
LMY = Lane-Mile-Years	Existing RSL Condition	16	20	18	26	20	16
	Predicted Performance Value - Post Fix	92	100	100	0.1	45	
	Design Terminal Performance Target	172	73	66	0.27	39	
	Density		7	15			
	% of structural distress that is bottom-up		90%				
	Cracking Quantity converted from CI - feet per mile		442	9235			
	Bottom-up fatigue cracking (%)		6%				
	Years until Design Terminal Performance Target is reached	30	21	15	30	17	15
	Life Extension (Chosen)						15
	Lane-Miles						9
	LMY Benefit						135
	Project Cost						$ 1,300,000
	$/LMY						$ 9,600

Directions: Fill in the Blue boxes only, and only where appropriate!!

Figure 6. Photo. RSL condition/target spreadsheet.

A synthesis of local calibration activities being undertaken by various highway agencies was conducted in 2013 and is provided in Appendix H. Tables 7 and 8 provide a summary of agency calibration coefficients for asphalt and concrete (JPC) pavements, respectively.

Table 7. Agency asphalt pavement calibration coefficients (adapted from Von Quintus et al. 2013, Pierce and McGovern 2014).

Distress	Factor	National	AZ	CO	MO	NC	NY	OH	OR	UT	WA	WI	WY
Fatigue	Bf1	1	249.0087	130.3674	1	—	—	1	1	1	-3.3	1	1
	Bf2	1	1	1	1	—	—	1	1	1	-40	1	1
	Bf3	1	1.2334	1.2178	1	—	—	1	1	1	20	1	1
Bottom-up cracking	c1	1	1	0.07	1	0.4372	0.50171	1	0.56	1	1	1	0.4951
	c2	1	1	2.35	1	0.1505	0.22719	1	0.225	1	0	1	1.469
	c3	6000	6000	6000	6000	—	—	6000	6000	6000	0	6000	6000
AC rutting	Br1	1	0.69	1.34	1.07	1.0175	0.59	0.51	1.48	0.56	0.6	1.0157	1.0896
	Br2	1	1	1	1	1	1	1	1	1	20.6	1	1
	Br3	1	1	1	1	1	1	1	0.9	1	8.9	1	1
Unbound base rutting	Bs1	1	0.14	0.4	0.01	0.7785	0.82	0.32	1	0.604	—	0.01	0.9475
Subgrade rutting	Bs1	1	0.37	0.84	0.4375	0.6616	0.74	0.33	1	0.4	—	0.5731	0.6897
Thermal cracking	Level 1 K	1.5	1.5	7.5	0.625	—	—	1.5	1.5	1.5	—	0.625	7.5
	Level 2 K	0.5	0.5	0.5	0.5	—	—	0.5	0.5	0.5	—	0.5	0.5
	Level 3 K	1.5	1.5	1.5	1.5	—	—	1.5	1.5	1.5	—	0.3	7.5
IRI	C1	40	1.2281	35	17.7	—	168.709	17.6	40	40	—	18.71	20.53
	C2	0.4	0.1175	0.3	0.975	—	-0.0238	1.37	0.4	0.4	—	0.04	0.4094
	C3	0.008	0.008	0.02	0.008	—	0.00017	0.01	0.008	0.008	—	0.085	0.00179
	C4	0.015	0.028	0.019	0.01	—	0.015	0.066	0.15	0.015	—	0.0197	0.015
Reflection cracking	C	1	2.55	2.5489	1	—	—	1	—	1	—	1	0.75
	D	1	1.25	1.2341	1	—	—	1	—	1	—	1	2.2

Table 8. Agency concrete (JPC) pavement calibration coefficients (adapted from Von Quintus et al. 2013, Pierce and McGovern 2014).

Distress	Factor	National	AZ	CO	FL	MO	NY	OH	UT	WI	WY
Transverse cracking	C1	2	2	2	2.8389	2	2	2	2	2	2
	C2	1.22	1.22	1.22	0.9647	1.22	1.22	1.22	1.22	1.22	1.22
	C4	1	0.19	0.6	0.564	1	0.2	1	0.6	1	0.6
	C5	-1.98	-2.067	-2.05	-0.5946	-1.98	-1.63	-1.98	-2.05	-1.98	-2.05
Joint faulting	C1	1.0184	0.0355	0.5104	4.0472	1.0184	—	1.0184	1.0184	1.15	0.5104
	C2	0.91656	0.1147	0.00838	0.91656	0.91656	—	0.91656	0.91656	0.91656	0.00838
	C3	0.0021848	0.00436	0.00147	0.002848	0.002185	—	0.002185	0.002185	0.004	0.00147
	C4	0.000883739	1.10E-07	0.008345	0.000883739	0.000884	—	0.000884	0.000884	0.000884	0.08345
	C5	250	20000	5999	250	250	—	250	250	250	5999
	C6	0.4	2.0389	0.8404	0.079	0.4	—	0.4	0.4	0.4	0.504
	C7	1.83312	0.189	5.9293	1.8331	1.83312	—	1.83312	1.83312	1.83312	5.9293
	C8	400	400	400	400	400	—	400	400	400	400
IRI	C1	0.8203	0.6	0.8203	0.8203	0.82	—	0.82	0.8203	4.0567	1.7
	C2	0.4417	3.48	0.4417	0.4417	1.17	—	3.7	0.4417	1.6275	1.32
	C3	1.4929	1.22	1.4929	2.2555	1.43	—	1.711	1.4929	0.7236	1.8
	C4	25.24	45.2	25.24	25.24	66.8	—	5.703	25.24	45.2388	35

Pavement Condition Survey Method

Figure 7 provides a summary of agency practices for conducting surface cracking surveys. Responses are summarized according to automated (includes semi- and fully-automated) surveys, manual (or windshield) surveys, moving toward or evaluating fully-automated surveys, a combination of manual and automated surveys, and unknown. The majority, if not all agencies conduct rut depth, faulting, and IRI measurements using automated equipment.

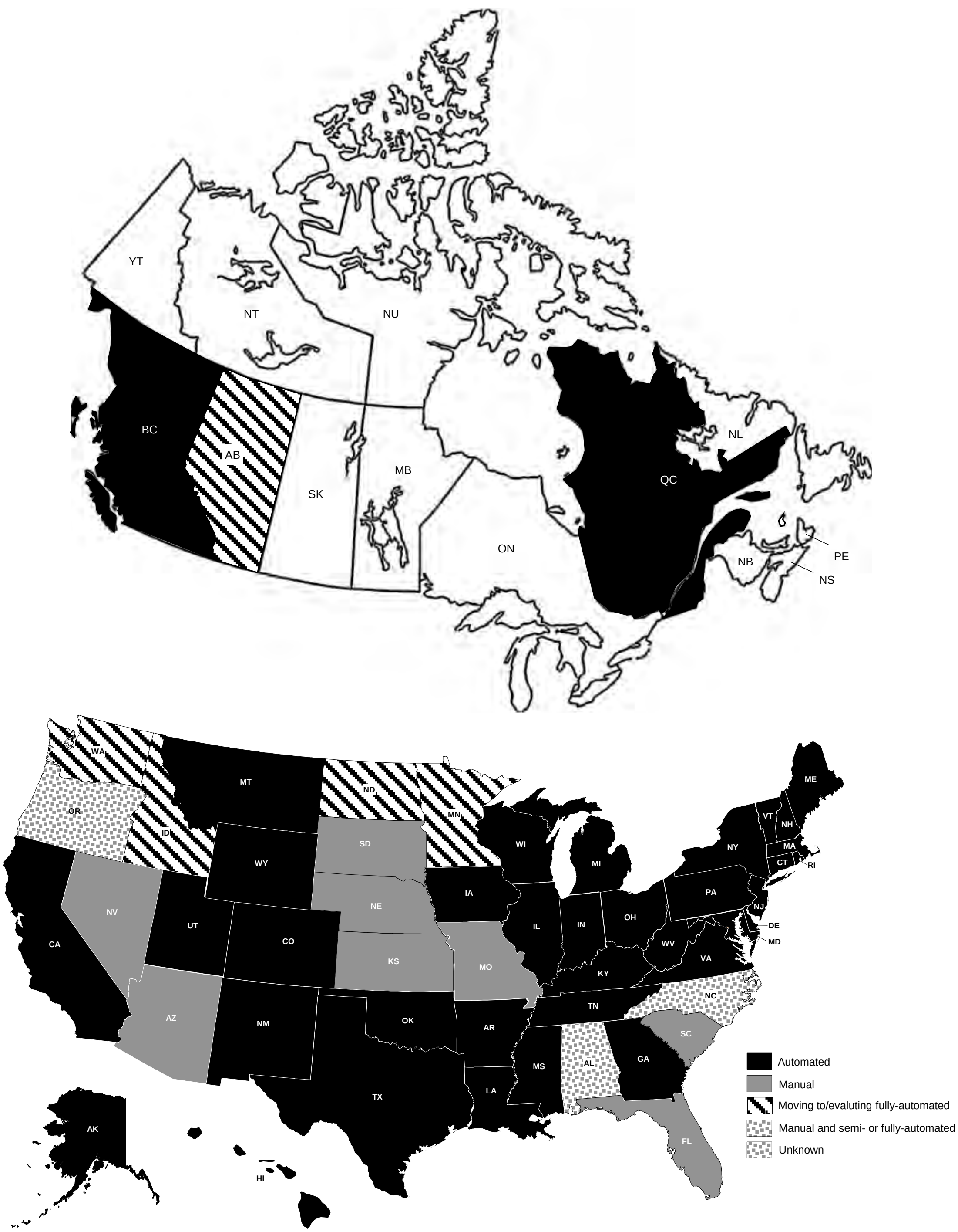


Figure 7. Photo. Summary of agency survey practices.

Local Calibration and Validation

The following provides a brief summary of agency efforts for evaluating and locally calibrating/validating the MEPDG performance prediction models.

- **Arizona DOT**. Arizona DOT, under project SPR-606, evaluated the MEPDG global models and conducted local calibration to Arizona conditions. Local calibration was based on 180 pavement sites with up to 20 years of pavement condition data. The calibration sites included 120 Arizona DOT Long-Term Pavement Performance (LTPP) sites, 36 Arizona DOT pavement management sections, 20 concrete pavement sites (Zaniewski 1986), and 4 sites from the Western Research Institute (WRI) sections (Farrar et al. 2006). All pavement sites used in the calibration process had detailed design, construction materials testing, and distress survey data. Dr. Darter noted that the pavement sites used in the calibration process should have at least 5 years of high quality pavement condition data (AASHTO *Calibration Guide* currently recommends 3 years of condition surveys spanning a 10-year period). The Arizona DOT has used the AASHTOWare Pavement ME Design™ software for pavement design evaluation since 2012, with the first pavement design conducted in 2013. The *Arizona DOT User Guide for AASHTO DARWin-ME* was developed as part of the SPR-606 project and is currently under review by the Arizona DOT. In addition, using AASHTOWare Service Units, basic training on software use has been provided to Arizona DOT staff.
- **Caltrans**. The MEPDG was adopted by Caltrans for rigid pavement design in 2005. The concrete pavement performance prediction models were calibrated based on in-service pavements (53 JPCP sections and 44 asphalt overlaid concrete sections). Concrete pavement design catalogs were developed using the NCHRP 1-37A MEPDG software based on California conditions including traffic, subgrade type, base type, shoulder type, and climate. The design catalog also includes load transfer, shoulder type, and granular base recommendations. The criteria used to develop the design catalog includes:
  - Failure criteria:
    - Transverse cracking – 10 percent cracked slabs.
    - Faulting – 0.10 inch.
    - IRI – 160 inches/mile (initial IRI – 63 inches/mile).
  - Materials:
    - CTE – 6 x 10^{-6} in/in/°F.
    - Surface absorptivity – 0.85 (default value).
    - Bond – no bonding between base and surface layer.
    - Joint spacing – 13.5 feet.
    - Unbound layer – default values.
    - Erodibility index for base layer – granular base = 3; asphalt concrete base (ACB) = 2; cement-treated base (CTB) = 1.
    - Dowel bar diameter – 1.5 inch (1.25 inch for slab thickness < 8.4 inches).
  - Reliability: 90 percent.
  - Design life: 40 years (assumes 2 percent slab replacement and/or diamond grinding).
  - Climate regions: coastal, desert, and low mountain.

  An example of the Caltrans rigid pavement design catalog is shown in figure 8 (note: thickness values shown in figure 8 are in feet).

Table 623.1B
Rigid Pavement Catalog (North Coast, Type I Subgrade Soil)(1), (2), (3), (4),(5)

TI	Rigid Pavement Structural Depth							
	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.70 AB		0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 JPCP 0.70 AB	
11.5 to 12	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 CRCP 0.35 HMA-A		0.80 JPCP 0.35 LCB	0.80 JPCP 0.25HMA-A	0.80 CRCP 0.40 HMA-A	
12.5 to 13	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.85 JPCP 0.50 LCB	0.85 JPCP 0.50 HMA-A	0.80 CRCP 0.50 HMA-A	
13.5 to 14	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	
14.5 to 15	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
15.5 to 16	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
16.5 to 17	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
> 17 < 9	1.00 JPCP 0.35 LCB 0.70 JPCP	1.00 JPCP 0.25 HMA-A 0.70 JPCP	0.90 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP	1.10 JPCP 0.35LCB 0.70 JPCP	1.10 JPCP 0.25 HMA-A 0.70 JPCP	1.00 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP

NOTES:
(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.
(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.
(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.
(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.
(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:
JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Figure 8. Table. Example of Caltrans rigid pavement design catalog (Caltrans 2012).

- **Connecticut DOT.** The Connecticut DOT is in the early stages of MEPDG evaluation and implementation. A sensitivity analysis of the MEPDG inputs and recommended hierarchical input levels has been conducted (Yut, Mahoney, and Zinke 2014). Future efforts include:
  - Assemble a MEPDG Implementation Team, develop a communication plan, conduct staff training, and define long-term plan for adopting MEPDG as Connecticut DOT design method (potentially within 12 months).

- Align Connecticut DOT distress data collection efforts with the MEPDG distress definitions (potentially within 24 months).
- Develop Connecticut DOT-specific MEPDG user guide, develop a central database(s) with required MEPDG input values, and calibrate and validate MEPDG performance prediction models to local conditions (potentially within 36 months).
- Develop design tables (future activity).

- **Georgia DOT**. The Georgia DOT developed a synthesis of thirteen agency calibration procedures. Georgia DOT is also in the process of developing a local calibration database.

- **Kentucky TC**. The Kentucky TC will be using the pavement segments that were designed using the AASHTOWare Pavement ME Design™ software as future calibration sites. The Cabinet is currently using the MEPDG default axle load spectra, Level 2/3 design inputs, and will be collecting additional input information (e.g., materials characterization) during pavement construction.

- **Mississippi DOT**. The Mississippi DOT is conducting a field study to collect deflection data using the FWD to backcalculate unbound layer moduli. The backcalculated layer moduli will be used in the local calibration process rather than the resilient modulus values derived from laboratory testing.

- **Montana DOT**. The Montana DOT initiated a research study to locally calibrate the flexible pavement performance prediction models to Montana conditions (Von Quintus and Moulthrop 2007). Local calibration consisted of the evaluation of fifty-five LTPP sites in surrounding states and Canada, thirty-four LTPP sites in Montana, and thirteen Montana DOT sites (include Superpave mixtures, pulverized base layers, and cement-treated base). The results of the analysis indicated:

  - The IRI, bottom-up fatigue cracking, and plant mix rutting prediction models closely reflect Montana conditions.
  - The top-down cracking model is unreliable and should be re-evaluated with the inclusion of the NCHRP 1-42A results.
  - The thermal cracking and unbound materials rutting performance prediction models are unreliable.
  - Cement-treated base coefficients should be used with caution due to the limited amount of fatigue cracking in the Montana pavement sections.

  Annual pavement condition distress surveys have been conducted on thirteen of the non-LTPP pavement segments. Periodic FWD testing is also being conducted on these same thirteen sites.

- **Nebraska DOR**. The Nebraska DOR has conducted a preliminary analysis of the AASHTOWare Pavement ME Design™ software for concrete pavements. In total, six concrete pavement projects were evaluated and the existing pavement condition compared to the predicted results (see table 9). The following provides a summary of findings:

  - Local calibration is needed prior to implementation. Specifically, the AASHTOWare Pavement ME Design™ software underpredicts the percent of cracked panels.

- Weather station selection impacts the predicted IRI values.
- Without including dowel bars at all transverse joints, performance criteria can't be attained by changing pavement thickness alone.
- Subgrade stabilization had little effect on the predicted pavement performance.

Table 9. Summary of Nebraska DOR comparison of actual to predicted distress.

Highway No.	JPCP Thickness	Aggregate Base Thickness	Subgrade Treatment	AADT	IRI[1] (in/mi)	Cracking[1] (%)	Faulting[1] (in)
275	10 inch (doweled)	4 inch	—	835	71 (87)	1 (0)	0.10 (0)
30	10 inch (doweled)	4 inch	Fly Ash	900	80 (96)	5 (0)	0.70 (0)
75	10 inch (doweled)	4 inch	Lime	765	123 (93)	8 (0)	0.04 (0)
81	10 inch (doweled)	4 inch	Lime	1250	94 (92)	10 (0)	0.04 (0)
I-80	14 inch (doweled)	4 inch	—	9000	165 (198)	6.5 (0)	0.13 (0.14)
2	10 inch (undoweled)	—	Prep only	355	137 (189)	7 (4.2)	0.07 (0.12)

[1] Actual distress (predicted distress).

- **Nevada DOT**. Nevada DOT is conducting two separate studies for local calibration; the University of Nevada-Reno is conducting the asphalt pavement models calibration and the University of Nevada-Las Vegas is conducting the calibration of the concrete pavement models. The asphalt pavement model calibration effort includes the calibration of polymer modified asphalt binder (SBS polymer and asphalt rubber) and validation using available distress and ride data. To date, calibration of the rutting models for the asphalt layer has been completed and local calibration of the fatigue and cracking models is underway. Local calibration of the concrete performance models was based on two projects located in Southern Nevada (I-15 and I-515). Additional efforts will be needed to finalize calibration of the IRI, cracking, and faulting models.
- **New Jersey DOT**. The New Jersey DOT conducted a research study to verify the asphalt concrete performance prediction models using level 2 and level 3 inputs (Siraj 2008). The research effort included data collection, evaluation of the accuracy of the input data, performance prediction, and comparison of predicted performance to field measured results. Pavement data (layer type, thickness, and materials) was obtained from the long-term pavement performance (LTPP) database and New Jersey DOT documents (e.g., as-built plans, quality control data, FWD data). Summary of findings include:
    - The MEPDG predicted rut depth, top down cracking, thermal cracking, and IRI, using level 2 and 3 inputs, was verified for New Jersey conditions.
    - The MEPDG predicted alligator cracking could not be statistically verified using level 2 traffic data and level 3 material inputs.

  A pavement design database was developed and screen shots of the program are shown in figure 9.

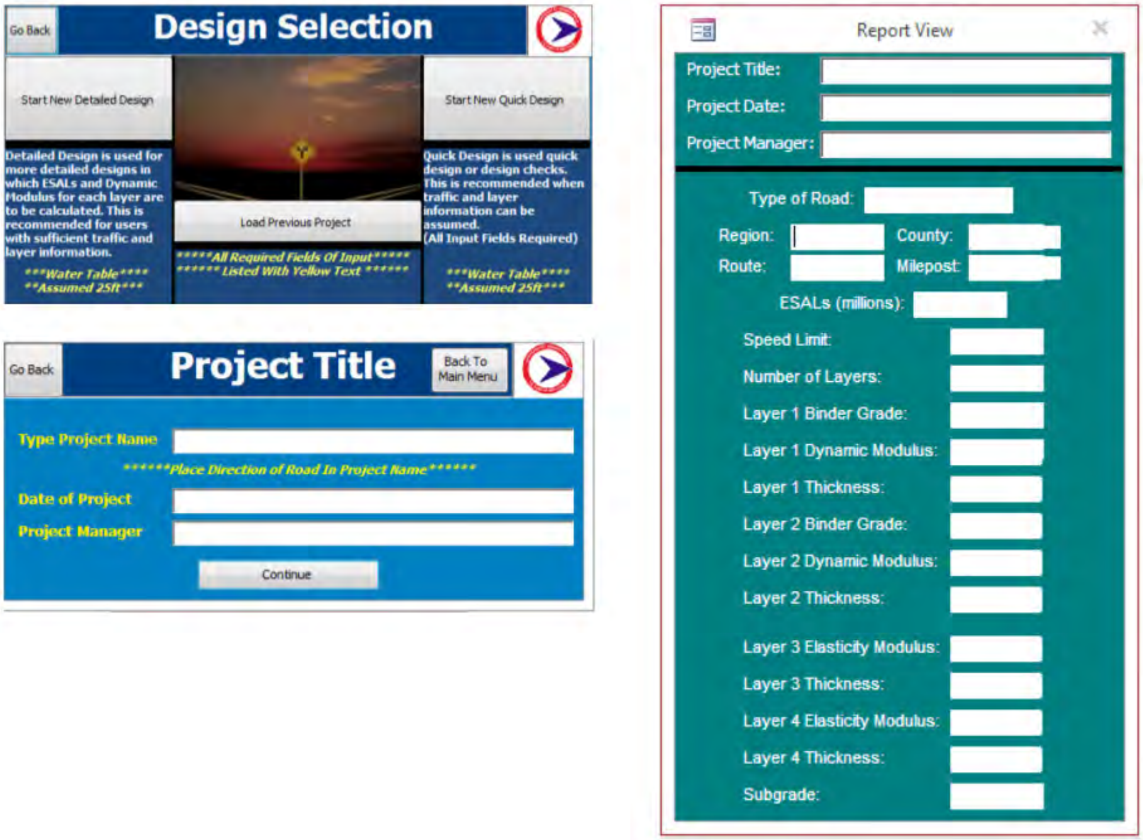


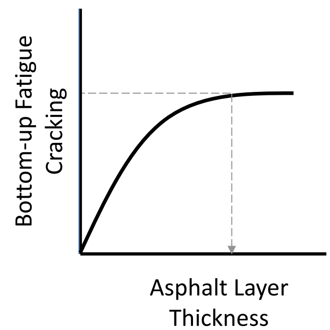
Figure 9. Photo. New Jersey DOT pavement design selection database.

- **New York State DOT**. The New York State DOT has constructed four concrete test sections across the state to evaluate material properties, pavement performance, and the impact of truck loading. Material testing, deflection testing, stress/strain measurements, and pavement performance (transverse cracking, faulting, and IRI) have been conducted and used to locally calibrate the JPCP performance prediction models (Sargand, Khoury, and Morrison 2012). New York State DOT is in the process of developing a JPCP design table and conducting a climate sensitivity analysis for JPCP.

  New York State DOT, through a study with the University of Texas-Arlington, is conducting research to develop design tables for flexible pavements (Romanoschi, Abdullah, and Bendana 2014). Local calibration utilized data obtained from a total of seventeen LTPP sites within New England. The asphalt pavement rut depth, alligator cracking, and IRI models have been calibrated to New York State conditions. The report has been finalized and is under review by the New York State DOT.

  In addition, New York State DOT has (or is) participating in several FHWA pooled fund studies including TPF-5(079) *Implementation of the 2002 AASHTO Design Guide for Pavement Structures*, TPF-5(121) *Monitoring and Modeling of Pavement Response and Performance*, and TPF-5(300) *Performance and Load Response of Rigid Pavement Systems*.

- **North Carolina DOT**. The North Carolina DOT conducted local calibration of the HMA pavement performance prediction models contained in the NCHRP 1-37A software. Since there have been changes to the models in the AASHTOWare Pavement ME Design™ software, they more than likely will need to recalibrate the HMA models. They have also started looking at the intermediate files to verify/evaluate the stress sensitivity of the developed designs. As soon as they have 5 years of concrete performance data, they will calibrate the JPCP performance prediction models. Site selection for re-calibration will include pavement segments that use the current materials specifications, have longer performance history, include only HMA pavement segments designed using Superpave mixes, and have complete datasets (e.g., construction, mix design, performance history). They have also developed a local calibration database. For validation, the Department queried the pavement management system (e.g., pavement age, progression of distress) to determine if the MEPDG prediction models are reasonably reflecting in-service pavement performance. They are currently conducting a research study, through the University of Illinois at Urbana-Champaign, to evaluate the rutting potential of aggregate materials and to develop and calibrate rutting damage models based on laboratory and field performance data. During the pavement design process, the North Carolina DOT is generating graphs based on total HMA layer thickness and bottom-up cracking and selecting the thickness level based on where the slope of bottom-up fatigue cracking approaches zero (see sketch). They also noted that documenting agency specification changes would be helpful to future generations. The North Carolina DOT developed a document to sell the MEPDG to the executive staff. This document compared the cost of WIM sites, material characterization, and so on to the improvement in pavement design and resulting performance.



- **Oregon DOT**. The Iowa State University has conducted local calibration using high and low traffic volumes, dry cold and wet mild climate conditions for asphalt pavements and a limited number of CRCP pavements. The results of the calibration effort include:
  - Due to the damage caused by studded tires and the use of less expensive aggregate materials for subgrade stabilization, Oregon DOT has disregarded the use of the MEPDG rut depth performance prediction models.
  - The thermal cracking model under predicted the field conditions; however, thermal cracking has not been an issue in Oregon since the implementation of performance grade (PG) binders in 2000.
  - The top-down cracking performance prediction model appears questionable and is being evaluated in an ongoing Oregon DOT research study.
  - The local calibration of the asphalt fatigue cracking performance prediction model resulted in small changes from the nationally calibrated model. One locally calibrated fatigue cracking performance prediction model is used for all asphalt pavements with more than 40 million (20-year) equivalent single axle loads (ESAL).

  - The nationally calibrated performance prediction models are used for all CRCP designs. Based on a comparison using four pavement sites, the resulting designs compared well with Oregon DOT experience.

- **Pennsylvania DOT**. The Pennsylvania DOT purchased AASHTOWare service units to receive introductory training on the MEPDG. Applied Research Associates (ARA) lead an MEPDG implementation planning meeting and provided recommendations on calibration site selection and developed an MEPDG implementation plan. Instrumented pavement sections constructed under the Superpave In-Situ Stress/Strain Investigation (SISSI) project are being used for local calibration (Solaimanian et al. 2006).
- **Quebec MOT**. Through the Canadian user group, Provinces have evaluated simulated designs and compared results. This effort has helped to get pavement designers to run the software, discuss the results, and improve the confidence level in the use of the AASHTOWare Pavement ME Design™ software.
- **Utah DOT**. Utah DOT conducted calibration and validation of the distress and IRI models using both LTPP and state highway pavement sections. In total, twenty-eight asphalt pavement and twenty-three JPCP segments were used in the calibration process. Utah DOT determined that for asphalt pavements the national alligator cracking model needs future calibration, the national transverse cracking model was valid for asphalt binders and mixtures used in Utah, the national rutting model over predicts field performance (by approximately 56 percent) and requires calibration, and the national IRI model was valid for Utah asphalt pavements. For JPCP, the MEPDG performance models were recalibrated using the “corrected CTE” values. Both the national transverse fatigue cracking and faulting models were valid for the NCHRP 20-07 calibration, and the national IRI model was valid for Utah JPCP.
- **Vermont AOT**. The initial MEPDG calibration effort for the Vermont AOT occurred in 2012 and a second effort was initiated in 2014. They are using data from five sites across the state to locally calibrate the MEPDG performance prediction models to Vermont conditions. At this time Vermont AOT is focusing on calibration of the IRI and rut prediction models. As new pavement sections are being constructed they are being included as MEPDG calibration sites.
- **Virginia DOT**. The Virginia DOT has conducted a number of MEPDG-related research studies and developed a manual for conducting pavement designs using the MEPDG (internal agency document). Pavement condition data are available for the years 2007 to 2013, primarily on the interstate and primary systems. Asphalt pavement performance prediction model calibration was conducted using newly constructed HMA pavement segments built after 2000 (representing Superpave mix designs) and with HMA layer thicknesses greater than 8 inches. The initial HMA performance prediction model calibration indicated that the MEPDG over predicts rut depth and under predicts bottom-up fatigue cracking.
- **Washington State DOT**. Local calibration has been conducted for new concrete pavements (Li et al. 2006) and for flexible pavements (Li, Pierce, and Uhlmeyer 2009). The primary findings from the local calibration include:

- Top-down cracking is a primary distress in asphalt pavement in Washington State; however, the MEPDG top-down cracking model does not adequately predict this distress type.
- Longitudinal cracking and studded tire wear are the primary distress types for concrete pavements in Washington State; however, neither of these distress types are modeled in the AASHTOWare Pavement ME Design™ software.

Washington State DOT has refined the pavement design catalog based on the 1993 AASHTO pavement design procedure, historical performance, and the MEPDG (Li et al. 2011). The Washington State DOT pavement design catalog is shown in table 10.

Table 10. Washington State DOT pavement design table (adapted from WSDOT 2011).

Design ESALs	HMA Pavement Thickness (in)	HMA Base[1] Thickness (in)	PCC Slab Thickness (in)	PCC Base Thickness (in)
< 5,000,000	6.0	6.0	8.0	4.2[1]
5,000,000 to 10,000,000	8.0	6.0	9.0	4.2[1] + 4.2[2]
10,000,000 to 25,000,000	10.0	6.0	10.0	4.2[1] + 4.2[2]
25,000,000 to 50,000,000	11.0	7.0	11.0	4.2[1] + 4.2[2]
50,000,000 to 100,000,000	12.0	8.0	12.0	4.2[1] + 4.2[2]
100,000,000 to 200,000,000	13.0	9.0	13.0	4.2[1] + 4.2[2]

1 Crushed surfacing base course.
2 Hot mix asphalt base.

- Dr. Darter noted that the standard deviation equations are just as important as the performance prediction model coefficients. The standard deviation equation impacts reliability.
- Cemex and ACPA provided a presentation on the comparison of national and local calibration results for JPCP models. To date, nineteen agencies have conducted local validation/calibration of the JPCP performance models, and eight of these agencies have changed one or more model coefficients. When compared to the national model, models that have been locally calibration result in 0.5 in or less difference in the required concrete slab thickness. The impact of curling due to higher CTE values can be mitigated by increasing the slab thickness and shortening the joint spacing. Dowel bars should be used for concrete slab thicknesses greater than 7.5 in. Depending on soil and climatic conditions, the MEPDG IRI design criteria cannot be met; however, reasonable slab thicknesses can be found to satisfy the cracking and faulting criteria. Since adding thickness to satisfy the IRI requirements is costly and not warranted, it was suggested that

agencies base the design on the lowest slab thickness that satisfies both the cracking and faulting requirements.

- As pavements designed using the MEPDG are constructed, tracking the construction process and evaluating the variability of the material test results could be beneficial.
- The HMA rutting model (Witczak model) is being evaluated by the AASHTO Joint Technical Committee on Pavements (JTCoP). JTCoP is collecting points of view from the pavement design community and will be conducting an NCHRP 20-07 study (similar to the CTE study, NCHPR 20-07, Task 327) to verify the rut depth performance prediction model. JTCoP will also be looking to calibrate the HMA rutting prediction model based on agency experience.

A summary of agency implementation activities is further summarized in table 11. See also Appendix I for a list of applicable agency reports and ongoing research projects.

Table 11. Agency MEPDG implementation activities.

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Alabama DOT	Completed CBR and HMA dynamic modulus testing, develop soils and HMA materials library	Developing automated file generation of axle load spectra from WIM data, identifying additional WIM sites	—	Develop calibration database	—	Training is being conducted through Auburn University	—
Alaska DOT&PF	HMA dynamic modulus testing, master curve; k1, k2, and k3 values for unbound base; classified base courses according to MR, percent passing No. 200 sieve, moisture content, and k1, k2, and k3. Developed materials catalog	Evaluate data from the twelve WIM sites (study not yet funded)	—	—	Alaska-specific mechanistic-empirical asphalt pavement design procedure; potentially use the MEPDG for comparison purposes	—	Developed Alaska-based ME design program, hesitant to implement MEPDG
Alberta Transportation	HMA dynamic modulus testing for most mix type and asphalt binder grade combinations	Traffic input data from WIM sites (calibrated monthly); installing new WIM site and relocating two others to collect more data	—	—	—	Canadian Guide	Implementation plan was completed several years ago, funding and staffing levels has hindered progress

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Arizona DOT	Completed characterization of asphalt materials, granular base, and subgrade soils	Completed traffic characterization	—	Evaluated global models and conducted local calibration using 180 sites with up to 20 years of condition data; conducted FWD testing and backcalculated layer moduli	Conducting parallel designs with DARWin and Structural Overlay Design for Arizona (SODA); using AASHTOWare Pavement ME Design™ on all approved projects	In the process of reviewing user guide; provided basic training to ADOT staff through AASHTOWare service units	All pavements designed by in-house staff are conducted with the MEPDG
Arkansas SHTD	Develop materials library, design input catalogs	Constructing WIM sites, portable WIM on secondary roads	—	Calibration completed for HMA performance prediction models and attempted to calibrate for JPCP but did not have enough data	Conduct additional concurrent designs	User Guide and Training was completed through the University of Arkansas. Additional training is planned.	—
Caltrans	Completed library of typical materials (concrete, bound and unbound based, subgrade soils)	Completed traffic database (1978 to current)	Completed climate database; conducted a sensitivity analysis	Rigid pavement models locally calibrated using data from in-service pavements; performance data from 1978-2004	Developed design catalog for use by Caltrans Design Engineers; compared results to other pavement design methods	Training and support for districts to be completed	Adopted ME pavement design methods for rigid pavements in 2005; use design tables

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Colorado DOT	Subgrade soils testing, but have not developed a soils database, CTE testing complete	Completed traffic database. Developed three clusters	Developed process for adding weather stations	Calibrated both asphalt and concrete performance models	AASHTOWare Pavement ME Design™ is used on all CDOT projects	Pavement Design Manual has been developed	As of July 2014, all designs are conducted with the MEPDG
Connecticut DOT	Plans to develop material database	Plans to develop traffic database	Completed sensitivity analysis of inputs	Developed recommendations for input levels and required resources to obtain those inputs; plans to calibrate and validate models within 2 to 3 years	—	Training materials have been developed through the University of Connecticut; future plans for staff training and user guide development	Plans to establish implementation team; plans to develop design table
Florida DOT	Database for HMA dynamic modulus and resilient modulus for soils, constructing concrete test road	—	Evaluating climate data to quantify impact on JPCP thickness	Evaluating new release to determine if recalibration is needed	—	—	JPCP only
Georgia DOT	—	—	—	Develop local calibration database	Conduct concurrent designs in 2015	User guide and training in progress	Under development

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Idaho TD	Flexible pavement database (asphalt binder, asphalt mixture, unbound base, and subgrade soils); future plans for PCC pavements (currently unfunded)	Twenty-seven WIM sites; traffic database	—	Calibration road map completed; local calibration for flexible pavements to be initiated in 2015 (2 year study); future plans (currently unfunded) for local calibration of PCC models	Use the MEPDG as a design check	Initial user guide prepared	Implementation roadmap, implementation plan for flexible pavements and user guide; full implementation expected in 2 to 3 years
Kentucky TC	—	Used Prep-ME for traffic data quality control, collecting additional WIM data, using default values until more data is collected	—	Identifying calibration sites, conducting site testing, reviewing historical condition data	Conduct concurrent designs using level 2/3 inputs	—	—
Louisiana DOTD	Materials characterization has been completed	Used Prep-ME for traffic data, constructing WIM sites	Determine distress threshold criteria, develop climate data file for each parish	In progress	Conducting concurrent designs, comparing results to DARWin	—	—

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Maryland SHA	Developed a catalog for asphalt, concrete and unbound material properties.	Completed a report on WIM implementation program	—	Currently performing a validation study of national models for Maryland conditions	New construction only; using AASHTO 1993 Pavement Design Guide as a design check	MEPDG chapter in design manual, training course conducted in 2012 for pavement engineers	Currently performing a validation study of national models for Maryland conditions
Mississippi DOT	HMA dynamic modulus completed; characterization of concrete, cement stabilized, unbound aggregate, and subgrades to be completed in 2015	Traffic characterization complete	Climate analysis conducted by NCAT	Site selection using pavement management data, FWD testing and backcalculation to begin in 2015	—	—	Initiated in State Study 163 and refined in State Study 170
Mississippi DOT	HMA dynamic modulus completed; characterization of concrete, cement stabilized, unbound aggregate, and subgrades to be completed in 2015	Traffic characterization complete	Climate analysis conducted by NCAT	Site selection using pavement management data, FWD testing and backcalculation to begin in 2015	—	—	Initiated in State Study 163 and refined in State Study 170

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Montana DOT	Asphalt binder, asphalt mixture, unbound base, and subgrade soil testing on new construction test sections; developing a GIS-based tool for accessing asphalt mixture properties on existing projects	Sixteen WIM sites (calibrated annually)	FWD testing is conducted statewide over a 5 year period and at the project-level	2007 research study established local calibration of flexible pavement models	AASHTO 1993 pavement design procedure (Microsoft Excel spreadsheet)	Field guide for flexible performance prediction models	Waiting for model updates based on NCHRP 1-42A results (top-down cracking)
Nebraska DOR	Asphalt materials and subgrade soils; includes data for all three input levels	Two WIM sites (used only by the State Patrol)	Conducted a parametric study to determine the importance of each input value and developed field instrumentation plan for data collection	—	—	—	Concrete pavement designs only
Nevada DOT	Completed asphalt material testing; evaluating asphalt mixtures using polymer modified binders; conducted concrete testing (two projects only)	Ongoing	—	Concrete model calibration complete; asphalt materials rutting model calibration completed; asphalt fatigue and cracking model calibration in progress	AASHTO 1993 Design is final; comparison designs will be conducted until agency is comfortable with MEPDG	—	All concrete pavement design will be conducted with the MEPDG; asphalt pavement design by July 2015

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
New Hampshire DOT	Default dynamic modulus values (Jackson et al. 2011)	—	—	—	Currently uses the 1972 AASHTO design procedure; have conducted comparative designs using MEPDG on one project	Attended FHWA MEPDG Workshop (2011)	—
New Jersey DOT	Materials database has been developed (includes dynamic modulus results on typical mixes)	—	—	—	Currently uses the AASHTO *1993 Pavement Design Guide* and DARWin 3.1 for all pavement designs; conducting comparative designs	—	—
New Mexico DOT	Developed E*, mix design, and soils database, conducting asphalt mix design testing (to be completed 2016),	Developing traffic database	Instrumented interstate asphalt pavement with strain and temperature gauges for validation	Working on asphalt model calibration	Comparison designs using MEPDG	—	In progress

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
New York State DOT	Completed dynamic modulus testing; asphalt materials database; resilient modulus testing on base course and subgrade soils; concrete materials database	Developed traffic database; use single statewide values for VCD, MDF, and AGPV	Developed climate database	Calibrated JPCP (based on four experimental pavement sections) and flexible pavement models (based on New England LTPP sites)	Modified existing design table based on MEPDG results (JPCP table complete, flexible pavement table under review)	—	Implement design table for both JPCP and flexible pavements by the end of 2015
North Carolina DOT	HMA materials testing completed, conducting concrete material testing	Completed clustering approach for traffic data, developed nine traffic files by functional class	Evaluate aggregate base versus full-depth asphalt sections, impact of subgrade Mr, include 20 years of climate data	May need to recalibrate HMA models, use MEPDG calibration coefficients and reliability levels, local calibration database in progress	Conducted designs using MEPDG since 2011, design using level 2 inputs	—	Completed
North Dakota DOT	—	—	—	Calibrated JPCP models	—	—	—
Oregon DOT	Asphalt pavement characterization (asphalt mixture, unbound base and subgrade soils); ongoing study to evaluate instrumented pavement sections	Sixteen WIM sites; used a "virtual" truck to develop needed traffic data inputs	—	Locally calibration of concrete pavement performance prediction models; evaluating asphalt pavement top-down cracking model	MEPDG used exclusively for concrete pavement designs, and for high traffic volume asphalt pavements	—	—

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Pennsylvania DOT	Plans to develop materials database	Plans to develop traffic database	Plans to develop default input values specific to Pennsylvania	—	Districts are conducting comparative designs with DARWin 3.1	User guide is under development; AASHTOWare service units to conduct introductory training	Implementation plan has been developed; full implementation anticipated within 18 months
Quebec MOT	Developed materials database	Developed traffic database	Developed climatic database	Developed calibration database	—	—	—
South Carolina DOT	RFP for HMA dynamic modulus and CTE testing	Evaluating need for additional WIM sites	—	Identify calibration sites, database plan, and instrument sites in progress	Concurrent designs planned using level 3 inputs	—	—
South Dakota DOT	Asphalt pavement characterization (asphalt mixture, unbound base, and subgrade soils; additional asphalt mixture characterization in progress	Sixteen WIM sites	Plans to use MERRA database to supplement climate data	—	—	—	Developed implementation plan
Tennessee DOT	HMA materials library in progress	Evaluating need for additional WIM sites	Sensitivity analysis in progress	In progress	Concurrent designs planned	—	Expected in 2016

Table 11. Agency MEPDG implementation activities (continued).

Agency	Materials	Traffic	Other	Calibration/ Validation	Pavement Design	User Guide & Training	Implementation Plan
Utah DOT	Completed untreated base course and soils resilient modulus testing; CTE testing complete	Completed traffic characterization	—	Completed for both asphalt and concrete pavement performance models	Side-by-side comparisons with DARWin since 2010	User guide; hands-on training for staff and consultants; presentations to upper management	Full implementation in 2011, except for Federal Aid – Local projects, (will be required by 2015)
Vermont AOT	—	—	—	In progress	Currently uses the AASHTO *1993 Pavement Design Guide* and DARWin 3.1 for all pavement designs	—	—
Virginia DOT	Materials library, subgrade classification in progress	Traffic library in progress, using one axle load distribution for all designs	—	Calibration of HMA performance models completed	Concurrent designs began in 2014	User guide completed (internal document), training in progress	—
Washington DOT	—	Evaluated axle load spectra data; sensitivity analysis of axle load spectra on pavement thickness	—	Calibrated flexible and rigid models based on MEPDG v0.6	Developed pavement design catalog based on 1993 AASHTO Guide, MEPDG, and agency experience	To be developed	Require recalibration; benchmark testing to determine impact of changes since MEPDG v0.6

Figure 10 provides the current MEPDG implementation status of State Highway Agencies. As previously noted, none of the Canadian Provincial governments have implemented the MEPDG at this time; however, Alberta, Manitoba, Ontario, Quebec, and the city of Edmonton are actively evaluating the MEPDG.

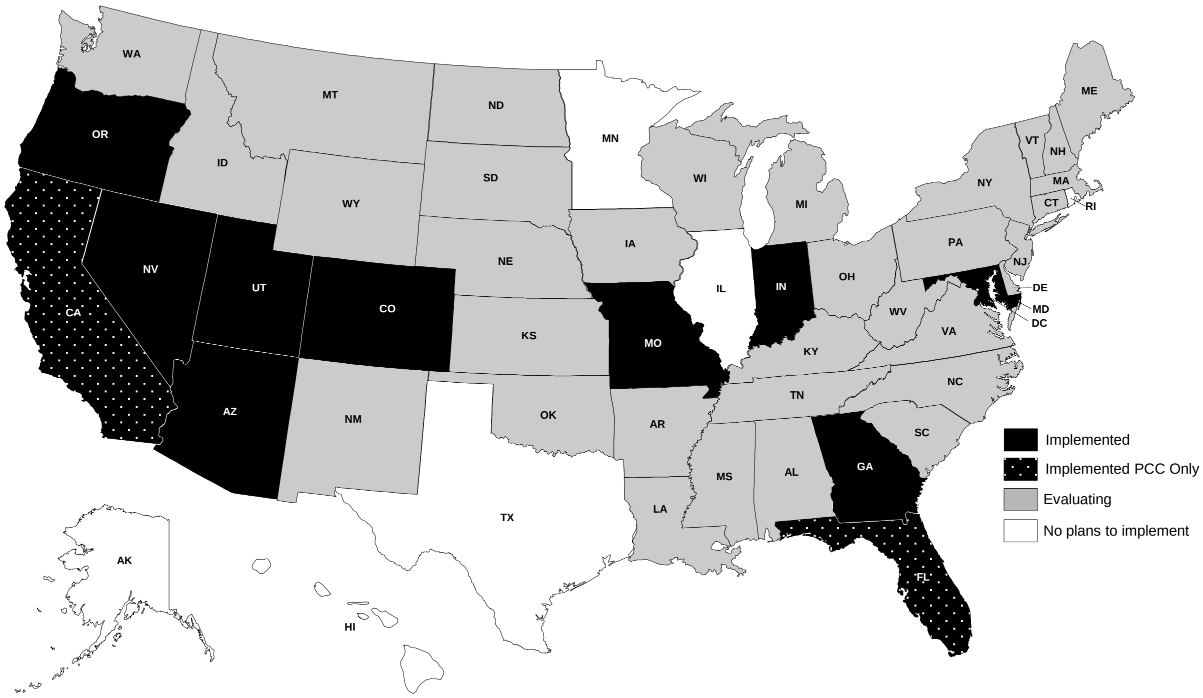


Figure 10. Photo. Summary of MEPDG implementation status.

Implementation Challenges, Issues, or Roadblocks

Agencies were asked to provide any challenges, issues, or roadblocks that may hinder implementation of the MEPDG. The following provides a summary of agency responses.

- **Arizona DOT**. Obtaining quality traffic data and collecting/processing WIM and other traffic data for use in level 1 analysis.
- **Caltrans**. Caltrans has identified a number of implementation issues including interpretation of the software results and ease-of-use with the software interface, deployment of the software to Caltrans Design Engineers (> 1,000 individuals), providing adequate and timely training on ME principles and software use, accommodation of local agencies, integration of ME design with life-cycle cost analysis, correlating ME design with pavement management, consideration of future preservation treatments, and revising construction specifications to correspond with ME design practices.
- **Montana DOT**. The Montana DOT's primary challenge with MEPDG implementation is staffing. There has been a significant number of retirements and turnover since the 2007 research effort resulting in a loss of internal knowledge with the MEPDG. The DOT also notes that equipment costs and staffing for the specialized testing equipment posed additional implementation challenges and roadblocks. In relation to pavement design, Montana DOT standard practice places chip seals on new asphalt pavement construction and maintenance overlays on existing pavements; however, neither of these

practices are accounted for in the MEPDG. The Montana DOT pavement network includes a significant number of low-volume roadways where the 1993 AASHTO pavement design procedure has worked well. Montana DOT is evaluating the benefits for implementing the MEPDG.

- **Nevada DOT**. Calibration of the asphalt fatigue and cracking models has been hindered due to the lack of distress on the selected field calibration sites. Determining if calibration should be based on functional classification, climate, or traffic. Limited weather data available with software download, only six weather stations for entire state of Nevada.
- **New Mexico DOT**. The DOT is very sensitive to reduction in pavement thickness without significant justification.
- **North Dakota DOT**. The MEPDG analysis indicates thinner (10 inch) concrete pavement thickness are sufficient for the DOT's design conditions; however, 12- to 13-inch concrete pavements have typically been constructed. It is unknown if the thinner sections will meet the performance expectations.
- **Utah DOT**. Multiple training sessions are needed to train staff on software operation.
- **Washington State DOT**. The Washington DOT identified a number of implementation issues including the need to develop an MEPDG user guide for Washington State, preparing input files for use by pavement design staff at headquarters and in each of the six regions, training pavement design engineers in ME principles and software use, and checking and responding to user feedback.
- Sun-setting DARWin has limited agencies ability to compare AASHTO 1993 and AASHTOWare Pavement ME Design™ results. PaveXpress (and others) can be used to for pavement designs using the AASHTO 1993 (http://www.pavexpressdesign.com/).
- Calibrating the JPCP models due to limited pavement segments with sufficient performance history.
- The IRI model predicts a more severe increase in IRI over time. Actual pavement performance shows a slower increase in IRI over time.
- There is a disconnect between pavement design inputs and what is included in the construction specifications; for example, dynamic modulus is typically not confirmed during construction.
- The backcalculation process requires significant knowledge and experience, which makes it difficult to use for most agencies.
- The North Carolina DOT is preparing a workshop on the results of the SPS-2 sites. This workshop may provide an opportunity to discuss developing regional JPCP performance prediction models.
- The more centralized the agency, the "easier" the implementation (Chris Wagner).
- Include MEPDG in university-level curricula.
- As contractors become more knowledgeable and efficient with the use of the MEPDG, agencies need to be able to respond to change orders that include a reduced pavement thickness.

Training and Documentation, Software, and Research Needs

A list of MEPDG and AASHTOWare Pavement ME Design™ training courses, workshops, and webinars is provided in Appendix J. The following provides a list of attendee identified training, software, and research needs:

Training and Documentation

- Develop guidelines for using M_r determined values from FWD testing.
- Develop training courses for ME fundamentals and design, and AASHTOWare Pavement ME Design™ software function and operation.
- Provide unlimited access to the educational version of the software. Suggestions include removing the annual license fee, expanding the database availability, providing access for “virtual” students, and developing a training camp on the MEPDG.
- Develop procedures on how to include the impact of preservation and how to include pavement sections with and without preservation treatments in the calibration process.
- Provide access to presentations (PDF files) from the FHWA webinars.
- Develop a high-level document for executive staff to help “sell” MEPDG implementation.
- Provide guidance on how to set up the calibration database (e.g., what items are needed) and demonstration of how the database is being used during the calibration (re-calibration) process. Develop a format for the local calibration database.
- Update the AASHTO *Manual of Practice* to reflect modifications and updates.
- Conduct a synthesis of highway agency calibration coefficients.
- Develop a website (potentially AASHTOWare) for accessing agency research reports and user guides related to the MEPDG and the AASHTOWare Pavement ME Design™ software (similar to asphaltfacts.com).
- Conduct a synthesis of agency design practices, for example, performance life, joint spacing, base type, and dowel bar size.
- Conduct a synthesis of agency implementation efforts using pavement management system data.
- Provide guidelines on how to incorporate unbound aggregate with recycled asphalt pavement (RAP).

Software

- Develop regional and national material and traffic databases. This would allow sharing of data between agencies and improve the calibration of the national performance prediction models.
- Provide a brief description of what is contained in all of the software generated temporary files; include descriptive column headings, units of measure, and so on.

- Provide the ability for agencies to reset the IRI, rut depth, and top-down cracking levels for prescribed future rehabilitation treatments.
- Provide a more comprehensive input file structure.
- Standardize the use of significant figures based on inputs.
- Document "tricks" for addressing pavement types not currently included in AASHTOWare Pavement ME Design™ Software.
- Provide a method for agencies to share implementation challenges, software issues, and resolution. It was suggested that this could be included on the AASHTOWare site.
- Include a warning in the AASHTOWare Pavement ME Design™ software that indicates when the user has input an unbound aggregate to subgrade soil ratio less than 2 (or 3).
- Provide status of software updates on AASHTOWare website (e.g., updates of current release, what will be included on next release).

Research

- Develop direct correlations between R-value and resilient modulus and California Bearing Ratio (CBR) and resilient modulus.
- Develop a concrete corner cracking model.
- Develop regionally-based JPCP performance prediction models.
- Improve rut depth prediction model for unbound aggregates and subgrade soils.
- Use the calibration coefficients for all agencies that have completed MEPDG model calibration and additional data collected for the LTPP sites to recalibrate the national models and possibly develop regional models.
- Develop a more efficient process for model calibration and recalibration. This could include generating and populating a calibration database, conducting the statistical analysis, and recommending calibration coefficients. It would be beneficial to automate as much of this process as possible.
- Compare pavement performance prediction to laboratory test results.
- Improve the methods for obtaining software inputs.
- Provide additional rehabilitation design options (e.g., hot in-place, cold in-place, full-depth reclamation, thin concrete overlays).
- Improve the unbound aggregate layer rutting model.
- Develop model for shrinkage cracking in asphalt pavements (southwest phenomenon).
- Incorporate ability to design thin concrete overlays (currently included in the AASHTOWare 2016 work plan).
- Develop test method for surface absorptivity (study underway with Ohio State and NCAT).

Peer Exchange Takeaways

The following provides a list of attendee identified peer exchange takeaways.

- Models need to be calibrated and recalibrated as additional data are obtained (calibration is a continuous process and is not a "once and done" effort).
- Training is necessary for successful MEPDG implementation.
- MERRA data can be used to complement the climate data that are currently included in the AASHTOWare Pavement ME Design™ software.
- Information provide on AASHTOWare current, ongoing, and future software enhancements was helpful.
- MEPDG will continue to provide a benefit in analyzing pavement structures and will be a better tool in the future with anticipated enhancements.
- All highway agencies are facing similar MEPDG implementation issues.
- Need to calibrate models using reasonable resilient modulus values.
- Require a modular ratio between unbound aggregate and subgrade soil of 2 to 3.
- "What you calibrate to is what you should use in design."
- Conduct CTE, modulus, and strength testing to characterize concrete materials.
- Evaluate how the change in inputs impact the final design results.
- Conduct FWD testing and backcalculation after construction to validate layer moduli.
- Capture "real" values for use in design-build projects.
- Validation and calibration of the Arizona and Utah DOT performance models was conducted without significant laboratory testing.
- Need to provide an interaction between pavement design and pavement management in the calibration process.
- Ensure that the right person is on the AASHTOWare list of licensee's. Several agencies indicated that they were unaware of updates or correspondences from AASHTOWare, which may imply that the applicable agency person is not listed as the primary contact.
- Evaluating required thickness versus the level of bottom-up cracking (North Carolina DOT process).
- Calibration process presented by North Carolina and Kentucky was very helpful.
- The values selected for reliability levels, types of performance prediction models used in the design/evaluation process, and distress threshold limits are similar amongst agencies.
- The need to create a database for local calibration (re-calibration).
- Documenting agency specification changes for use by future generations.
- Overlaying dry asphalt mixes that have top-down cracking results in poor overlay performance. Should mill and fill prior to the placement of the overlay.
- Work with other agencies to regionally calibrate the JPCP models.

- Need for fundamental training on mechanistic-empirical principals and design methodology.
- AASHTOWare Pavement ME Design™ software demonstration was the first opportunity of seeing the software in operation for several of the attendees.
- Beneficial to hear the efforts of other agencies in relation to the MEPDG.
- Helpful to know that other agencies have developed or will be developing design tables based on the MEPDG results.

Lessons Learned

- Plan for staffing continuity to minimize the impact of lost knowledge due to retirements and turnover.
- Need to sell the adoption/implementation of the MEPDG based on non-financial reasons. Obtaining executive buy-in is essential to the MEPDG implementation effort.
- Use other agency calibration coefficients as a starting point for local calibration.
- MEPDG calibration is not a "once and done" effort, and there is a need to develop a calibration database for long-term use.
- The North Carolina DOT indicated they should have waited for the production software before conducting local calibration. For research-related projects, be specific on what is expected to be the desired product. For example, specify that a database needs to be developed, populated, and provided to the agency with the results of the calibration process. Ensure that the calibration sites have a full range of distress types and severities. Conduct local calibration even though you may not have all the data. By at least starting the process you can identify what data you need.
- Mississippi DOT indicated that money should be spent up front to collect quality data for improving the pavement design process.
- Verify availability of needed data (e.g., traffic, materials, construction records, and performance data) prior to initiating the calibration process.
- If you don't know when a concrete pavement will be constructed, use July or August in the AASHTOWare Pavement ME Design™ software because it was noted that the impact of curling at high temperature in Colorado occurs during these months.
- It is important to question the inputs and outputs of the AASHTOWare Pavement ME Design™ software. If the output doesn't look right, question the input values used, and verify that they make engineering sense.
- Don't require the pavement section to meet all distress types over the performance life, except for thermal and fatigue cracking of asphalt pavements and faulting on concrete pavements. All other distress types should meet agency performance criteria. For example, Colorado DOTs performance criteria limit for asphalt pavement rutting and IRI is based on distress at year 12 and at year 27 for IRI and cracking on concrete pavements.
- AASHTOWare Pavement ME Design™ software predicts average distress, you get to pick the reliability level.

- Develop a comprehensive agency MEPDG user manual. As promotions and retirements occur, it is important to develop and update a thorough and complete MEPDG user manual to minimize a loss of knowledge and to shorten the learning curve. User manual can also be used to assist in staff and consultant training. Training is essential for conducting pavement rehabilitation designs.
- Access to good construction data and pavement condition data will significantly improve the calibration process. For this reason, it is also beneficial to include LTPP sites in the calibration process. It may be necessary to re-collect pavement condition data in accordance with the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2003).
- Trench studies can be a very valuable tool for confirming layer rutting.
- Work with the construction division and industry partners to align pavement design inputs and construction specifications.
- For calibration sites, develop a link between pavement management, pavement design, construction, and maintenance to track treatments and quantify performance.
- Nevada DOT has a very aggressive pavement preservation program (i.e., applying treatments when pavements are still in good to very good condition) and there are very few pavement sections with significant distress progression.

MEPDG User Group Pooled-Fund Study

FHWA provided a brief discussion on the pooled-fund study, *Regional and National Implementation and Coordination of ME Design* (http://www.pooledfund.org/Details/Solicitation/1365). A summary of project details include:

- Pooled fund will support two participants from each agency to attend one regional meeting and the national user group meeting each year. Current participating agencies include Alabama, Colorado, Iowa, Kansas, Kentucky, Maryland, Missouri, North Carolina, North Dakota, Ontario, Pennsylvania, Virginia, AASHTO, and FHWA.
- Pooled fund cost is $10,000 per person per year over a 5-year period. Cost includes travel to one regional peer exchange meeting and the annual national user group meeting each year.
- One to two people from each AASHTO region will be asked to participate on the advisory committee to help with meeting planning. The project details will be scoped in the June/July 2015 timeframe.
- Workshops on hot topics and/or training on key aspects of the MEPDG (and software) can be included in the regional and national user group meetings.
- For the national user group meeting, include invited presentations, regional presentations (e.g., materials, climate, traffic), and breakout sessions (e.g., traffic clustering, materials). In addition, pavement management personnel could be included as invitees.
- For additional information, contact Chris Wagner, christopher.wagner@dot.gov, (404) 562-3693.

The TAC MEPDG user group has been in-place for about 7 years and meets prior to the annual meeting and at least one other time during the year and through regularly scheduled conference calls. The leading Provinces participating in the MEPDG user group include Alberta, Manitoba, Ontario, and Quebec. The meetings and conference calls are used to share files, discuss results, and resolve issues as a group. The group also is working to expand the Canadian climate database, evaluate the performance model calibration using SI Units, and present papers and conduct workshops during Canadian conferences.

Agency Next Steps

Agencies were asked to provide a brief summary of next steps to further the implementation of the MEPDG in their agency. The following provides a list of agency responses.

- **Alabama DOT**. Develop local calibration database, and material testing at National Center for Asphalt Technology.
- **Alberta Transportation**. Inform executive staff of MEPDG implementation plans, include comparative design requirements for consultant designs for 2016/2017 contracts, and mine data from LTPP and research grade test sites.
- **Arizona DOT**. Finalize user manual and identify other implementation needs.
- **Arkansas DOT**. Coordinate with other groups (e.g., traffic, materials), develop materials database, review pavement management data to define condition thresholds, identify availability of electronic data files, and conduct additional concurrent designs.
- **Caltrans**. Use FWD deflection data to backcalculate concrete layer stiffness and load transfer efficiencies; conduct concrete flexural stiffness and strength testing; determine CTE from laboratory testing or identify typical values; and soil classification from triaxial laboratory data or derived from typical values.
- **Colorado DOT**. Evaluate additional implementation needs and conduct staff training.
- **Connecticut DOT**. Under the University of Connecticut Phase II research study, the sensitivity analysis of the Phase I effort will be expanded and the MEPDG distress prediction models will be validated using the state pavement management data. Also need to talk with upper management on the MEPDG and the needed steps for implementation and assign staff to specific task for evaluating the MEPDG and AASHTOWare Pavement ME Design™ software.
- **Florida DOT**. Continue evaluating climate data to determine why there is a difference in the JPCP thickness.
- **Georgia DOT**. Implementation project was completed in January 2015, complete the revisions to the MEPDG user guide, and conduct concurrent designs over the next year.
- **Idaho DOT**. Conduct material characterization of concrete pavements and develop database (likely to be initiated late 2015). Encourage region pavement designers to check pavement designs using the AASHTOWare Pavement ME Design™ software.
- **Kentucky TC**. Identify calibration sites, conducting testing on calibration sites, review historical condition data, expand data set to include more pavement management data, and conduct concurrent designs.

- **Louisiana DOTD**. Construct permanent WIM sites, improve climate data, continue with ongoing effort in local calibration, compare MEPDG results with DARWin results, and identify local distress criteria and thresholds.
- **Montana DOT**. Obtain additional pavement management system project records to determine local calibration coefficients for Montana DOT Grade D and S mixes. Conduct additional FWD testing for quantifying seasonal material effects. Montana DOT would also like to identify any MEPDG method changes that have occurred since the 2007 calibration effort, and staffing requirements, training resources, and MEPDG input levels.
- **Nebraska DOT**. Initiate Phase I study to evaluate LTPP sites to determine bias and error in performance prediction models. Phase II (additional calibration) and Phase III (develop user manual and conduct training) will be conducted at later dates.
- **Nevada DOT**. Conduct more rigorous distress data collection, conduct materials testing that is not currently included in mix design process, and work closely with the Nevada DOT Traffic Information Division to obtain the required traffic data.
- **New Jersey DOT**. Become more comfortable with the MEPDG flexible pavement design process and evaluate the applicability of the design procedure to composite pavements (asphalt over concrete).
- **New Mexico DOT**. Complete calibration of asphalt performance models and develop traffic database.
- **New York State DOT**. Develop design tables for rigid pavements and review the flexible pavement design tables (expected to occur within 18 months).
- **North Carolina DOT**. Improve climate data files (expected by end of November 2014), conduct concrete material testing (CTE, Young's modulus, and modulus of rupture on 18 mixes), and compare full-depth asphalt and asphalt over aggregate base performance (identify sites, which will also be used as future calibration sites).
- **North Dakota DOT**. Continue calibration of concrete pavement performance models and evaluate performance of pavement segments that were designed using the MEPDG.
- **Oregon DOT**. Purchase a multi-user license for software evaluation. Determine the impact of the NCAT evaluation of Oregon DOT instrumented pavement sections on calibration of performance models.
- **South Carolina DOT**. Conduct Phase I calibration study (12 to 18 months) to identify the number and location of calibration sites, develop a database and instrumentation plan for the calibration sites, currently released an RFP to conduct CTE and dynamic modulus testing (12 to 18 months), conduct Phase II study to develop database, discussing the possibility of adding WIM sites with the traffic division, and review the pavement management database for use in the calibration process.
- **South Dakota DOT**. Complete climate study and asphalt pavement material characterization, conduct comparison designs, and initiate local calibration of performance prediction models.
- **Tennessee DOT**. Develop a materials library, conduct local calibration (2-year study in progress), and conduct concurrent designs.

- **Utah DOT**. Continue training of agency staff and conduct local agency training.
- **Virginia DOT**. Conduct local calibration, identify policy items and review with stakeholders (e.g., reliability, limiting distress), and purchase service units for training.
- **Washington State DOT**. Purchase software license for in-house evaluation, refine the local calibration results for doweled JPCP and Superpave asphalt mixtures, and test and locally calibrate the HMA overlay of HMA and the HMA overlay of PCCP performance prediction models to Washington State conditions.

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APPENDIX B. TYPICAL MEETING AGENDA

Time	Topic
DAY 1	
8:00 – 9:30 a.m.	Welcome and Introductions
9:30 – 9:45 a.m.	BREAK
9:45 – 10:30 a.m.	Materials
10:30 – 11:15 a.m.	Climate
11:15 a.m. – 12:00 p.m.	Traffic
12:00 – 1:00 p.m.	LUNCH (on your own)
1:00 – 2:30 p.m.	Threshold limits, reliability, and hierarchical levels
2:30 – 2:45 p.m.	BREAK
2:45 – 4:30 p.m.	Calibration
4:30 – 5:00 p.m.	Day one key takeaways
DAY 2	
8:00 – 10:00 a.m.	Calibration
10:00 – 10:15 a.m.	BREAK
10:15 a.m. – 12:00 p.m.	Challenges/Issues/Roadblocks
12:00 – 1:00 p.m.	LUNCH (on your own)
1:00 – 2:00 p.m.	Additional needs
2:00 – 2:15 p.m.	BREAK
2:15 – 3:15 p.m.	Lessons learned
3:15 – 4:00 p.m.	User group
4:00 – 4:30 p.m.	SHA next steps
4:30 – 5:00 p.m.	Day two takeaways and closing remarks

APPENDIX C. MERRA CLIMATE DATA

TECHBRIEF

Evaluation of LTPP Climatic Data for Use in Mechanistic-Empirical Pavement Design Guide (MEPDG) Calibration and Other Pavement Analysis

The Long-Term Pavement Performance (LTPP) program is a large research project for the study of inservice pavements across North America. Its goal is to extend the life of highway pavements through various designs of new and rehabilitated pavement structures, using different materials and under different loads, environments, subgrade soil, and maintenance practices. LTPP was established under the Strategic Highway Research Program and is now managed by the Federal Highway Administration.

U.S. Department of Transportation

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http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/

Publication No. FHWA-HRT-15-026

FHWA Contact: Larry Wiser, HRDI-30, (202) 493–3079.

This document is a technical summary of the Federal Highway Administration report, *Evaluation of LTPP Climatic Data for Use in Mechanistic-Empirical Pavement Design Guide (MEPDG) Calibration and Other Pavement Analysis* (FHWA-HRT-15-019).

Objective

This TechBrief describes evaluating the use of the Modern-Era Retrospective Analysis for Research and Applications (MERRA) product as an alternative climatic data source for the Mechanistic-Empirical Pavement Design Guide (MEPDG) and other transportation infrastructure applications. The research was conducted from 2011 to 2014.

Introduction

The analysis methodology developed in the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) and accompanying software, Pavement ME Design®, emphasize the influence of climate on pavement performance (*1*). The temperature and moisture analyses performed by the MEPDG's Enhanced Integrated Climate Model (EICM) require air temperature, wind speed, percent sunshine, relative humidity, and precipitation values at hourly time intervals over the entire design life of the project.

Weather history information required by the MEPDG is typically obtained from ground-based operating weather stations (OWS) located near the project site. The MEPDG software includes a climate database of approximately 800 OWS throughout the United States, most located at commercial airports. If needed, climate data from multiple nearby stations can be interpolated as a virtual weather station (VWS).

MERRA (*2*), developed by the National Aeronautics and Space Administration (NASA), is a physically based global climate reanalysis product that combines computed model fields (e.g., atmospheric temperatures) with ocean-, airborne-, and satellite-based observations that are distributed irregularly in space and time. MERRA employs Gridpoint Statistical Interpolation (GSI) system over a vast number of observations. More than four million physical observations are processed during a typical 6-hour data assimilation cycle. MERRA data are provided from 1979 to the present at an hourly temporal resolution and a horizontal spatial resolution of 0.5 degrees latitude by 0.67 degrees longitude (approximately 50 km by 65 km at midlatitudes) at multiple elevations in the atmosphere.

Research

Statistical comparisons between MERRA climate data and those from various conventional ground-based sources for several hundred locations and comparisons of MEPDG performance predictions using MEPDG OWS and MERRA climate data for twenty locations distributed across the contiguous United States were performed.

A variety of data sources were examined in this study. Ground-based climate data provided as part of the MEPDG serve as the standard input for flexible and rigid pavement simulations using the Pavement ME Design® software. Additional data sources employed for comparisons with the MEPDG climate files include the Quality Controlled Local Climatological Data (QCLCD), United States Climate Reference Network (USCRN), and NASA's MERRA.

Table 1 summarizes the meteorological data evaluated in this study, both from the MEDPG climate files and from the other climate data sources described in the subsequent sections.

MERRA is capable of providing all weather history inputs required by the MEPDG and other current infrastructure applications. Table 2 contains the MERRA data elements used to develop MEPDG weather history inputs.

MERRA contains additional data elements useful for enhancements of current infrastructure applications and/or for the support of future applications. A complete listing of all MERRA data elements can be found at http://gmao.gsfc.nasa.gov/products/documents/MERRA_File_Specification.pdf.

A major advantage of MERRA over ground-based climate data sources is the uniform spatial coverage. The ground-based Automated Surface Observation System (ASOS) stations that provide much of the current ground-based climate data are mostly located at airports and therefore clustered along the east and west coasts of the United States and around major population centers. Many MERRA grid cells contain no ASOS weather stations, and most MERRA grid cells that do contain ASOS weather stations usually contain no more than one.

Table 1. Variables employed from each measurement product for use during analysis.				
Variable of Interest	**QCLCD**	**MEPDG**	**USCRN**	**MERRA**
Air Temperature	X	X	X	X
Dewpoint Temperature[1]				
Specific Humidity[1]	X	X	X	X
Wind Speed	X	X		X
Precipitation	X	X	X	X
Shortwave Radiation			X	X
Cloud Cover Fraction[2]		X		X
Sky Condition[3]	X			

X = Measurement / estimate is available at the majority of locations.
1 = Dewpoint temperature and specific humidity provide equivalent to humidity.
2 = Cloud cover fraction serves as a proxy for shortwave radiation.
3 = Sky condition serves as a proxy for cloud cover fraction, and hence, shortwave radiation.

Table 2. MERRA data elements used to develop MEPDG weather history inputs.		
Element	**Description**	**Units**
CLDTOT	Total cloud fraction	fraction
PRECTOT	Precipitation flux incident upon the ground surface	kg H_2O m^2 s^{-1}
PS	Surface pressure at 2 meters above ground surface	Pa
QV2M	Specific humidity at 2 meters above ground surface	kg H_2O kg^{-1} air
SWGDN	Shortwave radiation incident upon the ground surface	W m^{-2}
SWTDN	Shortwave radiation incident at the top of atmosphere	W m^{-2}
T2M	Air temperature at 2 meters above ground surface	K
U2M	Eastward wind at 2 meters above ground surface	m s^{-1}
V2M	Northward wind at 2 meters above ground surface	m s^{-1}

Statistical analyses were conducted between the different data sources relative to USCRN (i.e., USCRN treated as the reference measurement) for the approximately 17-year period of 1 July 1996 through 1 September 2013. This time period corresponds to the approximate temporal overlap of all available data sources used in this study. The emphasis of the statistical evaluation was on temperatures, as prior studies have shown that pavement performance was most sensitive to these climate inputs (*3, 4*). Wind speed and cloud cover are the next most sensitive climate inputs; however, the USCRN data do not contain these data elements and consequently they could not be evaluated. Although the MEPDG in its current form assumes no infiltration of surface water into the pavement layers, precipitation data from the various climate data products were nevertheless compared.

For near-surface air temperature, the average bias across all 275 collocated datasets evaluated was 0.63 °C for QCLCD vs. USCRN and

1.40 °C for the MERRA vs. USCRN comparisons. The spread of the MERRA bias distribution is slightly broader than for the QCLCD data; the average root-mean-square (RMSE) values were 2.04 °C for the QCLCD vs. USCRN and 3.28 °C for the MERRA vs. USCRN comparisons. Overall, both the QCLCD and MERRA data were different and warmer than the USCRN reference values, with the MERRA data being slightly more warm and variable.

Analyses were conducted for hourly precipitation rates between collocated USCRN, QCLCD, and MERRA stations. Both the QCLCD and MERRA data closely agree with USCRN precipitation measurements, but MERRA has 50 percent less average bias than does QCLCD (0.02 mm/hr vs. -0.03 mm/hr). Further, numerous QCLCD stations contain significant negative bias relative to USCRN, which is consistent with rain gauge "under catch" that is a known and pervasive problem with point-scale rain gauges. The RMSE is also slightly lower in the MERRA estimates (0.81 mm/hr vs. 0.90 mm/hr for QCLCD).

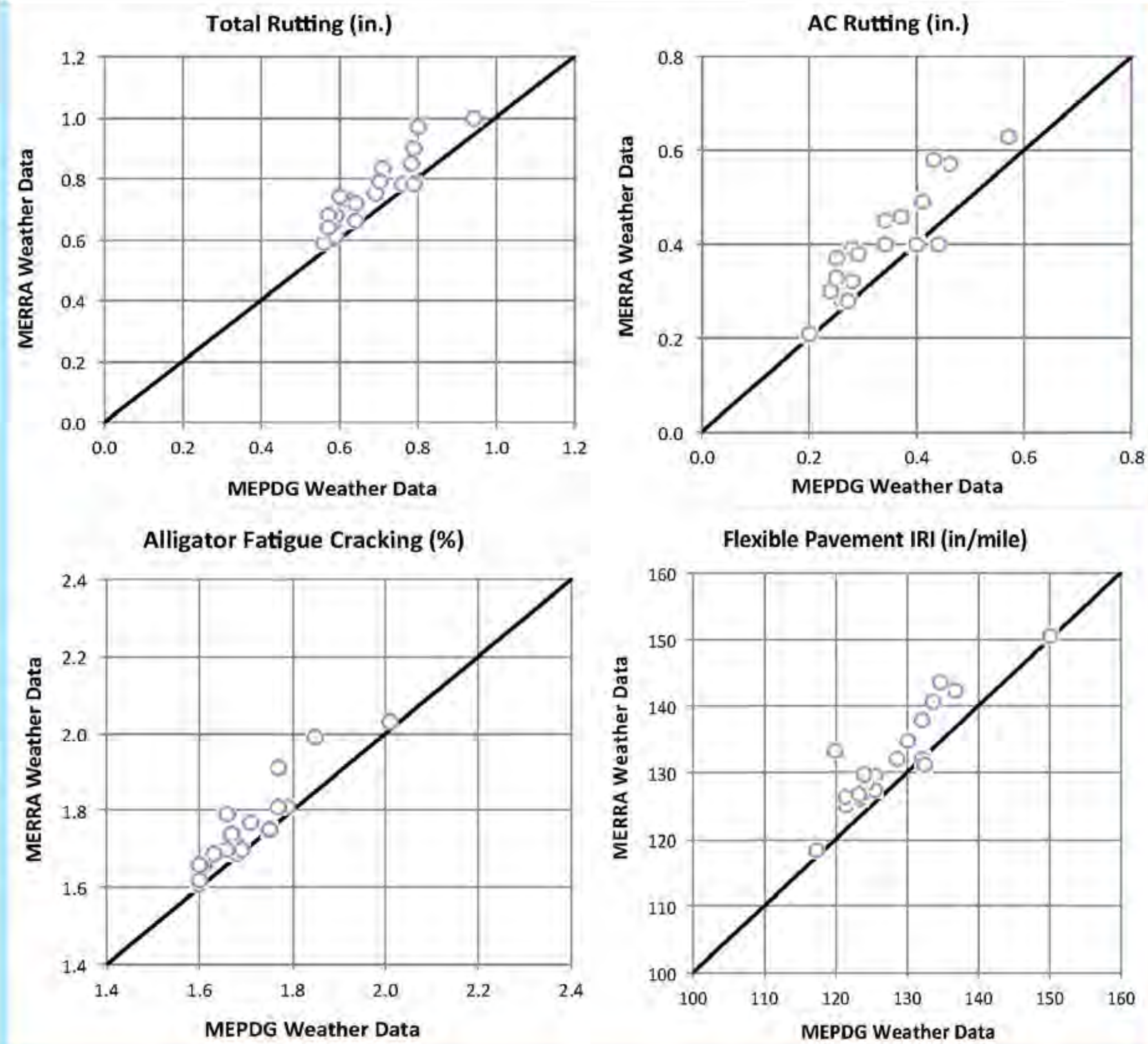
Pavement performance as predicted by the MEDPG models incorporated in the Pavement ME Design® software was evaluated using the MEPDG weather data files provided with the software, which are derived from the QCLCD and Unedited Local Climatological Data (ULCD) products from the National Climatic Data Center (NCDC) and the MERRA climate data for collocated sites and congruent time series. A total of 20 sites distributed randomly across the contiguous United States were analyzed.

It would have been ideal to evaluate MEPDG performance predictions using the USCRN "ground reference" data in addition to the MEDPG and MERRA weather time series. However, the USCRN data do not include the wind speed and cloud cover data required for the MEPDG models. Several attempts were made to synthesize these missing data from other sources, but none were satisfactory.

Both new flexible pavements and new jointed plain concrete pavements (JPCP) were analyzed. The pavement structures, traffic loads, material properties, and other inputs for the analysis correspond to the medium traffic cases for the sensitivity analyses described in Schwartz, et al and Ceylan, et al. (*5, 6*). All analyses were performed using Version 2.0 of the Pavement ME Design® software.

Comparisons of flexible pavement performance as predicted by the MEPDG using MERRA vs. MEPDG weather data are shown in figure 1 for total rutting, asphalt concrete (AC) rutting, alligator fatigue cracking, and roughness. Top-down longitudinal fatigue cracking was not considered because this model is generally viewed as unreasonably sensitive and unrealistic; a replacement for the current top-down fatigue cracking model is currently being developed in NCHRP Project 1-52.

Figure 1. Comparison of MEPDG flexible pavement predictions for total rutting, AC rutting, alligator fatigue cracking, and roughness (IRI) using MERRA vs. MEPDG weather data.



In all cases, the predictions are clustered tightly although not perfectly along the respective lines of equality and show a slightly higher prediction of distress for MERRA. This is consistent with the close but not perfect agreement found among these climate data time series in the statistical comparisons described previously.

Comparisons of rigid JPCP pavement performance as predicted by the MEPDG using MERRA versus MEPDG weather data show that these predictions are also clustered tightly although not perfectly along the respective lines of equality. This is also consistent with the close but not perfect agreement found among these climate data time series in the statistical comparisons described previously. The agreement between the MERRA vs. MEPDG weather data cases for rigid pavement performance is somewhat less than for flexible pavements. However, this is consistent with the fact that rigid pavement performance is more sensitive to shorter term (e.g., diurnal) temperature variations and thus to the differences between MERRA vs. MEPDG weather data over short time periods.

Conclusions

Results of and conclusions from the research include the following points:

The statistical comparisons of hourly temperature data, the meteorological variable most influential on pavement performance, found that the QCLCD and MERRA data have small and roughly comparable differences from the USCRN values. The mean biases in hourly temperatures computed for the QCLCD and MERRA data vs. the USCRN reference values averaged across 275 sites were 0.63 °C and 1.40 °C, respectively. The MERRA data are slightly warmer on average than the QCLCD values, but only by less than 1 °C.

Comparisons of predicted performance using the different sources of weather data are arguably the most relevant for pavement applications. Overall, the comparisons in MEPDG predicted performance for both flexible and rigid pavements using MERRA vs. MEPDG weather data are close and acceptable for engineering design. Based on the statistical comparisons among the various climate data sources, the agreement in predicted performance using MERRA vs. USCRN "ground truth" and/or MEPDG vs. USCRN would likely show similar scatter in agreement as seen in Figure 1. However, it is impossible to demonstrate this because the USCRN data lack the wind speed and cloud cover inputs required by the MEPDG software.

Both the statistical and performance prediction comparisons support the conclusion that MERRA is an acceptable source for climate data that can be used in place of conventional ground-based OWS sources.

Recommendations for LTPP

Based on the results of the research effort, it is recommended that:

1. The LTPP program uses the MERRA dataset as the basis for continuous hourly climate data histories for its test locations.

2. Using the MERRA dataset, LTPP should calculate the same derived computed climate statistics as shown in table 1.

3. The climate module in the LTPP Information Management System (IMS) should be expanded to contain MERRA data for the cells where LTPP test sections are currently located (thus MERRA data and OWS data will exist in the IMS and the users can select the dataset they wish to use).

Implementation of MERRA in the MEPDG

MERRA has shown great promise as a possible supplement or replacement for the weather data currently used in the MEPDG. The benefits, as outlined herein, have the potential to provide a more robust dataset with more granular spatial coverage and higher quality.

Based on the findings of this research, it is recommended that MERRA be considered for implementation within the

MEPDG. As part of this research, a tool is being created to extract MERRA data in a MEPDG-compatible format. This dataset could very easily (1) replace the current weather dataset or (2) be used as a complement to the OWS-based MEDPG dataset. In the case of option 1, the MERRA dataset could easily be formatted similarly to the existing dataset and an algorithm implemented in the MEPDG to select the appropriate MERRA cell(s) for a given project site. If option 2 is considered, the process described in option 1 would be implemented and a checkbox or similar data selection toggle could be presented to users so that they have a choice as to which dataset to use. In either case, the level of effort to implement MERRA in the MEPGD is relatively low and would require very little, if any, additional code to change the underlying MEDPG analytical engine.

A larger and potentially more significant change to the MEPDG analytical engine would be to permit use of the direct prediction of surface shortwave radiation from MERRA. Surface shortwave radiation is the key driver for pavement temperature variation. The MEPDG computes this using top-of-atmosphere solar radiation and an empirical relationship to incorporate diffusion and absorption through the atmosphere. The empirical relationship is both dated and calibrated only to northern tier continental U.S. States and Alaska. MERRA provides direct predictions of surface shortwave radiation, eliminating the empirical correction for diffusion and absorption. The changes to the EICM code to incorporate this would be modest. An additional benefit is that this would eliminate the need for the difficult-to-determine percent cloud cover input currently required by the MEPDG software.

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Additional Information

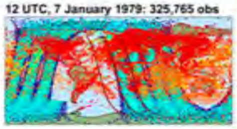
For additional information, contact Larry Wiser, Office of Infrastructure R&D, 6300 Georgetown Pike, McLean, VA 22101-2296; phone: 202–493–3079

Researchers — This study was performed by AMEC Environment and Infrastructure, Inc. and the University of Maryland, College Park. The Project Manager was Jonathan Groeger. The Principal Investigator was Dr. Charles Schwartz, P.E.
Distribution — This TechBrief is being distributed according to a standard distribution. Direct distribution is being made to the FHWA Divisions and Resource Center.
Availability — The report will be available in 2015 and may be obtained from the FHWA Product Distribution Center by email to report.center@fhwa.dot.gov, by fax to 301–577–1421, or by phone to 301–577–0818.
Key Words — MERRA, MEPDG, pavement performance, climate data, pavement design.

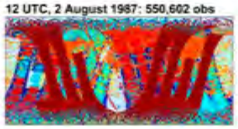
Data Assimilated for MERRA

The volume of data ingested during a 6-hourly assimilation cycle changes dramatically over time. During the EOS era, over 4 million observations are assimilated at one time.

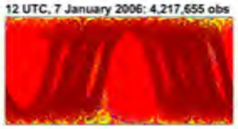
12 UTC, 7 January 1979: 325,765 obs



12 UTC, 2 August 1987: 550,602 obs



12 UTC, 7 January 2006: 4,217,655 obs



Conventional data & Satellite retrievals

Data Source/Type	Period	Data Supplier
Radiosondes	1970 – present	NCEP
PIBAL winds	1970 - present	NCEP
Wind profiles	1992/5/14 - present	UCAR
Conventional, ASDAR and MDCRS aircraft rep.	1970 - present	NCEP
Dropsondes	1970 - present	NCEP
PAOB	1978 – 2010/8	NCEP
GMS, METEOSAT, cloud drift IR & visible winds	1977 - present	NCEP
GOES cloud drift winds	1997 - present	NCEP
EOS/Terra/MODIS winds	2002/7/01 - present	NCEP
EOS/Aqua/MODIS winds	2003/9/01 - present	NCEP
Surface ship and buoy observations	1977 - present	NCEP
Surface land observations	1970 - present	NCEP
SSM/I V6 wind speed	1987/7 - present	RSS
SSM/I rain rate	1987/7 - present	GSFC
TMI rain rate	1997/12 - present	GSFC
QuikSCAT surface winds	1999/7 – 2009/9	JPL
ERS-1 surface winds	1991/8/5 – 1996/5/21	CERSAT
ERS-2 surface winds	1996/3/19 – 2001/1/17	CERSAT
SBUV ozone (V8 retrievals)	1978/10 - present	GSFC

Satellite radiance data

Data Source/Type	Period	Data Provider
TOVS (TIROS N, N-6, N-7, N-8)	1978/10/30 – 1985/01/01	NCAR
(A)TOVS (N-9, N-10, N-11, N-12)	1985/01/01 – 1997/07/14	NESDIS/NCAR
ATOVS (N-14, N-15, N-16, N-17, N-18)	1995/01/19 - present	NESDIS
EOS/Aqua	2002/10 - present	NESDIS
SSM/I V6 (F08, F10, F11, F13, F14, F15)	1987/7 - present	RSS
GOES Sounder T_B	2001/01 - present	NCEP

Find more information on MERRA at

http://gmao.gsfc.nasa.gov/merra

MERRA products are available online through the Goddard Earth Sciences Data and Information Services Center:

http://disc.sci.gsfc.nasa.gov/mdisc/data-holdings

MERRA was conducted at the NASA Center for Climate Simulation (NCCS).

The GMAO works to maximize the impact of satellite observations in the analysis and prediction of climate and weather through integrated Earth system modeling and data assimilation.

Global Modeling and Assimilation Office

Code 610.1

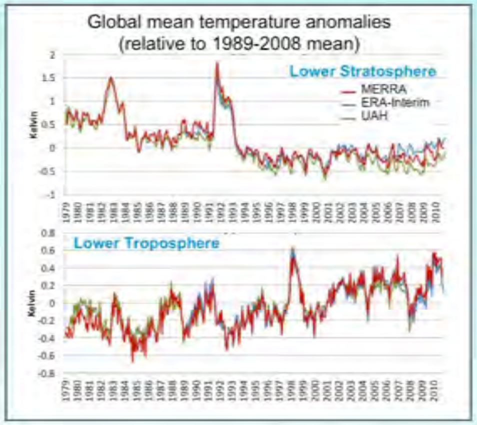
NASA/Goddard Space Flight Center

Greenbelt, MD 20771

http://gmao.gsfc.nasa.gov

MERRA

The Modern-Era Retrospective analysis for Research and Applications



Global Modeling and Assimilation Office

Goddard Space Flight Center

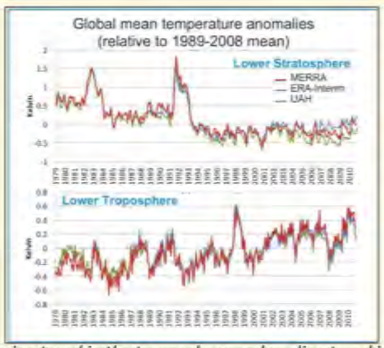
MERRA

The Global Modeling and Assimilation Office (GMAO) has used its GEOS-5 atmospheric data assimilation system (ADAS) to synthesize the various observations collected over the satellite era (from 1979 to the present) into an analysis that is as consistent as possible over time because it uses a fixed assimilation system. This contrasts with a weather -focused analysis where the system changes over time as improvements are implemented to improve weather forecasts. The goal of this historical re-processing - called MERRA, the **Modern-Era Retrospective Analysis for Research and Applications** - is a climate-quality analysis that places NASA's EOS observations into a climate context.

The MERRA System

MERRA is being conducted with version 5.2.0 of the GEOS-5 ADAS with a 1/2° latitude × 2/3° longitude × 72 layers model configuration.

A key development in the GSI, not available for the previous generation of reanalyses, has been the online bias correction for satellite radiance observations. Such corrections are needed to compensate for sensor drifts as well as to ensure that observations from different satellites, which have been calibrated independently, provide consistent measurements of our environment.



The warming trend in the troposphere and cooling trend in the stratosphere are evident in the MERRA and ERA-Interim results, and are in good agreement with the independent estimate based on MSU retrievals from the University of Alabama at Huntsville (UAH). From about 2004 onwards the additional data used in the reanalyses introduce larger discrepancies from the estimate using MSU/AMSU alone.

Products

MERRA has completed analysis of over 30 years of data and is now proceeding forward in near real-time as a climate analysis. Products are distributed through the GES DISC (http://disc.sci.gsfc.nasa.gov/MDISC/dataprods/merra_products.shtml) with several download options.

There are 26 product collections. Products are generated on three horizontal grids:

- Native ----------- (1/2º × 2/3º using model conventions)
- Reduced ------- (1¼º × 1¼º, dateline-edge, pole-edge)
- Reduced FV -- (1× 1¼º using model conventions)

3-D data are 72 model layers or 42 pressure levels.

Products include:

Analyzed Fields (u, v, t, q,O_3, ps): native grid, 6-hourly instantaneous fields, on model and pressure levels

Assimilated Fields: reduced grid, 3-hourly instantaneous fields on pressure levels

3-D Diagnostic Fields: reduced grid, 3-hourly time-averaged fields on pressure levels.

2-D Diagnostic Fields: native grid, hourly time-averaged fields

Products for Offline CTMs: various resolutions, frequencies and grids.

Ocean Surface Diagnostic Fields: native resolution, 1-hourly, 2D fields that can be used for ocean models

MERRA-Land Surface Diagnostic Fields: a supplemental product from a version of the Catchment Land Surface Model driven offline with MERRA forcing except that precipitation has been corrected with a global gauge-based data set from NOAA's Climate Prediction Center (see http://gmao.gsfc.nasa.gov/merra/news/merra-land_release.php).

Selected MERRA monthly mean products have been made available at PCMDI's Earth System Grid for CMIP5 model evaluations.

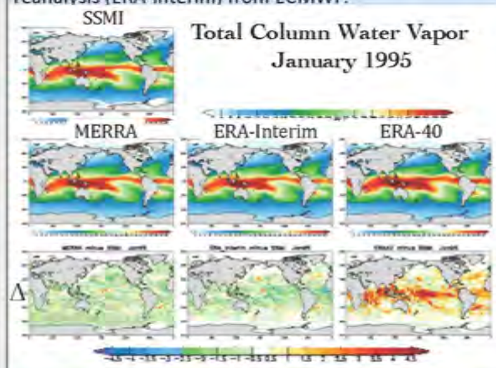
Resources

An atlas of MERRA climate with comparisons with other reanalyses and with gridded observations is available at: **http://gmao.gsfc.nasa.gov/ref/merra/atlas/.**

MERRA is documented in a set of publication that forms the *MERRA Collection* in the Journal of Climate: **http://journals.ametsoc.org/page/MERRA.**

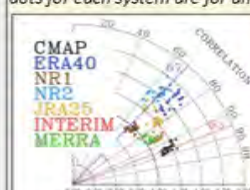
Some Results

Since MERRA focused on the hydrological cycle, our early evaluation of the system has looked at various aspects of the moisture distribution and variability, compared with the previous reanalyses and also the more recent reanalysis (ERA-Interim) from ECMWF.

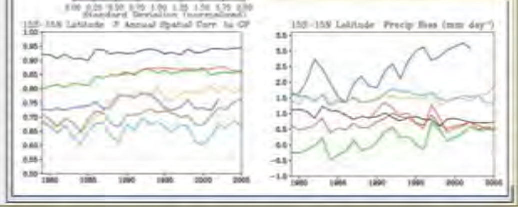


Monthly mean TCWV (kg m^{-2}) from reanalyses (2nd row) compared with that from SSMI (top row). The differences (reanalysis-SSMI) are shown in the bottom row. MERRA and ERA-Interim are very close and are slightly drier than the observations. Absolute differences are mostly smaller than 2 kg m^{-2}. In contrast, ERA-40 is much wetter than SSMI over much of the tropics.

The quality of the tropical (15S-15N) precipitation from the various reanalyses is evaluated by comparison with GPCP. A different observational product, CMAP, provides a baseline for the limit to how good a comparison can be expected. The comparisons are summarized in a Taylor diagram (upper plot) of the correlation and standard deviation normalized by GPCP. The dots for each system are for annual means of different years. A perfect comparison would be (1, 100).



The lower plots show time series of annual means of spatial correlation and mean bias. Clearly both MERRA and ERA-Interim have improved upon earlier reanalyses.



APPENDIX D. AGENCY PRESENTATIONS

AASHTOWare Pavement ME Design™ Task Force Update

Alberta

AAHTOWare Pavement ME Design™ Task Force Update Portland MEPDG Peer Exchange April 14-15 2015

Marta Juhasz, P.Eng.
Surfacing Standards Specialist
Alberta Transportation

Alberta

Software

AASHTOWare Pavement ME Design AASHTO

- **Most recent release v 2.1**
  - Released Summer 2014 (build date 7/29/2014)
  - Release notes: http://me-design.com/MEDesign/data/AASHTOWare%20Pavement%20ME%20Design%20Build%202.1.x%20Release%20Notes.pdf
- **Included**
  - Special Traffic Loading feature for flexible pavements
  - Backcalculation summary reports
  - Automatic Updater
  - Subgrade Modulus Sensitivity
  - Context sensitive Help

2

Alberta

Current Enhancements Underway

- **DRIP (posted to software web page) http://me-design.com/MEDesign/DRIP.html**
- **Reflection cracking (NCHRP 1-41)**
- **LTPP high quality traffic data (new traffic XMLs)**
- **Enhanced Climate data**
  - Also note LTPP Techbrief on MERRA data

3

Alberta

Current Enhancements Underway

- **MapME**
  - GIS data linkages for Climate, Traffic and Soils
- **API (Application Programming Interface) for ICM, JULEA and Project File**
- **New PCC Models and Coefficients**
- **Enhancements to be released July 2015**

4

Alberta

Recent Issues

- **Error in Freezing Index Calculation**
  - More of an issue for rigid pavements
  - To be corrected in next release
- **Automatic updater issue**
  - Patch issued already
- **Thermal Cracking**
  - Tensile strength in software is not temperature dependent
  - Will require recalibration
- **Code Review**
  - 3rd party was hired to review code as precursor to moving to Web-based software

5

Alberta

Code Review

- **Remember that software is legacy from different programmers/non-programmers, etc.**
- **Issues identified**
  - Dead code
  - Lack of commenting
  - Inconsistent naming practices
  - Hard coded constant numeric values
  - Inconsistent logic levels

6

FY2016 Enhancements

- **Priority 1: Code**
  - Code clean up
  - Mapping of US and SI units
  - Technical audit of code for engineering errors
  - Will include fix of thermal cracking issue
- **Thin Bonded Concrete Overlay**
- **Backcalculation Parts 1-3**
  - Part 1 – pre-processing tool
  - Parts 2 and 3 dependent on getting source code
- **Long term training on ME Design Principles**
- **Review of specific research**
  - Top-down, etc.

7

Future Enhancements

- **Long List including**
  - Moving to web
  - Tools to facilitate calibration

8

Other

- **Fall 2014 Survey**
- **Canadian User Group**

9

Questions?

10

Alaska DOT&PF – Characterization of Alaskan Transportation Materials for M-E Pavement Design

Characterization of Alaskan Transportation Materials for M-E Pavement Design

Steve Saboundjian, Alaska DOT&PF
Jenny Liu, University of Alaska Fairbanks

MEPDG Peer Exchange Meeting – Portland, OR, 4/14/15

Alaskan Paving Materials

1. Hot-mix asphalt (HMA) materials
2. Granular base course (GBC) materials
3. Asphalt Treated Base course materials
   a) Hot asphalt treated (HATB)
   b) Emulsion asphalt treated (EATB)
   c) Foamed asphalt treated (FATB)
   d) 50:50 blend of RAP & base (CABC)

HMA Characterization

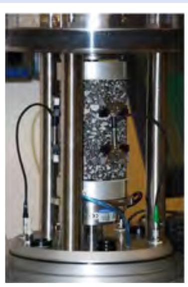
- Typical mixes in the 3 DOT&PF Regions
- Mix obtained at project site (windrow)
- Lab-reheated, compacted using SGC
- PMLC: Plant-Mixed, Lab-Compacted
- Cored and trimmed
- Dynamic modulus |E*| test (AASHTO T-342)
- Temperature (4) and Frequency (8) sweep

Typical Alaskan HMA

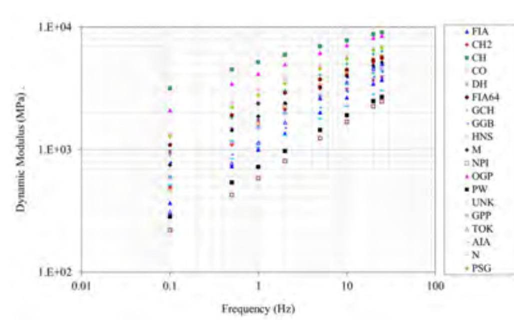
No.	Project	Abbreviation	Region	Mix Design Method	Other
1	AIA runway 7R_25L Rehabilitation	AIA	Central	SuperPave	
2	Chena Hot Springs Rd MP 24-56	CH	Northern	Marshall	
3	Chena Hot Springs	CH2	Northern	Marshall	
4	Fairbanks Cowles Street Upgrade	CO	Northern	Marshall	
5	East Dowling Road Extension and Recon.	D	Central	Marshall	Crumb Rubber HMA
6	Dalton Hwy. MP 175–197 Rehabilitation	DH	Northern	Marshall	
7	FIA Runway 1L_19R stage 3 (52-34)	FIA	Northern	Marshall	
8	FIA Runway 1L_19R stage 3 (64-34)	FIA64	Northern	Marshall	
9	Glenn Hwy MP 92-97 Cascade to Hicks Creek	GCH	Central	Marshall	
10	Glenn Hwy: Gambell to airport MP 0–1.5	GGB	Central	SuperPave	
11	Glenn Hwy: Hiland to Eklutna Resurfacing	GH	Central	Marshall	Crumb Rubber HMA
12	Glenn Hwy MP 34–42, Parks to Palmer Resurfacing	GPP	Central	SuperPave	
13	HNS Ferry Terminal to Union Street	HNS	Southeast	Marshall	
14	Minnesota Dr. Resurfacing: International Airport Rd to 13th	M	Central	SuperPave	
15	Parks Hwy MP 287–305 Rehabilitation	N	Northern	Marshall	
16	Richardson Hwy North Pole Interchange	NPI	Northern	Marshall	
17	Old Glenn Hwy: MP 11.5–18	OGP	Central	Marshall	
18	PSG Mitkof Hwy-Scow Bay to Crystal Lake Hatchery	PSG	Southeast	Marshall	
19	Palmer/Wasilla	PW	Central	Marshall	
20	Alaska Hwy MP 1267–1314	TOK	Northern	Marshall	
21	Unalakleet Airport Paving	UNK	Northern	Marshall	

4

Asphalt Mixture Performance Tester



|E*| at 21°C



|E*| Master Curves

$$\log_{10}|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log(f_R)}}$$

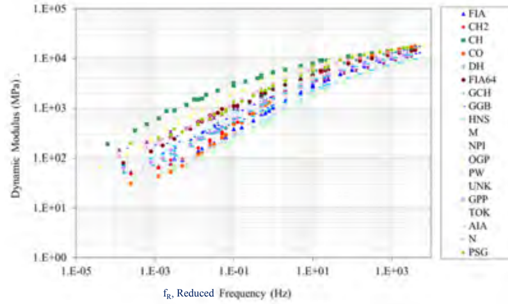
$$f_R = a(T) \cdot f$$

$$a(T) = 10^{aT^2 + bT + c}$$

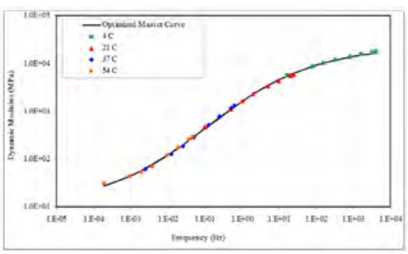
where

f	= loading frequency, Hz,
f_R	= reduced loading frequency at a reference temperature of 20°C, Hz,
$a(T)$	= shift factor as a function of temperature,
T	= temperature, °C,
$\|E^*\|$	= dynamic modulus of HMA, MPa, and
$\delta, \alpha, \beta, \gamma, a, b, c$	= regression constants.

|E*| Master Curves



Master Curve Coefficients



Master Curve Coefficients							R^2 (Logarithmic)	R^2 (Arithmetic)
δ	α	β	γ	a	b	c		
1.02628E+00	3.31607E+00	-6.43023E-01	7.04611E-01	-9.83274E-04	1.55817E-01	-2.82873E+00	0.99931	0.99641

Catalog of |E*| MCs & Coefficients

Mix Symbol	δ	α	β	γ	a	b	c
AIA	1.734E+00	2.789E+00	0.000E+00	4.930E-01	-1.250E-03	1.833E-01	-3.360E+00
CH	1.262E+00	3.082E+00	-5.030E-01	6.913E-01	-1.046E-03	1.572E-01	-2.811E+00
CH2	6.321E-01	3.684E+00	-1.641E+00	4.179E-01	-6.984E-04	1.563E-01	-3.021E+00
CO	1.026E+00	3.316E+00	-6.430E-01	7.046E-01	-9.833E-04	1.558E-01	-2.829E+00
DH	7.064E-01	3.937E+00	-5.968E-01	2.876E-01	-7.100E-04	1.476E-01	-2.795E+00
FIA64	1.324E+00	3.003E+00	-2.345E-01	6.161E-01	-8.290E-04	1.426E-01	-2.601E+00
FIA64	6.321E-01	3.759E+00	-9.884E-01	4.318E-01	-1.023E-03	1.640E-01	-3.054E+00
GCH	1.110E+00	3.172E+00	-9.097E-01	6.287E-01	-8.106E-04	1.466E-01	-2.668E+00
GGB	1.166E+00	3.095E+00	-7.500E-01	5.651E-01	-8.904E-04	1.536E-01	-2.882E+00
GPP	1.346E+00	2.945E+00	-5.011E-01	5.816E-01	-6.019E-04	1.342E-01	-2.547E+00
HNS	6.891E-01	4.172E+00	-2.520E-01	3.042E-01	-6.807E-04	1.455E-01	-2.776E+00
M	1.404E+00	2.847E+00	-6.420E-01	5.931E-01	-5.176E-04	1.312E-01	-2.526E+00
N	1.074E+00	3.213E+00	-6.469E-01	5.518E-01	-7.428E-04	1.440E-01	-2.700E+00
NPI	1.037E+00	3.469E+00	-1.355E-02	5.005E-01	-9.284E-04	1.490E-01	-2.697E+00
OGP	6.322E-01	3.746E+00	-1.367E+00	4.861E-01	-1.045E-03	1.797E-01	-3.214E+00
PSG	1.873E+00	2.491E+00	-5.309E-01	5.750E-01	-8.495E-04	1.536E-01	-2.880E+00
PW	7.990E-01	3.954E+00	-1.083E-01	3.771E-01	-1.075E-03	1.554E-01	-2.668E+00
TOK	1.427E+00	2.876E+00	-2.484E-01	8.156E-01	-1.205E-03	1.652E-01	-2.872E+00
UNK	1.421E+00	2.858E+00	-9.443E-01	5.838E-01	-6.977E-04	1.403E-01	-2.503E+00

Granular Base Course Characterization

- Base course material from the 3 Regions
- Fines content, P200 (3.15% - 10%)
- Moisture content (OMC-2%, OMC, OMC+0.7%)
- Temperature (-10°C --- 20°C)

AASHTO T-307 - Resilient Modulus

Repeated-load triaxial testing apparatus

Typical M_R Test Results

CP*(psi)	DS*(psi)	10% fines			8% fines			6% fines			3.15% fines		
		6.0%	5.3%	3.3%	6.0%	5.3%	3.3%	6.0%	5.3%	3.3%	6.0%	5.3%	3.3%
3	2.7	6.2	11.5	46.7	5.9	12.7		4.7	7.8	22.4	5.4	12.2	20.4
	5.4	9.0	13.0	30.8	9.3	14.2	33.5	7.2	10.8	20.6	8.2	14.9	21.8
	8.1	11.8	14.5	29.5	12.3	16.2	28.8	9.7	12.0	21.2	11.0	16.7	24.5
5	4.5	8.0	11.3	45.7	7.9	19.0	39.8	5.9	8.8	25.3	7.5	12.1	32.4
	9	12.4	16.8	39.0	12.9	17.8	35.4	10.1	12.4	26.0	11.8	19.4	32.3
	13.5	15.9	18.3	39.5	16.9	19.9	36.4	13.2	14.5	28.1	15.5	20.9	31.6
10	9	13.9	19.9	60.0	15.5	22.7	53.5	11.5	20.0	38.9	15.5	27.3	45.0
	18	20.2	23.7	57.4	21.7	26.2	52.1	16.6	19.2	40.5	20.8	28.8	47.8
	27	25.0	26.5	56.6	27.1	27.0	51.8	21.0	21.0	42.7	24.8	31.0	47.6
15	9	16.8	22.8	84.0	17.8	22.4	73.8	13.9	11.5	50.1	17.9	34.9	57.6
	13.5	18.2	22.6	69.4	19.7	23.4	61.9	13.7	22.4	48.0	17.9	29.1	51.2
	27	26.3	28.3	68.3	28.4	22.1	60.8	20.6	27.9	51.4	25.6	36.2	55.8
20	13.5	20.7	23.7	85.1	21.9	30.5	78.4	15.0	18.1	58.0	21.5	35.8	64.8
	18	22.5	25.8	79.2	24.5	30.4	71.4	15.4	21.9	57.2	22.5	36.5	62.4
	36	26.3	28.0	68.4	28.2	31.3	61.9	20.3	28.6	58.7	25.9	39.3	56.8

Modeling

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$

Moisture content	Fines content	k_1	k_2	k_3	R^2
3.30%	10%	1.138	0.806	-0.427	96.2%
	8%	1.095	0.780	-0.382	97.8%
	6%	1.032	0.740	-0.183	99.6%
	3.15%	1.118	0.731	-0.232	97.2%

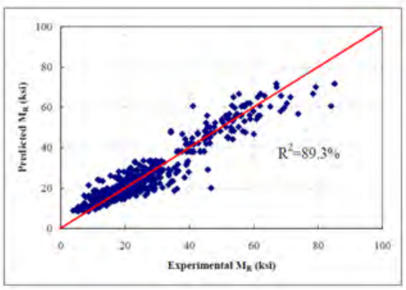
$$k_1 = 2.54 + 0.0537 * f_c - 0.3256 * W_s - 0.0073 * W_s * f_c$$

$$k_2 = 1.04 + 0.0354 * f_c - 0.1070 * W_s - 0.0071 * W_s * f_c$$

$$k_3 = -2.19 + 0.0154 * f_c + 0.4436 * W_s - 0.0049 * W_s * f_c$$

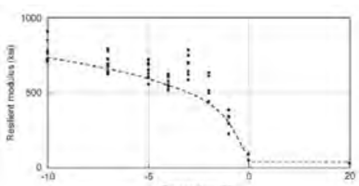
f_c = fines content (%), W_s = moisture content (%)

Measured vs Predicted M_R



Frost Heave Test Setup

Frozen M_R



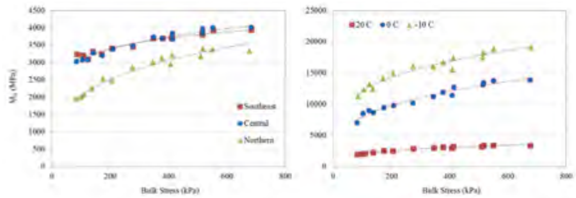
$$M_R = A_r^{-0.0371} e^{\frac{4.1014+0.7034}{T}} \sigma_d^{0.7346}$$

ATB Characterization

Mixture types	Binder	Aggregate Source	Binder Content	M_R Test
HATB	PG 52-28	Northern	2.5%, 3.5%, 4.5%	-10°C, 0°C, 20°C
		Central	2.5%, 3.5%, 4.5%	-10°C, 0°C, 20°C
		Southeast	2.5%, 3.5%, 4.5%	-10°C, 0°C, 20°C
EATB	CSS-1	Northern	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
		Central	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
		Southeast	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
FATB	Foamed Asphalt (PG 52-28)	Northern	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
		Central	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
		Southeast	1.5%, 2.5%, 3.5%	-10°C, 0°C, 20°C
RAP (50:50)	-	Northern	-	-10°C, -2°C, 20°C
		Central	-	-10°C, -2°C, 20°C
		Southeast	-	-10°C, -2°C, 20°C

HATB

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$



HATB

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$

$$k_1 = e^{1.1548+0.04736F-0.0596T-0.1723P_b}, \; k_2 = 0.2669 \text{ and } k_3 = -0.4109$$

$$M_R = e^{1.1548+0.04736F-0.0596T-0.1723P_b} P_a \left(\frac{\theta}{P_a}\right)^{0.2669} \left(\frac{\tau_{oct}}{P_a}+1\right)^{-0.4109} \qquad R^2 = 87\%$$

F = fractured surface, %

T = temperature, °C, and

P_b = percent binder by total weight, %.

Emulsion Treated Base

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$

$$M_R = e^{4.1203-0.0025F-0.0567T-0.0048P_b} P_a \left(\frac{\theta}{P_a}\right)^{0.0842} \left(\frac{\tau_{oct}}{P_a}+1\right)^{-0.2236} \qquad R^2 = 82\%$$

F = fractured surface, %

T = temperature, °C, and

P_b = percent binder by total weight, %.

Foamed-Asphalt Treated Base

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$

$$M_R = e^{-2.9060+0.0623F-0.0363T-0.2346P_b} P_a \left(\frac{\theta}{P_a}\right)^{0.0029} \left(\frac{\tau_{oct}}{P_a}+1\right)^{4.2850-0.0434F+0.0206T+0.1365P_b} \qquad R^2=80\%$$

F = fractured surface, %

T = temperature, °C, and

P_b = percent binder by total weight, %.

RAP - Base Course Blend (50:50)

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3}$$

$$M_R = e^{-0.0482+0.1294F-0.1660T} P_a \left(\frac{\theta}{P_a}\right)^{4.5056-0.00406F+0.0586T} \left(\frac{\tau_{oct}}{P_a}+1\right)^{0.15781-0.0099F-0.051T} \qquad R^2 = 87\%$$

F = fractured surface, %

T = temperature, °C

Catalog of Alaskan Paving Materials

1. Hot-mix asphalt (HMA) materials
2. Granular base course (GBC) materials
3. Asphalt Treated Base course materials
   a) Hot asphalt treated (HATB)
   b) Emulsion asphalt treated (EATB)
   c) Foamed asphalt treated (FATB)
   d) 50:50 blend of RAP & base (CABC)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Arizona – Materials

PAVEMENT DESIGN SECTION
MATERIALS GROUP
ADOT

Ashek Rana
Senior Pavement Design & Development Engineer
January, 2015

ADOT Pavement Design Practices

* AASHTO Pavement Design Guide
  - New Flexible Pavements
  - New Rigid Pavements

* Structural Overlay Design for Arizona (SODA)
  - Rehabilitation of existing Flexible Pavements

Mechanistic-Empirical Pavement Design

- In 1999, under SPR-402, a five research year project, ASU conducted characterization of Asphalt Binder, HMA Mix, Granular Base and Subgrade Soils
- Under SPR-672, Applied Research Associates (ARA), Inc., conducted a study on Arizona Traffic with characterization of traffic loadings, vehicle distribution, lane distribution, and other inputs
- Under SPR-606, verified the adequacy of the MEPDG global models and procedure for Arizona local condition and practices, if found to be inadequate, calibrated the global models, and also, validated the calibrated MEPDG models (Calibration & Implementation of the AASHTO ME Pavement Design Guide in Arizona)
- Developed "Arizona DOT User Guide for AASHTO Darwin-ME"
- Provided Basic Training to ADOT Staff

Calibration Sites

Total of 180 sections
- 120 LTPP
- 36 ADOT Pavement Management Sections
- 20 ADOT SPR 264 sections (concrete pavements)
- 4 ADOT WRI sections

All sites had detailed design, construction, materials testing and distress survey data.

New HMA & HMA/HMA Pavements

New JPCP & CRCP Sites

Composite (HMA overlaid JPCP & CRCP) Pavement

ADOT

Table 59. DARWin-ME local calibration coefficients for new HMA and HMA overlaid HMA pavement.

Model or Submodel Type	Model Coefficients	ADOT Local Calibration	Change from "Global" Models
Fatigue Damage Model (AC Fatigue)	K1	0.007566	No
	K2	3.9492	No
	K3	1.281	No
	BF1	249.0087232	Yes
	BF2	1	No
	BF3	1.233411397	Yes
Alligator Cracking Model (AC Cracking Bottom)	C1	1	No
	C2	4.5	Yes
	C4	6000	No
	AC Cracking Bottom Standard deviation	[illegible]	Yes
Reflection Cracking Model	a	[illegible]	No
	b	[illegible]	No
	c	2.55	Yes
	d	1.23	Yes
AC Rutting	K1	-3.35412	No
	K2	1.5606	No
	K3	0.4791	No
	BR1	0.69	Yes
	BR2	1	No
	BR3	1	No
	AC Rutting Standard Deviation	[illegible]	Yes
Base Rutting (Granular Subgrade Rutting)	K1 (Base)	2.03	No
	BS1 (Base)	0.14	Yes
	Base Rutting Standard Deviation	[illegible]	Yes
Subgrade Rutting (Fine Subgrade Rutting)	K1 (subgrade)	1.35	No
	BS1 (Subgrade)	0.37	Yes
	Subgrade Rutting Standard Deviation	[illegible]	Yes
HMA Transverse Cracking Model (Thermal Fracture)	Thermal Fracture Level 1K	1.5	No
	Thermal Fracture Level Standard Deviation	0.1468 * THERMAL + 65.027	No
	Thermal Fracture Level 2K	0.5	No
	Thermal Fracture Level Standard Deviation	0.2841 *THERMAL + 55.462	No
	Thermal Fracture Level 3K	1.5	No
	Thermal Fracture Level Standard Deviation	0.3972 * THERMAL + 20.422	No
HMA IRI Model	C1 (for Rutting)	[illegible]	Yes
	C2 (for Fatigue)	[illegible]	Yes
	C3 (for Transverse)	[illegible]	No
	C4 (for Site Factor)	[illegible]	Yes

ADOT

Table 60. DARWin-ME local calibration coefficients for new JPCP.

Model or Submodel Type	Model Coefficients	ADOT Local Calibration	Change from NCHRP 1-40D "Global" Models
PCC Fatigue Model	C1	2	No
	C2	1.22	No
JPCP Transverse Cracking Model	C4	0.19	Yes
	C5	-2.067	Yes
	PCC Cracking Standard Deviation	[illegible]	Yes
Faulting Model	C1	0.0355	Yes
	C2	0.1147	Yes
	C3	[illegible]	Yes
	C4	1.1E-07	Yes
	C5	20000	Yes
	C6	2.0389	Yes
	C7	0.1890	Yes
	C8	400	No
	PCC Faulting Standard Deviation	[illegible]	Yes
JPCP IRI Model	J1 (for Cracking)	0.60	Yes
	J2 (for Spalling)	3.48	Yes
	J3 (for Faulting)	1.22	Yes
	J4 (for Site Factor)	45.29	Yes
	PCC IRI JPCP Standard Deviation	5.4	No

Table 61. DARWin-ME local calibration coefficients for new CRCP.

Model or Submodel Type	Model Coefficients	ADOT Local Calibration	Change from NCHRP 1-40D "Global" Models
PCC Fatigue Model	C1	2	No
	C2	1.22	No
CRCP Punchout Model	C3	85	Yes
	C4	1.4149	Yes
	C5	-0.8061	Yes
	PCC Punchout Standard Deviation	1.5+2.9622*Pow(PO,0.4356)	Yes
CRCP IRI Model	C1	3.15	No
	C2	28.35	No
	PCC IRI CRCP Standard Deviation	5.4	No

ADOT

Table 64. Comparison of MEPDG "global" and Arizona specific model goodness of fit statistics.

Pavement Type	Distress/IRI Models	Global Models		Arizona DOT Calibrated Models	
		Global R^2, percent*	Global Model Standard Error, SEE*	Arizona R^2, percent	Arizona Standard Error, SEE
New HMA & HMA Overlay	Alligator cracking	8	14 percent	58	13 percent
	Transverse "thermal" cracking	NA	NA	Not Calibrated	Not Calibrated
	Total rutting	5	0.31 in	21	0.12 in
	IRI	30	19 in/mile	80	8 in/mile
New JPCP	Transverse cracking	20	9 percent	78	6 percent
	Transverse joint faulting	45	0.03 in	52	0.03 in
	IRI	75	25 in/mile	81	10 in/mile
ARFC/JPCP	Transverse cracking	Same as JPCP	Same as JPCP	Same as JPCP	Same as JPCP
	Rutting	Same as HMA	Same as HMA	Same as AZ HMA	Same as AZ HMA
	IRI	Same as HMA	Same as HMA	Same as AZ HMA	Same as AZ HMA
New CRCP	Punchouts	68%	5 PO/mile	Same as Global	2 AZ CRCP matched global predictions
	Slab/Base Friction	Established Values	NA	Same as Global	NA

*Global calibration coefficients using the Arizona database

ADOT DARWin-ME User Guide

Arizona DOT
USER GUIDE FOR
AASHTO DARWin-ME
Pavement Design Guide

Submitted to:
Arizona Department of Transportation
206 S. 17th Ave.
Phoenix, AZ 85007

Submitted by:
ARA
100 Trade Centre Dr., Suite 200
Champaign, IL 61820

July 19 2012

ADOT

ADOT DARWin-ME User Guide

- Overview of Manual
- General Information
- Performance Criteria
- Reliability
- Traffic Inputs
- Climate
- Materials
- Sensitivity
- Concrete
- Rehabilitation
- AZ Calibration Factors
- Example Problems

ADOT

Implementation of Pavement-ME

- ARA Training
  - Flexible Pavement (May & December, 2012 & August, 2013)
  - Rigid Pavement (June 2012 & March 2014)
- ADOT currently running Pavement-ME designs on all approved projects
- Training continuing with ARA (AASHTO Service Units)

ADOT

THANKS

ADOT

Arizona – PCCP Analysis

PCCP Analysis

AASHTO93 vs MEPDG

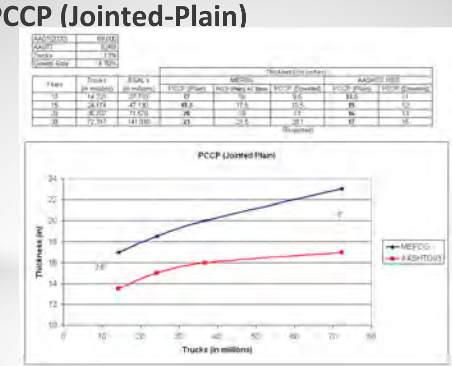
Presentation by

ADOT Pavement Design Group

January 21, 2015

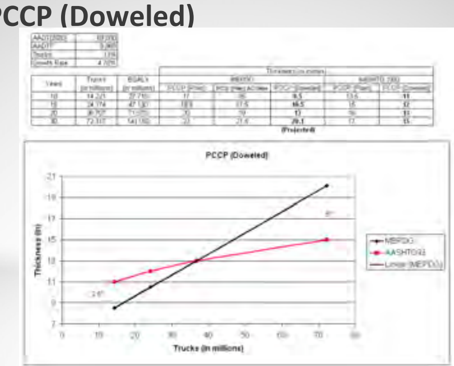
ADOT

PCCP (Jointed-Plain)



ADOT

PCCP (Doweled)

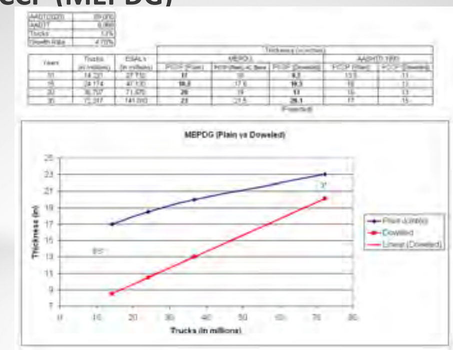


ADOT

PCCP (AASHTO93)

ADOT

PCCP (MEPDG)



Arizona – Traffic Inputs

MEPDG Traffic Inputs

Arizona Department of Transportation

Scott Weinland

Pavement Design Section

January, 2015

SPR-672 Development of a Traffic Data Input System in Arizona for the MEPDG

Stated Objectives:

- Identify MEPDG traffic data input needs.
- Evaluate current ADOT traffic data collection, storage, and analysis practices to determine whether the system can adequately meet MEPDG traffic data needs.
- Perform quality checks of existing traffic data to determine if they are reasonable and to identify anomalies.
- Develop a detailed action plan to satisfy future MEPDG traffic data needs.
- Document findings and recommendations.

Establishing Level 2 Traffic Inputs

Specific Steps:

- Step 1 - Traffic data identification and assembly.
- Step 2 - Traffic data processing, review, identification of anomalies and errors, and data cleansing.
- Step 3 - Statistical analysis to assign measured traffic data into subsets or natural groupings (called clusters) with similar characteristics and distribution patterns.
- Step 4 - Determination of optimum number of clusters in Arizona for each of the following traffic data types; MAF, VCD, Hourly Truck Distribution, ALD and Number of Axles per truck.
- Step 5 - Performance of a sensitivity analysis and interpretation of sensitivity results.
- Step 6 - Development of default statewide Level 2 traffic inputs for the MEPDG implementation in Arizona.

Establishing Level 2 Traffic Inputs

Step 1 - Traffic data identification and assembly

- 3 primary sources of traffic data for Arizona highways
  - Multimodal Planning Division (MPD) – Collects data primarily for transportation planning and traffic studies.
    - 15 WIM's (approximately ½ are operational)
    - 80 Classifiers (ATR's and AVC's)
    - 250 Permanent traffic counters
  - Enforcement and Compliance Division (ECD) – Formerly part of the Motor Vehicle Division (MVD) - traffic data primarily used for compliance. Data of limited value because most sites are within ports-of-entry and not capture all the traffic.
  - Long Term Pavement Performance (LTPP)
    - 32 sites with WIM's or Classifiers (AVC's)
- ARA found significant variations in traffic data collection practices, data accuracy, data storage practices and availability.

Establishing Level 2 Traffic Inputs

Step 2 - Traffic data processing, review, identification of anomalies and errors, and data cleansing

- Data Processing
  - Raw traffic data (MPD sources) was processed using ARA's Advanced Traffic Loading and Analysis System (ATLAS) software.
  - LTPP traffic data was received "post-processing" (in the form needed for input into the MEPDG).
- Data Review
  - Plots were generated to assess reasonableness of the data (% Trucks vs. Hour of Day, MAF vs. Month of Year, % Trucks vs. Vehicle Class, Number of Axles (Single, Tandem, etc.) vs. Vehicle Class, and % Single, Tandem, Tridem, Quad Axles vs. Axle Load).
  - Plots were then reviewed for consistency, accuracy , and completeness (Hourly Truck Distribution adds up 100 or MAF adds up to 12, Axle Load plots display distinct peaks as expected, and consistency from year to year.

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Establishing Level 2 Traffic Inputs

Step 2 - Traffic data processing, review, identification of anomalies and errors, and data cleansing (cont.)

- Identification of anomalies and errors
  - Data points and overall trends found to be inconsistent with expected trends were flagged (Note need to distinguish between unusual data, correct data, and incorrect data).
- Data Cleansing
  - Potential anomalous or erroneous data was removed from the database.

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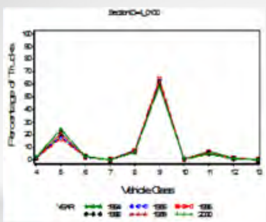
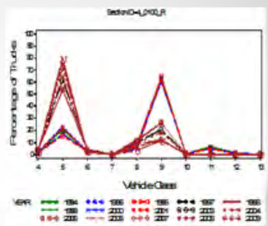
Establishing Level 2 Traffic Inputs

Identification of anomalies and errors using plots (Example 1)



Northbound US 93 MP 52.6 (pre - 09/2001)

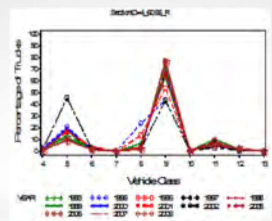


Northbound US 93 MP 52.6 (pre and post - 09/2001)

ADOT

Establishing Level 2 Traffic Inputs

Identification of anomalies and errors using plots (Example 2)



Southbound SR 85 MP 141.84

ADOT

Establishing Level 2 Traffic Inputs

Step 3 – Statistical analysis to assign measured traffic data into subsets or natural groupings (called clusters) with similar characteristics and distribution patterns

- Use of statistical analysis (Multivariate Hierarchical Distribution Patterns) to group data into clusters with "like" observations. Once clusters are established the can be applied to sites with similar characteristics (i.e. functional classification, location, predominant truck type, etc.)

Step 4 – Determination of optimum number of clusters in Arizona for each of the following traffic data types; MAF, Hourly Truck Distribution, VCD, ALD and Number of Axles per truck

- Determined using various statistical methods

ADOT

Establishing Level 2 Traffic Inputs

Step 5 – Performance of a sensitivity analysis and interpretation of sensitivity results

- Perform comprehensive sensitivity analysis using typical pavement sections (flexible and rigid) to determine if there are significant differences in pavement design due to the various clusters identified in Step 4
- Determine if clusters can be combined or eliminated if differences are insignificant.

Step 6 – Development of default statewide Level 2 traffic inputs for the MEPDG implementation in Arizona

- Establish MEPDG default inputs based on the average value for all sites that fall into a given cluster.

ADOT

Establishing Level 2 Traffic Inputs

Monthly Adjustment Factor (MAF)

- Analysis resulted in 1 cluster (average of all available sites).

Month	Statewide Default Monthly Adjustment Factors									
	VC4	VC5	VC6	VC7	VC8	VC9	VC10	VC11	VC12	VC13
January	0.99	0.87	0.85	1.11	0.90	0.86	1.03	0.69	0.62	1.23
February	1.03	0.97	0.90	0.87	0.94	0.92	0.95	0.78	0.85	0.96
March	1.02	0.99	0.92	0.94	1.02	0.94	0.88	0.85	0.98	0.84
April	0.97	0.91	0.94	1.13	0.92	0.93	0.91	0.81	1.00	0.91
May	0.96	0.95	0.91	0.78	0.92	0.93	0.83	0.97	0.91	0.79
June	0.89	0.96	0.93	0.96	0.93	0.98	1.00	1.13	1.13	0.79
July	0.91	0.98	0.92	0.64	0.91	0.92	0.84	1.13	0.95	1.00
August	0.95	0.99	1.01	0.86	0.93	1.08	0.95	1.25	1.20	0.74
September	1.05	0.95	0.90	0.84	0.90	0.90	0.82	0.96	0.91	0.67
October	1.06	1.01	1.05	1.00	1.08	1.00	0.96	1.00	0.99	1.05
November	1.10	1.24	1.35	1.25	1.40	1.25	1.42	1.14	1.22	1.41
December	1.05	1.19	1.33	1.63	1.14	1.27	1.42	1.30	1.24	1.60

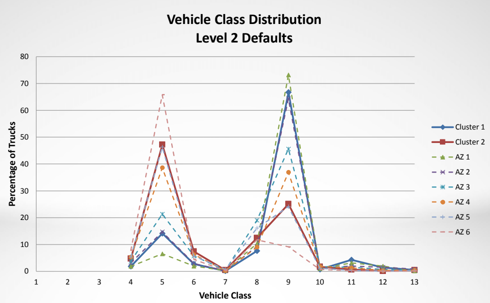
Establishing Level 2 Traffic Inputs

Vehicle Class Distribution

- Analysis resulted in 2 clusters.
  - Cluster 1 – Major Single Truck Trailer Route, Primarily Rural Principal Arterial
  - Cluster 2 – Intermediate Light and Single Trailer Route, Primarily Urban Principal Arterial, also Rural Minor Arterials
- ADOT will likely increase the number of clusters and create a cluster selection criteria based on relative percent of Single vs. Combo truck units (AZ-1 through AZ-6)
  - Due to uncertainties in assigning clusters based on functional classification
  - Pavement structural sections are highly sensitive to VCD

Establishing Level 2 Traffic Inputs

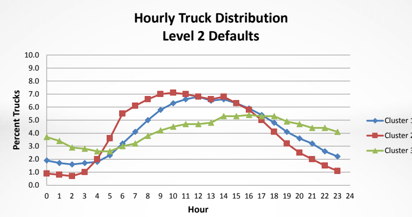
Vehicle Class Distribution (cont.)



Establishing Level 2 Traffic Inputs

Hourly Truck Traffic Distribution

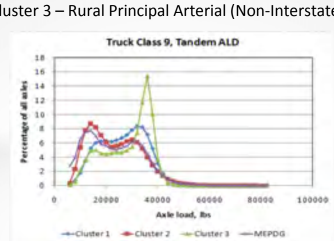
- Analysis resulted in 3 clusters.
  - Cluster 1 - Rural Highways
  - Cluster 2 – Urban Highways
  - Cluster 3 – Long Haul Sections of Rural Highways Across Desert



Establishing Level 2 Traffic Inputs

Axle Load Distribution (ALD)

- Analysis resulted in 3 clusters.
  - Cluster 1 - Rural Principal Arterial (Interstate)
  - Cluster 2 – Urban Freeways and Rural Minor Arterials/Collectors
  - Cluster 3 – Rural Principal Arterial (Non-Interstate)



Establishing Level 2 Traffic Inputs

Number of Axles per Truck

- Analysis resulted in 1 cluster (average of all available sites).

Vehicle Class	Axle Type			
	Single	Tandem	Tridem	Quad
4	1.34	0.75	0.00	0.00
5	2.14	0.00	0.00	0.00
6	0.95	0.95	0.00	0.00
7	0.33	0.02	0.26	0.07
8	2.61	0.49	0.00	0.00
9	1.20	1.84	0.00	0.00
10	0.98	1.01	0.86	0.06
11	4.78	0.00	0.00	0.00
12	3.88	0.98	0.03	0.14
13	1.29	1.90	0.19	0.14

Summary of ADOT Traffic Inputs

Traffic Input	Input Level	Source of Data
Initial AADTT	1	Site Specific (MPD or LTPP Traffic Tables)(AADT, T Factor%, Directional Distribution, Lane Distribution)
AADTT Growth Rate	1	Site Specific (MPD or LTPP Traffic Tables)
Monthly Adjustment Factor	2	Arizona Default – 1 cluster for entire state
Vehicle Class Distribution	2	Arizona Default – 2 clusters based on functional classification (pending change to 6 clusters based on % Singles & % Combos)
Hourly Truck Distribution	2	Arizona Default – 3 clusters based on location (rural, urban, or rural long haul)
Axle Load Distribution	2	Arizona Default – 3 clusters based on functional classification
Axles per Truck	2	Arizona Default – 1 cluster for entire state
Lateral Wander	2	Arizona Default – Average of 4 field measurements by ARA
Wheelbase	2	Arizona Default – WIM Data from 2 LTPP sites was analyzed to determine % trucks with Short, Medium and Long wheelbase.
Others	3	National Defaults (tire pressure, axle width, dual tire spacing, etc.)

ADOT

ADOT DARWin-ME Users Guide

Arizona DOT
USER GUIDE FOR
AASHTO DARWin-ME
Pavement Design Guide

Arizona Department of Transportation

ARA

ADOT

MEPDG Traffic Inputs
Moving Forward…

- Additional WIM Sites
  - ADOT's FY2015 Five-year construction program includes 15 new WIM sites.
  - Initial Feasibility Report issued for 30 additional WIM sites to be constructed during FY2016 .
- Challenges
  - Obtaining quality traffic data
  - Collecting and processing WIM and other traffic data for Level 1 analysis (SPR-672 included recommendations but implementation has many challenges).

ADOT

Thank You!

ADOT

Caltrans ME Design Implementation

Caltrans ME Design Implementation

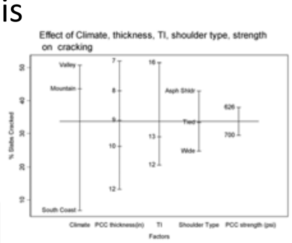
Mehdi Parvini, PhD, PE, PMP
Office of Rigid Pavement
California DOT

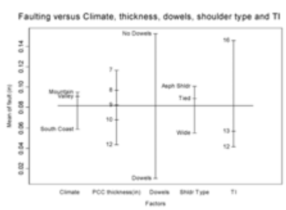
April 2015

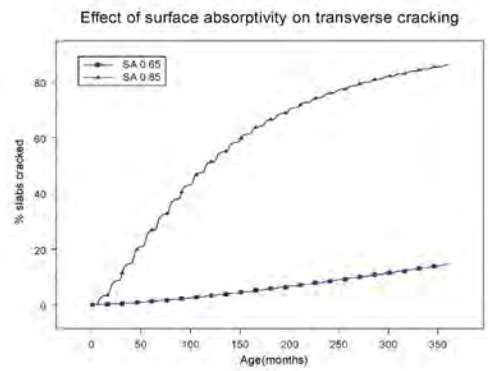
ME Implementation Status

- Adopted ME pavement design method in 2005 to replace the empirical design
- Use MEPDG for rigid pavement only
- The MEPDG was calibrated locally using data from in-service concrete pavements
- Led to the development of rigid pavement design catalog

Sensitivity Analysis

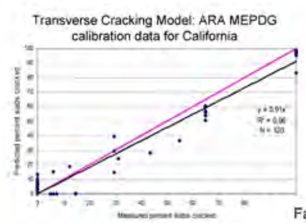


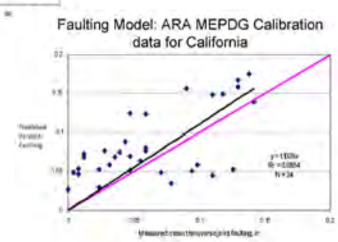




MEPDG JPCP Module Calibration

- JPCP: 53 sections
- ACOL rigid: 44 sections (for rehab models)
- Historic traffic and performance data
  - Traffic: 1978 to current year
  - Performance history from PMS data: 1978-2004
  - MEPDG calibration data set from ARA in Oct 06
- Compare California sections with National





Criteria for Design Catalog

- Failure Criteria (JPCP)
  - Transverse cracking: 10% slabs
  - Faulting: 0.1 inch
  - IRI: 160 in/mile
- Reliability: 90 %
- Design life
  40 years (2% slab replacement, grinding or both)
- 6 Climate Regions

Catalog Factorial

- Climate: Coastal, Desert & Low Mountain
- TI (MESALs): 9(1), 11(5.4), 13 (20) and 17(210)
- Spectra: Urban and Rural
- Base Type: ACB, CTB, Granular Base
- Subgrade: CH and SP
- Load Transfer: Dowels and no dowels
- Shoulder: Asphalt, Tied & Widened Lanes
- Granular subbase: Yes for CH and No for SP
- PCC Thickness: 7 to 14 inches based on traffic levels
- Total Number of Cases: 2160

Key Assumptions

- CTE assumed to be 6x10-6 /oF
- Surface absorptivity: Default value of 0.85
- No bonding between base & surface layer
- Joint Spacing: 13.5 ft
- Default values for unbound layers
- Erodibility Index of base
  3 for Granular bases, 2 for ACB, 1 for CTB

620-4 HIGHWAY DESIGN MANUAL
July 24, 2009

Table 622.1
Rigid Pavement Engineering Properties

Property	Values
Transverse joint spacing	13.5 ft average
Initial IRI immediately after construction	63 in/mile max
Reliability	90%
Unit weight	150 lb/ft³
Poisson's ratio	0.20
Coefficient of thermal expansion	6.0×10^{-6} / °F
Thermal conductivity	$1.25 \frac{Btu}{hr \cdot ft \cdot °F}$
Heat capacity	$0.28 \frac{Btu}{lbm \cdot °F}$
Permanent curl/warp effective temperature difference	Top of slab is 10 °F cooler than bottom of slab
Surface layer/base interface	Unbonded
Surface shortwave absorptivity	0.85
Cement type	Type II Portland Cement
Cement material content (cement + flyash)	24 lb/ft³
Water: cementitious material ratio	0.42
PCC zero-stress temperature	100.9 °F
Ultimate shrinkage at 40% relative humidity	537 microstrain
Reversible shrinkage (% of ultimate shrinkage)	50%
Time to develop ultimate shrinkage	35 days
Modulus of rupture or flexural strength (28 days)	625 psi
Dowel bar diameter	1.5 in (1.25 in for rigid pavement thickness < 0.70 ft)

HIGHWAY DESIGN MANUAL 620-9

Table 623.1B
Rigid Pavement Catalog (North Coast, Type I Subgrade Soil)(1) (2) (3) (4) (5)

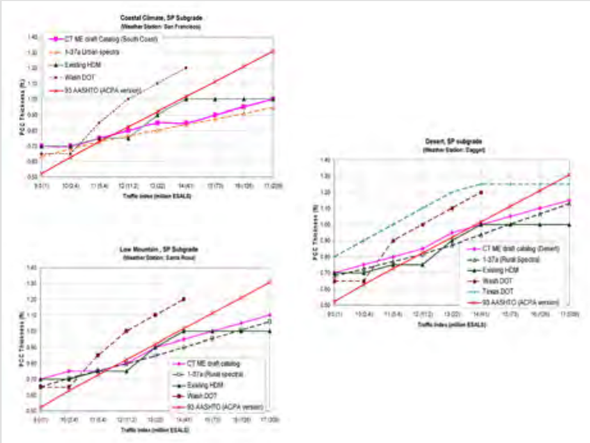
TI	Rigid Pavement Structural Depth: With Lateral Support (ft)				Without Lateral Support (ft)			
≤ 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.70 AB		0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 JPCP 0.70 AB	
11.5 to 12	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 CRCP 0.35 HMA-A		0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.80 CRCP 0.40 HMA-A	
12.5 to 13	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.85 JPCP 0.50 LCB	0.85 JPCP 0.50 HMA-A	0.80 CRCP 0.50 HMA-A	
13.5 to 14	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	
14.5 to 15	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
15.5 to 16	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
16.5 to 17	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
> 17 ≤ 9	1.00 JPCP 0.35 LCB 0.70 JPCP	1.00 JPCP 0.25 HMA-A 0.70 JPCP	0.90 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP	1.10 JPCP 0.35 LCB 0.70 JPCP	1.10 JPCP 0.25 HMA-A 0.70 JPCP	1.00 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP

NOTES:
(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.
(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.
(3) Portland cement concrete may be substituted for LCB when justified for constructability or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.
(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.
(5) Place a Bond Breaker between JPCP and LCB in all cases.

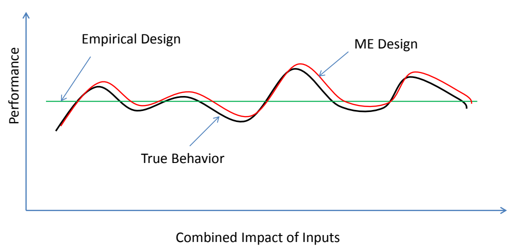
Legend:
JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Comparison Study

- In order to check the reasonableness of the catalog, it was compared to
  - Current Caltrans catalog
  - Washington and Texas designs for similar climate regions
  - Pavement Analysis software: ACPA
  - AASHTO 93 guide
  - Historical performance of the pavements



ME Design Benefit



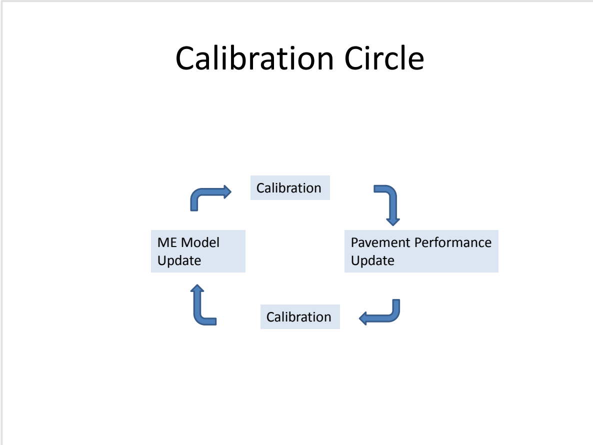
Challenges

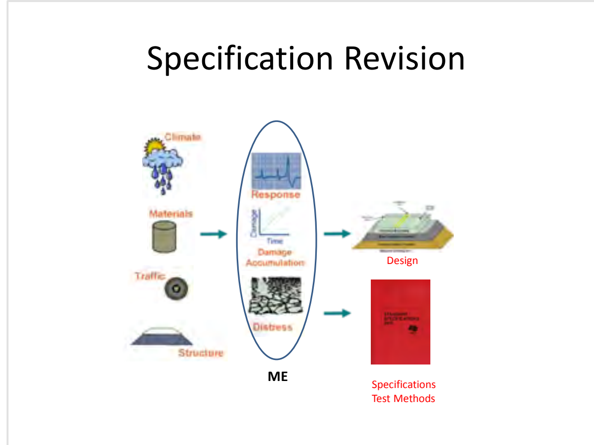
- Deployment
- Calibration
- ME and Spec. Revisions
- ME and LCCA Integration
- ME and PMS Relation
- Software

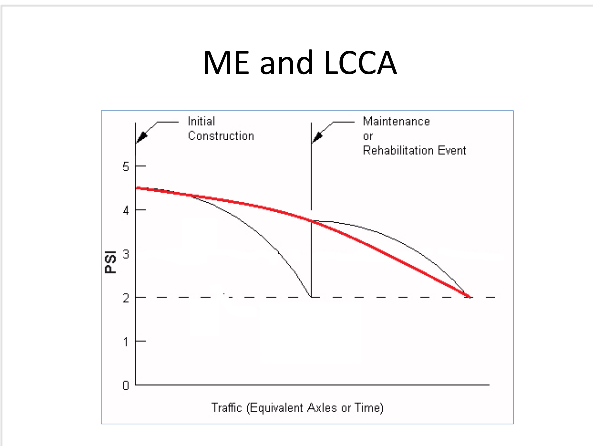
Deployment

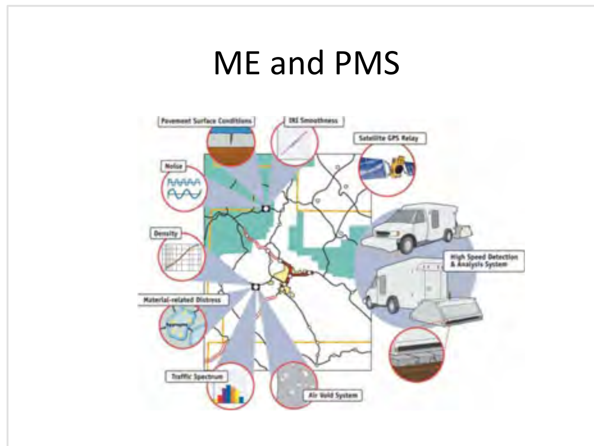
- Thresholds
- Users
- Training
- Locals
- License

Calibration, State or Region?

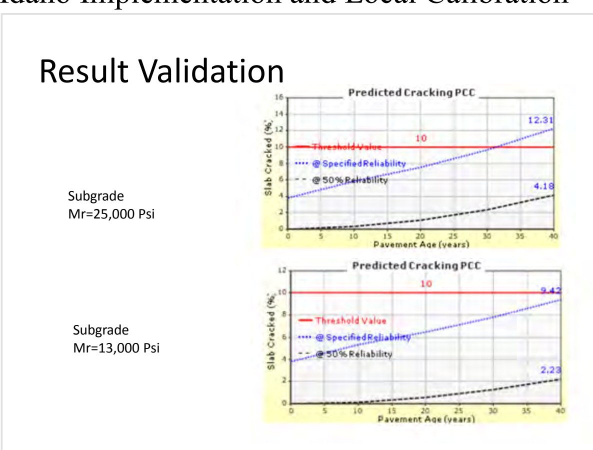


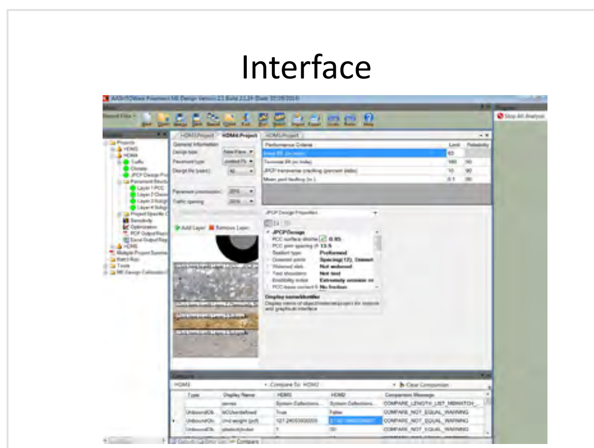






Idaho Implementation and Local Calibration – Materials





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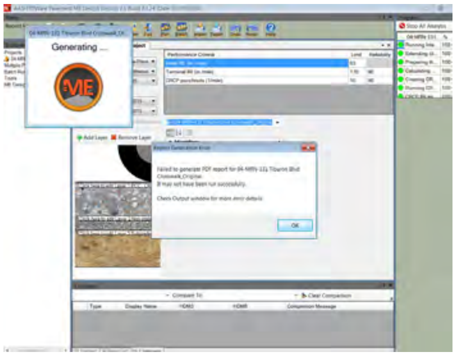
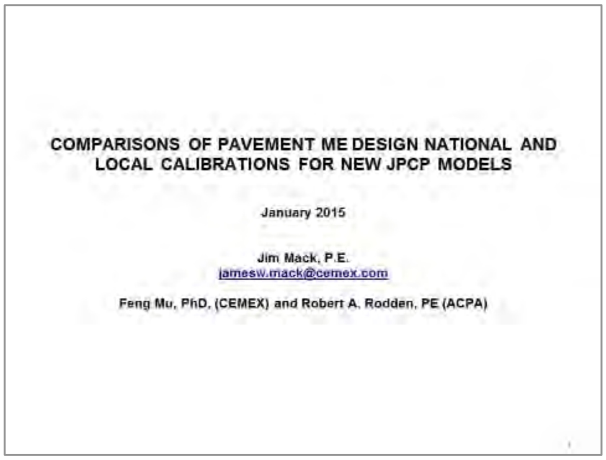
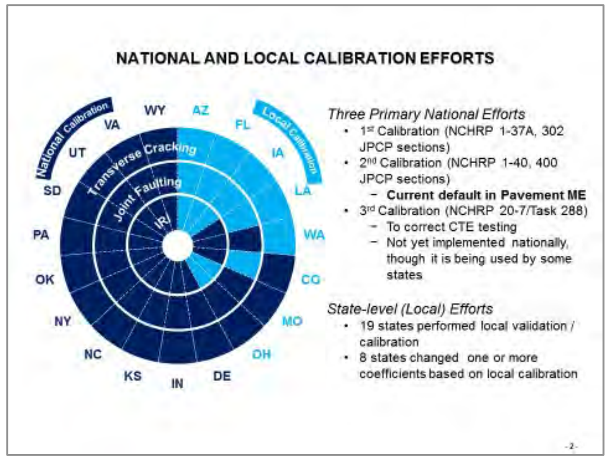


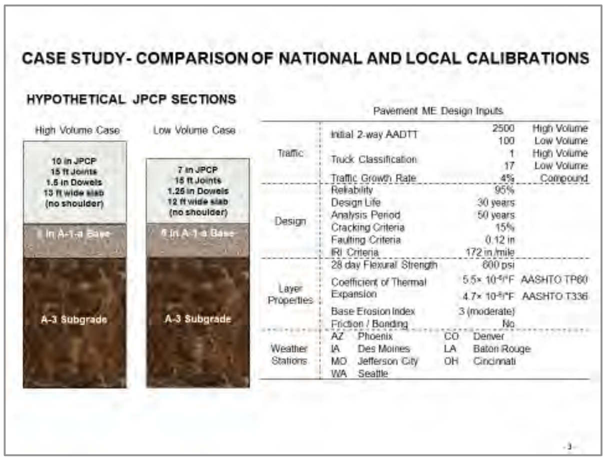
Thanks.

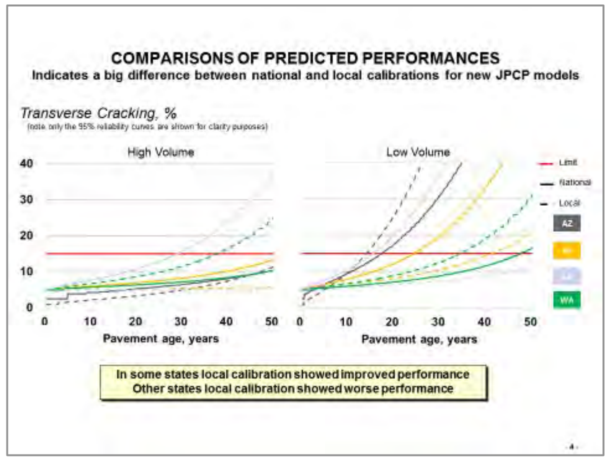
Questions?

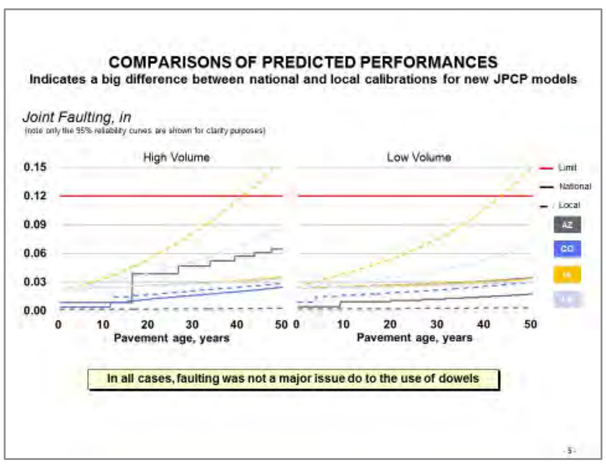
Cemex and ACPA – Comparison of Pavement ME Design National and Local Calibrations for New JPCP Models

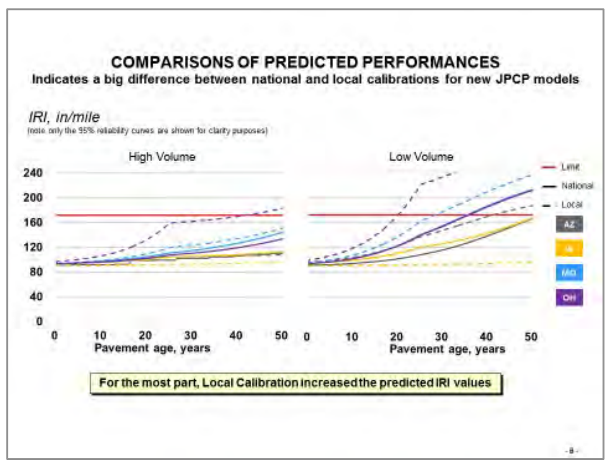


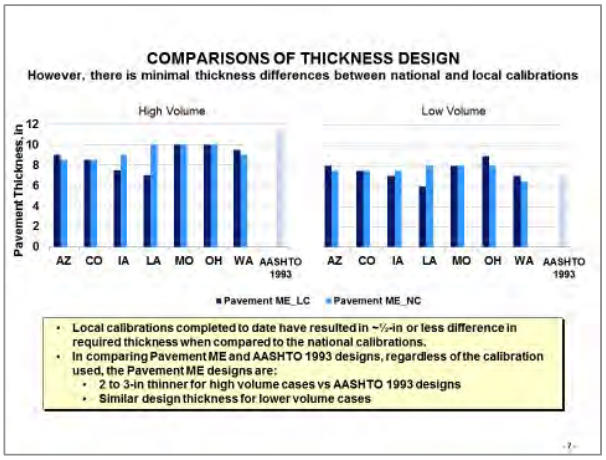


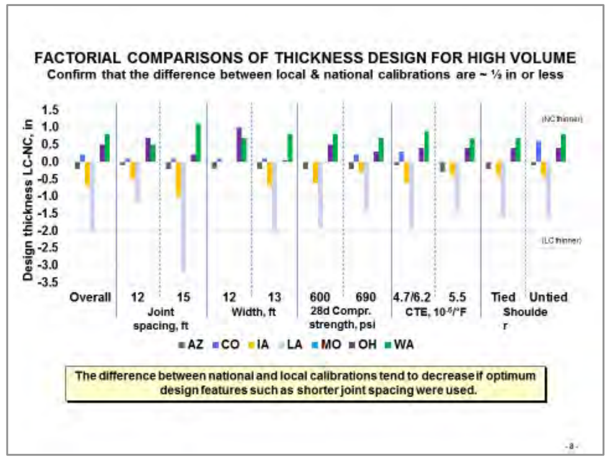


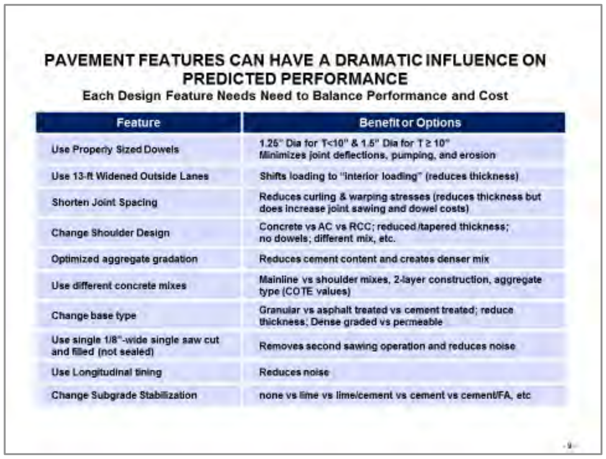




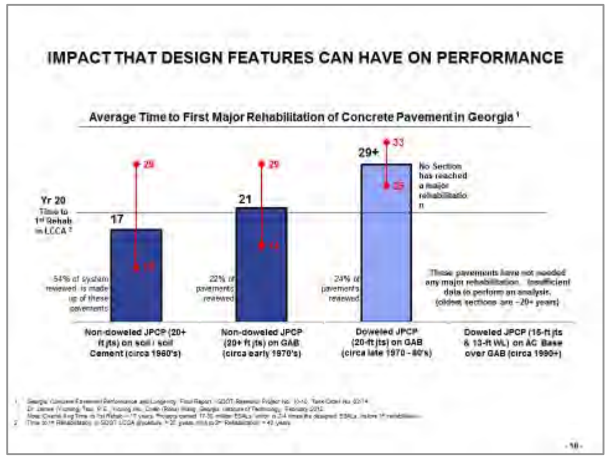


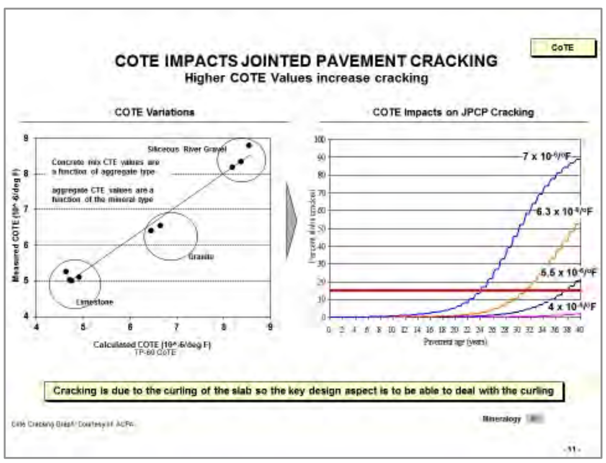


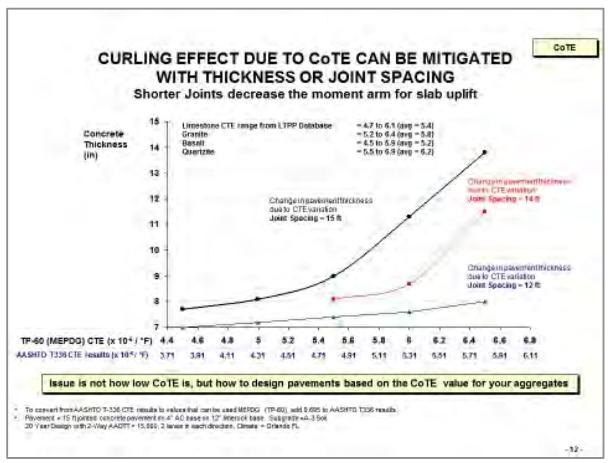


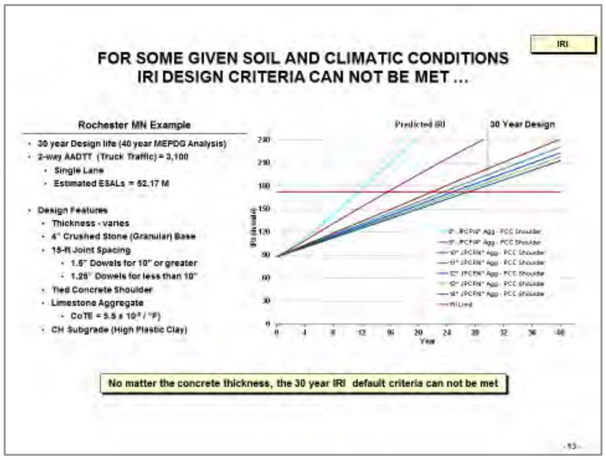


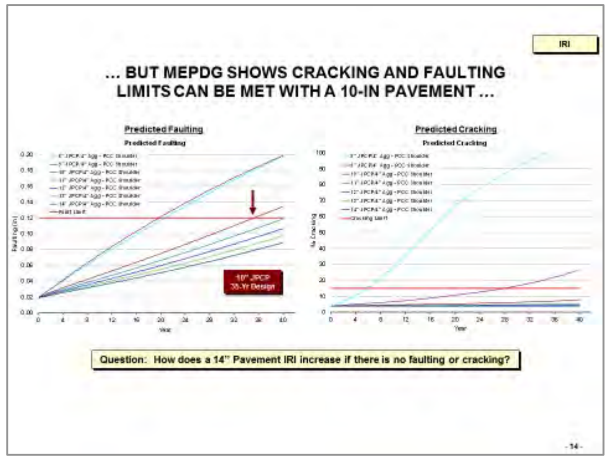
Feature	Benefit or Options
Use Properly Sized Dowels	1.25" Dia for T<10" & 1.5" Dia for T ≥ 10" Minimizes joint deflections, pumping, and erosion
Use 13-ft Widened Outside Lanes	Shifts loading to "interior loading" (reduces thickness)
Shorten Joint Spacing	Reduces curling & warping stresses (reduces thickness but does increase joint sawing and dowel costs)
Change Shoulder Design	Concrete vs AC vs RCC; reduced/tapered thickness; no dowels; different mix, etc.
Optimized aggregate gradation	Reduces cement content and creates denser mix
Use different concrete mixes	Mainline vs shoulder mixes, 2-layer construction, aggregate type (COTE values)
Change base type	Granular vs asphalt treated vs cement treated; reduce thickness; Dense graded vs permeable
Use single 1/8"-wide single saw cut and filled (not sealed)	Removes second sawing operation and reduces noise
Use Longitudinal tining	Reduces noise
Change Subgrade Stabilization	none vs lime vs lime/cement vs cement vs cement/FA, etc

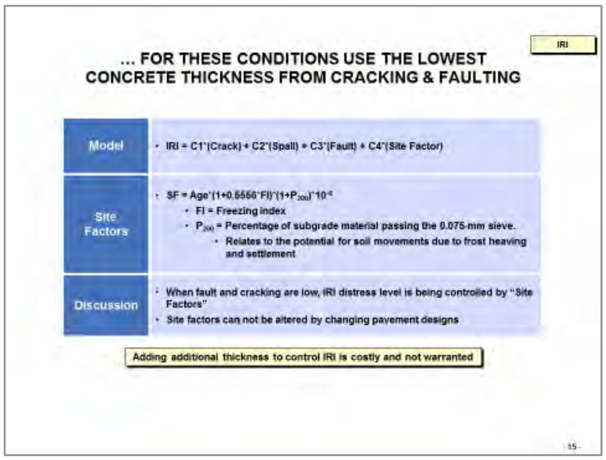


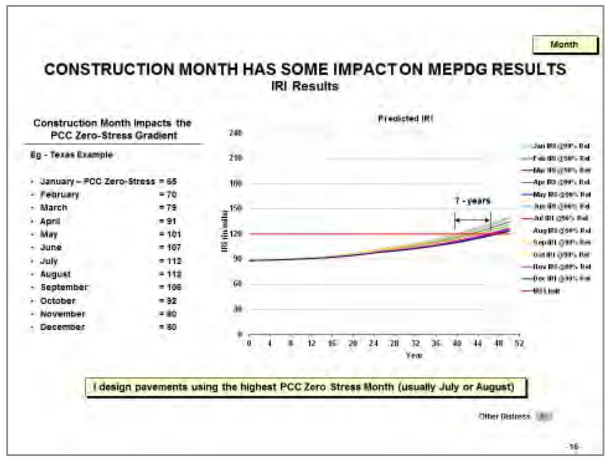


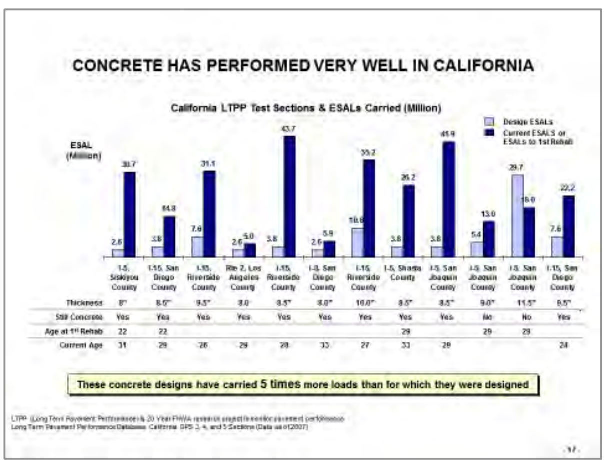


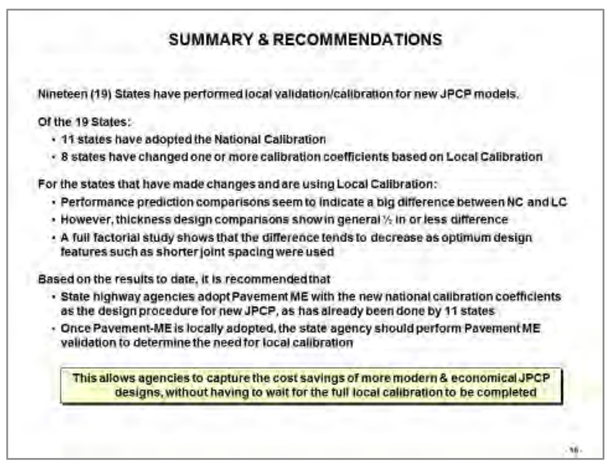

IRI
FOR SOME GIVEN SOIL AND CLIMATIC CONDITIONS IRI DESIGN CRITERIA CAN NOT BE MET …
Rochester MN Example
30 year Design life (40 year MEPDG Analysis)
2-way AADTT (Truck Traffic) = 3,100
Single Lane
Estimated ESALs = 52.17 M
Design Features
Thickness - varies
4" Crushed Stone (Granular) Base
15-ft Joint Spacing
1.5" Dowels for 10" or greater
1.25" Dowels for less than 10"
Tied Concrete Shoulder
Limestone Aggregate
CoTE = 5.5 x 10^-6 / °F)
CH Subgrade (High Plastic Clay)
Predicted IRI
30 Year Design
Year
IRI Limit
No matter the concrete thickness, the 30 year IRI default criteria can not be met
- 13 -


IRI
… BUT MEPDG SHOWS CRACKING AND FAULTING LIMITS CAN BE MET WITH A 10-IN PAVEMENT …
Predicted Faulting
Predicted Cracking
10" JPCP
30-Yr Design
Year
Question: How does a 14" Pavement IRI increase if there is no faulting or cracking?
- 14 -


IRI
… FOR THESE CONDITIONS USE THE LOWEST CONCRETE THICKNESS FROM CRACKING & FAULTING
Model
IRI = C1*(Crack) + C2*(Spall) + C3*(Fault) + C4*(Site Factor)
Site Factors
SF = Age*(1+0.5556*FI)*(1+P200)*10^-6
FI = Freezing index
P200 = Percentage of subgrade material passing the 0.075-mm sieve.
Relates to the potential for soil movements due to frost heaving and settlement
Discussion
When fault and cracking are low, IRI distress level is being controlled by "Site Factors"
Site factors can not be altered by changing pavement designs
Adding additional thickness to control IRI is costly and not warranted
- 15 -


Month
CONSTRUCTION MONTH HAS SOME IMPACT ON MEPDG RESULTS
IRI Results
Construction Month Impacts the PCC Zero-Stress Gradient
Eg - Texas Example
January – PCC Zero-Stress = 65
February = 70
March = 79
April = 91
May = 101
June = 107
July = 112
August = 112
September = 106
October = 92
November = 80
December = 80
Predicted IRI
7 - years
Year
IRI Limit
I design pavements using the highest PCC Zero Stress Month (usually July or August)
Other Distress
- 16 -


CONCRETE HAS PERFORMED VERY WELL IN CALIFORNIA
California LTPP Test Sections & ESALs Carried (Million)
Design ESALs
Current ESALS or ESALs to 1st Rehab
ESAL (Million)
I-5, Siskiyou County
I-15, San Diego County
I-15, Riverside County
Rte 2, Los Angeles County
I-15, Riverside County
I-8, San Diego County
I-15, Riverside County
I-5, Shasta County
I-5, San Joaquin County
I-5, San Joaquin County
I-5, San Joaquin County
I-15, San Diego County
Thickness
Still Concrete
Age at 1st Rehab
Current Age
These concrete designs have carried 5 times more loads than for which they were designed
- 17 -


SUMMARY & RECOMMENDATIONS
Nineteen (19) States have performed local validation/calibration for new JPCP models.
Of the 19 States:
11 states have adopted the National Calibration
8 states have changed one or more calibration coefficients based on Local Calibration
For the states that have made changes and are using Local Calibration:
Performance prediction comparisons seem to indicate a big difference between NC and LC
However, thickness design comparisons show in general ½ in or less difference
A full factorial study shows that the difference tends to decrease as optimum design features such as shorter joint spacing were used
Based on the results to date, it is recommended that
State highway agencies adopt Pavement ME with the new national calibration coefficients as the design procedure for new JPCP, as has already been done by 11 states
Once Pavement-ME is locally adopted, the state agency should perform Pavement ME validation to determine the need for local calibration
This allows agencies to capture the cost savings of more modern & economical JPCP designs, without having to wait for the full local calibration to be completed
- 18 -

ONE FINAL REQUEST

If your state has done Local Calibration, could you please send ACPA (rrodden@pavement.com) the reports with the Local Calibration values. We would like to continue reviewing the results based in local calibration efforts for concrete pavements. Similarly, if our information shown is not correct, please also let us know.

Thank You

- 19 -

Colorado – Local Calibration of Subgrade Soils

Local Calibration of Subgrade Soils

Objectives

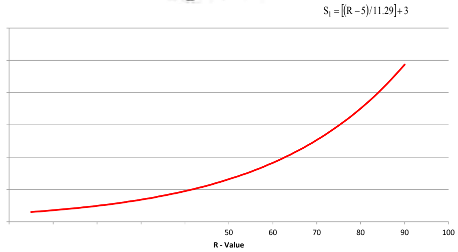
- CDOT's Studies
- Subgrade Characterization for M-E Design
- Modeling the Subgrade in M-E Design
- Where Does CDOT Go From Here

1989 Study of Colorado Soils

Colorado Procedure - 3102

$$M_R = 10^{[(S_1+18.72)/6.24]}$$

$$S_1 = [(R-5)/11.29]+3$$



2002 CDOT R-Value vs. M_r Study

A-2 Soils

R^2	R-Val	LL*	PI*	P-4*	P-10	P-40	P-200	Moist($M_{R(0)}$) *	Dens($M_{R(0)}$)
0.830	*	*	*	*	*	*	*	*	*
0.830	*	*	*	*	*	*	*	*	
0.154	*	*	*	*	*	*	*		
0.134	*	*	*	*	*	*			
0.134	*	*	*	*	*				
0.113	*	*	*	*					
0.093	*	*	*						
0.063	*	*							
0.035	*								
0.730								*	
0.325									*
0.730	*							*	
0.815	*			*				*	
0.816		*	*	*				*	

AASHTO T 190 with AASHTO T 307 at various moisture contents

FWD to Laboratory Ratios

Level 1 Design

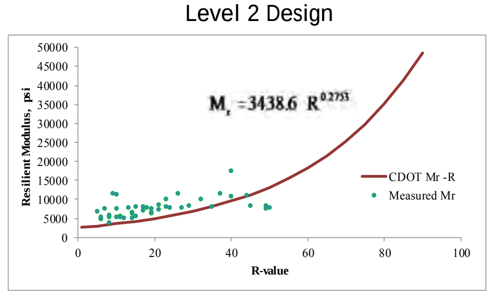
Layer Type	Location	$_R/M_r$
Unbound Granular Base and Subbase Layers	base/subbase between two stabilized or asphalt stabilized materials).	1.43
		1.32
		0.62
Embankment and Subgrade Soils		0.75
		0.52
		0.35

$_R$ = *Elastic modulus backcalculated from deflection basin measurements.*
$_r$ = *Elastic modulus of the in-place materials determined from laboratory repeated load resilient modulus test.*

From the Mechanistic-Empirical Pavement Design Guide Manual of Practice

Local M-E Design Calibration

Level 2 Design



AASHTO T 307

National M_r Values

Level 3 Design

	Resilient Modulus (M_r) at Optimum Moisture, psi	
	Flexible Pavements	Rigid Pavements
A-1-a	19,700	14,900
A-1-b	16,500	14,900
A-2-4	15,200	13,800
A-2-5	15,200	13,800
A-2-6	15,200	13,800
A-2-7	15,200	13,800
A-3	15,000	13,000
A-4	14,400	18,200
A-5	14,000	11,000
A-6	17,400	12,900
A-7-5	13,000	10,000
A-7-6	12,800	12,000

Modeling the Subgrade in M-E

Input for New Flexible and JPCP Designs

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	Level 2	Level 3
		Not Available		
		Not Available	-08	
	1	Not Available		
		Not Available		-E Design
		Not Available		-E Design
		Not Available	180 or T 99	
		Not Available	180 or T 99	
		Not Available		
		Not Available		2
		Not Available		

Modeling the Subgrade in M-E

Inputs for HMA Overlay of Existing Flexible Pavements

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	Level 2	Level 3
		FWD Deflection Testing and Backcalculated Resilient Modulus	**Section 4.4.3.2 Inputs**	
		Colorado Procedure 21-08		
	1	AASHTO T 195		
		Use software defaults		-E Design
		Use software defaults		-E Design
		AASHTO T 180 or T 99		
		AASHTO T 180 or T 99		
		AASHTO T 100		
		AASHTO T 215		2
		Not Applicable		

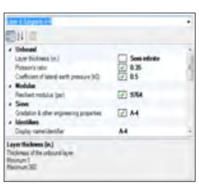
Modeling the Subgrade in M-E

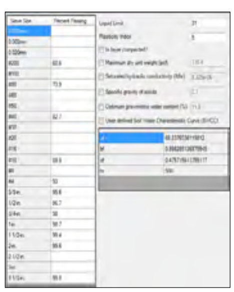
Inputs for Overlays of Existing Rigid Pavements

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	Level 2	Level 3
		FWD Deflection Testing and Backcalculated Dynamic k-value[3]	**Section 4.4.3.3 Inputs**	
		Colorado Procedure 21-08		
	1	AASHTO T 195		
		Use software defaults		-E Design
		Use software defaults		-E Design
		AASHTO T 180 or T 99		
		AASHTO T 180 or T 99		
		AASHTO T 100		
		AASHTO T 215		2
		Not Applicable		

Modeling the Subgrade in M-E

The top 8 feet of a pavement structure and subgrade can be divided into a maximum of 19 sublayers.





For a full-depth flexible or semi-rigid pavement placed directly on a thick embankment fill, the top 12 inches is modeled as an aggregate base layer, while the remaining embankment is modeled as the subgrade layer 1.

Modeling the Subgrade in M-E

Expansive Subgrade Soils

Plasticity Index	Depth of TreatmentBelow NormalSubgrade Elevation
10 – 20	2 feet
20 – 30	3 feet
30 – 40	4 feet
40 – 50	5 feet
More than 50	feet in height, or wasted

Frost Susceptible Soils

Stabilizing Agents

- **Lime Treated**
- **Cement Treated**
- **Fly Ash and Lime/Fly Ash Treated**

Geosynthetic Fabrics and Mats

Rigid Layer

Where Does CDOT Go From Here

Where Does CDOT Go From Here

2015
M-E Pavement Design
Manual

www.codot.gov/business/designsupport/materials-and-geotechnical/manuals/pdm/2015-pdm

Take Away

- AASHTO T 307 should be the preferred test method
- Old R-values should use the old equation
- Use the new resilient modulus equipment on select projects
- Use the level 3 M_r values for preliminary information only

Connecticut – M-EPDG Implementation Research Projects

M-EPDG Implementation Research Projects

Connecticut Department of Transportation (ConnDOT)

Connecticut Advanced Pavement Laboratory (CAP Lab) at UConn

Projected Timeline for the MEPDG Implementation

Implementation Step	Complete	Year 1 ?	Year 2 ?	Year 3 ?	Future Activity ?
1. Conduct sensitivity analysis of MEPDG inputs.	X				
2. Recommend MEPDG input levels and required resources to obtain those inputs.	X				
3. Assemble a ConnDOT MEPDG Implementation Team and develop communication plan.		?			
4. Conduct staff training.		?			
5. Develop formal ConnDOT specific MEPDG related documentation.			?	?	

Continued…

Projected Timeline for the MEPDG Implementation (cont'd)

Implementation Step	Complete	Year 1	Year 2	Year 3	Future Activity
6. Develop and populate a central database(s) with required MEPDG input values.		?	?	?	
7. Align distress data collection in CT with the MEPDG definitions.			?		
8. Calibrate and validate MEPDG performance prediction models to local conditions.			?	?	
9. Define long-term plan for adopting MEPDG as ConnDOT design method.		?			
10. Develop design catelog.					?

Connecticut Advanced Pavement Laboratory (CAP Lab)

- Research at the University of Connecticut CAP Lab via Memorandums of Understanding (MOU's) with ConnDOT:
  - Preparation of the Implementation Plan of the MEPDG in Connecticut – Phase I
    - Begin Date: July 2011
    - Final Report Publication Date: September 2014.
    - Completion Date: December 2014
  - Phase II – Expanded Sensitivity Analysis
    - Begin Date: January 2015
    - Scheduled Completion Date: June 2016

Phase I - Objectives

Prepare MEPDG Implementation Plans for ConnDOT.

- Identify design inputs
- Conduct Sensitivity Analysis for pertinent input ranges
- Develop training materials and guidelines for ConnDOT

Identification of Design Inputs

- Typical Pavement Designs
- Climatic Zones
- Main Traffic Variables
- Subgrade Properties
- Typical Pavement Structures

Typical Pavement Designs in CT

Three Typical Designs:

- Newly constructed AC pavements
- AC-overlaid AC pavements
- AC-overlaid rubbilized PCC pavements

Typical Maintenance/Preservation:

- 2 inch overlay placed over existing (with or without milling

Another Common Design:

- AC-overlaid PCC or repaired PCC

CT Climatic Zones

Summary of the MEPDG Climatic Data

Climate ID	Climate Name	Weather Station Locations	Elevation (ft)	Groundwater Depth (ft)
Climate I	Shore	Bridgeport New Haven Groton	11	20
Climate II	Inland	Hartford Willamantic	18 247	20
Climate III	Mount	Worcester, MA	1009	20

General Traffic Inputs

Highway Functional Class	Traffic Level	Design Life ESALs [million]	Initial AADTT [trucks]	Speed [mph]
Interstate HWY	Level 3 High	12.1	2500	70
Non-Interstate HWY	Level 3 Medium	4.8	1000	55
Local Arterial	Level 2	1.9	400	40

Subgrade Properties

Subgrade ID	Percent Passing #10 (Long1992)	Percent Passing #4 (Long1992)	AASHTO Class	Mix Dry Density	Resilient Modulus Range [psi] [NCHRP 2004]	Assigned Resilient Modulus [psi]
Soil A	75	8.7	A-1-b	123.3	6,000 – 16,000	10,000
Soil B	62	8.8	A-1-b	126.5	8,000 – 20,000	15,000
Soil C	50	11.2	A-1-b	142.5	10,000 – 30,000	20,000

Typical Pavement Structures

Design Parameter	Structure I (3+5+0)	Structure II (4+6+0)	Structure III (3+3+6)
HMA Layer Thicknesses [in]			
Surface HMA	3	4	3
Binder HMA	5	6	3
Base HMA	0	0	6
Asphalt Binder Inputs			
Surface AC Binder PG	64-22	64-22	64-22
Binder AC Binder PG	64-22	64-22	64-22
Base AC Binder PG	64-22	64-22	64-22
HMA Mix Properties[1]			
Surface AC Mix Type/ NMAS	S0.375	S0.375	S0.375
Binder AC Mix Type	S0.5	S0.5	S0.5
Base AC Mix Type	Granular Base A[2] + 2% PG 64-22		
Air Voids [percent]	4 (for all AC layers)		
Asphalt Binder Content[3] [percent]			
Surface AC	5.4-5.5	5.4-5.5	5.4-5.5[3]
Binder AC	4.8-4.9	4.8-4.9	4.8-4.9
Base AC	2	2	2

[1]See Table M.04.03-3 for gradation and volumetrics
[2]See Table 4.5 for granular base properties
[3]Depends on traffic level (See Table M.0.4.02-5)

Typical Pavement Structures

Pavement Structure 1 (level 2)	4 inches Superpave 0.5 inch, two equal lifts, on 6 inches Processed Aggregate Base, on 10 inches Subbase	Pavement Structure 4 (level 3)	4 inches Superpave 0.5 inch, two equal lifts, on 5 inches Superpave 1.0 inch, on 12 inches Subbase
Pavement Structure 2 (level 2)	3 inches Superpave 0.375 inch, two equal lifts, on 4 inches Superpave 1.0 inch, on 12 inches Subbase	Pavement Structure 5 (level 3)	4 inches Superpave 0.5 inch, two equal lifts, on 6 inches Superpave 1.0 inch, two equal lifts on 12 inches Subbase
Pavement Structure 3 (level 2)	4 inches Superpave 0.5 inch, two equal lifts, on 4 inches Superpave 1.0 inch, on 12 inches Subbase		

Note 1: The Superpave mix design levels are included above. The "levels" should not be shown on the typical sections. This information is to be included on the Superpave Notice to Contractor.

Note 2: The above pavement designs are based on calculations which assume that the subgrade on which they are constructed is composed of material suitable for roadway support.

Basic Granular Material Properties

Input	Grading A	Grading B	Grading C
Aggregate Gradation (Percent Passing Sieve)			
125 mm (5 in)	100	100	
90 mm (3.6 in)		90-100	
37.5 mm(1 ½ in)	55-100	55-95	100
19 mm(3/4 in)			15-80
6.3 mm(1/4 in)	25-60	25-40	25-60
4.15 mm (#4)	20-52	20-52	20-52
2 mm (#10)	15-45	15-45	15-45
0.425 mm (#40)	5-25	5-25	5-25
0.15 mm (#100)	0-10	0-10	0-10
0.075 mm (#200)	0-5	0-5	0-5
Plasticity Index	1	1	1
Assigned Modulus [psi]	30,000	25,000	20,000

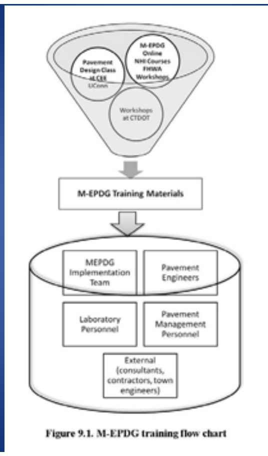
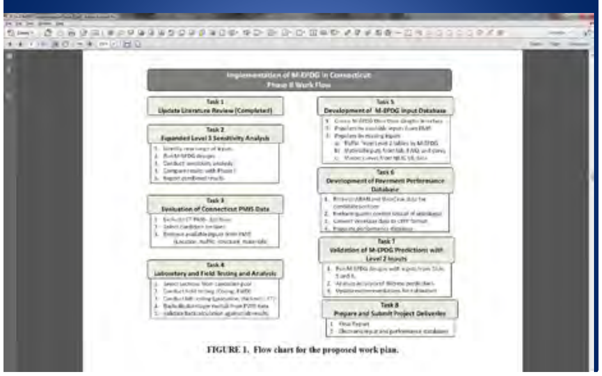


Figure 9.1. M-EPDG training flow chart

Phase II

- Expand the sensitivity analysis of MEPDG inputs
- Validate MEPDG distress predictive capabilities using State Pavement Management Data

Phase II Work Plan



Task 2: Expand Sensitivity Analysis

- Purchase two AASHTO Pavement-ME software licenses for use at UConn
- Wider range of inputs
  - Variable vehicle class distributions (VCD's)
  - Thinner AC pavements
  - Poorly graded subbase for newly constructed pavements
  - AC overlaid PCC pavements

Task 3: Evaluate CT PMIS Data

- Locations
- Functional Classification
- Traffic Volumes
- Pavement Structures
- Construction Type

Location

- Climate I – Shore
- Climate II – Inland
- Climate III – Northern Hills

Functional Classification

- Limited access roads (Interstates and divided CT routes)
- Unlimited access roads (undivided CT routes and major arterials)
- Low volume secondary roads (minor arterials and collectors)

Traffic Volume

- Level 1 traffic (< 0.3 million ESALs)
- Level 2 traffic (0.3 to 3 million ESALs)
- Level 3 traffic (3 to 30 million ESALs)
- Level 4 traffic (> 30 million ESALs)

Pavement Structure

- Thin AC
- Thick AC
- AC on PCC
- AC on AC on PCC

Construction Type

- New construction
- Minor rehabilitation/preservation (overlaid by "mill and fill")
- Structural rehabilitation (overlaid without milling)

Preliminary Pull of Sections

- Climate I – Shore
  - Limited access
    - Level 4 traffic – AC/AC/PCC: 2 sections
    - Level 3 traffic – AC/AC/PCC: 2 sections
  - Unlimited access
    - Level 2 traffic – AC/PCC – new: 4 sections
    - Level 2 traffic – AC/PCC – minor rehab: 4 sections
  - Low volume secondary
    - Level 1 traffic – Thick AC – new: 2 sections
    - Level 1 traffic – Thin AC – minor rehab: 2 sections

Preliminary Pull of Section

- Climate II – Inland
  - Limited access
    - Level 4 traffic – AC/AC/PCC: 2 sections
    - Level 3 traffic – AC/AC/PCC: 2 sections
  - Unlimited access
    - Level 2 traffic – AC/PCC – new: 4 sections
    - Level 2 traffic – AC/PCC – minor rehab: 4 sections
  - Low volume secondary
    - Level 1 traffic – Thick AC – new: 2 sections
    - Level 1 traffic – Thin AC – minor rehab: 2 sections

Preliminary Pull of Sections

- Climate III – Northern Hills
  - Limited access
    - Level 4 traffic – AC/AC/PCC: 2 sections
    - Level 3 traffic – AC/AC/PCC: 2 sections
  - Unlimited access
    - Level 2 traffic – AC/PCC – new: 4 sections
    - Level 2 traffic – AC/PCC – minor rehab: 4 sections
  - Low volume secondary
    - Level 1 traffic – Thick AC – new: 2 sections
    - Level 1 traffic – Thin AC – minor rehab: 2 sections

Sample of Sections Provided (48 provided in total)

Route Number	Direction	From Mile Point	To Mile Point	Length	Approximate Composition	Year of Last Resurfacing	Project Number
95	N	12.319	12.903	0.584	5" Superpave 0.50, on, 1" Superpave 0.375, on, Existing 9-10" Reinforced Concrete Pavement	2005	035-176
91	N	3.608	4.172	0.564	* 3" Bit. Conc. , on, 9" PCC,on, 6" Subbase	2004	092-580
15	N	42.182	42.757	0.575	*3.5" Bit. Conc., on, 8.5" PCC		173-334
95	N	92.597	93.128	0.531	4" Superpave 0.50, on, 1" Superpave 0.375, on, Existing Concrete	2002	094-202
12	N	3.902	4.302	0.400	* 2.75"Bit. Conc. , on, 2.75" Liquid, on 6.5" Granular	2012	058-235
1	N	50.700	51.012	0.312	* 2.25" Bit. Conc. on 8.5" PCC (1983) OR 2.25" Bit. Conc. on 7.5" Granular (2003)??	2008	043-122
34	E	16.980	17.350	0.370	* 2.25" Bit. Conc., on, 10.5" PCC	1997	106-107

…and that's all I have to say about MEPDG Implementation Research Projects in Connecticut

Florida – Florida Climate Effects

Florida Climate Effects

On Concrete Pavement Thickness

FHWA MEPDG PEER Exchange

November 5-6, 2014

Atlanta, GA

Florida Climatic Regions

Based on one half inch design thickness differences from MEPDG analysis

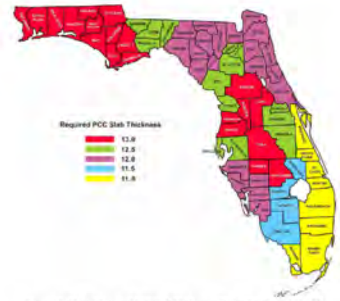
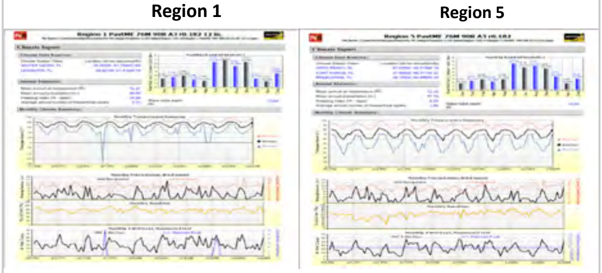


Figure 6.9: Map of Required PCC Slab Thickness at 90% reliability and 50 x 10⁶ Cumulative ESALs.

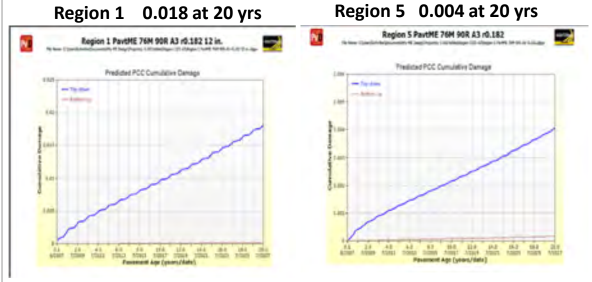
Pavement ME weather input summary

Region 1 Region 5

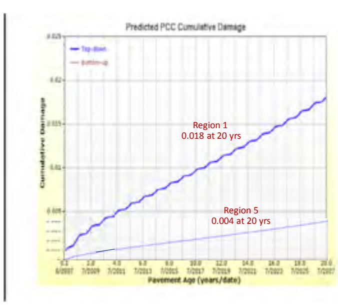


PCC Cumulative Damage

Region 1 0.018 at 20 yrs Region 5 0.004 at 20 yrs



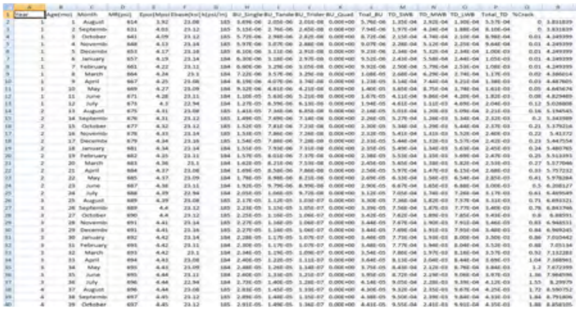
PCC Cumulative Damage

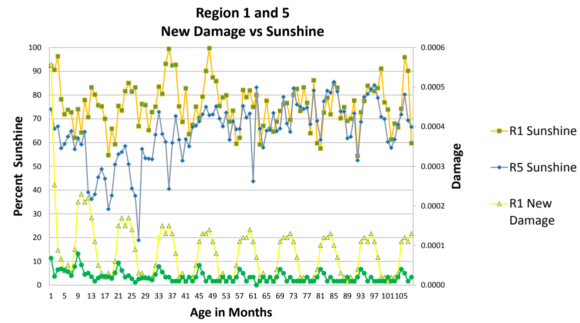


Monthly Climate Summary File

	A	B	C	D	E	F	G	H	I	J	K
1	#Monthly Climatic Summary										
2	#Temperature (degrees Farenheit)										
3	#Depths and precipitation in inches										
4	#Windspeed in miles per hour										
5	#Sunshine in percent										
6	Date	Minimum	Maximum	Average T	Maximum	Precipitat	Average V	Average S	Number V	Maximum	Frost Depth
7	Jul-96	69.1	96.1	81.9	23.1	4.14	5.9	92.2	17	0	
8	Aug-96	71.1	92	79.7	19.9	6.03	4.9	90.6	21	0	
9	Sep-96	64	95	79.5	23.1	4.39	4.7	96.1	18	0	
10	Oct-96	51.1	88	73.7	27	4.02	6.6	78.2	12	0	
11	Nov-96	46.9	87.1	66.7	29.8	0.69	7.9	71.9	9	0	
12	Dec-96	35	82	62.3	29	3.39	5.8	73.7	15	0	
13	Jan-97	32	82.8	61.7	29.1	2.59	5.7	72.7	15	0	
14	Feb-97	44.1	87.9	67.7	29.7	1.11	6.9	82	12	0	
15	Mar-97	53.8	87.9	72.8	27.8	2.8	6.7	74	11	0	
16	Apr-97	52	89.7	70.6	26	4.75	7.9	64.2	14	0	
17	May-97	58.9	93.8	77.1	23.1	1.4	6.1	77.9	13	0	
18	Jun-97	66.9	93.9	80.2	21.9	7.22	5.9	70.7	22	0	
19	Jul-97	73	97	81.5	23.1	6	4.6	83.2	29	0	
20	Aug-97	70	96	82.4	22.9	4.65	4.7	80.2	18	0	
21	Sep-97	69.1	93.8	80.7	22.7	8.21	5.5	75.7	15	0	
22	Oct-97	48.9	90.9	73.9	25.1	3.96	6.4	75.2	15	0	
23	Nov-97	45.9	84	67	23.9	3.17	6.2	70	11	0	
24	Dec-97	40.9	83	62.8	26	10.22	6.5	54.7	16	0	
25	Jan-98	40.9	83.9	63.6	23.9	2.97	7.2	65.8	16	0	
26	Feb-98	45	82.8	63.4	28.8	5.17	8.9	59.3	17	0	
27	Mar-98	40.9	87.9	63.8	28.7	4.41	8.4	75.4	8	0	
28	Apr-98	52	87	71.7	34	1.47	7.7	73.5	6	0	
29	May-98	61.1	93.9	78.4	28.1	0.69	5.8	81.6	12	0	
30	Jun-98	73	98.1	85	24	0.85	6.6	85	6	0	
31	Jul-98	73	99	82.6	19.9	9.07	4.2	81.5	21	0	
32	Aug-98	72	96	82.4	20.8	2.03	5	83.2	28	0	
33	Sep-98	69.1	93.8	79.5	19.9	8.8	6.6	66.9	24	0	
34	Oct-98	60	90.9	76.5	23	0.21	6.5	76.2	7	0	
35	Nov-98	50	86	71.4	23.1	0.92	4.6	75.8	11	0	
36	Dec-98	42.9	83.9	66.7	25.1	0.01	5.5	65.2	9	0	
37	Jan-99	30.9	82	63	29.2	2.77	6.1	72.8	15	0	
38	Feb-99	39.8	81.8	63.5	31	1.05	6.4	75.1	11	0	
39	Mar-99	45	82.9	65.2	32.8	0.65	6.7	83.5	8	0	
40	Apr-99	-72.4	93	73.5	135.9	1.27	6.3	80.5	10	6	

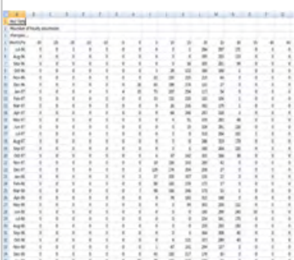
JPCP Cracking File



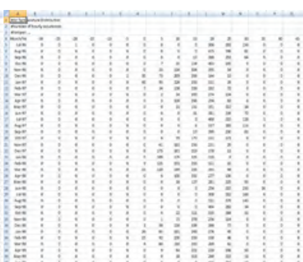


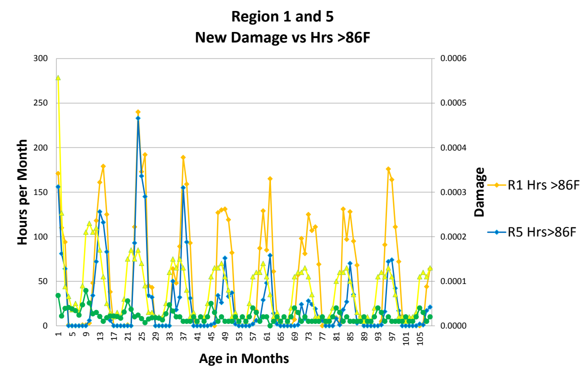
Air Temperature Distribution Files

Region 1



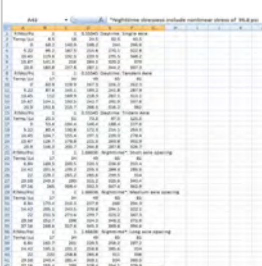
Region 5



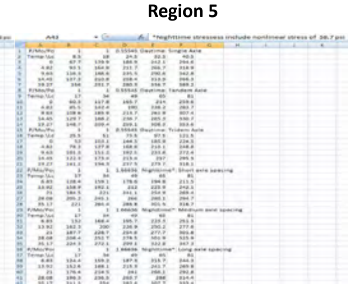


Stress Results Files

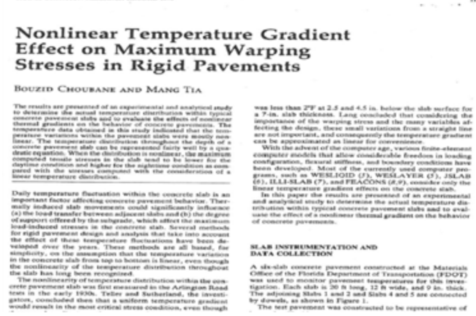
Region 1



Region 5



Nonlinear Temperature Gradient Paper (Choubane and Tia 1992)



TRANSPORTATION RESEARCH RECORD 1370

Nonlinear Temperature Gradient Effect on Maximum Warping Stresses in Rigid Pavements

BOUZID CHOUBANE AND MANG TIA

Questions?

Idaho Implementation and Local Calibration – Materials

MEPDG Peer Exchange Meeting
April 14-15, 2015
Portland, OR

Idaho Implementation and Local Calibration

Fouad Bayomy (UI) and Mike Santi (ITD)

University of Idaho

Implementation Steps

- Phase 1 – Materials and Traffic Database (2011-2013)

University of Idaho
Fouad Bayomy (PI)
Sherif El-Badawy (Co-PI)
Ahmed Awad (Grad Assistant)

ITD Project RP 193
http://itd.idaho.gov/highways/research/archived/reports/RP193Final.pdf

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Implementation Steps

- Phase 2 – Initial User Guide (v 1.1) and Calibration Road Map (2013-2014)

Applied Research Associates, Inc.
Jagannath Mallela,
Harold L. Von Quintus,
Michael I. Darter, and
Biplab B. Bhattacharya

ITD Project RP 211 A, B
http://itd.idaho.gov/highways/research/archived/reports/RP211MEPDGRoadMapFinal.pdf

http://itd.idaho.gov/highways/research/archived/reports/RP211UserGuideFinal.pdf

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Implementation Steps

- Phase 3 – Local Calibration – Flexible Pavements. Planned to start April 2015 over about 2 years

ITD Project RP 235
University of Idaho
Fouad Bayomy (PI)
Post Doc (TBD)
Grad Assistant(s) (TBD)

Expected to start May 2015

Implementation Steps

- Phase 1 – Materials and Traffic Database (2011-2013)_UI
- Phase 2 – Initial User Guide (v 1.1) and Calibration Road Map (2013-2014)_ARA
- Phase 3 – Local Calibration – Flexible Pavements. Planned to start April 2015 over about 2 years_UI
- Phase 4 – Materials Database for PCC Pavements
- Phase 5 – Local Calibration for PCC Models

Today's Presentation is focused on Phase 1 (RP 193)

Acknowledgement

- **UI Team**
  - PI
  - Dr. Sherif El-Badawy (Co-PI)
  - Mr. Ahmed Awad (Grad Student)
  - Admin Support by NIATT Staff and CE Office.
  - Idaho Asphalt Supply – Binder Testing
- **ITD**
  - Materials Office
    - Mike Santi
  - Traffic Office
    - Scott Fugit
    - Glenda Fuller
  - Districts
    - District Material Engineers & Supporting Staff
  - Research Office
    - Ned Parish
    - Inez Hopkins
- **FHWA**
  - Chris Wagner, PE
    FHWA, External Peer Reviewer

Main Project Tasks

- Study the Latest Version of the MEPDG Software (MEPDG Version 1.10).
- Review of Other State Agencies Implementation Efforts
- Material Database:
  - Binder
  - HMA
  - Unbound base/Subbase layers
  - Subgrade
- Develop Traffic Load Spectra
- Establish climatic factors
- MEPDG sensitivity analysis
- Performance and reliability
- Develop plan for local calibration and validation

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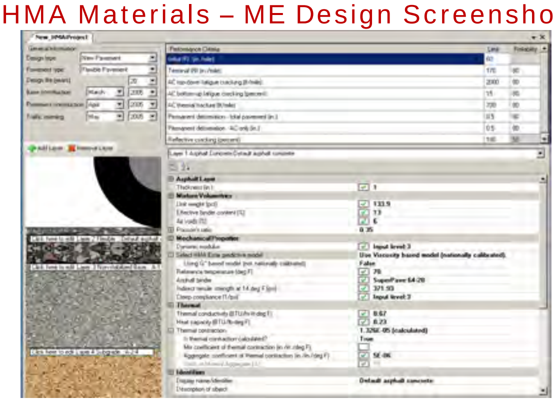
Task 3 - MEPDG Material Characterization

- Binder: G* and δ
- HMA Materials: Dynamic Modulus (E*)
- Unbound Base / Subgrade Layers:
  - R-Value / Mr, PI, Gradation.

HMA Materials – ME Design Screenshot



HMA Hierarchical Input Levels

ME Design includes three levels to obtain inputs to facilitate use and implementation.

Input Level	Determination of Input Values
1	Conduct comprehensive laboratory testing
2	Conduct limited laboratory testing and supplement with estimations using predictive equations
3	Use predictive equations with volumetric data

MEPDG Hierarchical Input Levels

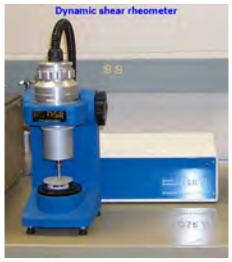
- **Level 1:** Highest level of data input accuracy. Laboratory or field measured data.
  - **EX: laboratory E* for HMA, G*& δ for binder, Mr for Base/SG.**
- **Level 2:** Intermediate level of accuracy. Correlations with other properties.
  - **EX: Mr = 1155 +555*R**
- **Level 3:** Lowest level of data input accuracy. Typical default values (best estimates).
  - **EX: for A-1-a soil Mr = 38,000 psi**

Important Consideration !

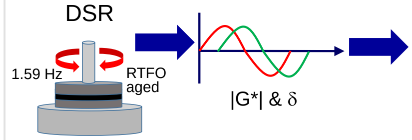
- *The same approach and models are used at ALL input levels.*
- *The only difference is the amount of laboratory testing required.*

MEPDG Binder Characterization

- Levels 1 and 2: **G*** **and** δ at 10 rad/sec (RTFO)
- Level 3: choose binder grade



Binder A-VTS Relationship

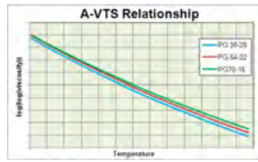


Temp	|G*|	δ

$$\eta = 1000 * \frac{G^*}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628}$$

$$\log(\log(\eta)) = A + VTS.\log(T_R)$$

Conversion of G* & δ to viscosity



MEPDG Binder Characterization (MEPDG Level 1)

- 9 PG grade binders typically used in Idaho.
- G* and δ at 10 rad/sec were determined using DSR at 40, 70, 100 & 130 ºF.
- All tests were done by Idaho Asphalt Supply.

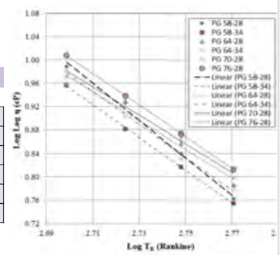
PG 58-28
PG 58-34
PG 64-22
PG 64-28
PG 64-34
PG 70-22
PG 70-28
PG 70-34
PG 76-28

Example: PG 76-28

PG 76-28

Superpave Binder Test Data on RTFO Aged Samples (Level 1)

Temp (°F)	At Angular Frequency = 10 rad/sec	
	G* (Pa)	Delta (°)
40	2.20E+07	42.28
70	2.19E+06	59.11
100	1.34E+05	58.16
130	1.86E+04	63.63



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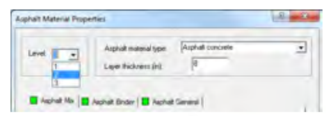
HMA Material Characterization Input Levels

Input Level	Description
1	Conduct E* (Dynamic Modulus) testing
2 & 3	Use E* predictive equations with volumetric data



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HMA Material Characterization

ITD Field Mixes Investigated

Mixture Type						
SP1	SP2	SP3		SP4	SP5	SP6
SP1-1	SP 2-1	SP 3-1	SP 3-5-4	SP 4-1	SP 5-1	SP 6-1
	SP 2-2 *	SP 3-2	SP 3-5-5	SP 4-2	SP 5-2	SP6-2
		SP 3-3	SP 3-6 *	SP 4-3	SP 5-3	
		SP 3-4	SP 3-7 *	SP 4-4 *	SP 5-4	
		SP 3-5-1	SP 3-8 *			
		SP 3-5-2	SP 3-9 *			
		SP 3-5-3	SP3-10 *			
1	2	14		4	4	2
Total Number of Mixtures = 27 Mixtures						

** From ITD Project No. RP 181 "Development and Evaluation of Performance Tests to Enhance Superpave Mix Design and its Implementation in Idaho"*

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Typical Idaho Superpave Mix Requirements

ITD Mixture Type	SP1	SP2	SP3	SP4	SP5	SP6
a	< 0.3	0.3 -< 1	1 -< 3	3 -< 10	10 - < 30	≥ 30
	40	35	30	30	30	30
b % Minimum	50/-	65/-	75/60	85/80	95/90	
Void Content of	--	40	40	45	45	45
	35	35	40	45	45	50
c	--	10	10	10	10	10
Gyrations for Nini Gyrations for Ndes Gyrations for Nmax	6 40 60	6 50 75	7 75 115	8 90 160	8 100 160	9 125 205
	<91.5	≤ 90.5	≤89.0	≤89.0	≤89.0	≤89.0
Gmm@	96.0	96.0	96.0	96.0	96.0	96.0
	≤98.0	≤98.0	≤98.0	≤98.0	≤98.0	≤98.0
	4.0	4.0	4.0	4.0	4.0	4.0
f	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2
e	70-80	65-78	65-75	65-75	65-75	65-75

Source: University of Idaho

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Experimental Work

GS Determination from Servopac Gyratory Compactor data

E* Testing using AMPT Machine

E* Testing Data (MEPDG level 1)

- *2-replicates per mix.*
- ***4** Temperatures & **6** Frequencies*

Temperatures	Frequency, Hz
40ºF (4.4ºC)	25
70ºF (21.1ºC)	10
	5
100ºF (37.8ºC)	1.0
130ºF (54.4ºC)	0.5
	0.1

Example of MEPDG level 1 Input Data of SP4-2 Mixture

Asphalt Mix Dynamic Modulus

Temp (°F)	Mixture E* (psi)					
	0.1	0.5	1	5	10	25
14	1.83E+06	2.11E+06	2.23E+06	2.46E+06	2.55E+06	2.66E+06
40	9.58E+05	1.24E+06	1.37E+06	1.67E+06	1.80E+06	1.93E+06
70	2.48E+05	4.05E+05	4.86E+05	7.23E+05	8.39E+05	1.00E+06
100	5.95E+04	1.02E+05	1.28E+05	2.26E+05	2.84E+05	3.77E+05
130	1.45E+04	2.45E+04	3.07E+04	5.87E+04	7.72E+04	1.17E+05

- ✓ At least E* matrix (3-temperatures * 3-frequencies).
- ✓ Minimum temperature within 10 to 20 ° F
- ✓ Maximum temperature within 125 to 135° F.
- ✓ E-values = 10,000 to 5,000,000 psi.

NCHRP 1-37A η*-based Model

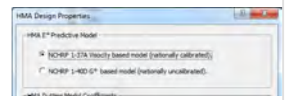
(Andrei, Witczak and Mirza's Revised Model, 1999)

$$\log_{10} E^* = -1.249937 + 0.02923\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a - 0.82208\frac{V_{beff}}{V_{beff}+V_a} + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.00547\rho_{34}}{1+e^{(-0.603313-0.313351\log f - 0.393532\log \eta)}}$$

Where,

^{5} psi;

^{6} poise;



a

beff

34 ;

38 ;

4 ; and

200

NCHRP 1-40D *G**-based Model

(Witczak, El-Basyouny & El-Badawy Revised from Bari's Model, 2007)

$$\log_{10} E^* = 0.02 + 0.758\left(|G_b^*|^{-0.0009}\right)\times\left(\begin{array}{l}6.8232 - 0.03274\rho_{200} + 0.00431\rho_{200}{}^2 + 0.0104\rho_4 - 0.00012\rho_4{}^2 \\ + 0.00678\rho_{38} - 0.00016\rho_{38}{}^2 - 0.0796V_a - 1.1689\left(\frac{V_{beff}}{V_a+V_{beff}}\right)\end{array}\right) + \frac{1.437 + 0.03313V_a + 0.6926\left(\frac{V_{beff}}{V_a+V_{beff}}\right) + 0.00891\rho_{38} - 0.00007\rho_{38}{}^2 - 0.0081\rho_{34}}{1+e^{(-4.5868-0.8176\log|G_b^*|+3.2738\log\delta_b)}}$$

Where,

- E^* = HMA dynamic modulus, psi,
- $|G_b^*|$ = dynamic shear modulus of binder, psi,
- δ_b = phase angle, degrees,
- V_a = air voids in the mix, %,
- V_{beff} = effective binder content, by volume, %,
- ρ_{34} = cumulative % retained on the 3/4 in sieve,
- ρ_{38} = cumulative % retained on the 3/8 in sieve,
- ρ_4 = cumulative % retained on the No. 4 sieve, and
- ρ_{200} = % passing the No. 200 sieve.

Idaho GS-based E* Model

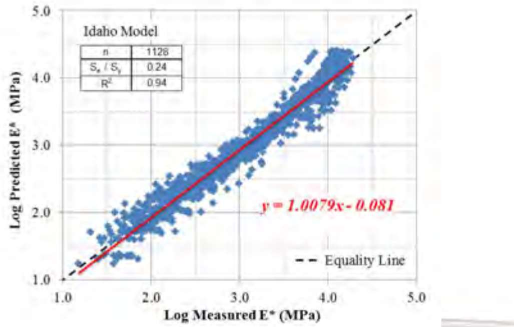
(Bayomy & Abu Abdo 2009)

$$E^* = 1.08\left(\frac{\rho_w . G^* . GS . G_{mb}}{P_b(1-P_b)}\right)^{0.558}$$

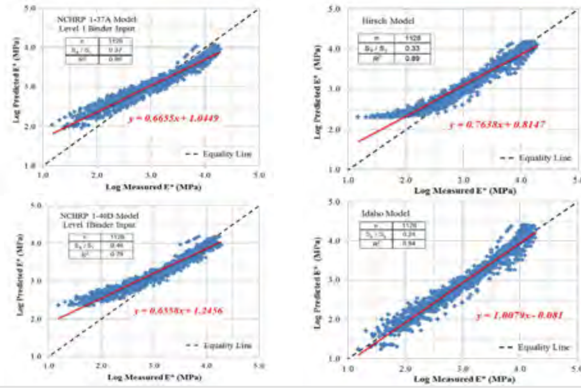
where,

- E* = dynamic modulus of the mixture, MPa
- G* = dynamic shear modulus for RTFO aged binder, MPa
- P_b = binder content by mix weight
- GS = Gyratory Stability, kN.m
- G_{mb} = bulk specific gravity of the mix
- ρ_w = Density of water, kg/m³

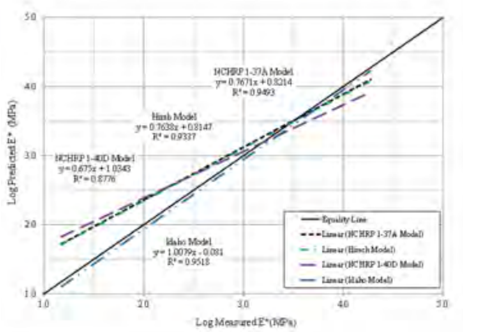
Measured Vs. Predicted E* using Idaho Model



Comparison of Witczak, Hirsh and Idaho E* Models



Comparison of MEPDG E* Predictions



Source: University of Idaho

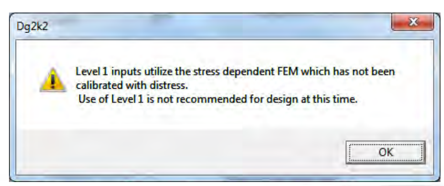
G-Stab 2010 and *E-Star2010*

G-Stab 2010 Software *E-Star 2010* Software

Unbound Materials and Subgrade Soils

Level 1 Inputs

- MEPDG recommends against the use of Level 1 inputs for unbound materials. This level requires input of K1, K2 and K3 of Mr vs. stress relationship.



Level 2 Strength Properties

Strength/Index Property	Model
CBR	$M_r = 2555(CBR)^{0.64}$
R-value	$M_r = 1155 + 555R$
AASHTO layer coefficient	$M_r = 30000\left(\frac{a_i}{0.14}\right)$
PI and gradation*	$CBR = \frac{75}{1+0.728(wPI)}$
DCP*	$CBR = \frac{292}{DCP^{1.12}}$

ITD Unbound/Subgrade Material Characterization (MEPDG Level 2 inputs)

Two Models Developed:

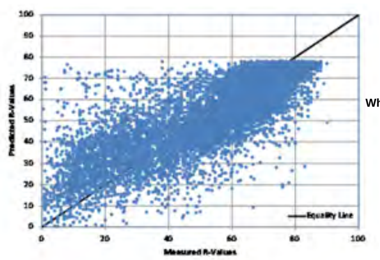
- **R-Value** Models based on ITD Database
- **M_r-R-Value** Model based on literature Mr-Data

R-Value Model:

- Historical ITD R-Values database **(from 1953 to 2008).**
- **8233** points with soil classification, P200 and PI.
- Collected by Dr. Stanley Miller (UI)
- **ITD-PR 185, NIATT KLK 553:** Developing Statistical Correlations of Soil Properties with R-Value for Idaho Pavement Design.

Developed R-Value Model

Measured versus Predicted R-Values

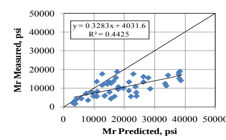


$R = 10^{(1.893 - 0.00159*P200 - 0.022*PI)}$

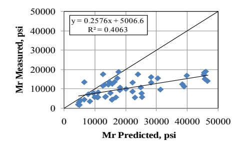
Where:

P200 = % Passing Sieve No. 200
PI = Plasticity Index

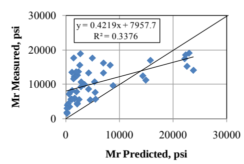
Comparison of Different M_r-R-Value Models based on M_r Literature Data



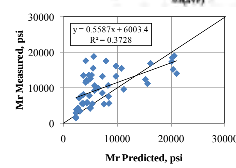
Asphalt Institute Model (Mr=1155+555*R)



ADOT Model:



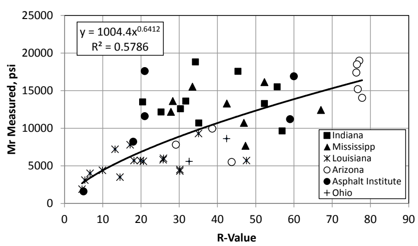
WSDOT Model [$Mr = 720.5\,(e^{(0.0521*R)}-1)$]



ITD Model (Log Mr = (222+R)/67)

2010 M_r-R-Value Model

M_r (psi) = $1004.4\,(R)^{0.6412}$



The majority of this literature soils are fine grained materials.

Recommend Default R-Values and Ranges for Unbound Granular Materials and Subgrade Soils for MEPDG Level 3 for Idaho

Soil Type	Recommended R	Recommended R Range	
		Lower Bound	Upper Bound
OH	32	15	49
OL	44	30	58
CH	15	3	26
MH	28	12	45
CL	27	12	41
CL-ML	45	31	60
ML	60	47	73
SC	35	17	54
GC	38	20	56
SC-SM	53	35	70
GC-GM	60	46	73
SM	66	52	80
GM	72	59	84
SP-SC	15	1	32
SW-SC	71	62	80
SP-SM	74	64	84

Recommended Typical Values and Ranges of PI for ITD Unbound Granular Materials and Subgrade Soils

Soil Type	Recommended PI	Recommended PI Range	
		Lower Bound	
OH	21	17	24
OL	7	1	12
CH	39	23	56
MH	24	14	34
CL	15	8	21
CL-ML	5	4	7
ML	1	0	2
SC	16	6	25
GC	13	7	20
SC-SM	5	4	6
GC-GM	5	4	7
SM	0	0	1
GM	0	0	1
SP-SC	16	3	29
SW-SC	10	2	19
SP-SM	0	0	0

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Recommended Typical Values and Ranges of LL for ITD Unbound Granular Materials and Subgrade Soils

Soil Type	Recommended LL	Recommended LL Range	
		Lower Bound	Upper Bound
OH	62	57	66
OL	33	26	40
CH	65	46	85
MH	67	53	81
CL	35	25	45
CL-ML	27	23	31
ML	24	20	28
SC	36	24	49
GC	33	25	40
SC-SM	25	20	30
GC-GM	26	21	30
SM	23	16	29
GM	24	16	31
SP-SC	46	13	79
SW-SC	26	24	29
SP-SM	-	-	-

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In Summary

- **For HMA** – E* database and volumetric properties. Recommendation for E* models
- **For Binders** – G* and Delta
- **For Unbound Materials** – R-value Model, Mr vs.R-Value Model, and Typical values and ranges for R-Value, LL and PI.

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What is Missing?

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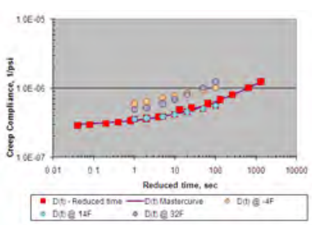
Creep Compliance, Tension

Input requirements

Loading Time (sec)	Creep compliance (1/psi)		
	Low Temp -4°F	Mid Temp 14°F	High Temp 32°F
1			
2			
5			
10			
20			
50			
100			

D(t) Mastercurve



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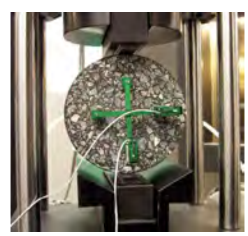
Indirect Tensile Strength

- Definition: Measured strength when HMA is subjected to indirect tension (by applying compressive load diametrically)
- Test protocol: AASHTO T 322
- Tested at 14°F
- Required input for TCMODEL



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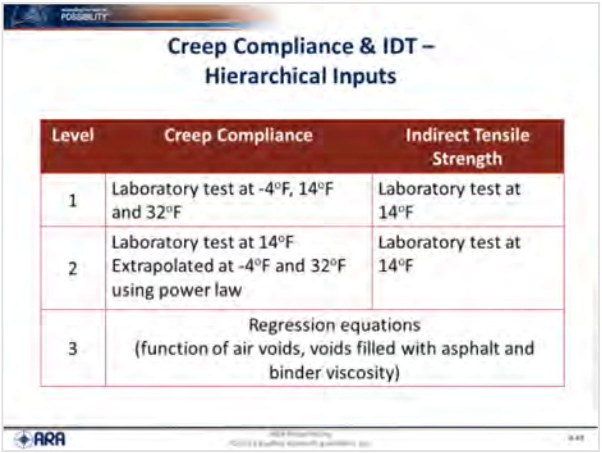
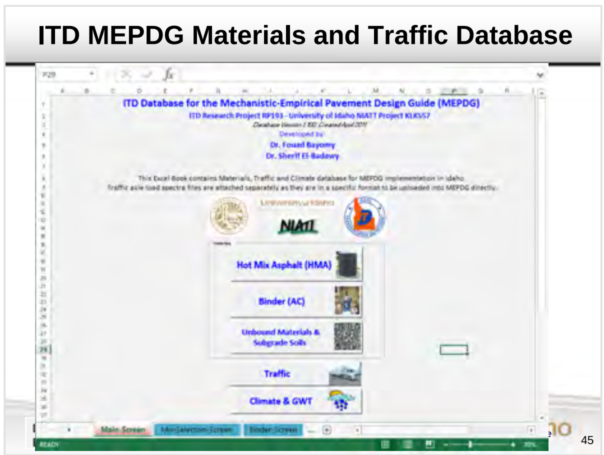
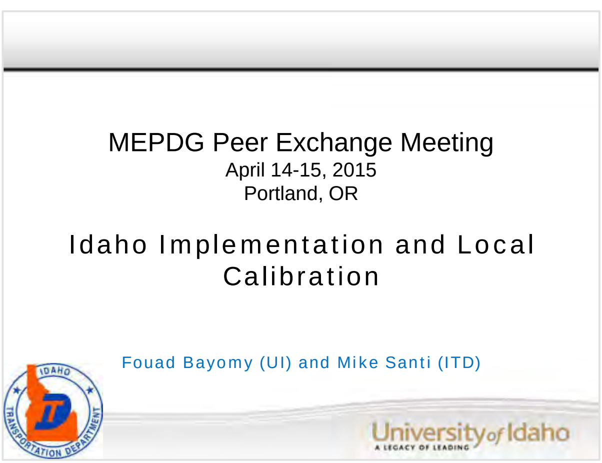
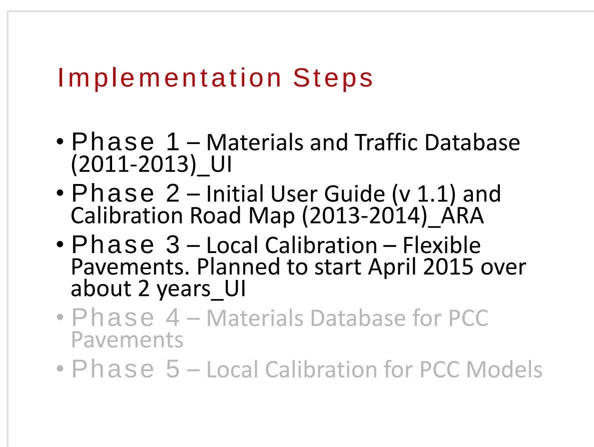




Idaho Implementation and Local Calibration – Traffic





Implementation Steps

- **Phase 1** – Materials and Traffic Database (2011-2013)_UI

ITD Project RP 193
http://itd.idaho.gov/highways/research/archived/closed.htm

Main Project Tasks

- Study the Latest Version of the MEPDG Software (MEPDG Version 1.10).
- Review of Other State Agencies Implementation Efforts
- Material Database:
  - Binder
  - HMA
  - Unbound base/Subbase layers
  - Subgrade
- Develop Traffic Load Spectra
- Establish climatic factors
- MEPDG sensitivity analysis
- Performance and reliability
- Develop plan for local calibration and validation

Task 4 - MEPDG Traffic Characterization

- Traffic inputs in the MEPDG are very **comprehensive** and more **sophisticated** compared to older design methodologies.
- It relies on the traffic axle load spectra data which requires continuous WIM data measurements.
- 3 hierarchical input levels.

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MEPDG Traffic Input Levels

Input Level	of Traffic	Data	Weight Data
Level 1	Very Good	Site-Specific, Continues	Site-Specific
Level 2	Fair	Site-Specific, short	Regional Summaries (TWRGs)
Level 3	Poor	No Actual Class Data	Statewide Summaries

Department of Civil Engineering University of Idaho A LEGACY OF LEADING 6

MEPDG Major Traffic Inputs

Four basic traffic input categories are required by MEPDG as follows:

– **Volume**
– **Classification**
– **Weight**
– **General**

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Traffic Inputs in ME Design

Traffic Volume
- Annual Average Daily Truck Traffic
- Growth Factor (by truck class)
— linear, compound
- Highway Capacity Limits
  - New feature in ME Design
  - To enforce a cap of design traffic volume based on lane capacity limits

Traffic Volume Adjustment
- Vehicle Class Distribution
- Monthly Adjustment
- Hourly Truck Distribution

Axle Load Distribution
- Load distribution by axle type
— single, tandem, tridem & quad

Design Lane Features
- Number of lanes in design direction
- Directional distribution factor
- Lane distribution factor
- Operational speed

General Traffic Inputs
- Number of Axles Per Truck
- Axle Spacing
- Truck Class 8-13 Wheelbase
- Lateral Wander
- Tire spacing and pressure

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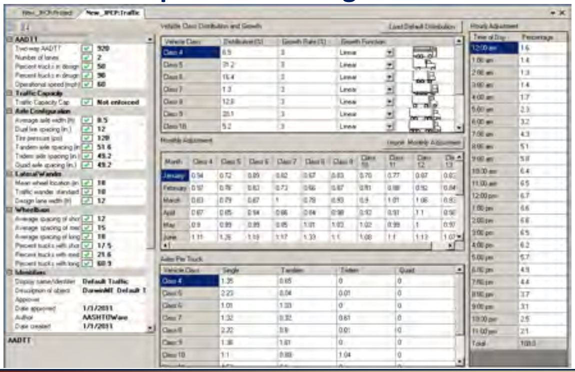
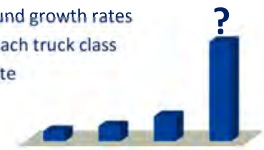
Traffic Inputs – ME Design Screenshot



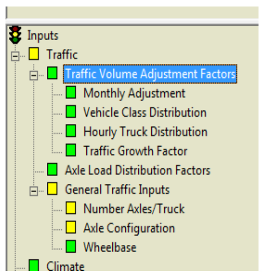
Traffic Volume

- Annual Average Daily Truck Traffic
  - Base year truck volume counts
  - Source includes WIM data, manual or automated vehicle counters
  - Use site-specific truck volume
- Traffic Growth Factor
  - Choice of linear or compound growth rates
  - Separate growth rate for each truck class
  - Use site-specific growth rate



MEPDG Traffic Inputs

- **Base Year Truck Traffic Volume.**
  - AADTT
  - No. of Lanes in Design Direction
  - % Trucks in Design Direction.
  - % Trucks in Design Lane
  - Speed.
- **Traffic Volume Adjustment Factors**
  - Monthly Adjustment.
  - Vehicle Class Distribution.
  - Hourly Truck Distribution.
  - Traffic Growth Factors.
- **Axle Load Distribution Factors.**
- **General Traffic Inputs.**
  - Number of Axles per Truck.
  - Axle Configuration
  - Wheel Base.



ID	Functional Classification	Rout	Mile post	Nearest City
79	Principal Arterial -Interstate (Rural)	I-15	27.7	Downey
93	Principal Arterial -Interstate (Rural)	I-86	25.05	Massacre Rocks
96	Principal Arterial -Other (Rural)	US-20	319.2	Rigby
115	Principal Arterial -Interstate (Rural)	I-90	23.37	Wolf Lodge
117	Principal Arterial -Interstate (Rural)	I-84	231.7	Cottrell
118	Principal Arterial-Other (Rural)	US-95	24.1	Mica
119	Principal Arterial-Other (Rural)	US-95	85.2	Samuels
128	Principal Arterial -Interstate (Rural)	I-84	15.1	Black canyon
129	Principal Arterial-Other (Rural)	US-93	59.8	Gerome
133	Minor Arterial (Rural)	US-30	205.5	Filer
134	Principal Arterial -Other (Rural)	US-30	425.785	Georgetown
135	Principal Arterial -Other (Rural)	US-95	127.7	Mesa
137	Principal Arterial -Other (Rural)	US-95	37.075	Homedale
138	Principal Arterial -Other (Rural)	US-95	22.72	Marsing
148	Principal Arterial -Other (Rural)	US-95	363.98	Potlatch
155	Minor Arterial (Rural)	US-30	229.62	Hansen
156	Minor Arterial (Rural)	SH-33	21.94	Howe
166	Principal Arterial -Interstate (Rural)	I-84		
169	Principal Arterial -Other (Rural)	US-95	56.002	Parma
171	Principal Arterial -Interstate (Rural)	I-84	114.5	Hammett
173	Principal Arterial -Interstate (Rural)	I-15	177.86	Dubois
179	Principal Arterial -Interstate (Rural)	I-86B	101.275	American Falls
185	Principal Arterial-Other (Rural)	US-12	163.01	Powell
192	Principal Arterial-Other (Rural)	US-93	16.724	Rogerson
199	Principal Arterial-Other (Rural)	US-95	441.6	Alpine

Idaho WIM Sites

(25 WIM Sites)

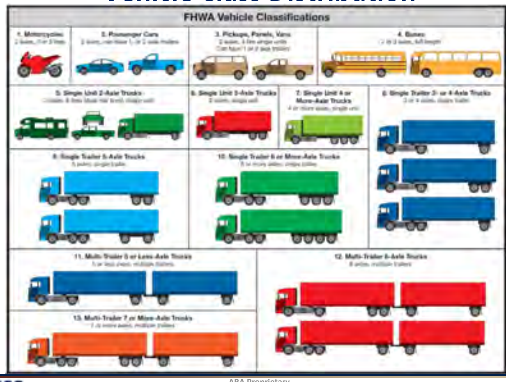
Idaho WIM Sites

Hammett WIM
I-84 @ MP 114.500
0.5 Mi. E of Jct I84B, E Hammett IC
Segment Code 001010

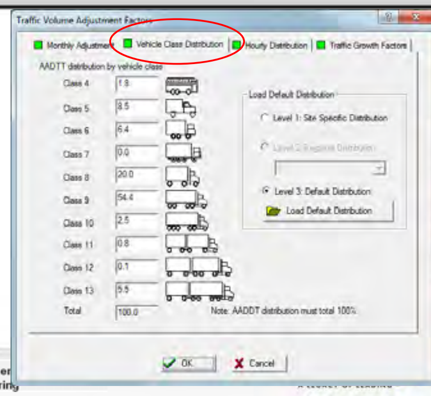
Idaho MEPDG Traffic Characterization

- *WIM Data:*
  - *Classification Data (C-Cards),*
  - *Weight Data (W-Cards)*
- ***TrafLoad*** software (NCHRP project 1-39) was used for processing the WIM data to generate MEPDG traffic inputs.
- ***25 WIM*** *Sites data was analyzed. (2008-2009).*
- 23 WIM sites were successfully analyzed by the *TrafLoad* for the weight data.
- Only 21 WIM sites ➔continuous classification data.

Vehicle Class Distribution



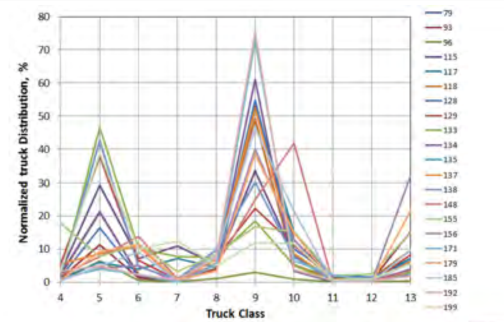
MEPDG Vehicle Class Distribution



Normalized Truck Distribution by Truck Class (level 1)

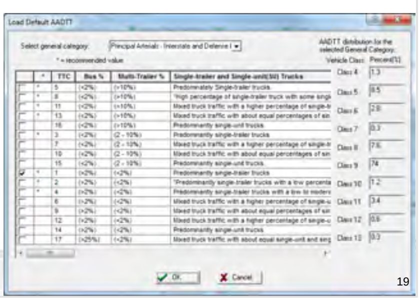
	FHWA Truck Class									
	4	5	6	7	8	9	10	11	12	13
79	1.77	21.20	2.13	0.50	8.35	49.07	5.19	1.11	1.01	9.67
93	0.99	11.21	1.31	0.11	4.09	52.90	12.73	0.76	0.59	15.33
96	0.23	6.45	0.55	0.07	1.04	3.02	0.82	0.05	0.07	0.37
115	2.62	29.15	7.15	10.82	5.31	33.57	7.92	0.26	1.03	2.18
117	1.03	5.96	3.86	7.20	4.56	52.35	15.06	1.45	1.33	7.20
118	1.77	21.20	2.13	0.50	8.35	49.07	5.19	1.11	1.01	9.67
128	1.25	16.44	1.75	0.22	5.49	54.73	9.96	2.28	1.54	6.34
129	5.10	37.84	6.61	0.64	7.29	22.21	11.36	0.45	0.17	8.33
133	1.34	46.53	10.18	7.73	7.54	18.56	5.12	0.08	0.01	2.92
134	2.15	21.28	1.90	0.36	5.51	61.01	3.43	0.19	0.27	3.91
135	1.84	42.40	4.74	0.82	9.71	30.16	7.54	0.53	0.08	2.19
137	5.37	8.56	10.73	0.32	6.94	52.33	8.71	0.61	0.18	6.26
138	1.84	42.40	4.74	0.82	9.71	30.16	7.54	0.53	0.08	2.19
148	2.11	7.69	13.66	1.16	5.02	24.87	41.78	0.00	0.12	3.59
155	17.94	7.73	11.46	3.10	8.46	16.75	15.21	2.07	2.33	14.95
156	1.01	4.00	5.12	0.00	4.96	39.99	12.72	0.00	0.08	32.12
171	1.14	3.82	2.39	0.03	5.18	72.76	6.35	2.23	0.58	5.54
179	0.42	9.27	11.36	0.70	3.27	38.55	14.79	0.08	0.00	21.57
185	0.26	4.77	9.10	0.45	8.05	46.29	21.53	0.00	0.00	9.55
192	3.40	4.90	2.18	0.60	7.24	75.47	3.68	0.50	0.26	1.78
199	2.98	38.76	9.94	12.49	5.12	11.90	11.67	0.68	1.06	5.40

Normalized Truck Distribution by Truck Class

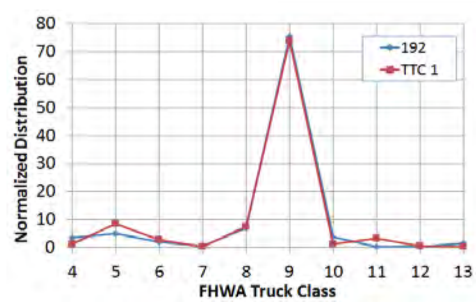


Truck Traffic Classification Groups (TTC), (Level 3)

- MEPDG recommends grouping the vehicle class distribution using 17 TTC groups.
- TTC grouping system is based in the distribution of four truck groups: VC 4, VC 5, VC 9, and VC 13.



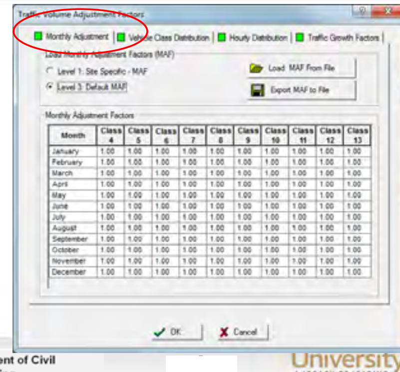
Idaho TTC Groups



Idaho Truck Traffic Classification Groups

WIM Site ID	TTC Group
79	7
93	5
96	12
115	9
117	N/A
118	14
128	4
129	13
133	14
134	3
135	12
137	4
138	N/A
148	N/A
155	N/A
156	N/A
171	3
179	N/A
185	N/A
192	1
199	N/A

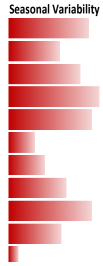
Monthly Adjustment Factors (MAF)



Monthly Adjustment Factors – Idaho Averages

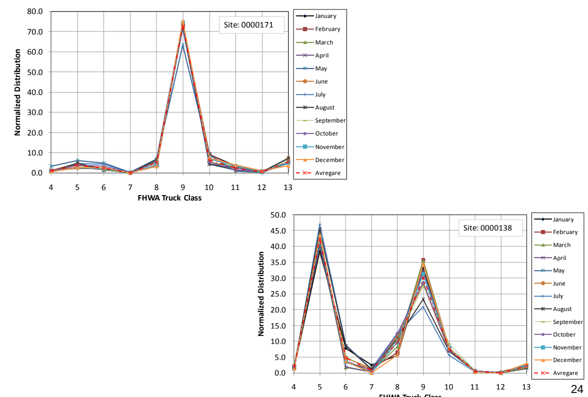
Month	Jan	Feb	March	April	May	June	July	Aug	Sep	Oct	Nov	Dec
Class 4	0.74	0.83	0.77	0.91	1.12	0.99	1.49	1.46	1.31	0.94	0.72	0.71
Class 5	0.86	0.82	0.8	0.85	0.98	1.01	1.33	1.21	1.14	1.08	0.99	0.91
Class 6	0.91	0.87	0.83	0.86	0.9	0.84	1.3	1.45	1.29	1.26	0.75	0.74
Class 7	1.04	0.63	0.75	1.2	1.63	0.72	1.09	1.21	0.98	0.62	0.58	0.98
Class 8	0.64	0.67	0.86	1	1.07	1.17	1.53	1.42	1.18	1.03	0.79	0.63
Class 9	0.98	1	0.95	0.95	0.95	0.94	0.97	0.98	1.06	1.16	1.07	1
Class 10	0.88	0.96	1.1	1.1	1.1	0.84	0.85	1.01	1.08	1.13	0.92	1.02
Class 11	0.9	0.91	0.97	0.93	0.57	0.62	0.68	0.81	0.88	0.6	0.82	1.03
Class 12	0.93	0.67	1.48	0.79	1.2	0.69	1.08	0.96	0.56	0.76	0.67	1.06
Class 13	1.12	0.96	1.01	0.88	0.8	0.81	0.88	0.99	0.93	1.13	1.09	1.42
Default	1	1	1	1	1	1	1	1	1	1	1	1

Seasonal Variability

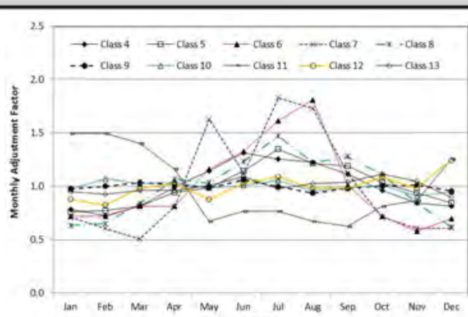


Less than 0.85 | 0.85 – 1.15

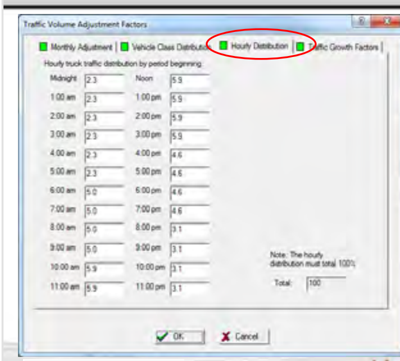
Monthly Normalized Truck Distribution by Class



MAF at WIM Site 79



Hourly Truck Distribution



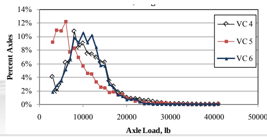
This parameter represents the percentage of truck traffic for each hour of the day.

For flexible pavements, hourly truck distribution factors have negligible impact on the predicted distresses and IRI.

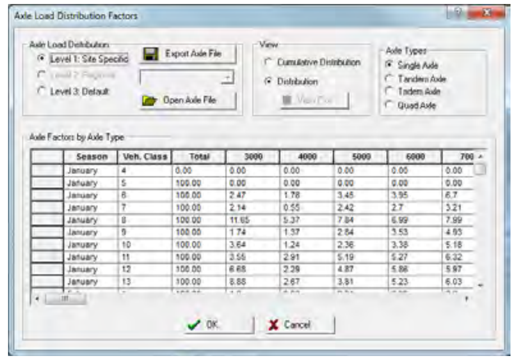
Thus, MEPDG default hourly truck distribution factors can be used.

Axle Load Distribution Factors

- The axle load distribution factors (spectra) present the percentage of the total axle applications within each load interval for each axle type (single, tandem, tridem, and quad) and vehicle class (class 4 to 13).
- Load interval for each axle type:
  - o Single axles : 3,000 lb to 41,000 lb at 1,000 lb intervals.
  - o Tandem axles: 6,000 lb to 82,000 lb at 2,000 lb intervals.
  - o Tridem axles : 12,000 lb to 102,000 lb at 3,000 lb intervals.
  - o Quad axles : 12,000 lb to 102,000 lb at 3,000 lb intervals.
- ALS can only be determined form WIM data.



MEPDG Axle Load Distribution Factors



Season	Veh. Class	Total	3000	4000	5000	6000	700
January	4	0.00	0.00	0.00	0.00	0.00	0.00
January	5	100.00	0.00	0.00	0.00	0.00	0.00
January	6	100.00	2.47	1.78	3.45	3.95	6.7
January	7	100.00	2.14	0.55	2.42	2.7	3.21
January	8	100.00	11.65	5.37	7.84	6.99	7.99
January	9	100.00	1.74	1.37	2.84	3.53	4.93
January	10	100.00	3.64	1.24	2.36	3.38	5.18
January	11	100.00	3.55	2.91	5.19	5.27	6.32
January	12	100.00	6.68	2.29	4.87	5.86	5.97
January	13	100.00	8.88	2.67	3.81	5.23	6.03

Axle Load Spectra for MEPDG

- Site-Specific, ***(Level 1)***.
- Truck Wight Road Groups (TWRGs), ***(Level 2).***
- Statewide Average ***(Level 3).***

Quality of WIM Data

- The quality of WIM data is always questionable.
  - – WSDOT reported that out of **38** WIM site data, only **12** possessed good data.
  - – Arkansas reported that out of **55** WIM site data, only **10** sites provided good weight data.

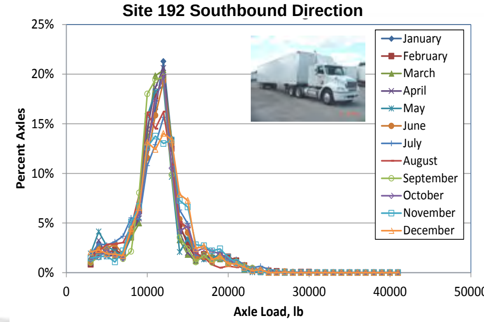
Idaho WIM Data Quality Check

- FHWA recommends these two quality checks on Class 9 truck axle weights:
  - – Regardless of the GVW, the **steering axle weight** (single axle) should peak between **8,000** and **12,000** pounds.
  - – **Tandem axle** weight for the **fully loaded** truck should peak between **30,000** and **36,000** pounds.

*Out of the **23** investigated WIM sites, only **14** WIM sites were found to comply with the quality checks.*

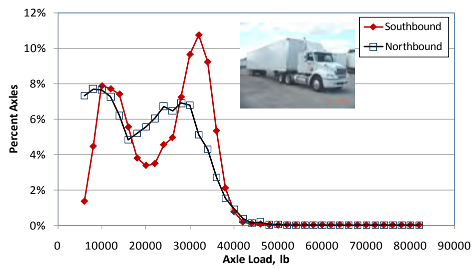
Axle Load Spectra (ALS)

Monthly Variation in Single Axle Spectra for Class 9 Truck at WIM Site 192 Southbound Direction



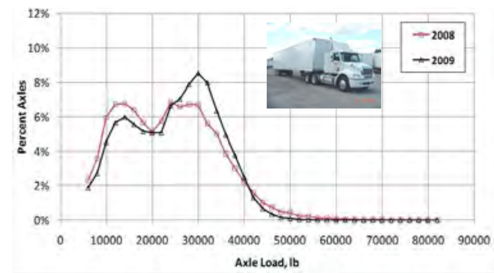
Axle Load Spectra (ALS), Cont'd

Comparison of the Southbound and Northbound Annual Tandem Axle Load Spectra for Class 9 Truck at WIM Site 169



Axle Load Spectra (ALS), Cont'd

Comparison of 2008 and 2009 Tandem Axle Load Spectra for Class 9 Truck at WIM Site 137

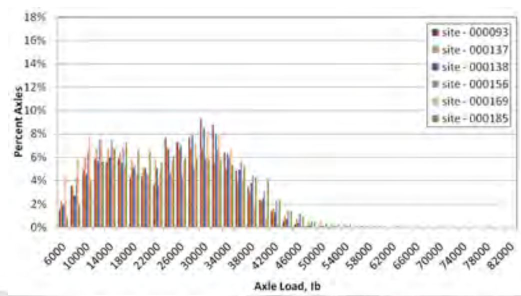


Idaho Axle Load Spectra

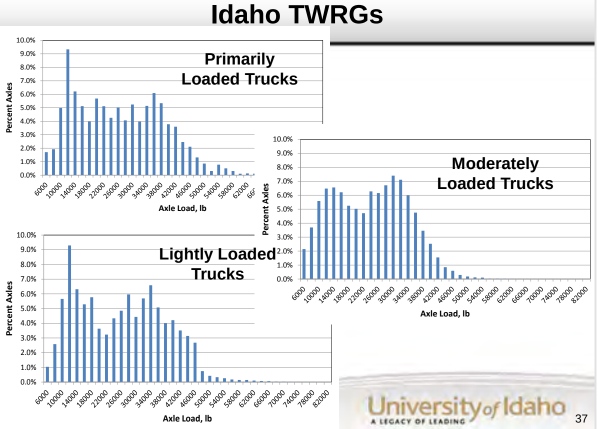
- Site-specific ALS (MEPDG level 1).
- Traffic Weight Road Groups (TWRGs), (MEPDG level 2).
- Statewide ALS (MEPDG level 3).

Idaho Truck Weight Road groups (TWRGs)

TWRG axle load distributions are summary load distributions that represent axle loads found on roads with similar truck weight characteristics (similar axle load distributions).



Idaho TWRGs



Idaho TWRGs

WIM Sites Associated with Idaho Truck Weight Road Groups

	WIM Station
	79, 117, 134, 148, 155
	93, 137, 138, 156, 169, 185
Loaded	96, 129, 192

Idaho Axle Load Spectra

- Site-specific ALS (MEPDG level 1).
- Traffic Weight Road Groups (TWRGs), (MEPDG level 2).
- Statewide ALS (MEPDG level 3).

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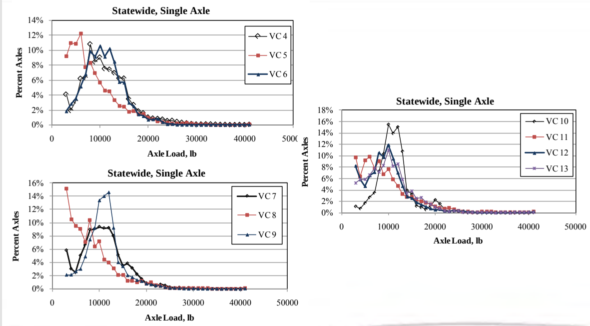
Statewide Single ALS by Truck Class



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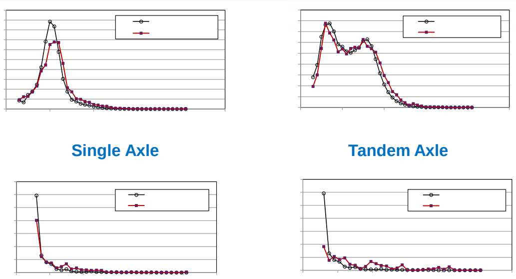
Comparison of Statewide and MEPDG National Default ALS for VC 9



Single Axle

Tandem Axle

Tridem Axle

Quad Axle

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General Traffic Inputs

- Wander (Default).
- No. of axles per truck category.
- Axle configuration (Default).
- Wheelbase (Default).

General Traffic Inputs

Lateral Traffic Wander

Mean wheel location (inches from the lane marking): 18

Traffic wander standard deviation (in): 10

Design lane width (ft): (Note: This is not slab width): 12

Number Axles/Truck | Axle Configuration | Wheelbase

	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

OK Cancel

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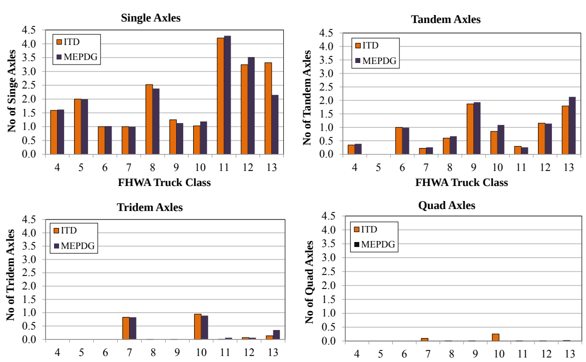
General Traffic Inputs Number of Axles Per Truck



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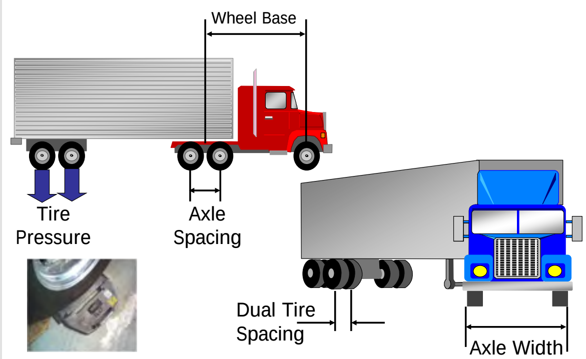
General Traffic Inputs Axle Configuration Parameters



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Kentucky – MEPDG Implementation Status

Kentucky MEPDG Implementation Status

MEPDG Peer Exchange Meeting, Atlanta

Kentucky Transportation Cabinet
Kentucky Transportation Center
November 5-6, 2014

Our Vision of Implementation

- Kentucky Currently has ME process
- Use Current Data and System Framework as much as possible
- Target Level 2/3 Designs
- Calibration/Verification sections post Superpave implementation 2000

Matrix of Calibration/Verification Sites

Flexible(HMA) Sampling Template

Subgrade or Soil (Note 1)	Binder Type	Pavement Type: (1) New or Reconstruction — (1a) Conventional (10% Asp Design)	(1) New or Reconstruction — (1b) Full Depth with 4" DGA [2014 Test]	(1) New or Reconstruction — (1b) Full Depth with 4" DGA [2013 Test]	(2) HMA-Overlay — (2a) HMA/HMA	(2) HMA-Overlay — (2b) HMA/PCP (2b1) Intact PCC	(2) HMA-Overlay — (2b) HMA/PCP (2b2) Break-n-Seat, Crack-n-Seat	(2) HMA-Overlay — (2c2) Rubblized PCC
Non-Stabilized	PG 64-22		V9 - 40 Garrard - KY 52 (MP 5.368-8.76) V18 - 106 Taylor - KY 210 (MP 0-7.577)	V16 - 105 Scott - KY 32 (MP 0.014-3.714)				
	PG 76-22/PMA		V17 - 69 Lincoln - US 150 (MP 8.456-11) V40 - 114 Warren - US 231 (MP 5.9-7.3)					
	Marshall			V29 - 94 Lewis - US 127 (MP 2.383-7.445)				
Stabilized	PG 64-22							
	PG 76-22/PMA							
	Marshall		V36 - 106 Shelby - KY 55 (MP 12.4-20.254)	V5 - 37 Franklin - US 127 (MP 0.913-6.101) V13 - 105 Scott - US 460 (MP 12.217-18.599)				

Note 1: Alternate option: Coarse Grained (A-1 to A-3) & Fine Grained (A-4 to A-7)

Other stratification options:

HMA Thickness (<4", 4-6", >6" or <=6" or >6")

Base Type (Granular DGA/CSA, Drainable Base, Treated CTB/LCB/Fly-Ash/ATB)

Rigid(JPCP) Sampling Template

Base Type	Dowels	Edge Support	Pavement Type: (1) New or Reconstruction	(2) JPCP-Overlay — (2a) JPCP/HMA	(2) JPCP-Overlay — (2b) JPCP/JPCP (Unbonded)	
Granular (DGA/CSA)	YES	Yes - Tied PCC & Widened	Greenbelt (KY715) Franklin US127 MP6.8-8.7 Donaldson National TPK			
		No - HMA Shoulder	Audubon Pkwy			
	NO	Yes - Tied PCC & Widened				
		No - HMA Shoulder				
Drainable Base (Treated or Untreadted)	YES	Yes - Tied PCC & Widened				
		No - HMA Shoulder				
	NO	Yes - Tied PCC & Widened				
		No - HMA Shoulder				

Other stratification options:

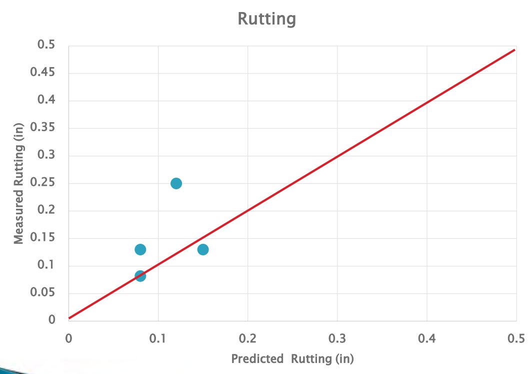
Thickness (<=10" or >10")

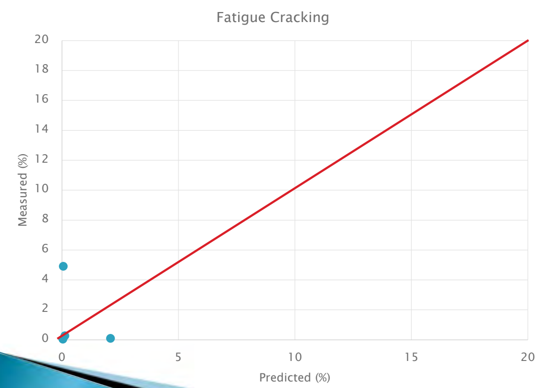
Dowel Size: (1.25", 1.5")

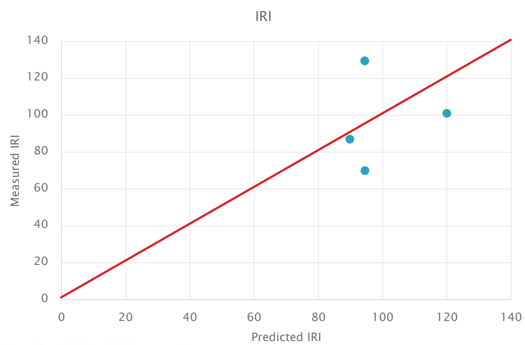
Subgrade: (1) Coarse Grained (A-1 to A-3) & Fine Grained (A-4 to A-7) OR (2) Treated & Non-Treated

Calibration/Verification

- Currently underway
- Field work completed on four sites
- Additional sites identified
- Original Construction Information
  - Materials properties
  - Initial rideability
- Field distress collection
  - IRI, cracking, rutting
- In-situ material sampling, FWD testing
- Compare with Pave ME prediction







Implementation

- KYTC identified the following for successful implementation
  - Implementation plan,
  - materials and traffic libraries,
  - KYTC-specific user input guide,
  - Identify key stake holders, department divisions and industry
  - continuous verification, calibration and validation
- Implementation time frame early 2016 for new construction projects

Implementation Cont.

- Periodic meeting with key staff and external partners
  - Paving industry associations
  - Engineering consultants
- KYTC Pavement Maintenance (PM) team has begun to collect automated pavement distress with video imaging
- Working with the PM team to use the data for calibration by correlating the automated data to manually collected data.
- KY design staffs envision “Concurrent Designs” to facilitate the implementation process.

Challenges/Impediments/Issues:

- Time and effort to collect and obtain consistent field distress data for consistent time-history distress records.
- Calibration calls for at least 3 distress observations/measurements over an 8-10 year period.
- Data conversion to fit LTPP/MEPDG format (Agency/KYTC distress definition vs. LTPP vs. HPMS),

Challenges/Impediments/Issues:

- Complexity of Data Inputs and analysis methods of PaveME (Software complexity)
- Availability of needed data and defining input levels for routine design and calibration
- Lack of resources for in-house local calibration
- For calibration, most of the states are blessed by having working LTPP segments. Kentucky is not in that group. (No LTPP sites to incorporate them in calibration dataset)

Pavement Distress Data

- Current system measures extent/severity for many thresholds
  - How to translate to MEPDG units
- How do we determine distress thresholds?
  - Pavement Management data by facility type?
- Is there a need to change how distress data is evaluated in the future?
  - Automated vs. Extent/Severity?

Challenges for Production Use

- KY currently has a high level of confidence/reliability in designs (>95%)
- Historically we have very little fatigue cracking
  - How to develop realistic distress thresholds with similar level of confidence?
- What will routine design look like?
  - PaveME vs. Design Catalog?
  - How can we simplify inputs?
    - Standard templates?
- Anticipate thickness reductions from current system

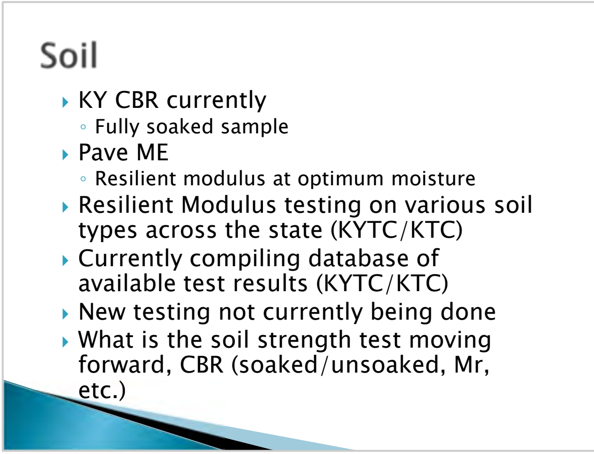
Kentucky Default Design Input Guide

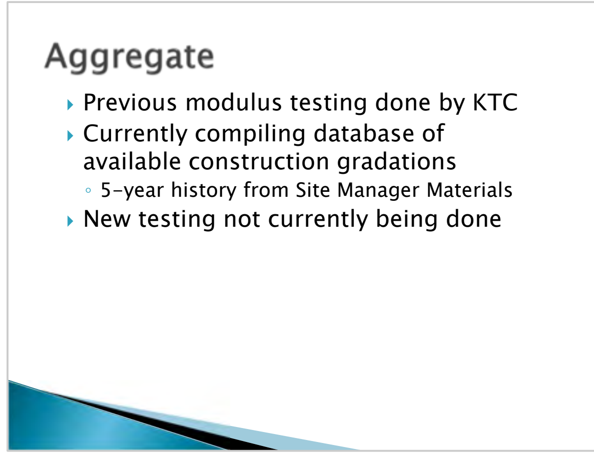
Traffic

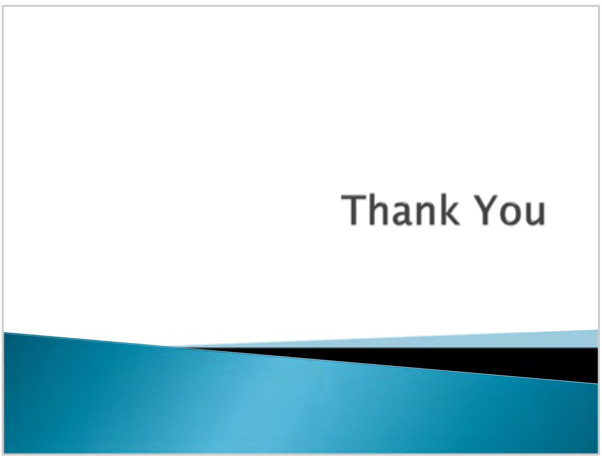
- Current design process develops KY ESALs
- Challenges
  - WIM site location
  - Adequate WIM data (by facility type)
- What information is important?
  - Hourly/Seasonal
  - **Truck volume**
  - Vehicle Class Distribution
  - Load Spectra
  - **Growth Rate**

Bound Materials

- Typical default input information developed from historical data
- Using information from AASHTO SiteManager Materials (5-year history)
- Detailed laboratory testing has not been conducted to date
  - E* testing on selected mixtures underway


Soil
KY CBR currently
Fully soaked sample
Pave ME
Resilient modulus at optimum moisture
Resilient Modulus testing on various soil types across the state (KYTC/KTC)
Currently compiling database of available test results (KYTC/KTC)
New testing not currently being done
What is the soil strength test moving forward, CBR (soaked/unsoaked, Mr, etc.)


Aggregate
Previous modulus testing done by KTC
Currently compiling database of available construction gradations
5-year history from Site Manager Materials
New testing not currently being done


Thank You

Maryland – MEPDG Implementation in Maryland: Traffic, Climate, and Materials

Status of MEPDG inputs - Traffic

- Consultant completed report on traffic implementation program
- Currently, MD does not have enough WIM sites
- Funding to construct additional WIM sites is limited
- Working with Motor Carrier Division (MCD) to develop WIM sites that serve mutual needs.
- Potential to upgrade qualified ATR sites to WIM.

Status of MEPDG inputs – Traffic (Contd.)

- Anticipate using Level 2 for most projects
- Level 1 for high-significance projects
- Level 3 for low-significance projects
- Existing axle load distribution data is of reasonable quality
- Currently estimating traffic volume, percent trucks and vehicle class distribution
- Prep-ME® is envisioned as the primary data processing tool when more WIM data becomes available.

Status of MEPDG inputs - Climate

- 4 weather stations in MD
- 2 are complete, 2 have missing information
- Using data from contiguous states (VA, DC, WV, PA, DE) depending on project location and interpolating climate data
- Will look into MERRA-NASA climate data when it is available
- Will check the June 2015 data (additional 3 to 5 years of weather data is anticipated)

Status of MEPDG inputs – Material Properties

- University of MD completed a study in 2010
- Asphalt binders:
  - Currently no level 1 or 2 data
  - Sensitivity to predicted pavement performance appears <u>to be slight</u>
  - Based <u>only</u> on this criterion, collection of Level 1 or 2 data has <u>little purpose</u> EXCEPT….

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures:
  - Potential for substantial differences in predicted performance using Level 1 vs. Level 2/3 dynamic modulus data, hence there is a need for SHA to collect Level 1 Dynamic Modulus values
  - Input of Level 1 data requires input of Level 1/2 binder data

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - Witczak predictive equation for Level 2/3 dynamic modulus:
    - Not intended for SMA mixtures, a very common premium mix in MD
    - Does not differentiate among different dense graded mixtures

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - AMPT and UTM-25 general purpose test systems for measuring dynamic modulus, creep compliance, and low temperature tensile strength properties.

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - Thermal Cracking generally not a major distress in MD
  - If the appropriate binder grade is specified, the MEPDG does not predict any significant thermal cracking.

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - Hence, Level 3 adequate for Maryland:
    - Creep Compliance
    - Low temperature tensile strength
    - Aggregate Coefficient of thermal contraction
  - Level 3 converts dynamic modulus and other mixture properties to the above three parameters

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - Detailed sensitivity analysis will be conducted during the local calibration efforts to understand the influence of Dynamic Modulus on pavement performance
  - Collecting Dynamic Modulus data as part of the AMPT pooled fund study

Status of MEPDG inputs – Material Properties (Contd.)

- HMA Mixtures (Contd.):
  - Level 3 data for:
    - Thermal conductivity
    - Heat Capacity
    - SSA (Has a major influence on predicted performance, but no method to measure it yet)

Status of MEPDG inputs – Material Properties (Contd.)

- PCC Mixtures:
  - All physical data (w/c ratio, cement type etc.) routinely measured on individual projects
  - No level 1 data. 28-day strength parameters will be measured for JPCP paving projects

Status of MEPDG inputs – Material Properties (Contd.)

- PCC Mixtures (Contd.):
  - Until accepted testing standards available for SSA, Level 3 data acceptable
  - Thermal conductivity can be measured in the lab, but Level 3 acceptable, since this is relatively fixed for PCC

Status of MEPDG inputs – Material Properties (Contd.)

- PCC Mixtures (Contd.):
  - Performance predictions closely agree for Level 1 versus Level 3 inputs for 28-day E_c and MOR, so Level 3 should be suitable for most designs
  - Predicted slab cracking (and consequently IRI) for JPCP appears to be highly sensitive to the input level for E_c and MOR
  - Faulting and LTE insensitive to input level

Status of MEPDG inputs – Material Properties (Contd.)

- PCC Mixtures (Contd.):
  - CTE is an important, but difficult to measure, parameter for pavement performance:
    - Faulting
    - IRI
    - Slab Cracking
  - Weighted average method in MEPDG for Level 2 or Level 3

Status of MEPDG inputs – Material Properties (Contd.)

- PCC Mixtures (Contd.):
  - Shrinkage properties:
    - No acceptable testing protocols for ultimate shrinkage strain, time to 50% shrinkage)
    - Use Level 3 defaults

Status of MEPDG inputs – Material Properties (Contd.)

- Unbound materials:
  - Lot of M_r data available. Continue to collect data in the future
  - Reasonable amount of data for subgrade (A-4, A-6) and subbase (A-2-4)
  - Gaps in data for granular base materials (A-1-a, A-1-b) and poor subgrade soils (A-7-5, A-7-6)

Status of MEPDG inputs – Material Properties (Contd.)

- Unbound materials (Contd.)
  - Level 2/3 data acceptable for hydraulic properties:
    - Little impact on predicted performance
    - Empirical correlations in terms of gradation and plasticity parameters in MEPDG provide sufficient accuracy

Mississippi – Field Data Collection and Evaluation Study

Mississippi Field Data Collection and Evaluation Study

William Barstis, P.E.
Pavement Research Engineer
Research Division
Mississippi Department of Transportation

Introduction

- MDOT and BCD finalizing plans for an upcoming field testing/sampling project
- Objective: Obtain data to locally calibrate performance models in the AASHTOWare Pavement ME Design software
- Some of these models are currently in the software and will be locally calibrated for Mississippi materials and conditions:
  - HMA rutting
  - HMA bottom-up fatigue cracking

Collect Data for Potential Local Calibration of other Performance Models

- Currently in the software but MDOT wants to wait until the gurus agree on them before locally calibrating
  - HMA top-down fatigue cracking
  - Reflection cracking of fatigue and thermal cracks
- Completely new models that may be added in the future:
  - Shrinkage in CSM base layers
  - Reflection of transverse cracks

TRB 2015

- Lectern Session 276 – Incorporation of Cementitiously Stabilized Materials in Mechanistic-Empirical Pavement Design Guide
- Workshop 864 – Recent Advancements in Mechanistic Evaluations of Flexible Pavements

Outline

- Selection of pavement sample sections
- Measure pavement temperature with depth
- Perform FWD tests
- Evaluate pavement distresses
- Obtain samples for laboratory testing
- Site Report

Selection of Sample Sections Based on Typically Constructed Pavement Types (SS 163)

Subgrade Type	Binder Type	Mix Type	Pavement Type											
			Conventional [1]			Semi Rigid [2]			Deep Strength [3]			AC overlay over		
			AC Thickness											
			L	M	H	L	M	H	L	M	H	AC	JCP	CRCP
Non Stabilized	Modified	D [4]												
		S [4]												
	PG	D												
		S												
Stabilized	Modified	D												
		S												
	PG	S												
		D												

Notes: [1] Flexible pavements with thick unbound base/subbase layers
[2] AC pavements with cement- or lime-treated base/subbase layers
[3] AC pavements with asphalt treated base/subbase layers
[4] D: Dense HMA mix, S: Superpave HMA mix

Table 3.4. Test sections selected from the Mississippi LTPP sections.

Sub-grade	Binder	Mix type	AC thickness < 6"	AC thickness 6" – 9"	AC thickness > 9"	AC overlays of AC (surface preparation) Yes	AC overlays of AC (surface preparation) No	AC overlays of AC (surface preparation) Recycled	AC overlays of JCP[6]	AC overlays of CRCP[7]
Conventional Pavements [1]										
Stabilized	PG	D								
		S								
	Mod	D								
		S								
Non Stabilized	PG	D	A310-1, A320-1, A330-1, A350-1				A310-2, A320-2, A350-2		7012-1	
		S								
	Mod	D								
		S								
Semi Rigid Pavements [2]										
Stabilized	PG	D			3094-1		3094-2		3097-1	3099-2
		S								
	Mod	D								
		S								
Stabilized	PG	D	3083-1, 3090-1, 3085-1	3081-1, 3087-1	3089-1, 2807-1	3081-2, 3087-2, 2807-2				
		S								

Non-LTPP Pavement Sections

- 67 Flexible and Semi Rigid pavement analysis sections
- SS No. 263 – Collection and Evaluation of Core Data for the MEPDG for Overlayed and New Pavements
- SS No. 264 – District Traffic Control Support
- SS No. 265 – Research Division Support

Selection of Specific Sample Section

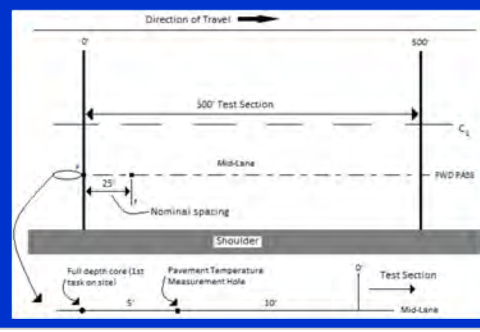
Analysis section – same pavement structure throughout length

Distresses collected on two 500-ft sample sections per mile of analysis section

One 500-ft sample section selected from each analysis section to perform extensive field testing/sampling

Specific 500-ft sample section selected based on average of each of the following three types of cracking:
Alligator, Longitudinal, Transverse

Location of FWD Tests and Pavement Temperature Measurement



FWD - Load and Deflection Data

FWD Tests

- Conducted at 25-ft interval
- Sensor spacing: 0", 8", 12", 18", 24", 36", 48", 60",72"
- Four load levels: 6k, 9k, 12k, 16k
- Two drops at each load level
- Collect time history on second drop at each load level

Use of FWD Test Data in Current MDOT Local Calibration

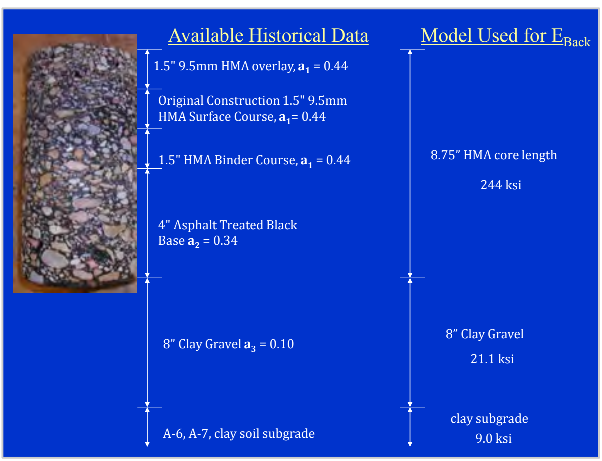
- Peak load and peak deflection data from first drop at each load level used to backcalculate any unbound pavement layers and subgrade modulus values
- Use backcalculated modulus values rather than laboratory derived resilient modulus values for local calibration
- Develop materials library of typical backcalculated modulus values for each type subgrade soil and unbound layer for future pavement design

FWD Time History Data – Potential to Implement Current and Future Research

- 23rd Annual FWD User Group Meeting (Oct 6-8, 2014 in Indianapolis, Indiana)
- Halil Ceylan presented "Backcalculation of Asphalt Concrete Dynamic Modulus Master Curve Coefficients from Enhanced FWD Data Using Neural Networks: A Preliminary Study
- www.FWDUG.org 2014 (Indianapolis, Indiana)

Characterizing Existing HMA Layers for HMA Overlay Design

- Current approach:
- Pages 3.6.39 - 3.6.41 "Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures" (2004)
- "Existing asphalt bound layers will be treated as a single layer in the overlay design".



Data obtained from Cores

- "The Level 1 characterization requires field cores to obtain the undamaged modulus master curve and backcalculated modulus from NDT analysis to obtain initial damage level and damaged modulus master curve".
- Data needed from field cores are then used in Dynamic Modulus predictive equation to establish the undamaged master curve:
  - Air void content, asphalt content, gradation
  - A and VTS parameters for the ASTM viscosity temperature susceptibility relationship determined from recovered binder

Undamaged and Damaged HMA Dynamic Modulus Master Curve

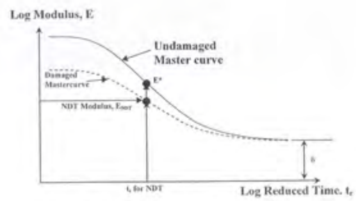


Figure 3.6.12 HMA layer damage computation, Level 1.

Halil Ceylan, et. al. Research

- Development of Asphalt Dynamic Modulus Master Curve Using Falling Weight Deflectometer Measurements, TR-659
- Research by Iowa State University for Iowa Department of Transportation
- Use FWD deflection time history data and in-situ pavement temperature gradient measurements to directly evaluate damaged dynamic modulus master curve
- Eliminate need to characterize undamaged dynamic modulus master curve using core data.

Distress Survey

- Emphasis placed on identifying type and source of cracking
- Mechanistic-Empirical Pavement Design Guide – A Manual of Practice (2008)
- "One reason for the relatively high error terms for both load related fatigue cracking prediction equations (Eqs. 5-7 and 5-9) is that none of the LTPP test sections included in the calibration effort were cored or trenched to confirm whether the fatigue cracks started at the top or bottom of the HMA layers."

Objective of distress survey – Characterize each crack by its specific causal mode and then appropriate to the corresponding performance model

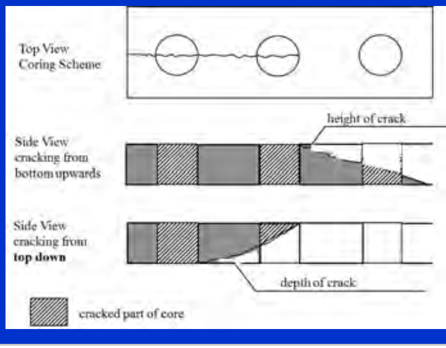
Determine Origin of each Crack

- Bottom-up cracks in HMA layers (originating at bottom of HMA)
- Top-down cracks in HMA layers (originating at top of HMA)
- Thermal cracks in HMA layers
- Block cracking
- Reflected cracks subdivided into origin of the reflected crack; i,e., from the underlying HMA layer, cementitious stabilized material (CSM) base layer, or stabilized subgrade layer

For sample sections including CSM base or stabilized subgrade layer evaluate if reflected crack originates from:

- Fatigue crack within CSM base layer
- Shrinkage crack within CSM base layer
- Fatigue crack within stabilized subgrade layer
- Shrinkage crack within stabilized subgrade layer

Cores to Evaluate Top-Down vs. Bottom-Up Cracking - SS No. 255



Reduce Number of Cores

- SS No. 255, “A Synthesis Study of Noncontact Nondestructive Evaluation of Top-Down Cracking in Asphalt Pavements”
- Objective – identify a nondestructive and noncontact technology operating at highway speed to determine top-down vs. bottom-up cracking
- Findings – “No highway-speed remote sensing technology is available in practice that can scan pavement surface and map crack propagation through the asphalt layer.” (December , 2013)

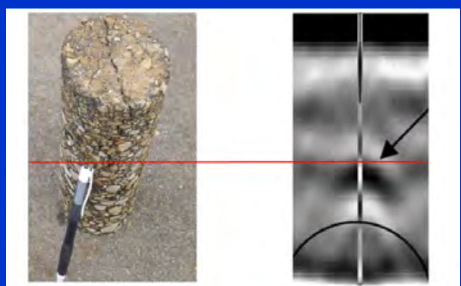
Ground Penetrating Radar (GPR)

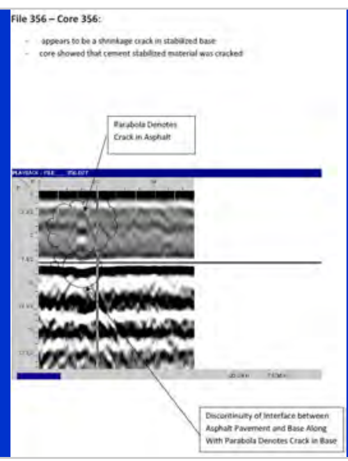
- Dr.Waheed Uddin (University of Mississippi), Robert Varner, and Howard Hornsby (Burns, Cooley, & Dennis, Inc)

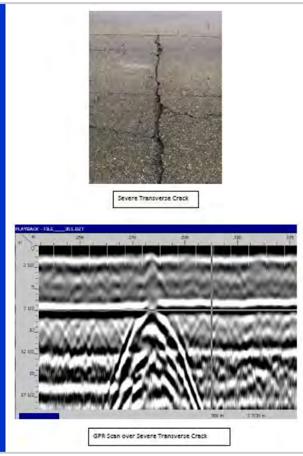
Feasibility Study

- See if BCD’s GPR can be used in current field testing/sampling project to evaluate cracks in each of the 67 sample sections (February 2012)
- Performed on SR 25 in Leake County
- Pavement structure constructed in 2000:
- 2” 12.5 mm HMA
- 2.5” 19 mm HMA
- 3” 25 mm HMA
- 6” lime-fly ash stabilized soil base layer

Correlate Top-Down Crack in Core to Distinguishing Feature within GPR Trace

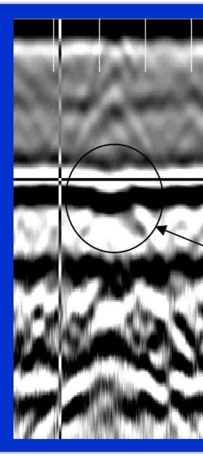






Not Perfect

- Parabola in GPR trace indicates a crack in the base layer
- Core did not reveal a definite crack in that layer
- GPR HMA layer thickness prediction and measured core thickness were the same



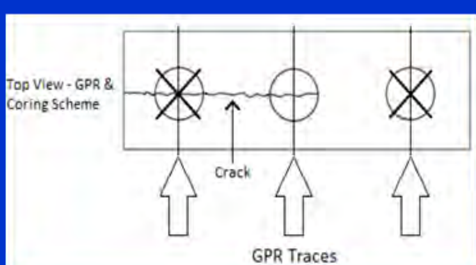
Data Supports use of GPR to:

- Determine if crack extends through full depth of asphalt
- Location of cracks in stabilized base layer
- Location of HMA stripping
- Determine total thickness of HMA

Plan to Characterize Cracks using GPR

- First 5 or 6 sample sections evaluated in project - develop experience base correlating type crack to GPR trace via cores
- Subsequent sample sections – Rely on engineering judgment and GPR to categorize type cracks and maybe use one core per sample section

At least reduce number of cores from 3 to 1

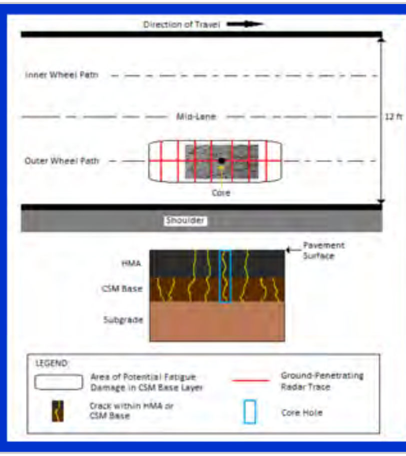


CTB (CSM Base) Layer Fatigue Equation

Equation (5-10b in MEPDG MOP)

FC_{CTB} = Area of fatigue cracking, sq. ft.
Transfer function constants C_1, C_2, C_3, C_4 never calibrated at national level
Primary reason MDOT has to go through local calibration process

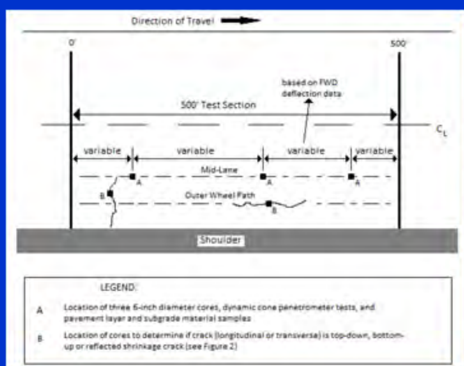
$$FC_{CTB} = C_1 + \frac{C_2}{1 + e^{(C_3 - C_4 Log(DI_{CTB}))}}$$



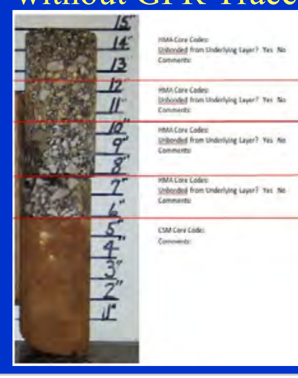
Two Distress Survey Maps

- Surface of pavement
- CSM base layer
  - Shrinkage cracks
  - Fatigue cracks

Core at Select FWD Test Points



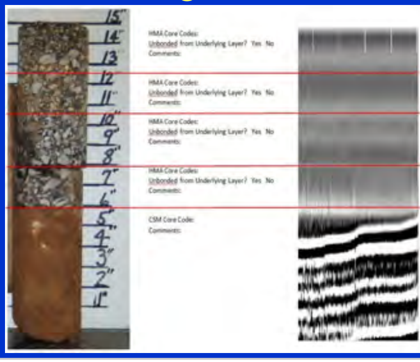
Display of Cored Material without GPR Trace



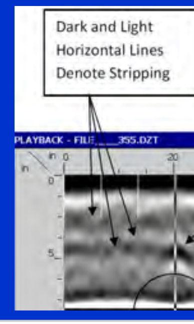
1. Intact LFA Core; Excellent Condition

6. LFA Material Extracted from Core Hole is Soft and Crumbly

Display of Cored Material including GPR Trace



Adjust Material Sampling Location Based on GPR Trace to avoid Stripped HMA Areas



MATERIALS SAMPLING & FIELD TESTING PLAN FOR NEW CONSTRUCTION & OVERLAY PROJECTS – NON-LTPP SECTIONS

Test Section Identification: 4933　Log Mile: 5.964
Highway/Route Number: MS-16　Latitude: N 32.77561°
County: Neshoba　Longitude: W 89.26800°

Pavement Cross Section (Planning Pavement Sampling Operation):

Layer & Material Type		Layer Thickness, inches	
		Plan	Field
1	Original AC Surface – HT 9.5mm Mix	1.5	
2	Original AC Surface – HT 12.5mm Mix	4	
3	Original AC Surface – HT 19mm Mix	4.5	
4	Crushed Stone – Drainage Course	4	
5	Lime-Fly Ash – Class 9, Group C	6	
6	Treated Subgrade – Lime/Lime-Fly Ash	6	
7	Subgrade – A-6, A-7	Infinite	

Material Sampling/Laboratory Testing Plan Summary (Layer thickness should be measured for each layer in the pavement structure during the sampling operation):

Material/Layer Type	Material Properties				
1 – Original AC Surface	T 166 (3) T 209 (1)	T 308 (1)	T 30 (1)		
2 – Original AC Surface	T 166 (3) T 209 (1)	T 308 (1)	T 30 (1)		
3 – Original AC Surface	T 166 (3) T 209 (1)	T 308 (1)	T 30 (1)		Modified AASHTO T 322 (9)
4 – Crushed Stone Drainage Course					
5 – Lime-Fly Ash Base	D 6752 (2)	CSM IDT (2)	D 6951 (1)		
6 – Treated Subgrade	D 6752 (1)	CSM IDT (1)	D 6951 (2)		
7 – Untreated Subgrade	T 265 (3) D 7263 (2)	T 89 (3) T 90 (3)	T 272 (3)	T 88 (3)	D 6951 (3)

Modified AASHTO T 322

- AASHTO T 322 – Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device
- Performed on 9 - 6" diameter specimens from each of 15 sample sections
- Test temperatures – 40 ^{0}F, 60 ^{0}F, 80 ^{0}F
- Rate of loading – 2" per minute
- Record horizontal and vertical deformation time history

$$N_{Allowable} = k_{f1}\left(\beta_{f1}\right)k_1'\left(\frac{1}{\varepsilon_t}\right)^{k_{f2}\beta_{f2}}\left(\frac{1}{E_{ac}}\right)^{k_{f3}\beta_{f3}}$$

CSM IDT

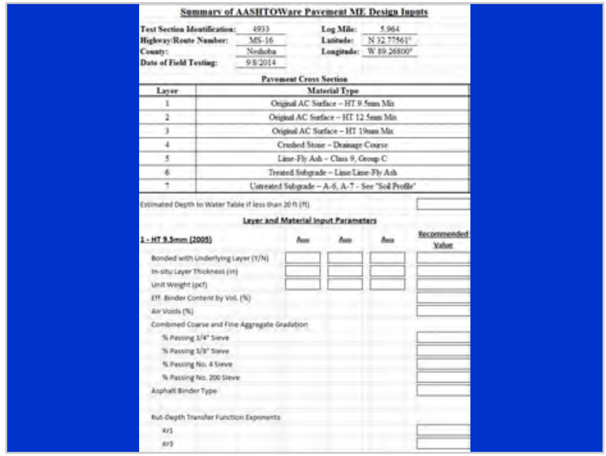
- Will provide indirect tensile strength (IDT)
- Estimate CSM modulus of rupture (MR)
  MR = 2 * IDT
- Estimate unconfined compressive strength (UCS)
  UCS = 8.33 * IDT
- Estimate elastic modulus (E or M_r) from UCS
  - E = 1200 * UCS (soil cement)
  - E = (500 + UCS) * 1000 (lime-fly ash)
  - M_r = (0.124 * UCS + 9.98) * 1000 (lime)

Site Report

- "Summary of AASHTOWare Pavement ME Design Inputs" form
- Soil Profile
- Distress Survey Map of Existing Pavement Surface
- CSM Base Layer Fatigue and Shrinkage Cracking Survey Map
- "Distress Survey for New HMA and HMA Overlay Pavements'' form
- Drainage survey

Site Report continued

- Three figures, one from each of three FWD select coring locations
  - Picture of cored material
  - Core classification(s)
  - GPR trace if collected for given location
- Engineering evaluation
- In-situ pavement temperature measurements
- Figure(s) of GPR trace correlated to type crack if collected for given sample section
- Field and laboratory test results



Summary of AASHTOWare Pavement ME Design Inputs

Test Section Identification: 4933 **Log Mile:** 5.964
Highway/Route Number: MS-16 **Latitude:** N 32.77561°
County: Neshoba **Longitude:** W 89.26800°
Date of Field Testing: 9/8/2014

Pavement Cross Section

Layer	Material Type
1	Original AC Surface – HT 9.5mm Mix
2	Original AC Surface – HT 12.5mm Mix
3	Original AC Surface – HT 19mm Mix
4	Crushed Stone – Drainage Course
5	Lime-Fly Ash – Class 9, Group C
6	Treated Subgrade – Lime Lime-Fly Ash
7	Untreated Subgrade – A-6, A-7 - See "Soil Profile"

Estimated Depth to Water Table if less than 20 ft (ft)

Layer and Material Input Parameters

1 - HT 9.5mm (2005) Recommended Value

Bonded with Underlying Layer (Y/N)
In-situ Layer Thickness (in)
Unit Weight (pcf)
Eff. Binder Content by Vol. (%)
Air Voids (%)
Combined Coarse and Fine Aggregate Gradation
% Passing 3/4" Sieve
% Passing 3/8" Sieve
% Passing No. 4 Sieve
% Passing No. 200 Sieve
Asphalt Binder Type
Rut-Depth Transfer Function Exponents
Kr1
Kr3

QUESTIONS?

Montana DOT Implementation of the AASHTO Mechanistic-Empirical Design Guide in Utah

Montana Department of Transportation- Local Calibration for the Implementation of the MEPDG

Darin Reynolds, P.E., Greg Zeihen, P.E. & Andy White, EIT
MDT Surfacing Design

PAVEMENT DESIGN

The new pavement will be built in the future, on subgrades often not yet exposed or accessible; using materials not yet manufactured from sources not yet identified; by a contractor who submitted the successful "low dollar" bid, employing unidentified personnel and procedures under climatic conditions that are frequently less than ideal.

Why People Visit Montana

Need for a Better Design Method

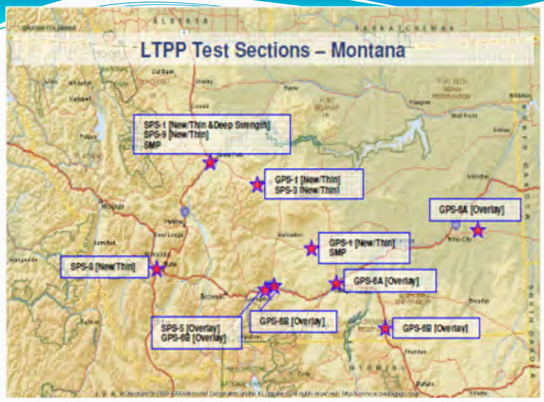
- AASHTO '93 based on limited, site-specific information.
- NCHRP Project 1-37A to use state-of-the-art distress prediction models based on mechanistic-empirical design principles.
- MDT Recognized the need for a new design method, and a need to verify and calibrate models to local conditions.

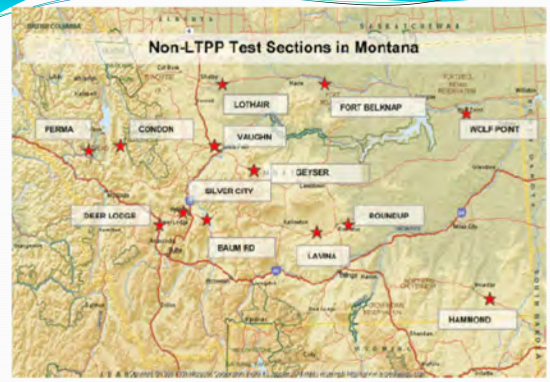
How To Proceed?

- MDT Submitted an RFP for technical assistance to transition to M-E design and develop calibration and prediction strategies compatible with the new NCHRP (AASHTO 2002) design methodology.
- Awarded to Fugro-BRE (now simply Fugro): FHWA/MT-07-008/8158-1 Mechanistic-Empirical Pavement Design Guide Flexible Pavement Performance Prediction Models for Montana: Vol I-III (http://www.mdt.mt.gov/research/projects/pave/pave_model.shtml)

Fugro-BRE 's Plan

- Literature review of distress prediction models
- Develop matrix of pavement types versus climate
  - Site data from 55 LTPP sites in surrounding states and Canada
  - Site data from 34 LTPP sites in Montana
  - Site data from 13 specific pavement types around Montana





Non-LTPP Sites

- Included to address specific pavement types in Montana
  - Superpave mixes
  - Pulverized base
  - Cement-treated base

LTPP and Non-LTPP Sections

Type of Structure	Conventional Flexible	Deep Strength & Full Depth	Includes Drainage Layer	Semi-Rigid Pavement	Pulverized Pavement	HMA Overlay of Semi-Rigid	HMA Overlay	Total Number Test Sections	Total Less SPS-3 Sites
Non-LTPP Test Sections	Baum Road* Lothair E* Silver City W			Geyser E Hammond NW Lavina W Perma Roundup E Wolf Point S	Condon N Deer Lodge/Beckhill	Fort Belknap	Vaughn N*		
	3			6	2	1	1	13	10
LTPP Test Sections	30-0113 30-0114 30-0805 30-0806 30-0901 30-0902 30-0903 30-1001 30-8129	30-0115 30-0116 30-0117 30-0118	30-0119 30-0120 30-0121 30-0122 30-0123 30-0124				30-0502 30-0503 30-0504 30-0505 30-0506 30-0507 30-0508 30-0509 30-0560 30-0561 30-6004 30-7066 30-7075 30-7076 30-7088		
	9	4	6				15	34	34
Total	12	4	6	6	2	1	16	47	44

*SPS-3 (Superpave) Sites: Baum Road, Lothair E, Vaughn N

Materials

- Previously-constructed sections
  - Twenty foot deep bores (10” dia.)
    - Obtain layer thickness
    - Check for water table/rigid layer
    - Sample unbound layers
  - Fourteen cores taken at each site
    - At each end of each 500’ test section
      - One 10-in core
      - Six 6-in cores
  - Jils Falling-weight deflectometer tests every 100 feet

Materials

- Newly constructed sections
  - Sample basic materials
    - Binder
    - Crushed aggregate
    - Plant mix from windrow
    - Crushed aggregate course
    - Subgrade
  - Jils Falling-weight deflectometer tests every 100 feet

Summary of Tests

- Hot-Mixed Asphalt Pavement Layers
  - Indirect Tensile Resilient Modulus
  - Indirect Tensile Strength
  - Low Temperature Creep Compliance
  - Low Temperature Indirect Tensile Strength
  - Aggregate Gradation, Asphalt Content, Max. Theoretical Density, Bulk Density
  - Penetration/Viscosity of Asphalt Cement

Summary of Tests (cont.)

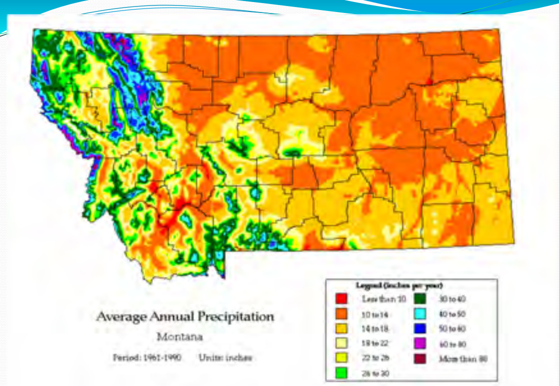
- Unbound Base/Subbase and Subgrade
  - Resilient Modulus
  - Moisture/Density Relationship Using Modified Compaction
- Cement Treated Base
  - Elastic Modulus
    - Calculated from indirect tensile test and seismic test on some samples
  - Compressive Strength

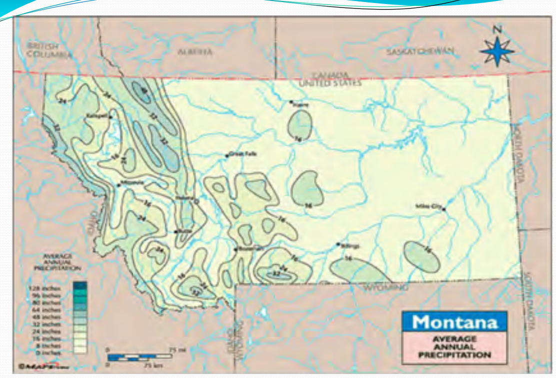
Material Properties Needed for Performance Prediction Modeling

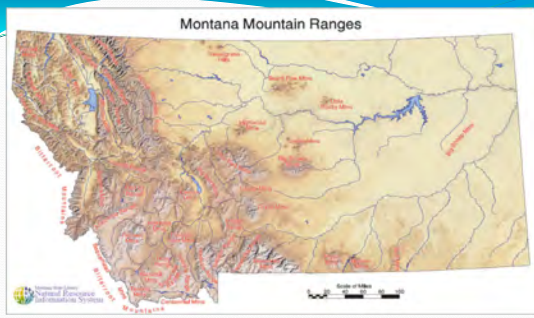
Layer/Material	Property	Input Level	Purpose
HMA	Dynamic Modulus	1	Used for fatigue & rutting predictions.
	IDT Creep Compliance	1,2	Used for transverse cracking predictions.
	IDT Strength	1,2	
	IDT Strain at Failure		Information purposes for calibration.
	Air Voids	1,2,3	Used for calculating dynamic modulus & predicting fatigue cracking.
	Effective Asphalt Content by Volume	1,2,3	
	Density	1,2,3	Used to estimate coefficient of thermal contraction.
	Gradation	2,3	Used for calculating dynamic modulus.
	Asphalt Performance Grade	3	Used to provide default values for other input parameters.
Unbound Aggregate Base & Subgrade Soil	Resilient Modulus	1	Used to predict fatigue cracking and rutting.
	Dynamic Cone Penetrometer	2	Estimate resilient modulus of unbound layers and subgrade.
	Deflection Basins	1,2,3	Determine modulus of layer in existing pavements.
	Gradation	1,2,3	Used to estimate water content over time.
	Water Content	1,2,3	Estimate resilient modulus over time.
	Dry Density	1,2,3	Estimate resilient modulus over time.
	Soil-Water Characteristic Curve	1,2,3	Estimate resilient modulus & changes in water content over time.
	Atterberg Limits	1,2,3	
	Gradation	1,2,3	
	Classification	1,2,3	Used to provide default values for other input parameters.
CTB	Flexural Strength	1,2,3	Used to estimate fatigue cracking.
	Elastic Modulus	1,2,3	Used to estimate fatigue cracking.
	Compressive Strength	2	Used to estimate flexural strength.

Climate

- Montana is a large state: General climate is dry/hard freeze
- Complicated with microclimates
  - Extremely wet-Northwest, Glacier Park vicinity
  - Extremely dry-West-central, Southwest, and east
  - Roads over passes and at high elevations
- Transverse cracks are a significant issue







MEPDG Climate Inputs

- Sensitivity analysis indicated problems with climate data
- Generate more climate files and check climate inputs into MEPDG
- Transverse cracking model did not work well-possibly because of this

Traffic

- Weigh-In-Motion (WIM) sites statewide
  - Collection of spectral data used by new software is standard practice

Threshold Limit Reliability

- MDT still needs to hold a policy discussion to establish design criteria and reliability for routine pavement design with the MEPDG.
- Currently Use Default Values
- Abrasion of Chip Seals is significant in some areas in Montana. In our typical 7 year life of a chip seal, wear contributes to 0.2”-0.4” of PMS Rutting.

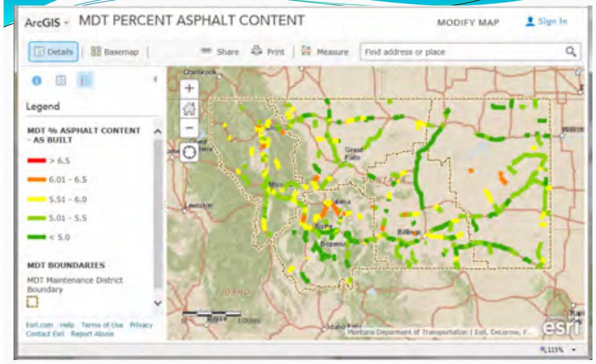
MEPDG Hierarchical Levels

- Level 1 requires laboratory or field material testing
- Level 2 permits inputs from limited testing, agency database, or correlations
- Level 3 has the lowest level of accuracy, with inputs based on national or state averages or engineering experience

Calibration

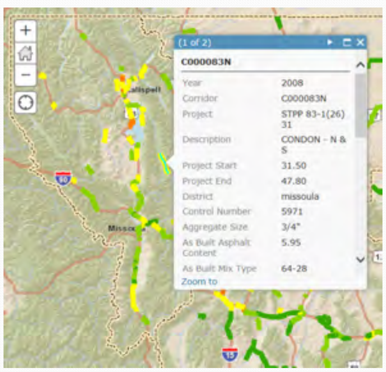
- Fugro-BRE report and MDT sensitivity analysis indicates that local calibration coefficients should be used whenever possible.
- MDT is working on GIS map with asphalt contents for all projects in the state since 2000.

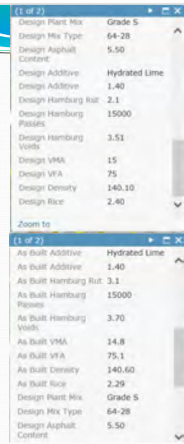
Localized Calibration Coefficients – Historical Mix Information



http://bit.ly/1uxZN4g

Historical Mix Info:





Calibration

-Sensitivity Analysis of 2002 Software

- Ride prediction model gives reasonable results
- Alligator cracking model gives reasonable results
- Top-down cracking model is <u>unreliable</u>
  - Should not be used until reformulated (NCHRP 1-42A)
- Plant mix surface rutting model give reasonable results
- Unbound materials rutting models are <u>unreliable</u>
- Thermal cracking model is <u>unreliable</u>
  - Climate data is suspect
- Cement treated base coefficients should be used with caution, because insufficient fatigue cracking has occurred on existing sections for calibration

Challenges, Issues, and Roadblocks

- Montana has 147,164 square miles with 24,000 lane miles of roads and five transportation financial districts
- Surfacing Design staff:
  - One supervisor – designs for one district and reviews all five districts
  - Two design engineers with two districts apiece
  - Research Position eliminated
  - Retirement/Turnover – loss of knowledge
- Expense of equipment for specialized testing and additional personnel needed

Challenges, Issues, and Roadblocks (cont.)

- Neither the unbound materials rutting nor the longitudinal and transverse cracking models (2002 software) work well in Montana
- MDT's standard practice of chip sealing new pavements, and placing maintenance overlays are difficult to account for in the new modeling software
- MDT uses Hamburg test for rutting and moisture resistance and these results cannot be incorporated into new modeling software (how can we incorporate this into the design software?)
- A significant portion of Montana's network is low volume, for which the 1993 AASHTO works well. Why change?

MDT Sensitivity Analysis

- Design inputs that have a substantial effect on pavement distress prediction
  - AADT
  - Axle Load Spectra
  - Climate
  - PMS Thickness
  - PMS Air Voids
  - PMS Effective Binder Content
  - PMS Poisson's Ratio
  - Pulverized Base Course
  - Base Course Resilient Modulus
  - Subgrade Resilient Modulus
  - Local Calibration Coefficients

MDT Sensitivity Analysis (cont.)

- Design inputs that have a lesser effect on pavement distress prediction
  - Vehicle Class Distribution
  - Depth to Water Table
  - PMS Dynamic Modulus (Surprising?)
  - PMS Gradation
  - Base Course Poisson's Ratio
  - Subgrade Type (???)

Additional Needs

- Gather a substantial amount of PMS project records to determine which mixture properties should be used to **calculate local calibration coefficients** for routine pavement design for both grade D and S mixes. *(Mapping In Progress)*
- Research to better correlate R-Value and resilient modulus
  - R-value is an index test and is twice removed from Mr through CBR
- Develop guidelines for use of FWD resilient modulus
- Obtain multiple data points for seasonal FWD values *(Mapping in Progress)*
- Outside Training for new staff
- Additional Staff

Lessons Learned

- Lost a tremendous amount of MEPDG experience through retirement, turnover. Plan for continuity.
- Research should be done to generate more climate files for the MEPDG in Montana to better represent the different climatic conditions in the state.
  - During sensitivity analysis it was noted that different climate stations had the same data: this may be the reason the thermal cracking model is unreliable.

Status of Transition to MEPDG

- MDT is currently using an in-house developed spreadsheet based on AASHTO 1993. We are waiting on model updates (NCHRP 1-42A).
- MDT is continuing distress surveys of thirteen non-LTPP sites yearly, and periodically conducting FWD tests on them.
- MDT is developing an asphalt map that will have mix design information linked to road sections.

MDT Objectives

- Identify changes to the Method since our Calibration
- Identify staffing requirements of other DOT's
- Identify outside training resources
- Identify Level of Inputs practical for MDT


Questions?
BEGIN
BAUM

Nebraska DOT – Asphalt Modulus Testing and Sensitivity Analysis

INITIAL MEPDG SOFTWARE EVALUATION IN NEBRASKA 2009

Thank You!

- Matt Beran and Lieska Halsey
- They conducted the many runs that helped us evaluate the MEPDG software in 2009.

PCC PROJECTS ANALYZED 2009

- Hwy. 275 – Waterloo NW
- Hwy. 30 – Columbus East
- Hwy. 75 – Nebraska City South
- Hwy. 81 – Geneva N & S
- I-80 – Harrison to Q St.
- Hwy. 2 – Litchfield to Hazard

Using level 3 inputs.

PCC PROJECTS ANALYZED 2009

- Hwy. 275 – Waterloo NW
- Hwy. 30 – Columbus East
- Hwy. 75 – Nebraska City South
- Hwy. 81 – Geneva N & S
- I-80 – Harrison to Q St.
- Hwy. 2 – Litchfield to Hazard

Using level 3 inputs.

Hwy. 275 Waterloo NW

- 2002 – 4 Lane Expressway
- 10" Doweled PCC
- Unbound Foundation Course – 4" Thickness
- 15' Width of Slab
- 16'-6" Long. Joint Spacing
- ADTT = 835

Comparison Existing Condition vs Prediction

	Roadway Condition	MEPDG Prediction
IRI (in/mile)	71	87
Cracking (%)	1	0
Faulting (in)	0.01	0

Varied Thickness 6"-10"

	Trial of different PCC Thicknesses					
	Input	**Run 1**	**Run 2**	**Run 3**	**Run 4**	**Run 5**
	Reliability	85	85	85	85	85
	PCC Layer Thickness	10	9	8	7	6
	Dowel Bar Diameter	1	1	1	1	1
L3	28 day PCC Modulus of Rupture	710	710	710	710	710
L3	28 Day Compressive Strength	-	-	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000	4100000	4100000
35 Years	IRI	119	109	111	120	141
	Cracking	4	5	7	15	35
	Faulting	0.1	0.0	0.0	0.0	0.0
50 Years	IRI	195	149	152	164	197
	Cracking	5	6	11	30	54
	Faulting	0.137	0.0	0.0	0.0	0.0

Vary Properties of Concrete

10" Thickness - Concrete Properties Varied

	Input	Run 14	Run 15	Run 16	Run 17	Run 18	Run 19	Run 20	Run 21	Run 22	Run 23	Run 24	Run 25	Run 26
	Reliability	85	85	85	85	85	85	85	85	85	85	85	85	85
	PCC Layer Thickness	10	10	10	10	10	10	10	10	10	10	10	10	10
	Dowel Bar Diameter	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
L3	28 day PCC MOR	-	-	-	-	710	750	800	800	650	600	600	550	550
L3	28 Day Strength	5000	4500	4000	3500	-	-	-	-	-	-	-	-	-
L3	Elasticity	4.29	4.07	3.83	3.59	4.1	4.1	4.1	4.1	4.1	4.1		4.1	3.5
35 Years	IRI	111	116	124	139	105	102	100	101	113	141	125	203	144
	Cracking	8	11	16	27	4	4	3	4	9	34	17	93	32
	Faulting	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
50 Years	IRI	148	155	168	187	138	133	127	132	150	196	168	242	194
	Cracking	12	19	30	50	5	4	3	5	14	39	31	106	57
	Faulting	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	

Hwy. 30 Columbus East

- 2002 4-Lane Expressway
- 10" Doweled PCC
- Unbound Foundation Course 4" Thickness
- Stabilized Subgrade Type Fly Ash
- 15' Width of Slab
- ADTT=900

Comparison Existing Condition vs Prediction

	Roadway Condition	MEPDG Prediction 85%
IRI (in/mile)	80	96
Cracking (%)	5	0
Faulting (in)	0.07	0

Varied Thickness' 8"-10"

	Input	Run 17.5	Run 14	Run 15
	Reliability	85	85	85
	PCC Layer Thickness	10	9	8
	Dowel Bar Diameter	1.5	1.25	1.25
L3	28 day PCC Modulus of Rupture	710	710	710
L3	28 Day Compressive Strength	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000
35 Years	IRI	152	147	166
	Cracking	4	5	8
	Faulting	0.0	0.0	0.0
50 Years	IRI	197	191	215
	Cracking	5	7	13
	Faulting	0.0	0.0	0.0

Varied Weather Stations

Input	Run 8	Run 9
Reliability	85	85
PCC Layer Thickness	8	8

		1.5" Dowel 15' Slab Norfolk	1.5" Dowel 15' Slab Omaha
35 Years	IRI	147	137
	Cracking	8	7
	Faulting	0.0	0.0
50 Years	IRI	194	180
	Cracking	12	11
	Faulting	0.0	0.0

1.5"Dowels not typical for 8" PCC at NDOR

Hwy. 75 Nebraska City South

- 2003 4-Lane Expressway
- 10" Doweled PCC
- Unbound Crushed Concrete Foundation Course, Thickness 4"
- Lime Stabilized Subgrade
- 15' Slab Width
- ADTT=765

Hwy. 75 Nebraska City South

	Roadway Condition	MEPDG Prediction 85%
IRI (in/mile)	123	93
Cracking (%)	8	0
Faulting (in)	0.04	0

Vary Thickness' 6"-10"

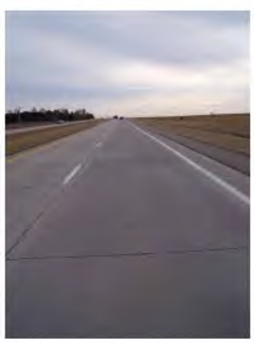
	Input	Run 1	Run 2	Run 3	Run 4	Run 6
	Reliability	85	85	85	85	85
	PCC Layer Thickness	10	9	8	7	6
	Dowel Bar Diameter	1.5	1.25	1.25	1.25	1.25
L3	28 day PCC Modulus of Rupture	710	710	710	710	710
L3	28 Day Compressive Strength	-	-	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000	4100000	4100000
	Additional Comments					Stabilized Soil
85% Reliability						
35 Years	IRI	145	148	151	160	186
	Cracking	4	5	6	[illegible]	[illegible]
	Faulting	0.0	0.0	0.0	0.0	0.0
50 Years	IRI	191	193	196	[illegible]	[illegible]
	Cracking	4	5	9	[illegible]	[illegible]
	Faulting	0.015	0.0	0.0	0.0	0.0

Stabilized Subgrade

	Input	Run 1	Run 5	Run 9
	Reliability	85	85	85
	PCC Layer Thickness	10	10	10
	Dowel Bar Diameter	1.5	1.25	1.5
L3	28 day PCC Modulus of Rupture	710	710	710
L3	28 Day Compressive Strength	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000
	Additional Comments	Subgrade Preparation	Added Stabilized Soil	Same as Run 5 with 1.5 dowels
85% Reliability				
35 Years	IRI	145	147	147
	Cracking	4	4	4
	Faulting	0.0	0.0	0.0
50 Years	IRI	191	194	194
	Cracking	4	4	4
	Faulting	0.015	0.0	0.0

Hwy. 81 Geneva N & S

- 2004 4-Lane Expressway
- 10" doweled PCC
- 4" Foundation Course
- Lime Stabilized Subgrade
- 15' Slab Width
- ADTT=1250



Hwy. 81 Geneva N & S

	Roadway Condition	MEPDG Prediction 85%
IRI (in/mile)	94	92
Cracking (%)	10	0
Faulting (in)	0.04	0

Varied Thickness' 7"-10"

	Input	Run 1	Run 2	Run 3	Run 4
	Reliability	85	85	85	85
	PCC Layer Thickness	10	9	8	7
	Dowel Bar Diameter	1.5	1.25	1.25	1.25
L3	28 day PCC Modulus of Rupture	710	710	710	710
L3	28 Day Compressive Strength	-	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000	4100000
35 Years	IRI	144	147	150	160
	Cracking	4	4	6	16
	Faulting	0.0	0.0	0.0	0.0
50 Years	IRI	189	192	194	212
	Cracking	4	5	9	29
	Faulting	0.018	0.0	0.0	0.0

Compare Stabilized to SG Preparation

	Input	Run 1	Run 5	Run 6	Run 10
	Reliability	85	85	85	85
	PCC Layer Thickness	10	10	10	10
	Dowel Bar Diameter	1.5	1.5	1.5	1.5
L3	28 day PCC Modulus of Rupture	710	710	710	710
L3	28 Day Compressive Strength	-	-	-	-
L3	Modulus of Elasticity	4100000	4100000	4100000	4100000
	Additional Comments	No Stab. Subgrade	Lime Mr = 30000	Lime Mr = 50000	Lime Mr=30000 12' wide
Output					
35 Years	IRI	144	143	143	158
	Cracking	4	4	4	5
	Faulting	0.0	0.0	0.0	0.0
50 Years	IRI	189	189	189	212
	Cracking	4	4	0	7
	Faulting	0.018	0.0	0.0	0.1

I-80 Harrison to Q St.

- 1999 6-lane
- 14" Non-dowelled PCC
- 4" Crushed Concrete Foundation Course
- ADTT=9000

I-80 Harrison to Q St.

	Roadway Condition	MEPDG Prediction 90%
IRI (in/mile)	165*	198*
Cracking (%)	6.5	0
Faulting (in)	0.13*	0.14

*Diamond Ground - Numbers lower now

Non-Dowels vs Dowels

	Input	Run 1	Run 2
	Reliability	90	90
	PCC Layer Thickness	14	14
	Dowel Bar Diameter	-	1.5
L3	28 day PCC Modulus of Rupture	710	710
L3	28 Day Compressive Strength	-	-
L3	Modulus of Elasticity	4100000	4100000
35 Years	IRI	296	220
	Cracking	4	4
	Faulting	0.3	0.2
50 Years	IRI	309	267
	Cracking	5	5
	Faulting	0.427	0.2

Hwy. 2 Litchfield to Hazard

- 1990
- 9" Non-doweled PCC
- Subgrade Preparation
- ADTT=355

Hwy. 2 Litchfield to Hazard

	Roadway Condition	MEPDG Prediction 80%
IRI (in/mile)	137	189
Cracking (%)	7	4.2
Faulting (in)	0.07	0.12

Varied Thickness' and Strengths

	Input	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9
	Reliability	80	80	80	80	80	80	80	80	80
	PCC Layer Thickness	9	10	8	9	9	9	9	9	9
	Dowel Bar Diameter	-	-	-	-	-	-	-	-	-
L3	28 day PCC Modulus of Rupture	710	710	710	710	-	-	-	-	-
L3	28 Day Compressive Strength	-	-	-	-	3500	4000	4500	5000	5500
L3	Modulus of Elasticity	4100000	4100000	4100000	4530911	3586616	3834254	4066840	4286826	4496061
35 Years	IRI	268	231	340	283	346	334	318	[illegible]	285
	Cracking	21	4	80	38	102	[illegible]	73	86	40
	Faulting	0.2	0.2	0.3	0.2	0.2	0.2	0.2	0.2	0.2
50 Years	IRI	[illegible]	289	486	358	395	392	325	374	[illegible]
	Cracking	37	6	47	60	109	103	99	79	62
	Faulting	0.249	0.2	0.3	0.3	0.2	0.2	0.2	0.2	0.3

Summary

- Conducted some preliminary software runs.
- Local calibration is needed. Actual % Panels Cracking is under-estimated in the MEPDG prediction.
- Weather stations selection does effect IRI.
- Without dowel bars, performance criteria can't be attained by changing pavement thickness.
- Our MEPDG runs showed that Stabilizing the Subgrade had little effect on pavement performance.
- Moving forward we need a local calibration done prior to implementing AASHTOware Pavement ME.

Nevada DOT AASHTO ME Implementation Progress in Nevada

AASHTO ME Implementation Progress in Nevada

Michele Maher, P.E.
Yathi Yatheepan, P.E.
Nevada Dept. of Transportation

nevadadot.com

Topics

- Progress in asphalt pavement design calibration efforts
- Progress in concrete pavement design calibration efforts
- Current status of AASHTO ME implementation
- Future efforts

FHWA MEPDG PEER EXCHANGE MEETING
JANUARY 20-22, 2015

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Local Calibration for Asphalt Pavement Design

- University of Nevada, Reno is leading the effort.
- Two phases:
  – Calibration for polymer modified binder
  – Field validation based on available distress and ride data

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Local Calibration for Asphalt Pavement Design

- Calibration for polymer modified binder
  – Nationally calibrated model is based on unmodified and modified asphalt
  – Nevada uses PG 64-28NV/NVTR in the northern part of the state and PG 76-22NV/NVTR in southern part
  – NV-Binder is modified with SBS polymer (2-3% by wt.)
  – NVTR-Binder is modified with tire rubber (Min 10% by wt.)
  – Local calibration for tire rubber modified mix has been initiated recently
  – Models calibration is based on statewide data

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Map of Sampled Projects

Presented by Dr. Elie Hajj during 2014 Nevada Transportation Conference

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Local Calibration for Asphalt Pavement Design

- Progress
  – Completed asphalt and unbound materials rutting model calibration for new construction and overlay
  – Fatigue and cracking models calibration is in progress
  – Lack of fatigued and cracked pavement data on the selected sections is an issue

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Local Calibration for Concrete Pavement Design

- University of Nevada, Las Vegas led the effort
- Based only on two projects on I-15 and I-515 in Southern Nevada
- Two aggregate sources and three mix proportions were used
- Further efforts are necessary to calibrate the models for IRI, distress, and faulting

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Current Status

- AASHTO 93 is still in use for pavement design
- AASHTO ME implementation is in the process
- Darwin 3.1 (AASHTO 93 Design) will be phased out as computers are replaced
- Goal: Full scale AASHTO ME implementation by July 1, 2015

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Future Efforts Needed

- AASHTO ME local calibration/validation is a continuous process
- More rigorous distress data will be collected
- Testing required for MEPDG design that are not currently performed as part of mix design process will need to be in place, or will be collected separately
- Work closely with Traffic Information Division to get the required traffic data

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Questions

- Our Questions
  - Did any states try to incorporate RWIS data into AASHTO ME weather database?
  - Did any states have automated distress data collection in place? If yes, are you satisfied with the quality? Who is the Vendor?
- Your Questions

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JANUARY 20-22, 2015

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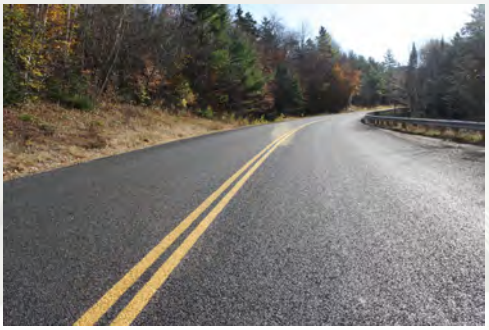
New Hampshire – MEPDG Activities

MEPDG PEER EXCHANGE

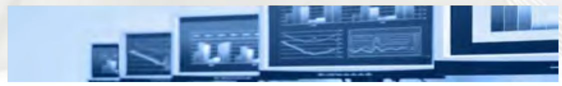
May 13-14, 2015 - Albany, NY



Eric Thibodeau – Pavement Management Chief

MEPDG ACTIVITIES

1. Research Projects
2. Training
3. Comparative Designs



Research Projects

- NETC 06-03 Establishing Default Dynamic Modulus Values for New England (2011)
  - 3-4 mixes from each state (except MA)
- NHDOT RAC Project 14282S (2011)
  - Pavement Instrumentation for local calibration
  - Instrumented a section of divided highway
  - Strain gages, pressure cells, axle sensor array, WIM, moisture probes, RWIS

Research Projects – Cont.

- NETC 06-01 New England Verification of NCHRP 1-37A MEPDG (2011)
  - Included NYSDOT
  - Focused on Level 2 & 3 Inputs only

Training

- FHWA MEPDG Workshop (2011)
  - Focused around NCHRP Version 1.1 software
  - Design Examples

Comparative Designs

- I-93 Salem-Manchester 10418C (2007)
  - Used NCHRP 1-37A Version 1.1 software
    - Thermal cracking predictions inoperable
  - Compared against:
    - AASHO 1972 Interim (Current Practice)
    - AASHTO 1993
    - AI Perpetual Pavement (PerRoad Version 3.0)

New Jersey – Performance Prediction Verification

MEPDG PEER EXCHANGE MEETING
MAY 13th,2015, ALBANY, NY

NJDOT – Performance Prediction Verification

Author: Nusrat Siraj Morshed, P.E.
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VERIFICATION OF ASPHALT CONCRETE PERFORMANCE PREDICTION USING LEVEL 2 AND LEVEL 3 INPUTS OF MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE FOR FLEXIBLE PAVEMENTS OF THE STATE OF NEW JERSEY

Author: Nusrat Siraj Morshed, P.E

Advisor: Yusuf Mehta, Ph.D., P.E.
Associate Professor,
Rowan University

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Robert Blight
Antoinette Morency
Sharad Rana
Eileen Sheehy
Joseph Beke

Outline

- Background
- Problem statement
- Objectives
- Hypothesis
- Research approach
- Literature Review
- Comparisons of results to measured data for
  - Permanent deformation (rutting)
  - Bottom-up fatigue (alligator cracking)
  - Top-down fatigue (longitudinal cracking)
  - Thermal cracking
  - IRI (International Roughness Index)
- Summary
- Conclusion

www.trb.org/mepdg/guide

Background

- M-EPDG is an evolving software
- Regardless of the input level, the damage models remain the same.

www.trb.org/mepdg

Levels of Inputs

Level	Level of accuracy	General Input Sources
Level 1	Highest	Site specific data
Level 2	Intermediate	Agency database
Level 3	Minimal	Default or user defined

- **Problem Statement**

The predicted performance from M-EPDG for New Jersey roads need to be verified before implementation.

- **Objectives**

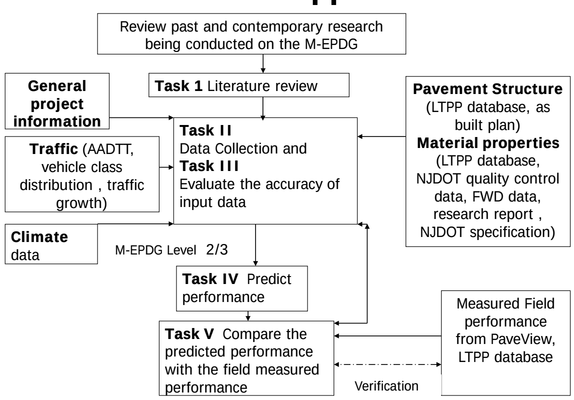
To evaluate the accuracy of the pavement performance predicted in the state of New Jersey using the M-EPDG software with level 2 and level 3 inputs.

To develop the process of verification that can be followed by any state agency or research institution.

- **Hypothesis**

The predicted pavement performance from M-EPDG can be verified using level 2 traffic input and level 3 material input.

Research Approach



Studies conducted on M-EPDG based on field measured data

Author	Conclusions
Muthadi et al., 2008	•The M-EPDG predicted rutting values matched very well with the measured values for the LTPP sections. •The M-EPDG predicted rutting did not match well to the NCDOT measured rutting.
Kang et al., 2008	•Occasionally, distress quantities appeared to increase then drop back down.

The predicted performance that are compared with the measured Performance

- Rutting
- Alligator cracking (Bottom-up fatigue).
- Longitudinal cracking (Top-down fatigue).
- Thermal cracking
- Roughness (IRI - International Roughness Index)

Summary of the Sections

	Section	M.P	AADTT*
North Region	Route 183 S	1.3 - 1.8	365
	Route 94 S	21.8 - 22.3	550
	Route 124 E	4.0 - 4.2	625
	Route 159 E	0.1 - 0.3	728
	Route 23 S (LTPP 1030)	23.9	875
	Route 15 N (LTPP 1003)	10	1463
	Route 139 W	0.4 - 1.1	2170

* Average Annual Daily Truck Traffic

Summary of the Sections Cont.

	Section	M.P.	AADTT
Central Region	Route 64 S	0.0 - 0.2	409
	Route 202 S (LTPP 1033)	4.1	626
	Route 70 W	55.8 - 57.9	739
	Route 35 S	21.4 -21.7	1182
	Route 31 S	8.7 - 9.4	1746
	Route 31 S	5.9 - 6.3	1883
	Route 195 E (LTPP 1011)	10.2	2868
	Route I-195 W (LTPP 0508)	10.8	3300
	Route 95 S (LTPP 6057)	1.2	4740

* Average Annual Daily Truck Traffic

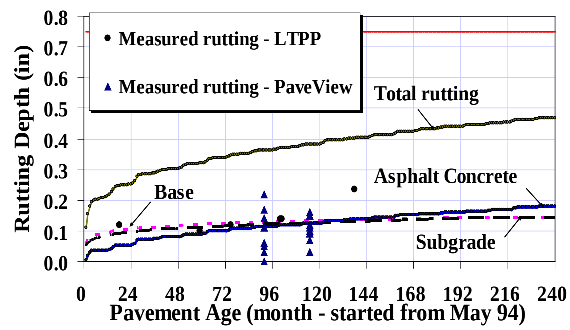
Summary of the Sections Cont.

	Section	M.P.	AADTT
South Region	Route 9 S	45.4 - 48.1	201
	Route 322 W	37.0 - 37.2	532
	Route 322 W	37.3 - 40.8	532
	Route 49 W	3.3 - 5.1	666
	Route 70 E	12.4 - 12.6	1780
	Route 55 N (LTPP 1638)	57.5	2050
	Route 55 S (LTPP 1034)	58.5	2050
	Route 40 E	47.4 - 47.5	2150
	Route 55 N (LTPP 1031)	36.4	2860

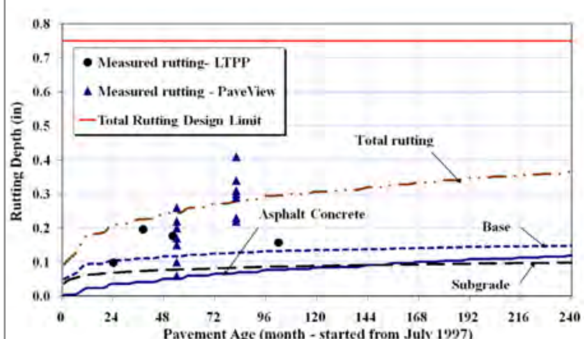
* Average Annual Daily Truck Traffic

Comparison of Measured Rutting vs. Predicted Rutting

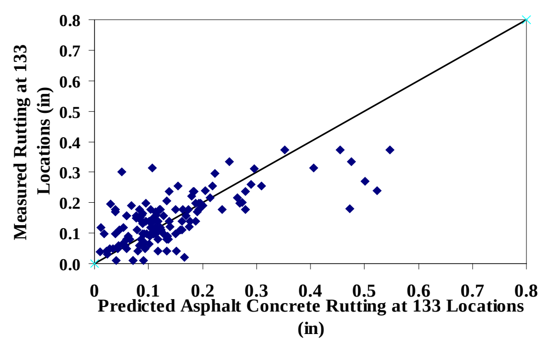
Route 15 (1003)



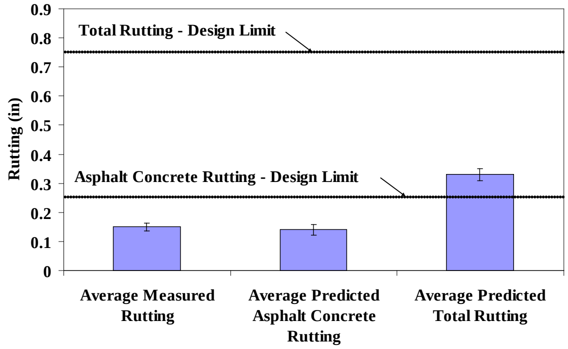
Route 23 S (1030)



Measured and predicted rutting for 25 New Jersey sections



Average measured and predicted rutting for 25 New Jersey sections

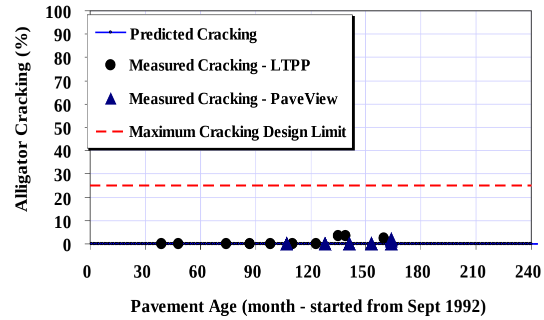


Summary of the analysis - Rutting

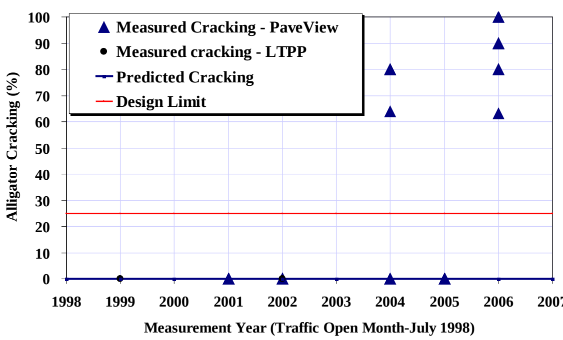
- Due to over prediction of subgrade rutting in the state of New Jersey sections, especially for older sections and with little history of subbase and subgrade rutting (State agency, Personal Communications, 2008), it can be concluded that measured field rutting was reflected primarily in asphalt concrete layer.

Comparison Of Measured Alligator Cracking vs. Predicted Alligator Cracking

Route 195 (LTPP section 0508)



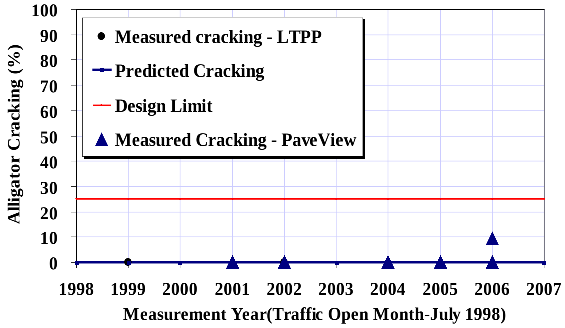
Route 195 (LTPP section 1011) Before correction



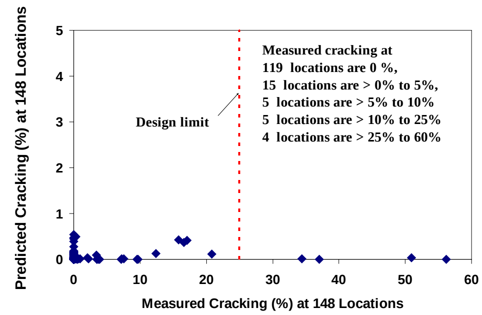
Raw measured PaveView data for LTPP section 1011: 2004

MP	Multiple Crack (Slight) %	Load Multiple Crack (Slight) %
9.7	100	100
9.8	100	100
9.9	100	100
10.0	80	80
10.1	0	0
10.2	0	0
10.3	0	0
10.4	0	0
10.5	0	0
10.6	0	0

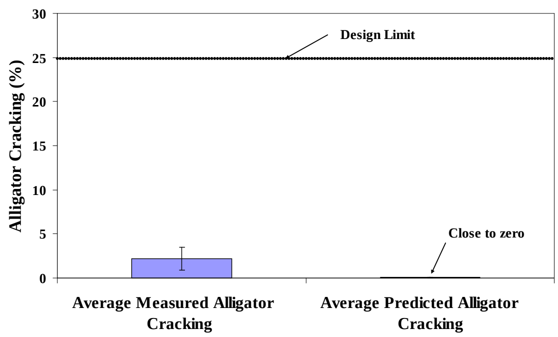
Route 195 (LTPP section 1011) After correction



Measured and predicted alligator cracking for 25 New Jersey sections



Average measured and predicted alligator cracking for 25 New Jersey sections



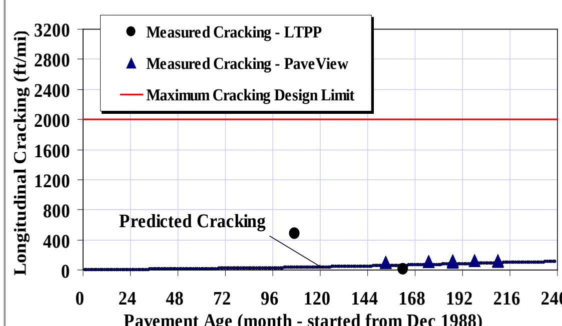
Summary of the analysis - Alligator Cracking

- The standard error of the design guide model based on 461 observations was 6.2% (Design Guide, Appendix II, 2004).
- The standard error of this research based on 148 observations was 0.66% (after correction) and 1.70% (before correction).

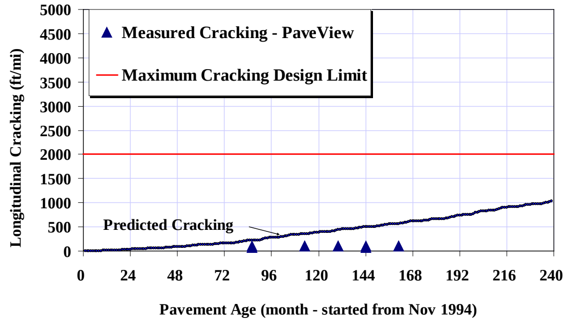
Results

Longitudinal Cracking (Top-Down Fatigue)

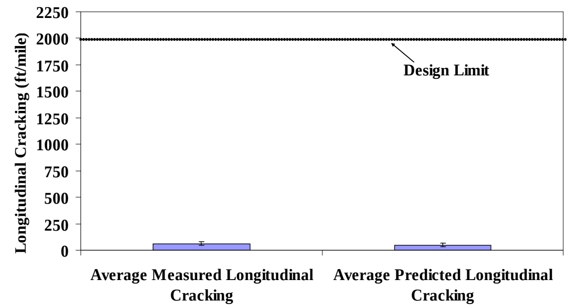
Route 95 (LTPP section 6057)



Route 159 E (MP 0.1 – 0.3)



Average measured and predicted longitudinal cracking for 25 New Jersey sections

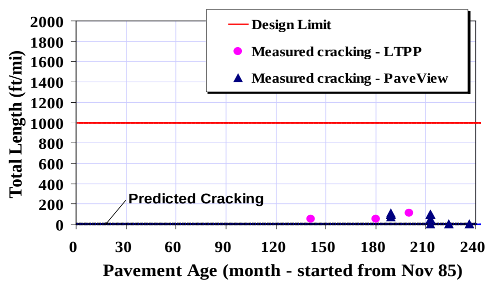


Summary of the analysis - Longitudinal Cracking

- The standard error of the design guide model based on 414 observations was 1242 ft/mile (Design Guide, Appendix II, 2004).
- The standard error of this research based on 145 observations was 12 ft/mile.

Comparison Of Measured Thermal Cracking vs. Predicted Cracking

Route 55 S (1034)



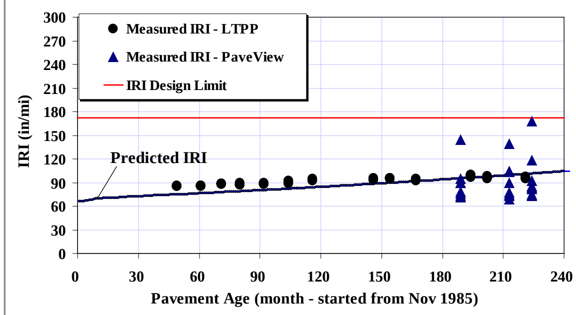
Summary of the analysis - Thermal Cracking

- The level 3 prediction error of the model based on 156 observations was 86.5 ft/mile (Design Guide, Appendix HH, 2004).
- The prediction error of this research based on 144 observations was 6.1 ft/mile.

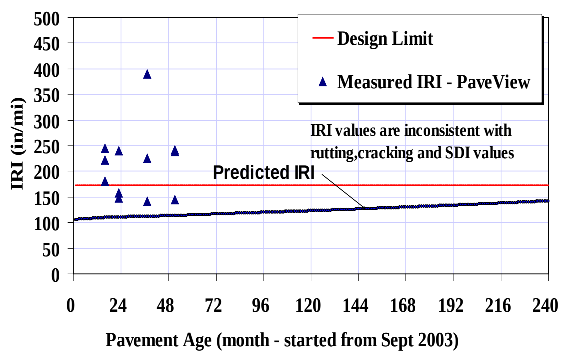
Results

Roughness

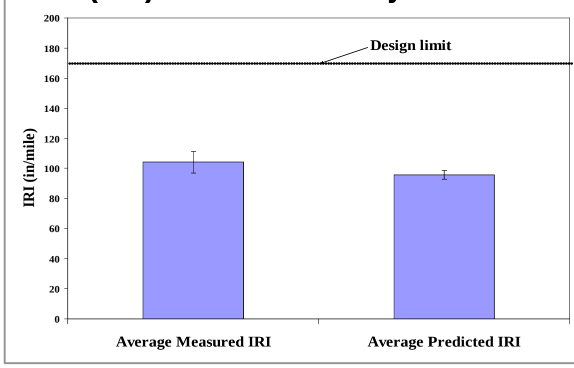
Route 55 S (LTPP section 1034)



Route 35 (MP 21.4 - 21.7)



Measured and predicted roughness (IRI) for 25 New Jersey sections



Conclusion

- The rutting, longitudinal cracking, thermal cracking and roughness (IRI) predicted performance from M-EPDG is verified for level 2 traffic input and level 3 material input for the state of New Jersey.
- The alligator cracking predicted performance from M-EPDG is not verified statistically for level 2 traffic input and level 3 material input for the state of New Jersey.
- The challenges of verification process using field measured data is demonstrated.
  - This process of verification will provide a tool to the other state agencies and other researchers to facilitate the process of verification.

Questions

New Mexico State of State Asphalt Testing and Calibration

FHWA SOUTHWEST STATES MEPDG PEER EXCHANGE MEETING

NMDOT – State of State Asphalt Testing and Calibration

New Mexico Department of Transportation

Jeffrey S. Mann, PE
State Pavement Engineer
Pavement Management and Design Bureau
NMDOT
January 20, 2015

NMDOT MEPDG History

- Pre 2006
  - John Tenison – Materials Bureau Chief largely drove effort
    - Presented to Executive Implementation Plan
    - Marginal Support
      - Minimal commitment to testing, calibration, ect to implement MEPDG
    - LTPP Involvement
    - Retires ~2004(2005)
- Post 2006
  - Several Research Projects

NMDOT MEPDG History

- Post 2006
  - Implementation Plan Developed 2006 – 2007 Through NMDOT Pavement and Materials Committee
    - Recognized that NMDOT Materials Lab not sufficient
  - 2007 – MEPDG Database (MEPDG V 0.9)
    - UNM tasked to collect materials, traffic, climate data to develop database to support MEPDG. Some initial rutting calibration.
  - 2008 – Fatigue Endurance Limit (FEL) and Perpetual Pavement
    - UNM tasked to determine fatigue endurance limit – essentially verified what other research has determined
  - 2009 – Traffic Data Collection and Reporting
    - UNM tasked to review NMDOT process and procedures consistent w TMG and make recommendations for improvement to meet MEPDG requirements
  - 2010 – Dynamic Modulus and Resilient Modulus of Soils
    - Initial Testing of HMA mixes local to Albuquerque and some resilient modulus testing

NMDOT MEPDG History

- Post 2006
  - 2010 - Study and Evaluation of Materials Response in Hot Mix Asphalt Based on Field Instrumentation
    - Instrumentation Section I40, MP 141, west of Albuquerque
    - Measuring load and temperature impacts to full depth HMA section
    - Developing WIM program to interpret and develop Axle Load Spectra based on current TMG
    - Calibration Effort
  - 2012 – Statewide E* and G* Testing
    - 54 Mixes tested in laboratory (ongoing).
  - 2013 – Advanced Statewide Calibration
    - 1 Project Per District within State of NM
    - Materials, construction data collection and lab testing
    - Monitor performance through Pavement Management
  - 2013 – Optimal Use of FWD and GPR
    - Purchase GPR Equipment (3 Units, 400 mHz, 900 mHz and 2 gHz)
    - Evaluate the use of FWD and provide procedural improvements for use with MEPDG
  - 2013 – Pavement Management Combined with Pavement Design
    - Pavement Management and Design Bureau

NMDOT Current MEPDG Policy and Moving Forward

- MEPDG 2.1
  - Side by Side Comparison with 1993 Designs
  - Level 3 with some Level 2 inputs
    - Specifically Traffic Volume Data
    - Specific Binder Data
  - Design Manual of Practice in Progress
  - 2015 (HOPEFULLY) Implementation
- Moving Forward
  - Creep Compliance
  - CTE – further concrete studies
  - More WIMS
  - Improvement in Climatic Data
    - ME has only 9 NM sites
  - Continued FWD Usage
  - Continued Calibration
  - HMA Rehabilitation – Mechanical Properties of CIR, FDR, Cold Central Plant Recycling
  - Thin Bonded Overlays
  - Mechanical Stabilization (Geogrid – Biaxial, Triaxial)
  - OGFC

Dynamic Modulus Testing of NMDOT Superpave Mixes for the Implementation of Mechanistic-Empirical Pavement Design Guide

Input Levels in MEPDG for AC Material Characterization

Level 3

Requires the designer to estimate the most appropriate design input values for the material property based on experience with little or no testing.

Level 2

Estimated through correlations with other material properties that are measured in the laboratory or field.

Level 1

The most current implementable procedure available, normally involving comprehensive laboratory or field tests.

Advantages Includes:

Greater flexibility for the engineer consistent with the size, cost, and overall importance of the project.

Allows agencies to develop initial design methodology consistent with its internal technical capabilities.

Provides a very convenient method to increase an agency's technological skill gradually over time.

Ensures the development of the most accurate and cost efficient design consistent with agency financial and technical resources.

Input Levels in MEPDG for AC Material Characterization

MEPDG Input Levels	Description
1	• Conduct E* (dynamic modulus) laboratory test (NCHRP 1-28A) at loading frequencies and temperatures of interest for the given mixtures • Conduct binder complex shear modulus (G*) and phase angle testing on the proposed asphalt binder (AASHTO T315) at ω = 10 rad/s (1.59 Hz) over a range of temperatures. • From binder test data estimate Ai-VTSi for mix compaction temperature. • Develop mastercurve for asphalt mixture that accurately defines the time-temperature dependency including aging.
2	• No E* laboratory test required. • Use E* predictive equation. • Conduct G*-δ on the proposed asphalt binder (AASHTO T315) at ω = 10 rad/s (1.59 Hz) over a range of temperatures. The binder viscosity or stiffness can also be estimated using conventional asphalt test data such as Ring and ball softening point, absolute and kinematic viscosities, or using the Brookfield viscometer. • Develop Ai-VTSi for mix compaction temperature. • Develop mastercurve for asphalt mixture that accurately defines the time-temperature dependency including aging.
3	• No E* laboratory test required. • Use E* predictive equation. • Use typical Ai-VTSi values provided in the Design Guide software based on PG, viscosity, or penetration grade of the binder. • Develop mastercurve for asphalt mixture that accurately defines the time-temperature dependency including aging.

Objectives of the Research Project

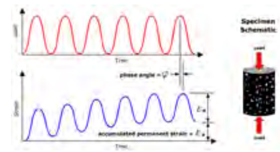
- Collect asphalt mixtures and binders from Department sources.
- Compact asphalt samples to be representative of the actual field compaction level.
- Conduct *E** tests in the laboratory.
- Determine *E** master curves and shift factors
- Conduct frequency sweep dynamic shear tests in the laboratory
- Develop a Microsoft Excel® spreadsheet and populate the spreadsheet with statewide materials dynamic modulus data, mix volumetric data, and asphalt binders data to calibrate an existing E* predictive equation or develop a new E* predictive equation of Department mixes for MEPDG Level 2 analysis.

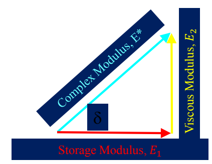
Introducing Dynamic Modulus (*E**)

- Represents the time-dependent stiffness characteristic
- Main input property of HMA in MEPDG
- It is the ratio of peak stress to peak recoverable strain under oscillatory loading
- *E** can be decomposed in to storage and loss moduli
- Storage Modulus: measures the elastic portion of the response $E' = \frac{\sigma_0}{\varepsilon_0}\cos\delta$
- Loss Modulus: measures the viscous response / the energy dissipated as heat $E'' = \frac{\sigma_0}{\varepsilon_0}\sin\delta$
- Complex Modulus is then:

$$E^{*\prime} = E' + i\,E''$$

- Dynamic modulus = absolute value of complex modulus $|E^*| = \sqrt{(E')^2 + (E'')^2} = \sqrt{\left(\frac{\sigma_0}{\varepsilon_0}\right)^2(\sin^2\delta + \cos^2\delta)} = \frac{\sigma_0}{\varepsilon_0}$
- And the Phase Angle: $\tan\delta = \frac{E''}{E'}$





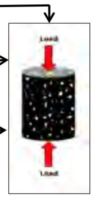
AASHTO T 342

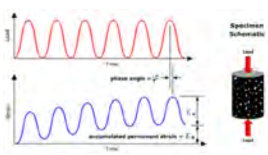
- AASHTO T 342 – "Standard method of test for determining dynamic modulus of hot-mix asphalt concrete mixture".
- AASHTO T 342 Fundamentals:

Cyclic Loading

Test Frequencies: 0.1, 0.5, 1, 5, 10, and 25 Hz

Test Temperatures: 10, 4.4, 21.1, 37.8, and 54.4˚C



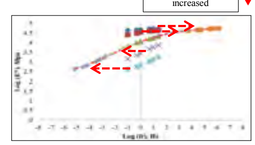


- The test method can be used to determine both dynamic modulus and phase angle

Development of Dynamic Modulus Master Curve

- Time-Temperature Superposition Principle
  - Is a concept typically used to determine temperature-dependent mechanical properties of linear viscoelastic materials like asphalt concrete from known properties at a reference temperature.
  - For LVE material, the curves of the instantaneous modulus as a function of time or frequency for asphalt concrete do not change shape as the temperature is changed but appear only to shift left or right.
  - This facilitates the idea that a mastercurve at a given temperature can be used as the reference to predict the modulus at various temperatures by applying only a shift operation.

elastic moduli of asphaltic materials → Increase with loading rate or frequency / decrease when the temperature is increased



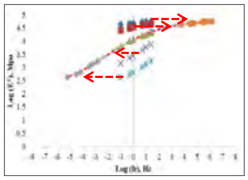
- The application of the time-temperature superposition principle typically involves-
  - Experimental determination of loading frequency-dependent curves at several temperatures for a small range of selected frequencies.
  - The computation of a translation factor to correlate these properties for the temperature and frequency range
  - Development of a mastercurve based on experimental data showing the effect of frequency for a wide range of frequencies
  - The application of the translation factor to determine temperature-dependent moduli over the whole range of frequencies

Construction of Dynamic Modulus Master Curve

- The amount by which dynamic modulus data is shifted to fit in a smooth curve at a reference temperature is referred to as shift factor, $a(T)$.
- Shifting is achieved by dividing the loading time in the time domain or multiplying the loading frequency in the frequency domain by the shift factor to get the reduced time or reduced frequency.

Reduced frequency: $f_r = f * a(T)$; $\log(f_r) = \log(f) + \log[a(T)]$

Otherwise, reduced time: $t_r = \dfrac{t}{a(T)}$; $\log(t_r) = \log(t) - \log[a(T)]$



- Shifting is achieved by dividing the loading time in the time domain or multiplying the loading frequency in the frequency domain by the shift factor to get the reduced time or reduced frequency.
- Mastercurve can be developed for any reference temperature chosen arbitrarily.
- At reference temperature the shift factor is 1 and therefore its logarithm is zero.
- Advantages of Mastercurve involves:
  - It reduces the three dimensional data (dynamic modulus, loading time/frequency and temperature) in to two dimensional data by eliminating the temperature variable. This makes it easy to compare test results conducted at different conditions.
  - The possibility of interpolation to get intermediate data within the test data range.
  - Evaluating other material functions (i.e. relaxation modulus or creep compliance) by interconversion technique.

The Experimental Shift Factor Function & Dynamic Modulus Mastercurve Fitting

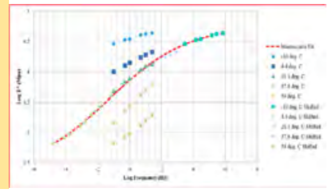
- Several equations available to fit shift factor trend with temperature. The most common, relatively easy one is the Second degree polynomial.

$$\log a_T = aT^2 + bT + c$$

- The function that is predominantly used for developing mastercurve for dynamic modulus data is the sigmoid function.

$$\log(|E^*|) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log(f_r)}}$$

- δ is minimum modulus value
- α is span of modulus values
- β, γ are shape parameters
- The parameter, γ indicates how steep the function is i.e. how fast the modulus is changing from the minimum value to the maximum.
- β represents the horizontal position at which the rate of change of modulus changes from positive to negative.
- δ is associated with the minimum value of asphalt mix modulus generally caused by high temperature.
- The largest modulus which is associated with binder modulus at very low temperature is represented in the sigmoidal function by the sum of parameters δ and α.

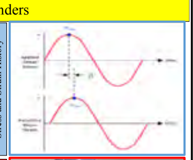


Frequency Sweep Dynamic Shear Tests on Typical NM Asphalt Binders

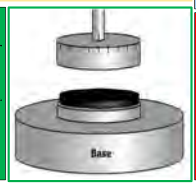
DSR

- DSR measures the complex shear modulus (G^*) and phase angle (δ) of a binder specimen.
- Complex shear modulus (G^*) is the total resistance to deformation of the binder specimen when repeatedly sheared.
- The phase angle (δ) is the time-lag between the applied shear stress and the resulting shear strain.
- DSR test is used to characterize the elastic and viscous behavior of asphalt binder at medium to high temperatures.
- Test temperature is determined by the actual anticipated temperature of the region where the binder will be used.
- DSR test uses a thin asphalt binder sample sandwiched between two circular plates.
- The lower plate is fixed.
- The upper plate oscillates back and forth across the sample at a specified frequency to create a shearing action.
- As a standard practice, the specified loading rate of 10 rad/second (1.59 Hz) is used to simulate the shearing action corresponding to a traffic speed of 55 mph (90 km/hr).

Stress and Strain History



Binder with top and bottom plate



DSR Setup

The DSR Test Standard: AASHTO T 315

- AASHTO T 315 suitable for use when the dynamic shear modulus varies between 100 Pa and 10 MPa.
- This range in modulus is typically obtained between 6 and 88°C at an angular frequency of 10 rad/s, dependent upon:
  - **The grade,**
  - **Test temperature, and**
  - **Conditioning (aging) of the asphalt binder.**
- The test temperature, specimen size and plate diameter depend upon the type of asphalt binder being tested.
- Unaged asphalt binder and rolling thin-film oven (RTFO) residue are tested at the high temperature using a specimen of 1 mm thick and 25 mm in diameter.
- PAV residue is tested at lower temperatures. These lower temperatures make the specimen quite stiff, which results in small measured phase angles (δ). Therefore, a thicker sample, 2 mm in thickness with a smaller diameter of 8 mm is used so that a measurable phase angle (δ) can be determined.
- Again, test temperatures greater than 115°F (46°C) use a sample 1 mm thick and 25 mm in diameter. On the other hand, while the test temperatures are in between 39°F and 104°F (4°C and 40°C), a specimen with 2 mm in thickness and 8 mm in diameter is used.
- The required stress or strain amplitude depends upon the value of the complex shear modulus of the asphalt binder being tested.
- Stress amplitudes have been selected to ensure that the measurements are within the region of linear viscoelastic behavior.
- The test specimen in maintained at the test temperature to within ±0.1°C.

The G* Mastercurve

- $|G^*|$ test is conducted in a <u>strain controlled mechanism.</u>
- The shear stress is measured by applying a <u>preselected strain level.</u>
- The <u>applied strain level used was 1.0%.</u>
- This was selected so that the <u>strain level must be measureable</u> to the DSR compliance while <u>taking in to consideration the maximum stress that can be applied by the DSR equipment.</u>
- The Dynamic Shear Modulus MC Fitting Equation

$$\log(|G^*|) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log(f_r)}}$$

  - α, β, γ , and δ are the fitting parameters, and f_r is the reduced frequency.

- Shift Factor Fitting Equation for $|G^*|$ MC

- Williams-Landel-Ferry equation (WLF) equation is an empirical equation associated with time-temperature superposition of the $|G^*|$ data.

$$\log a_T = -\frac{C_1(T - T_r)}{C_2 + (T - T_r)}$$

- T_r is the reference temperature, C_1 and C_2 are positive constants that depend on the material and the reference temperature.

E* Predictive Equation in MEPDG

Bitumen viscosity | Loading frequency | Air void content | Asphalt content

$$\log|E^*| = 3.750063 + 0.029232\,\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\,\rho_4 - 0.058097\,V_a - 0.802208\left(\frac{V_{beff}}{V_{beff} + V_a}\right) + \frac{3.871977 - 0.0021\,\rho_4 + 0.003958\,\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\,\rho_{34}}{1 + e^{(-0.603313 - 0.313351\log(f) - 0.393532\log(\eta))}}$$

Aggregate gradation parameters

- The equation above is nationally calibrated for the level 3 input in MEPDG.
- For more accurate Level 2 MEPDG design or analysis, new model or calibration of existing model is required.
- This requires a number of HMA or WMA sample used by the Department to be tested in the laboratory for E*.
- Thus, ensuring enough sample tested for the region, a new model or calibrated existing model can be developed.

Test Matrix for Current Research Project

- Test Matrix for E* Testing

Superpave (SP) mixes with Nominal Maximum Size (NMS) of the aggregate	Mix or aggregate Sources with typical mineral aggregate	Performance grade (PG) of asphalt binders
1. SP – II (NMS = 25.0 mm) 2. SP – III (NMS = 19.0 mm) 3. SP – IV (NMS = 12.5 mm)	1. District 1 – typical of basalt, limestone 2. District 2 – typical of caliche aggregates 3. District 3 – typical of limestone 4. District 4 – typical of granite, limestone 5. District 5 – typical of rhyolite, granite 6. District 6 – typical of limestone	1. PG 64-22 or lower 2. PG 70-22 3. PG 76-22 or higher
Total Number of Mixes (target) = 3 SPs x 6 Aggregate Sources x 3 PGs = $\boxed{\text{54 Mixes}}$		

Test Matrix for Current Research Project

- Test Matrix for G* Testing

No.	Performance Grade (PG)	Source
1	PG 64-22	HollyFrontier Refining and Marketing LLC, or if not available, from other source depending on the technical panel discretion
2	PG 70-22	HollyFrontier Refining and Marketing LLC, or if not available, from other source depending on the technical panel discretion
3	PG 76-22	HollyFrontier Refining and Marketing LLC, or if not available, from other source depending on the technical panel discretion
4	PG 64-22	NuStar Energy or Western Refining, or if not available, from other source depending on the technical panel discretion
5	PG 70-22	NuStar Energy or Western Refining, or if not available, from other source depending on the technical panel discretion
6	PG 76-22	NuStar Energy or Western Refining, or if not available, from other source depending on the technical panel discretion

The E* Database

- The objective is to develop a Microsoft Excel® spreadsheet and populate the spreadsheet with statewide materials dynamic modulus data, mix volumetric data, and asphalt binders data to calibrate an existing E* predictive equation or develop a new E* predictive equation of Department mixes for MEPDG Level 2 analysis.
- Up to this quarter, 12 different asphalt concrete mixes are incorporated in the database.
- The developed database includes 1 main module listing all the samples, and 6 sub-modules for six NMDOT districts.

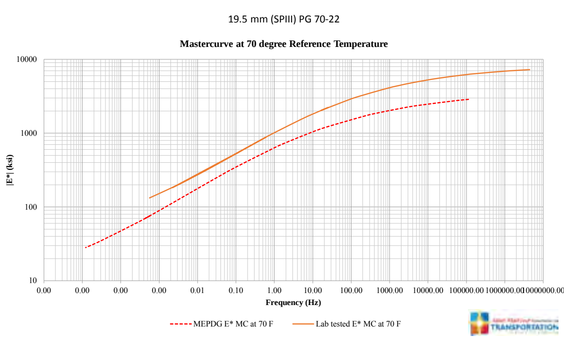
Main Module

Typical Sub-module

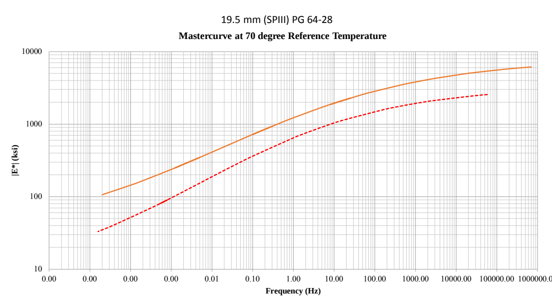
Mastercurve Parameters of all the Mixes Tested

District ID	Super-pave Gradation	Binder PG grade (Specified/Used)	Mix Type	RAP Fraction (%)	Number of Specimen tested	Specimen ID	Maxi-mum Specific Gravity	Bulk Specific Gravity	Air Void %	Asphalt Content (%)	α	β	δ	γ
D-1	SP – IV	76-22/70-22	WMA	35.0	3	1 2 3	2.489	2.341 2.339 2.342	6.0 6.0 5.9	4.7	2.7755	-0.8565	2.055	-0.455
D-4	SP – III	70-22/70-22	HMA	0.0	5	1 2 3 4 5	2.478	2.352 2.344 2.347 2.334 2.352	5.1 5.4 5.3 5.8 5.1	4.2	2.45	-0.3	2.37	-0.53
D-6	SP – III	70-22/70-22	HMA	0.0	3	1 2 3	2.488	2.348 2.341 2.339	5.6 5.9 6.0	4.4	2.80	-0.65	2.0	-0.40
D-3	SP - III	76-22/70-22	HMA	35.0	3	1 2 3	2.573	2.444 2.444 2.439	5.0 5.0 5.2	4.4	2.45	-0.52	2.32	-0.4
D-2	SP - III	70-22/58-28	HMA	35.0	3	1 2 3	2.471	2.323 2.330 2.323	6.0 5.7 6.0	4.5	1.74	-0.55	2.80	-0.68
D-3	SP-IV	70-22/64-22	HMA	25.0	3	2 3 4	2.424	2.279 2.279 2.298	6.0 6.0 5.2	5.0	2.05	-0.40	2.55	-0.50
D-5	SP-IV	70-22/70-22	HMA	25.0	3	2 3 4	2.424	2.279 2.279 2.298	6.0 6.0 5.2	5.0	2.06	-0.41	2.54	-0.50
D-5	SP - III	58-28/58-28	HMA	30.0	3	4 5 7	2.510	2.362 2.359 2.364	5.9 6.0 5.8	4.1	2.62	-0.45	2.27	-0.42
D-1	SP – III	76-22/64-28	WMA	35.0	3	4 5 6	2.348	2.217 2.228 2.214	5.6 5.1 5.7	5.7	2.62	-0.8565	2.13	-0.42
D-6	SP - III	76-28/76-28	WMA	0.0	3	2 3 5	2.407	2.272 2.287 2.287	5.6 5.0 5.0	5.8	2.11	-0.10	2.50	-0.60
D-6	SP - III	76-28/76-28	HMA	15.0	3	3 4 5	2.492	2.342 2.345 2.342	6.0 5.9 6.0	4.9	2.57	-0.50	1.30	-0.45
D-4	SP - III	64-28/64-28	HMA	0.0	3	1 2 6	2.564	2.410 2.410 2.410	6.0 6.0 6.0	5.6	2.27	-0.20	1.51	-0.50
D-4	SP – III	70-22/70-22	WMA	0.0	3	1 2 3	2.459	2.317 2.310 2.312	5.8 6.0 6.0	4.5	2.62	-0.81	1.25	-0.50
D-4	SP - III	76-22/76-22	WMA	0.0	3	1 2 3	2.478	2.337 2.347 2.346	5.7 5.3 5.3	4.2	3.06	-0.95	0.98	-0.35

Level 3 E* Comparison to Level 1 E* Lab



Level 3 E* Comparison to Level 1 E* Lab



Study and Evaluation of Materials Response in Hot Mix Asphalt Based on Field Instrumentation

MEPDG Background

Mechanistic-Empirical → AASHTO PAVEMENT ME

Purely Empirical → AASHTO (1993)

Primary Inputs
- Individual traffic
- Moduli of all layers
- Climate

Design Criteria
- Separate criteria for each type of damage

Primary Inputs
- Generalized Traffic
- Modulus of soil

Design Criteria
- Serviceability Index

Bottom-Up Alligator Cracking

Actual wheel load, *i* =period, *j*=load group

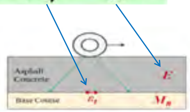
$$D=\sum_{i=1}^{p}\sum_{j=1}^{m}\frac{n_{i,j}}{N_{f,i,j}}$$

Related to amount of alligator cracking (sq-ft)

Allowable load

Local Parameters, need to be calibrated.

$$N_f = 0.00342(10^M)(0.007566)\beta_{f1}\left(\frac{1}{\varepsilon_t}\right)^{3.9492\beta_{f2}}\left(\frac{1}{E}\right)^{1.281\beta_{f3}}$$



Rutting

Asphalt Layer	Unbound Layers

f(depth) Temperature Number of load

$$\frac{\varepsilon_p}{\varepsilon_r}=k_z\beta_{r1}10^{-3.1552}T^{1.734\beta_{r2}}N^{0.39937\beta_{r3}}$$

Local Parameters, need to be calibrated.

intercept strain f(water, Modulus) f(water)

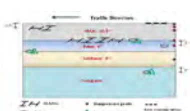
$$\Delta_p=\beta_{s1}k_1\varepsilon_v h\left(\frac{\varepsilon_0}{\varepsilon_r}\right)e^{-\left(\frac{\rho}{N}\right)^{\beta}}$$

resilient strain

average vertical elastic strain

Local Parameters, need to be calibrated.

Interstate 40 (I-40) Instrumentation Section in New Mexico



Profile of the sensors

Milling the old pavement

Pressure place installation

Strain gauges installation

Axle Sensing strips

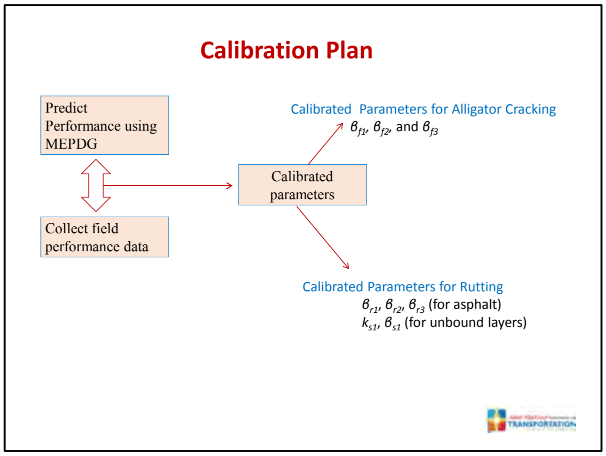
Section overview

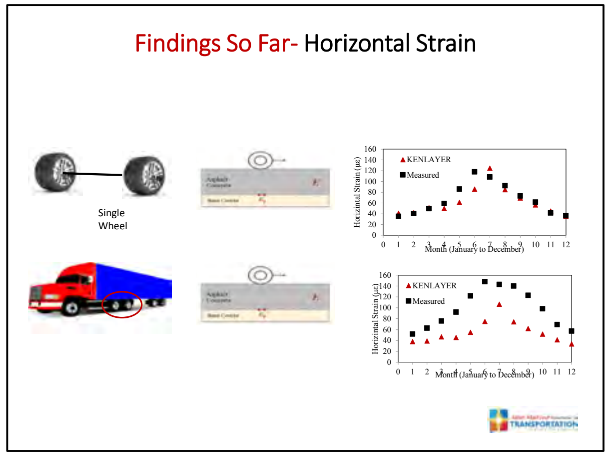
Goals and Objectives of the Calibration Project

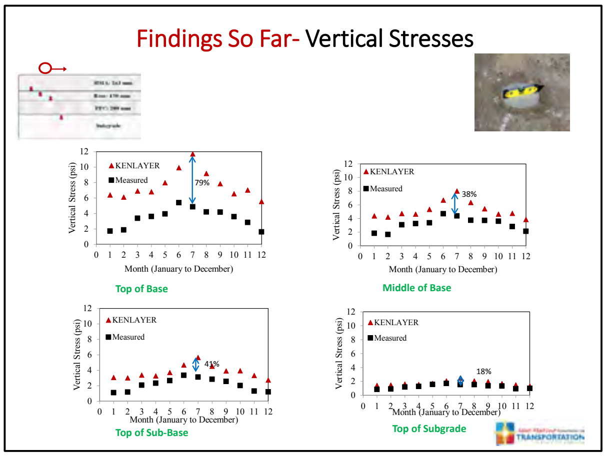
I-40 Instrumentation Section

Objectives: Continuous stress-strain data; Performance data; WIM Data; Weather and temperature data

Goals: MEPDG Prediction and Measured σ-ε Match ?; Calibration of MEPDG



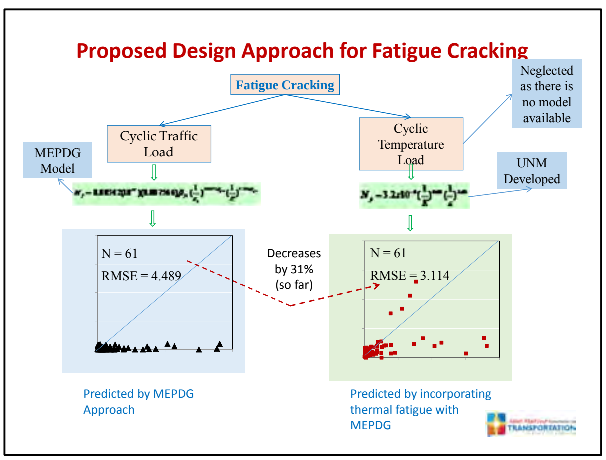




Findings So Far-Predicted vs. Measured Performances

Service Life (Yrs)	Alligator Cracking		Rutting	
	Predicted	Measured	Predicted	Measured
0	0	0	0	0
1.67	0.0113	0	0.234	0.05

The comparison will be continued next several years to calibrate the local parameters.



Advanced Statewide Calibration

Objectives

- Collect Materials from several reconstruction projects, including:
  - Binders
  - Base Coarse and Subgrade
  - Aggregates for HMA/WMA
  - RAP, Versabind, and other additives and modifiers
- Collect Construction Data in situ:
  - Moisture contents
  - Gmm/Gmb
  - Density
  - etc.
- Conduct Lab Testing: Mr, E*, FEL, G*/δ, Gradation, etc.
- Use collected and Lab data to calibrate MEPDG for New Mexico Conditions

Data Collection

- Field Cores, field collection, and field testing crucial
- Construction Team data not always accurate, where you want it, or available
- Whole project or section calibration? Crucial question to answer
- Spatial and depth uniformity can not be assumed

Lab Testing

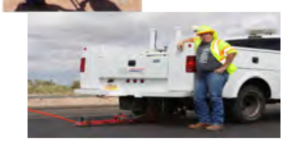
Challenges:

- Geogrid adjustments (Using before and after field testing)
- RAP adjustments (using extracted binder for lab testing)
- Rocks in subgrade causing gradation difficulties as seen in table
- Mr adjustments for subgrade gradation issues and RAP in Base Coarse

	150+50	149+50	150+50	149+50
Sieve	Wt.	Wt.	%	%
1.5"-8"	?	?	?	?
1"	21.1	250.9	1.01	9.85
¾"	38.5	227.1	1.84	8.92
½"	89.2	222.4	4.26	8.73
3/8"	52.9	142.3	2.53	5.59
#4	181	265.7	8.65	10.43
#8	148.2	202.1	7.08	7.93
#10	50.8	66.6	2.43	2.61
#30	300.9	456.3	14.38	17.92
#40	118.3	147.9	5.65	5.81
#50	159.8	152	7.64	5.97
#80	297.2	189	14.20	7.42
#100	101.8	46.6	4.86	1.83
#200	533.1	178.1	25.47	6.99
Sum	2092.8	2547	100	100

Field Testing

- DCP, FWD, and Clegg Hammer added for Uniformity verification
- Field Testing Invaluable for Spatial and Depth Uniformity and thus chosen calibration section length
- Helpful for approximate determination of in situ strength and stiffness



UNM DCP Testing, Naomi and Construction Team Member (top)
NMDOT FWD testing, Sean Bottom

Our Experience

What worked

- Having investigators in the field
- Multiple field tests, collection of in situ cores and base/subgrade directly from calibration section
- Developing reliable contacts in construction team
- Staying in close contact with construction team

What did not work

- Depending on only one or two construction contacts
- Depending solely on construction companies proctors, gradation values, etc
- Using entire construction site for calibration, a specific section must be chosen according to uniformity
- 100 ft makes a difference!

Other Difficulties and Questions

- Determine which version of MEPDG calibrating
  - Newer MEPDG uses different tests and inputs than older versions
- Finding enough projects that are purely new construction and not rehabilitation or bridge reconstruction
- Considering the range of climates in New Mexico is one project from each district sufficient? Should each district have its own calibration?

FHWA SOUTHWEST STATES MEPDG
PEER EXCHANGE MEETING

QUESTIONS?

Jeffrey S. Mann, PE
State Pavement Engineer
Pavement Management and Design Bureau
NMDOT
January 20, 2015

Jeffreys.mann@state.nm.us

New York – MEPDG Model Calibration Effort for JPCP in New York State

MEPDG Model Calibration Effort for JPCP in New York State

MEPDG PEER EXCHANGE MEETING
MAY 13-14, 2015
ALBANY, NY
OHIO UNIVERSITY

NYSDOT MEPDG Implementation

- National Pooled Fund Studies: TPF5-(079), TPF-5(121), and TPF-5(300)
- Ohio University
- University of Texas at Arlington

Local Calibration of JPCP Ohio University

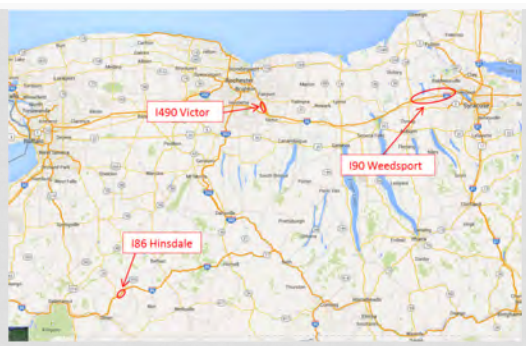
- Globally Calibrated
- Local calibration needed for Implementation of MEPDG in NYS

Data Collection

Experimental Pavements in NYS

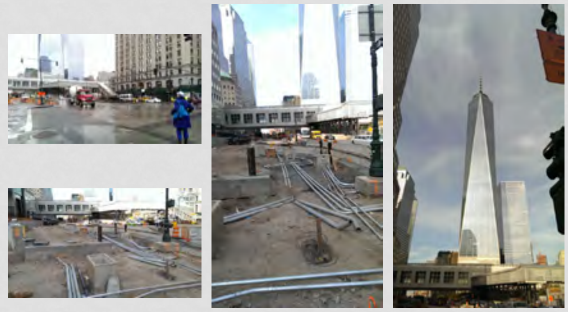
- I490 Victor
- I86 Hinsdale
- I90 Weedsport
- Rte 9A, NYC

Location Map



Location Map for Rte 9A

RTE 9A



Data Collected

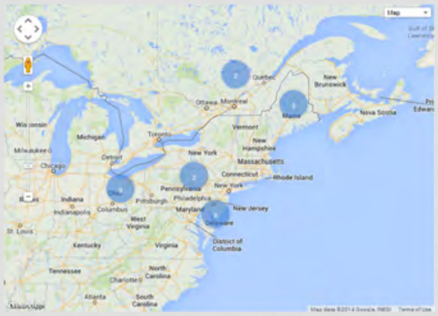
- Design Features
- Material Properties
- FWD Test
- Distress Survey

Database for MEPDG

- Material libraries
  - Individual Material
  - Traffic
  - Climate
- Material data from experimental pavements has been stored.

MEPDG JPCP Model Calibration

Selected Roadway Segments for Calibration



Selected Roadway Segments

#	Section ID	Type	Route	State	County	Lanes	Longitude	Latitude	Analysis Period
1	23-3013	GPS3	I-295	ME	Cumberland	2	-70.00343	43.92516	1988-2007
2	23-3014	GPS3	I-295	ME	Sagadahoc	2	-69.98491	43.93650	1988-2007
3	42-1623	GPS3	I-180	PA	Lycoming	2	-76.95766	41.24602	1988-2008
4	42-3044	GPS3	I-78	PA	Berks	2	-75.91678	40.57606	1988-
5	89-3015	GPS3	Rt 40	QC	Maurice	2	-72.35417	46.47318	1988-2001
6	89-3016	GPS3	Rt 40	QC	Maurice	2	-72.23617	46.56555	1988-2005
7	39-3013	GPS3	US 68	OH	Brown	1	-83.88985	38.88109	1987-1993
8	39-3801	GPS3	US 7	OH	Belmont	2	-80.74814	39.96576	1987-
9	10-0201	SPS2	US 113	DE	Sussex	2	-75.43926	38.84385	1996-2012
10	10-0202	SPS2	US 113	DE	Sussex	2	-75.43938	38.85564	1996-2012
11	10-0203	SPS2	US 113	DE	Sussex	2	-75.43932	38.84989	1996-2012
12	10-0204	SPS2	US 113	DE	Sussex	2	-75.43945	38.8654	1996-2012
13	10-0205	SPS2	US 113	DE	Sussex	2	-75.43923	38.84111	1996-2012
14	10-0206	SPS2	US 113	DE	Sussex	2	-75.43934	38.85367	1996-2012
15	10-0259	SPS2	US 113	DE	Sussex	2	-75.43942	38.86727	1996-2012
16	10-0260	SPS2	US 113	DE	Sussex	2	-75.43919	38.8395	1996-2012
17	39-0203	SPS2	US 23	OH	Delaware	2	-83.0741	40.41442	1995-
18	39-0209	SPS2	US 23	OH	Delaware	2	-83.07398	40.40511	1995-2012
19	39-0211	SPS2	US 23	OH	Delaware	2	-83.07406	40.41034	1995-
20	39-0260	SPS2	US 23	OH	Delaware	2	-83.07384	40.39455	1995-
21	39-0261	SPS2	US 23	OH	Delaware	2	-83.07403	40.40724	1995-
22	39-0262	SPS2	US 23	OH	Delaware	2	-83.07418	40.4203	1995-
23	39-0263	SPS2	US 23	OH	Delaware	2	-83.07423	40.42285	1995-

Data Collection

LTPP Database (infopave.com)

- Distresses
  - Transverse Cracking
  - Transverse Joint Faulting
- Smoothness (IRI)
- Material/Structure
- Traffic
- Climate
- Other Design Features

Calibration Flowchart

Execute MEPDG

Distress Predictions

Assess Bias

Determine Local Calibration Coefficients to Eliminate Bias

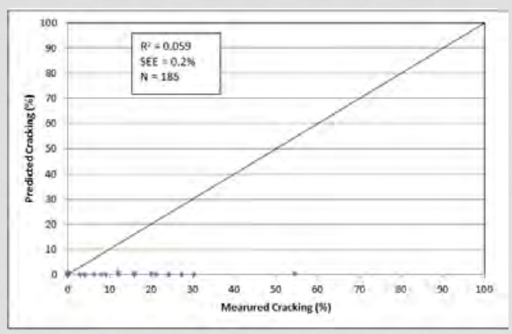
Assess Standard Error

Verify Adequacy

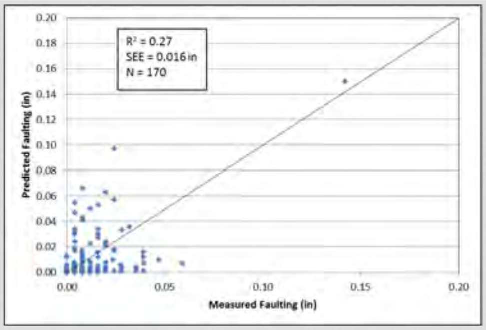
Assess Local Bias

Performance Indicator	Bias (p-value)	Standard Error	R^2	Hypothesis H_0:y_i-x_i=0	Comment
Transverse Cracking	<0.0001	0.2 (%)	0.059	Reject	Bias
Faulting	0.113	0.016 in	0.27	Accept	No Bias
IRI	0.187	17.7 in/mi	0.78	Accept	No Bias

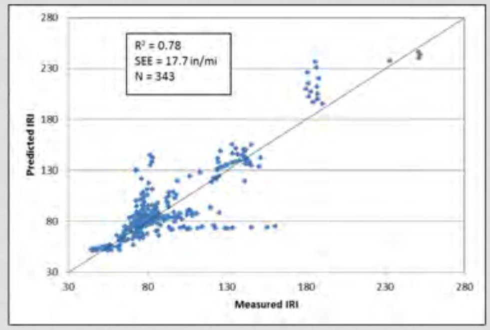
Comparison of Predicted and Measured Transverse Cracking Using the Global Calibration Values



Comparison of Predicted and Measured Faulting Using the Global Calibration Values



Comparison of Predicted and Measured IRI Using the Global Calibration Values



Determine Local Calibration Factors

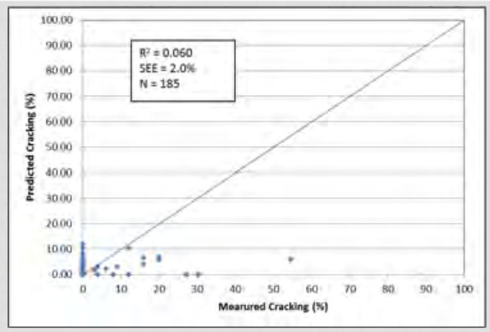
$$CRK = \frac{100}{1 + C_4 FD^{C_5}}$$

where: CRK = Predicted amount of bottom-up or top-down cracking (fraction);
FD = Fatigue damage;
C4 = Calibration constant, 1.0;
C5 = Calibration constant, -1.98.

C1 and C2 are related to Fatigue damage.

Cracking	C1	C2	C4	C5	SSE
Global Coefficients	**2**	**1.22**	**1**	**-1.98**	**8923.7**
Local Coefficients	**2**	**1.22**	**0.2**	**-1.63**	**8139.8**

Comparison of Predicted and Measured Cracking Using the Local Calibration Values



Eliminate Local Bias of Distress Prediction Models

Performance Indicator	Bias (p-value)	Standard Error	R^2	Hypothesis H_0:y_i-x_i=0	Comment
Transverse Cracking	0.061	2.0 (%)	0.06	Accept	No Bias
Faulting	0.113	0.016 in	0.27	Accept	No Bias
IRI	0.079	17.6 in/mi	0.79	Accept	No Bias

Preliminary Results

- A typical JPCP project design was run using local calibration factors.
- The result was compared with that using global calibration factors.
- The inputs for this project are largely based on data obtained from I90
- The result shows that cracking prediction at 50 years is increased using local calibration factors.
- Cracking @ 50 years using global calibration factors is 27% , while using local calibration factors is 72 (90% Reliability).
- The design life for cracking based on local calibration factors are shortened from 33 years to 24 years.

JPCP Design Catalog Development

- Currently under development
- Sensitivity of climate is under investigation

Summary

- Comprehensive concrete pavement data from test sections in NYS has been collected.
- A set of local calibration factors of JPCP has been developed for New York State.
- Verification with data from test sections showed that local factors increased cracking prediction over design life.
- More concrete pavement sections from NYS with sufficient data would increase significance.
- JPCP design catalog for NYS is currently under development using local calibration factors.

Thank You!

New York – Development of Design Tables for Flexible Pavements for New York State DOT

UNIVERSITY OF TEXAS ARLINGTON

DEVELOPMENT OF DESIGN TABLES FOR FLEXIBLE PAVEMENTS FOR NEW YORK STATE DOT

Stefan Romanoschi
Ali Qays Abdullah

May 14th, 2015

1

Premise for The Work

- NYSDOT aims to implement the ME Design Method for the design of new and rehabilitated flexible and rigid pavement structures
- The AASHTOWare Pavement ME models must be calibrated to NYSDOT conditions
- The AASHTOWare Pavement ME software will be used only in the Main Office for special projects. It is too complex and requires extensive input data to be used in the Regional Offices for routine designs.
- A simplified design method is needed so it can be used in Regional Offices. The designer will select only several major inputs and will obtain the design.

2

End-Product

- A simple, MEPDG-based design procedure to be used by the Pavement Design Engineer in the Regional Office
- A guidance for use of the procedure and selection of input values

3

TASKS

1. Develop a database of inputs (material properties, traffic) required by MEPDG
2. Calibrate the flexible pavement models using LTPP data in NE States (NY, PA, NJ, CT, MA, RI, VT, ME, NH)
3. Develop design tables to be used by designers in regional offices
4. Consult with the Regional Offices to check the reasonableness of the design tables
5. Develop training materials and train the personnel on the use of the procedure

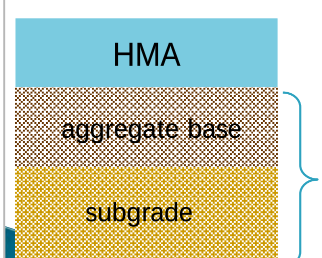
4

Flexible Pavements

MATERIAL CHARACTERIZATION

Flexible Pavement Materials

- *Modulus of Elasticity*

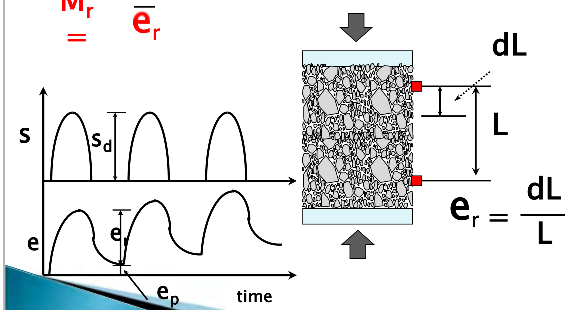


Asphalt Mixtures
Dynamic Modulus
AASHTO TP62

Unbound Materials
Resilient Modulus
NCHRP 1-28A
AASHTO T307

Dynamic Resilient Modulus of Asphalt Mixture

$M_r = \frac{s_d}{e_r}$



$e_r = \frac{dL}{L}$

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Dynamic Resilient Modulus

- Five temperatures
- Six frequencies
(0.1Hz,0.5Hz, 1Hz, 5Hz, 10 Hz, 25Hz)
- Five pulses for each frequency

8

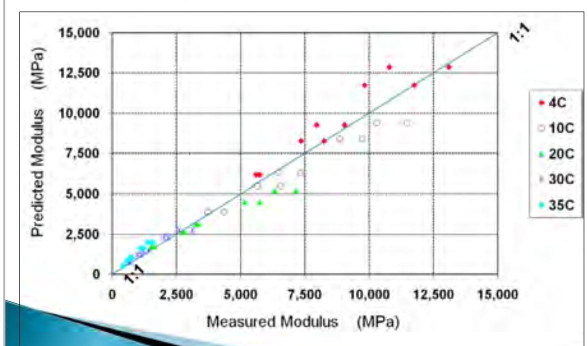
Witczak Equation

$$\log|E^*| = -1.249937 + 0.029232P_{200} - 0.001767(P_{200})^2 + $$
$$+0.002841P_4 - 0.058097V_a - 0.802208\frac{V_{beff}}{(V_{beff}+V_a)} + $$
$$+\frac{[3.871977 - 0.0021P_4 + 0.003958P_{38} - 0.000017(P_{38})^2 + 0.00547P_{34}]}{1+e^{(-0.603313-0.313351\log f - 0.393532\log \eta)}}$$

E* = Asphalt Mix Dynamic Modulus, in 10^5 psi
η = Bitumen viscosity in 10^6 poise
f = Load frequency in Hz
V_a = % air voids in the mix, by volume
V_{beff} = % effective bitumen content, by volume
P34 = % retained on the ¾ inch sieve,
P38 = % retained on the 3/8 inch sieve,
P4 = % retained on the # 4 sieve,
P200 = % passing the # 200 sieve

9

Measured vs. Predicted Dynamic Modulus NYSDOT Region 5 Mix 1



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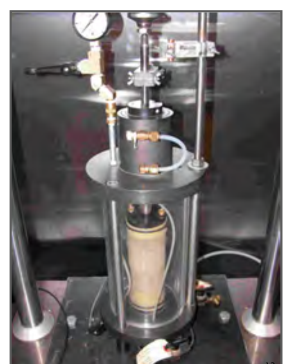
Unbound Materials (Aggregates and Subgrade)

- Resilient Modulus
  - Level 3 Defaults
  - Level 2 Correlations
  - Level 1 Materials specific testing
- Variability
  - None
  - Seasonal Values
  - EICM

11

Triaxial Resilient Modulus M_R

- 90%, 95%, 100% MDD
- Three moisture contents
- AASHTO T 307 Protocol



12

NYSDOT TRAFFIC DATA ANALYSIS

Traffic Data Collection

- **Vehicle Classification sites**
  - Vehicle Class Distribution (VCD)
  - Monthly Distribution Factors (MDF)
  - Hourly Distribution Factors (HDF)
- **WIM sites**
  - Axle Groups Per Vehicle (AGPV)
  - Axle Load Spectra

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Traffic Data

- Hierarchical Approach
  - Level 1 – site specific values
  - Level 2 – regional average values
  - Level 3 – statewide average or default average values

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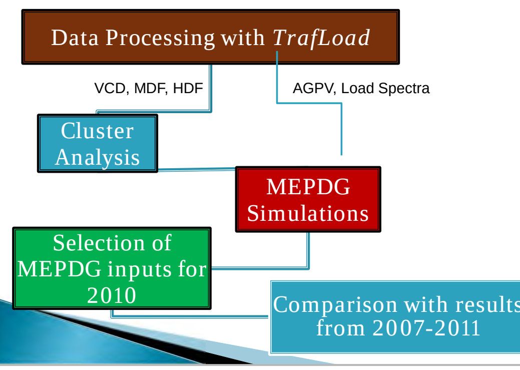
Research Methodology



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Data Collection & Assembly

- Extraction of Data with TrafLoad (2007-2011)

Year	2007	2008	2009	2010	2011
No. of vehicle classification sites for VCD , HDF (MDF)	55 (38)	75 (38)	57 (34)	52 (52)	45 (45)

Year	2007	2008	2009	2010	2011
Number of WIM sites (AGPV & ALDF)	12	12	14	19	14

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Cluster Analysis

- Definition

  Utilization of a hierarchical mathematical algorithm which classifies the sites on the basis of similarity of traffic characteristics
- Hierarchical
  - Ward's Method
- Average values for each cluster

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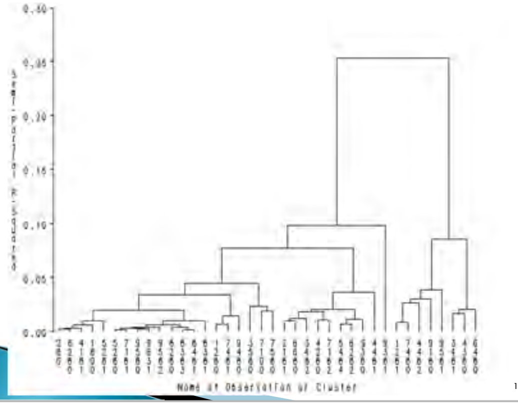
Cluster Analysis – Branch Plot



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MEPDG Runs – Flexible Pavement

- Typical Flexible Pavement Structure
  - 4.0 inch HMA - 9.5mm SM
  - 8.0 inch HMA – 19 mm SM
  - 12.0 inch granular base layer
  - A-7-6 subgrade soil.
- Climatic file: Messina, NY
- Design Life: 15 years
- Distresses:
  - Total Rut Depth (inch) &
  - Delta IRI= $IRI_t - IRI_0$

Figure: Rutting

20

MEPDG Runs – Rigid Pavement

- Typical Rigid Pavement Structure
  - 12.0 inch JPCP slab
  - 10.0 inch granular base layer
  - A-7-6 soil subgrade soil
- Climatic file: Messina, NY
- Design Life: 15 years
- Distresses:
  - Transverse Cracking (% cracked area) &
  - Delta IRI= $IRI_t - IRI_0$

Figure: Transverse Cracking

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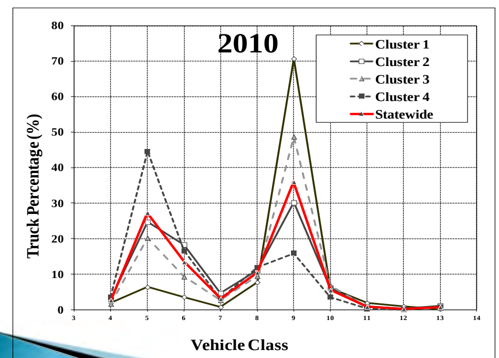
VCD Input



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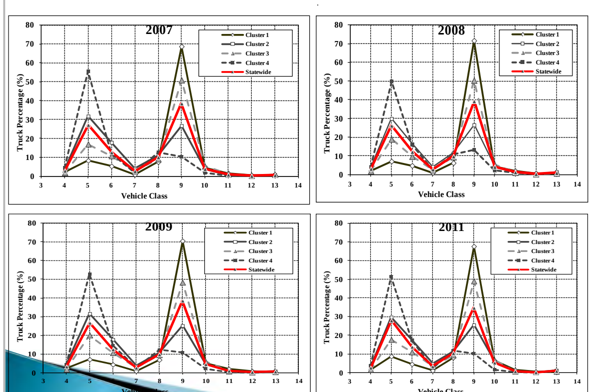
VCD Input



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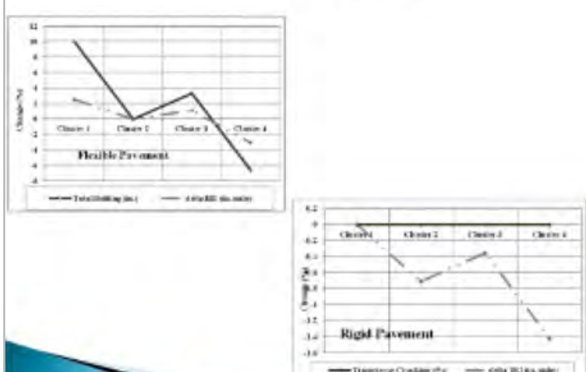
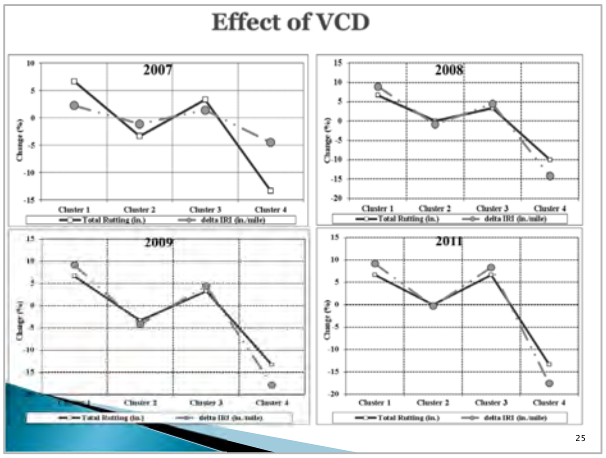
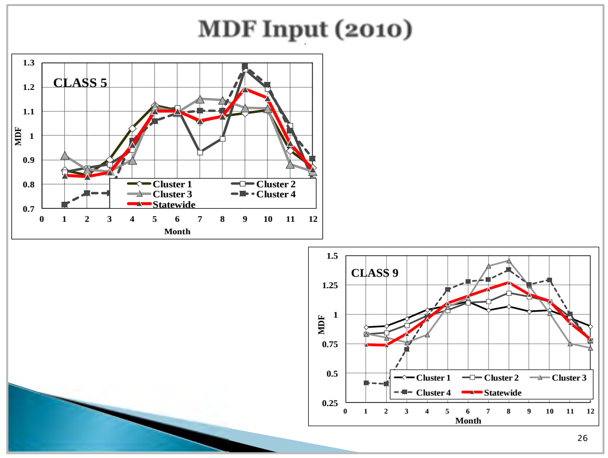
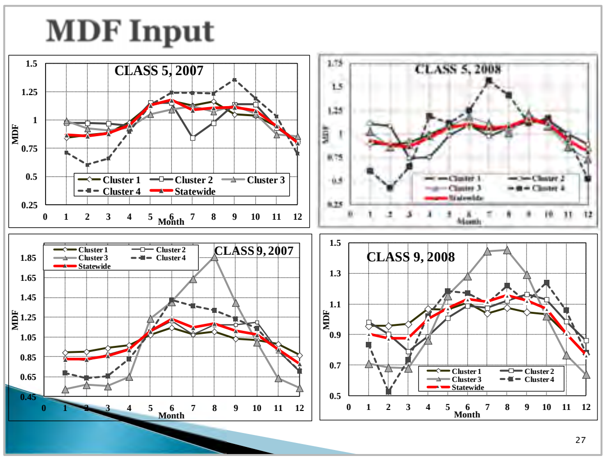
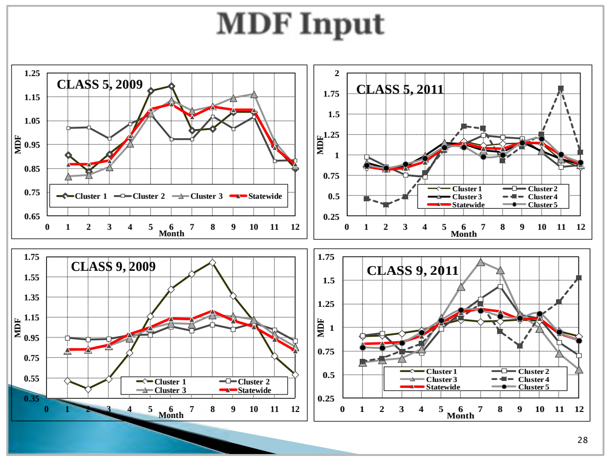
Effect of VCD (2010)

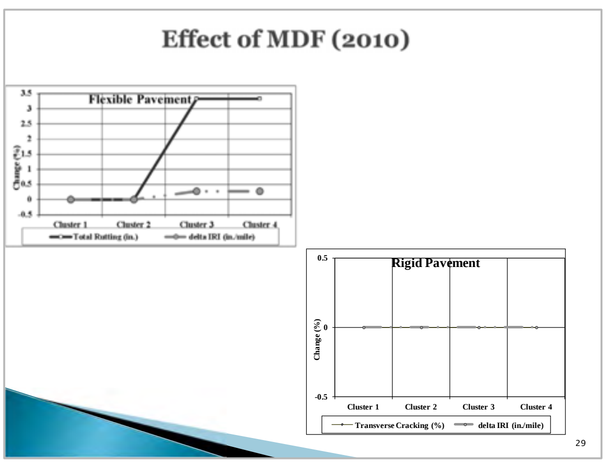


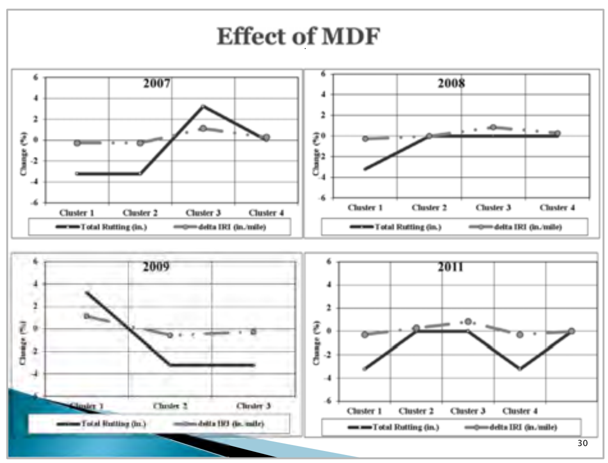

Effect of VCD
2007
2008
2009
2011
Change (%)
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Total Rutting (in.)
delta IRI (in./mile)
25


MDF Input (2010)
CLASS 5
CLASS 9
MDF
Month
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Statewide
26

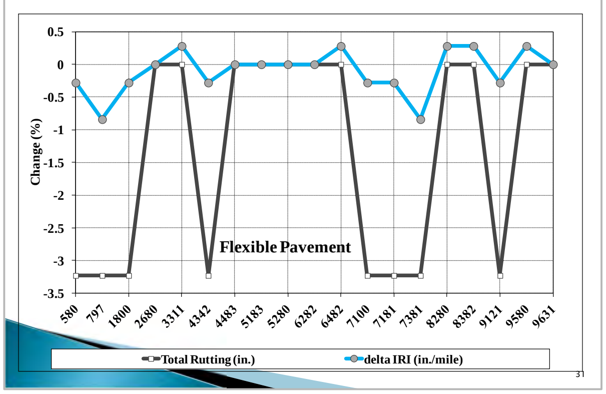

MDF Input
CLASS 5, 2007
CLASS 5, 2008
CLASS 9, 2007
CLASS 9, 2008
MDF
Month
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Statewide
27


MDF Input
CLASS 5, 2009
CLASS 5, 2011
CLASS 9, 2009
CLASS 9, 2011
MDF
Month
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Cluster 5
Statewide
28


Effect of MDF (2010)
Flexible Pavement
Rigid Pavement
Change (%)
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Total Rutting (in.)
delta IRI (in./mile)
Transverse Cracking (%)
29

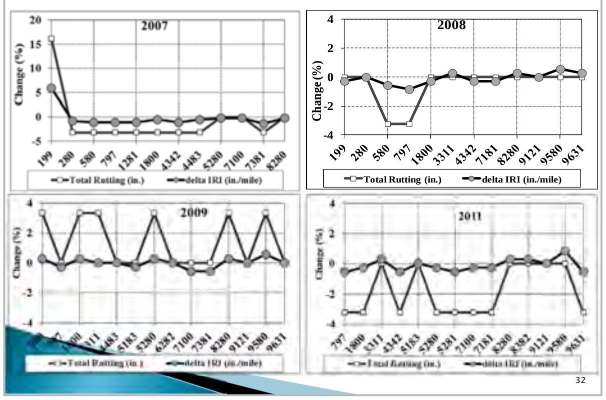

Effect of MDF
2007
2008
2009
2011
Change (%)
Cluster 1
Cluster 2
Cluster 3
Cluster 4
Total Rutting (in.)
delta IRI (in./mile)
30

Effect of AGPV (2010)



31

Effect of AGPV



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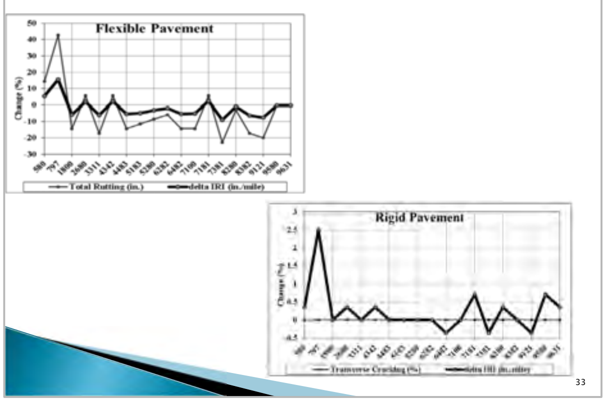
Effect of Axle Load Spectra (2010)



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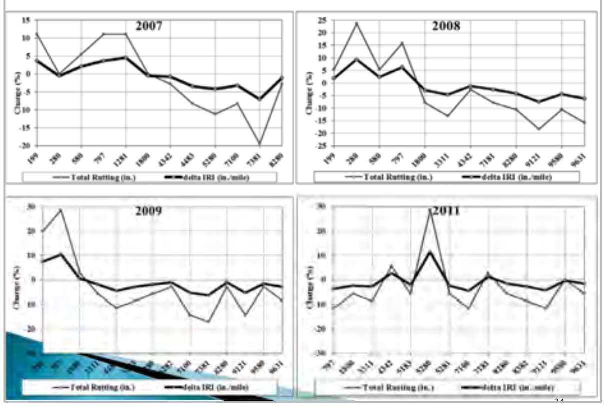
Effect of Axle Load Spectra



Comparison of Statewide Average Values

Predicted Distresses (VCD)	Years					Mean	Standard Deviation	Coefficient of Variation
	2007	2008	2009	2010	2011			
Total Rutting (in)	0.30	0.30	0.26	0.30	0.30	0.29	0.018	6.1%
Delta IRI	35.5	35.6	33.9	35.6	35.5	35.22	0.740	2.1%

Predicted Distresses (MDF)	Years					Mean	Standard Deviation	Coefficient of Variation
	2007	2008	2009	2010	2011			
Total Rutting (in)	0.31	0.31	0.31	0.30	0.31	0.31	0.004	1.5%
Delta IRI	35.7	35.70	35.7	35.6	35.7	35.68	0.045	0.1%

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Comparison of Statewide Average Values

Predicted Distresses (AGPV)	Years					Mean	Standard Deviation	Coefficient of Variation
	2007	2008	2009	2010	2011			
Total Rutting (in)	0.31	0.31	0.30	0.31	0.31	0.31	0.004	1.5%
Delta IRI	35.80	35.80	35.6	35.7	35.7	35.72	0.084	0.2%

Predicted Distresses (Axle Load Spectra)	Years					Mean	Standard Deviation	Coefficient of Variation
	2007	2008	2009	2010	2011			
Total Rutting (in)	0.36	0.38	0.35	0.35	0.35	0.29	0.013	3.6%
Delta IRI	37.90	38.60	37.5	37.5	37.40	35.22	0.497	1.3%

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Conclusions

- Though clusters are well defined, cluster averages do not significantly affect predicted pavement performance.
- Use of statewide average values for VCD, MDF, AGPV & Axle Load Spectra is recommended.
- Even though clusters are not the same, the same conclusion was drawn when traffic data from other years was analyzed.
- No significant difference in effect on pavement performance if statewide average values for different years are used.

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Development of the Design Tables

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Research Objective

- Calibrate the distresses models in AASHTOWare Pavement ME 2.1 to the local conditions of North Eastern region of the U.S.
- Develop a simple design procedure for new flexible pavement structures for New York State Department of Transportation (NYSDOT)
- Develop design tables for each region of Upstate and Downstate New York
- Observe the climate effects on the design tables

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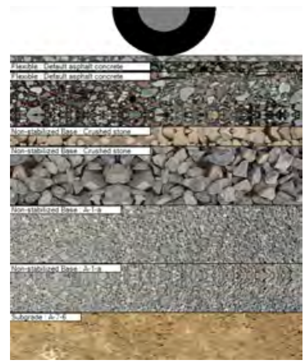
New York State Department of Transportation Current Practice

- HMA Layers
  - HMA Surface Course Layer = 1.75 in
  - HMA Binder Course Layer = 2 in
  - HMA Base Course Layer = Varied
- ATPB Layer = 4 in
- Subbase Course Layer = 12 in
- Selected Subgrade Layer = Varied
- Subgrade Layer = 24 in - Optional
- Infinite Subgrade Layer



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NYSDOT Current Practice = CPDM

Mr =28 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 2	165	0
2 < ESALs <= 4	175	0
4 < ESALs <= 8	200	0
8 < ESALs <= 13	225	0
13 < ESALs <= 23	250	0
23 < ESALs <= 45	250	150
45 < ESALs <= 80	250	300
80 < ESALs <= 140	250	450
140 < ESALs <= 300	250	600

Mr = 34 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 4	165	0
4 < ESALs <= 7	175	0
7 < ESALs <= 13	200	0
13 < ESALs <= 23	225	0
23 < ESALs <= 40	250	0
40 < ESALs <= 70	250	150
70 < ESALs <= 130	250	300
130 < ESALs <= 235	250	450
235 < ESALs <= 300	250	600

Mr = 41 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 6	165	0
6 < ESALs <= 11	175	0
11 < ESALs <= 20	200	0
20 < ESALs <= 35	225	0
35 < ESALs <= 60	250	0
60 < ESALs <= 110	250	150
110 < ESALs <= 200	250	300
200 < ESALs <= 300	250	450

Mr = 48 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 8	165	0
8 < ESALs <= 16	175	0
16 < ESALs <= 30	200	0
30 < ESALs <= 50	225	0
50 < ESALs <= 85	250	0
85 < ESALs <= 160	250	150
160 < ESALs <= 300	250	300

Mr = 55 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 12	165	0
12 < ESALs <= 20	175	0
20 < ESALs <= 40	200	0
40 < ESALs <= 65	225	0
65 < ESALs <= 115	250	0
115 < ESALs <= 215	250	150
215 < ESALs <= 300	250	300

Mr = 62 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 15	165	0
15 < ESALs <= 30	175	0
30 < ESALs <= 50	200	0
50 < ESALs <= 90	225	0
90 < ESALs <= 150	250	0
150 < ESALs <= 300	250	150

New York State Department of Transportation Current Practice

- Full depth asphalt pavement structure
- Superpave asphalt mixture

Location	Location by Counties	Standard PG Binder Grades (Material Designation)	Polymer Modified PG Binder Grades (Material Designation)
Upstate	All Counties not Listed under Downstate	64S-22 (702-64S22)	64V-221,2 (702-64V22)
Downstate	Orange, Putnam, Rockland, Westchester, Nassau, Suffolk Counties and City of New York	64H-22 (702-64H22)	64E-22 (702-64E22)

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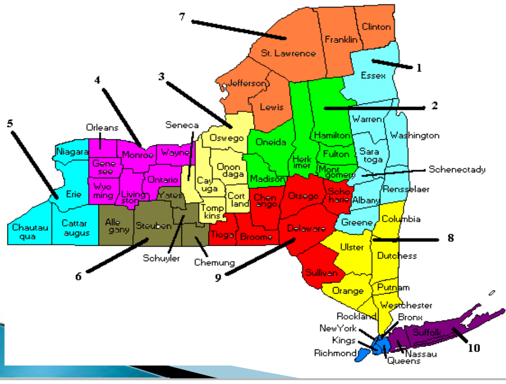
LOCAL CALIBRATION



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AASHTOWare Framework

- Permanent Deformation Model

$$\Delta_{p(HMA)} = \beta_{r1} * K_z * \epsilon_{r(HMA)} * 10^{k_1'} * (T)^{\beta_{r2} * k_2'} * (N)^{\beta_{r3} * k_3'}$$

$$\Delta_{P(Base)} = \beta_{gb} * K_{s1} * \epsilon_v * h_{soil} * \left(\frac{\varepsilon_0 \epsilon}{\varepsilon_r}\right) * e^{-\left(\frac{\rho}{n}\right)^{\beta}}$$

$$\Delta_{P(Subgrade)} = \beta_{sg} * K_{s1} * \epsilon_v * h_{soil} * \left(\frac{\varepsilon_0 \epsilon}{\varepsilon_r}\right) * e^{-\left(\frac{\rho}{n}\right)^{\beta}}$$

- Alligator Cracking Model

$$FC_{Bottom} = \left(\frac{6000}{1 + e^{(C_1 * C_1' + C_2 * C_2' * \log 10(D * 100))}}\right) * \left(\frac{1}{60}\right)$$

- International Roughness Index (IRI)

$$IRI = IRI_0 + C_1 * (SF) + C_2 * (FC_{Bottom}) + C_3 * (TC) + C_4 * (RD)$$

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AASHTO Local Calibration Guide

1. Select Hierarchical Input Level
2. Develop Local Experimental Plan
3. Select Roadway Segments
4. Distress Evaluation and Extraction
5. Sample Size Estimation for Each Performance Model

- Bias

$$N \geq \left(\frac{t * Sy}{et}\right)^2 \qquad N = \left(\frac{z_{\frac{\alpha}{2}} * \delta}{E_T}\right)^2$$

- Precision

$$\frac{Se}{Sy} \geq \left[\frac{X\alpha^2}{n-1}\right]^{0.5}$$

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AASHTO Local Calibration Guide

6. Asses Local Bias

$$H_0: \sum(y\ measured - y\ predicted) = 0$$

7. Eliminate of Local Bias
   - Regression Analysis
   - Optimization Approach
8. Validation of the Local Calibrated Models
   - Traditional (Split Sample) Approach
   - Jack Knife Testing

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2. LTPP Extracted Data

- Traffic Data
- Structural Data and Materials Properties Data
- Climatic Data
- Distresses Data
  - Total Rutting
  - Alligator Cracking
  - Thermal Cracking
  - Longitudinal Cracking
  - International Roughness Index (IRI)

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Data Assembly

- Total of 29 flexible LTPP pavement test sections obtained from LTPP database
- Out of 29 sections
  - 21 GPS-1 pavement test sections
  - 08 GPS-2 pavement test sections
- Only 17 sections retained for the calibration process

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1. Selected LTPP Sites

State Code	State	SHRP ID	Total Lanes	Structural Type	Construction Date	
					1	2
9	Connecticut	1803	2	Flexible	7/1/1988	1/17/1995
23	Maine	1001	4	Flexible	7/1/1988	6/6/1995
23	Maine	1009	2	Flexible	7/1/1988	8/22/1993
23	Maine	1028	2	Flexible	7/1/1988	5/12/1992
25	Massachusetts	1003	2	Flexible	6/1/1988	6/7/1988
34	New Jersey	1003	4	Flexible	8/1/1988	4/8/1994
34	New Jersey	1011	4	Flexible	7/1/1988	4/28/1998
34	New Jersey	1030	4	Flexible	12/1/1988	2/24/1991
34	New Jersey	1031	4	Flexible	7/1/1988	4/4/1996
34	New Jersey	1033	4	Flexible	7/1/1988	9/11/1997
34	New Jersey	1034	4	Flexible	12/1/1988	–
34	New Jersey	1638	4	Flexible	12/1/1988	–
42	Pennsylvania	1597	2	Flexible	8/1/1988	6/12/1990
42	Pennsylvania	1599	2	Flexible	8/1/1988	6/1/1999
50	Vermont	1002	2	Flexible	8/1/1988	–
50	Vermont	1004	2	Flexible	8/1/1988	10/6/1998
50	Vermont	1681	2	Flexible	6/1/1989	9/8/1991
50	Vermont	1683	2	Flexible	6/1/1989	9/23/1991
Missed Traffic Data and Unreliable Performance Data						
23	Maine*	1012	4	Flexible	7/1/1988	–
23	Maine*	1026	2	Flexible	7/1/1988	9/26/1996
25	Massachusetts*	1002	6	Flexible	6/1/1988	6/5/1988
25	Massachusetts*	1004	4	Flexible	8/1/1988	6/1/2001
33	New Hampshire*	1001	4	Flexible	8/1/1988	8/1/2001
36	New York*	1008	4	Flexible	5/1/1989	8/25/1989
36	New York*	1011	4	Flexible	6/1/1988	9/14/1993
36	New York*	1643	2	Flexible	5/1/1989	10/12/1989
36	New York*	1644	2	Flexible	5/1/1989	6/19/1996
42	Pennsylvania*	1605	2	Flexible	8/1/1988	6/14/1995
42	Pennsylvania*	1618	2	Flexible	12/1/1988	8/27/1989

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Traffic Data

- Traffic data extracted from LTPP database
- Traffic data includes:
  - AADTT
  - Monthly Adjustment Factor (MAF)
  - Vehicle Class Distribution (VCD)
  - Axle Load Distribution Factors (ALDF)
- General Traffic Inputs: Default MEPDG values
- GF estimated with compound growth formula:

$$AADTT_X = AADT_1 * (GR)^{AGE}$$

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Structural Data (LTPP)

- Extracted from LTPP database
- Structural Data Includes:
  - Gradation Analysis
  - Binder Gradation and Content
  - Base Layer Soil Data
  - Subgrade Layer characterization
  - Layer Thickness

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Climatic File Generation

1. Ann. Avg. Precipitation (LTPP, 1985-1996)
2. Ann. Avg. Precipitation (MEPDG, 1996-2006)
3. Comparison of Ann. Avg. Precipitation values
4. Pairs of approximate Ann. Avg. Precip values from LTPP and MEPDG were selected
5. Hourly Precip, Temp, Wind speed and % Sunshine were copied from MEDPG to LTPP
6. Hourly temp was adjusted based on daily averages of temperature

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Hourly Temperature Adjustment

$$T_{[\text{July3 1992, 2:00 PM}]} = T_{[\text{July3 2001, 2:00 PM}]} + (T_{[\text{Avg. July 3 1992] MEPDG}} - T_{[\text{Avg. July 3 2001] LTPP}})$$

- Example:
  - $T_{[\text{July3 1992@ 2:00 PM}]} = 47.3°F$
  - $T_{[\text{Avg. July 3 1992}]} = 39.01°F$
  - $T_{[\text{Avg. July 3 2001}]} = 37.94°F$

$$T_{[\text{July3 1992, 2:00 PM}]} = 47.3 + (39.01 - 37.94) = 48.35°F$$

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3. Sample Size Estimation for Distress Prediction
   - Models:
     - Bias

$$N = \left(\frac{z_{\frac{\alpha}{2}} * \delta}{E_T}\right)^2$$

Pavement Type	HMA New Pavement			
Performance Model	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Performance Indicator Threshold (@ 90% Reliability) (δ)	10%	0.4 in	500 ft/mile	225 in/mile
Standard Error of Estimate (SEE)	5.30%	0.16 in	83 ft/mile	18.6 in/mile
Tolerable Bias (E_T)	8.70%	0.27 in	136 ft/mile	31 in/mile
Minimum No. of Researches Required for Validation & Local Calibration	4	6	36	74
Number of the LTPP Sections Used	17	18	17	17

$E_T = SEE * Z_{\frac{\alpha}{2}}$ $Z_{\alpha/2} = 1.64$

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- Precision

$$\frac{Se}{Sy} \geq \left[\frac{X\alpha^2}{n-1}\right]^{0.5}$$

Performance Models	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Based on Maximum Measured Values				
Sy	8.43%	0.23 in	1860 ft/mile	35 in/mile
Se	8.54%	0.24 in	1996 ft/mile	54 in/mile
Se/Sy	1	1.05	1.07	1.56
(X²α/(n-1))^0.5	1	1.05	1.07	1.28
Minimum No. of Sites Required for Validation & Local Calibration	325	225	249	17
Number of LTPP sections used	17	18	17	17
Based on Full Set of Measured Data				
Sy	6.99%	0.17 in	1662.7 ft/mile	40.98 in/mile
Se	13.89%	0.24 in	80.3 ft/mile	320.59 in/mile
Se/Sy	1.99	1.37	0.05	7.82
(X²α/(n-1))^0.5	1.64	1.39	0.14	1.64
Minimum No. of Sites Required for Validation & Local Calibration	2	5	10,000,000	2
Number of LTPP sections Used	17	18	17	17

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4. Assess and Eliminate Local Bias

Distress type	Calibration	Total SSE	Bias	Se\Sy	R^2	Hypothesis; Ho:Σ(Meas.-Pred.) = 0
Alligator Cracking	Before	3,645	-3.2	1	0.0006	Reject; P<0.05
	After	2,766	0.21	0.96	0.07	Accepted; P>0.05
Total Rutting	Before	11.5	0.056	1.37	0.55	Reject; P<0.05
	After	8.8	0.04	1.21	0.56	Reject; P<0.05
Thermal	Before	234,373,333	129.1	1.116	0.31	Reject; P<0.05
	After	-	-	-	-	-
IRI	Before	754,583	-24.7	7.82	0.09	Reject; P<0.05
	After	115,777	-6	1.053	0.87	Accepted; P>0.05

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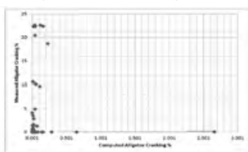
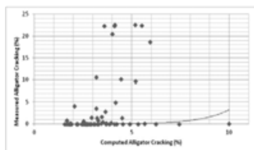
- Alligator Cracking

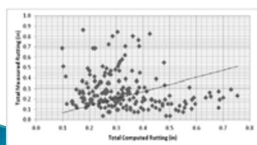


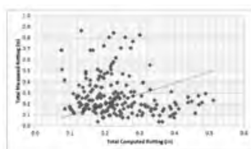
Before Calibration



After Calibration

- Rutting Model





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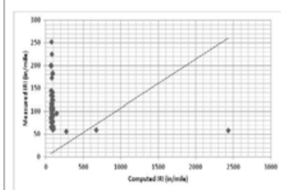
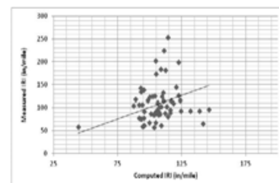
- IRI Model



Before Calibration



After Calibration

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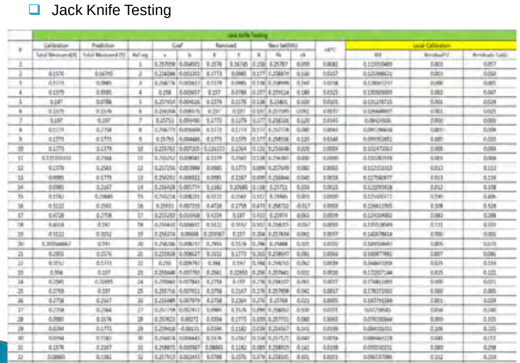
5. Performance Models Validation

- Jack Knife Testing



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Calibration Coefficients

Distress	Layer	Calibration Coeff.	M-E PDG National	M-E PDG NYSDOT (Momin, 2012)	AASHTOWare Pavement ME NYSDOT (Abdullah, 2015)
Permanent Deformation	HMA	β_{r1}	1	0.436	0.59
		β_{r2}	1	1	1
		β_{r3}	1	1	1
	Base	βr_{GB}	1	2.0654	0.82
	Subgrade	βr_{SG}	1	1.481	0.74
Alligator Cracking	HMA	C_1	1	-0.06883	0.50171
		C_2	1	1.27706	0.22719
Longitudinal Cracking	HMA	C1	7	-1	7
		C2	3.5	2	3.5
		C3	1,000	1,856	1,000
IRI	HMA	C1	40	51.6469	168.709
		C2	0.4	0.000218	-0.0238
		C3	0.008	0.0081	0.00017
		C4	0.015	-0.9351	0.015

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Development of the Design Tables

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Development of Design Cases

Design Cases Conditions:

- The pavement structures for new flexible pavement classified as Principal Arterial – Interstate.
- Design life of 15 years
- Design reliability of 90%
- Water table of 10 feet

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Design Considerations

NYSDOT CPDM for Mr 28 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 2	165	0
2 < ESALs <= 4	175	0
4 < ESALs <= 8	200	0
8 < ESALs <= 13	225	0
13 < ESALs <= 23	250	0
23 < ESALs <= 45	250	150
45 < ESALs <= 80	250	300
80 < ESALs <= 140	250	450
140 < ESALs <= 300	250	600
NYSDOT CPDM for Mr 41 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 6	165	0
6 < ESALs <= 11	175	0
11 < ESALs <= 20	200	0
20 < ESALs <= 35	225	0
35 < ESALs <= 60	250	0
60 < ESALs <= 110	250	150
110 < ESALs <= 200	250	300
200 < ESALs <= 300	250	450
NYSDOT CPDM for Mr 55 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 12	165	0
12 < ESALs <= 20	175	0
20 < ESALs <= 40	200	0
40 < ESALs <= 65	225	0
65 < ESALs <= 115	250	0
115 < ESALs <= 215	250	150
215 < ESALs <= 300	250	300

NYSDOT CPDM for Mr 34 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 4	165	0
4 < ESALs <= 7	175	0
7 < ESALs <= 13	200	0
13 < ESALs <= 23	225	0
23 < ESALs <= 40	250	0
40 < ESALs <= 70	250	150
70 < ESALs <= 130	250	300
130 < ESALs <= 235	250	450
235 < ESALs <= 300	250	600
NYSDOT CPDM for Mr 48 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 8	165	0
8 < ESALs <= 16	175	0
16 < ESALs <= 30	200	0
30 < ESALs <= 50	225	0
50 < ESALs <= 85	250	0
85 < ESALs <= 160	250	150
160 < ESALs <= 300	250	300
NYSDOT CPDM for Mr 62 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Subgrade Thickness (mm)
ESALs <= 15	165	0
15 < ESALs <= 30	175	0
30 < ESALs <= 50	200	0
50 < ESALs <= 90	225	0
90 < ESALs <= 150	250	0
150 < ESALs <= 300	250	150

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Developing Procedure

1. General Information

General Information
Design type: New Pavement
Pavement type: Flexible Pavement
Design life (years): 15
Base construction: May 2015
Pavement construction: June 2016
Traffic opening: September 2016

2. Design Criteria and Reliability

Performance Criteria	Limit	Reliability
Initial IRI (In/mile)	60	-
Terminal IRI (In/mile)	225	90%
AC Longitudinal Cracking (ft/mile)	2000	90%
AC Fatigue Cracking (Percent)	10	90%
AC Thermal Cracking (ft/mile)	500	90%
Permanent Deformation–Total Rutting (in)	0.75	90%
Permanent deformation–AC only (in)	0.25	90%

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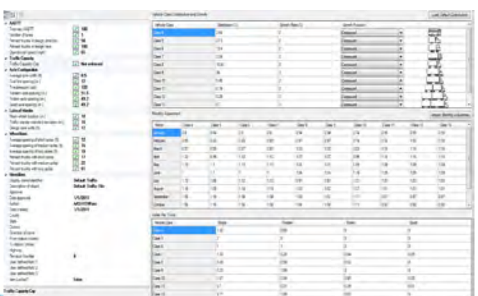
3. Traffic Inputs

- Average Statewide Traffic Data of the year 2010



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4. Climatic Data

July /1996 to February /2006 Import ICM Verify Weather

Date/Hour	Temperature (deg F)	Wind Speed (mph)	Sunshine (%)	Precipitation (in.)	Humidity (%)	Water Table (ft)
7/1/1996 12:00:00 AM	64	7	0	0	93	10
7/1/1996 1:00:00 AM	64	7	0	0	93	10
7/1/1996 2:00:00 AM	64	7	0	0	93	10
7/1/1996 3:00:00 AM	64	7	0	0	93	10
7/1/1996 4:00:00 AM	64	6	0	0	93	10
7/1/1996 5:00:00 AM	64	7	0	0	96	10
7/1/1996 6:00:00 AM	66	7	0	0	90	10
7/1/1996 7:00:00 AM	66.9	6	0	0	87	10
7/1/1996 8:00:00 AM	70	9	0	0	84	10
7/1/1996 9:00:00 AM	72	8	100	0	79	10
7/1/1996 10:00:00 AM	75.9	9	100	0	72	10
7/1/1996 11:00:00 AM	81	7	50	0	62	10
7/1/1996 12:00:00 PM	84.9	9	50	0	48	10
7/1/1996 1:00:00 PM	84.9	10	75	0	43	10
7/1/1996 2:00:00 PM	84.9	13	100	0	43	10
7/1/1996 3:00:00 PM	82.9	13	100	0	43	10
7/1/1996 4:00:00 PM	82	14	100	0	41	10
7/1/1996 5:00:00 PM	80.1	10	100	0	44	10
7/1/1996 6:00:00 PM	77	8	100	0	45	10
7/1/1996 7:00:00 PM	72	7	100	0	46	10
7/1/1996 8:00:00 PM	70	5	100	0	53	10
7/1/1996 9:00:00 PM	68	5	100	0	59	10

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Location at AASHTOWare Climatic Stations					Water Table Depth (ft)
County	Station ID	Longitude	Latitude	Region	
Saratoge	Albany (14735)	-73.803	42.748	1	10
Warren	Glens Falls (14750)	-73.61	43.341	1	10
Oneida	Utica (94794)	-75.384	43.145	2	10
Onondaga	Syracuse (14771)	-76.103	43.109	3	10
Monroe	Rochester (14768)	-77.677	43.117	4	10
Erie	Buffalo (14733)	-78.736	42.941	5	10
Chautauqua	Dunkirk (14747)	-79.272	42.493	5	10
Niagara	Niagara Falls (04724)	-78.945	43.107	5	10
Steuben	Dansville (94704)	-77.713	42.571	6	10
Chemung	Elmira/Corning (14748)	-76.892	42.159	6	10
Allegany	Wellsville (54757)	-77.992	42.109	6	10
St. Lawrence	Massena (94725)	-74.846	44.936	7	10
Clinton	Plattsburgh (94733)	-73.523	44.687	7	10
Jefferson	Watertown (94790)	-76.022	43.992	7	10
Orange	Montgomery (04789)	-74.265	41.509	8	10
Dutchess	Poughkeepsie (14757)	-73.884	41.627	8	10
Westchester	White Plains (94745)	-73.708	41.067	8	10
Nassau	Farmingdale (54787)	-73.417	40.734	10	10
Suffolk	Islip (04781)	-73.102	40.794	10	10
Suffolk	Shirley (54790)	-72.869	40.822	10	10
New York	New York (94728)	73.967	40.783	11	10
Queens	New York (94789)	-73.796	40.655	11	10
Queens	New York (14732)	-73.881	40.779	11	10

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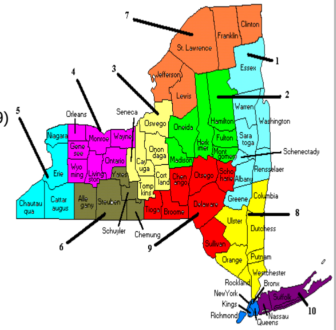
Region 9 – Virtual Stations

- R 1- Albany (14735)
- R 6 - Elmira (14748)
- R 8 - Montgomery (04789)
- R 3 - Syracuse (14771)
- R 2 - Utica (94794)



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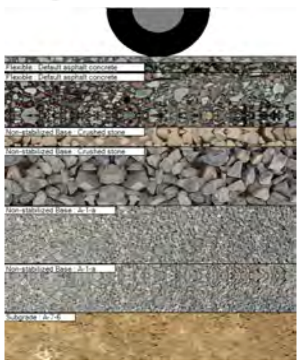
5. Pavement Structural Configuration

- HMA Layers
  - HMA Surface Course Layer = 1.75 in
  - HMA Binder Course Layer = 2 in
  - HMA Base Course Layer = Varied
- ATPB Layer = 4 in
- Subbase Course Layer = 12 in
- Selected Subgrade Layer = Varied
- Subgrade Layer = 24 in - Optional
- Infinite Subgrade Layer



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6. Asphalt Concrete Properties

Aggregate Gradation data for Upstate			
Sieve #	% passing	Layer	Nominal Maximum Aggregate Size
3/4"	100	Top	9.5mm
3/8"	100		
No.4	82		
No 200	4		
3/4"	92	Binder	19 mm
3/8"	67		
No.4	49		
No 200	2		
3/4"	86	Base	25mm
3/8"	67		
No.4	43		
No 200	5		
Aggregate Gradation data for Downstate			
Sieve #	% passing	layer	Nominal Maximum Aggregate Size
3/4"	100	Top	12.5mm
3/8"	89		
No.4	60		
No 200	4		
3/4"	78	Binder	19 mm
3/8"	63		
No.4	48		
No 200	5		
3/4"	65	Base	37.5mm
3/8"	56		
No.4	34		
No 200	4		

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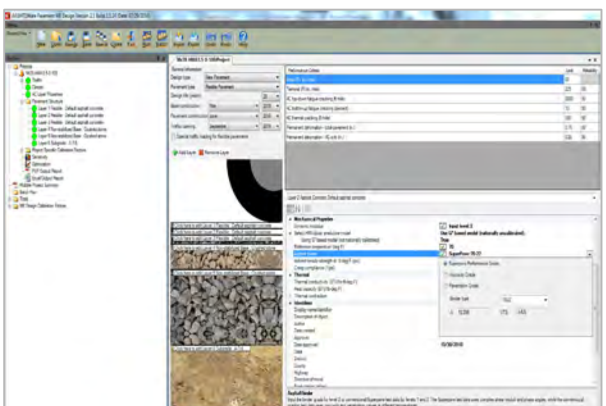

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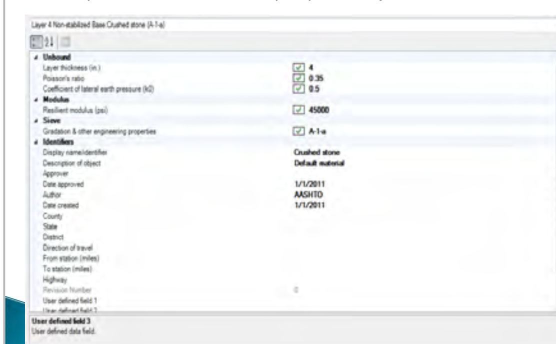
7. Aggregate Gradation Unbound Granular Layers

- Asphalt Treated Permeable (ATB) Base Layer



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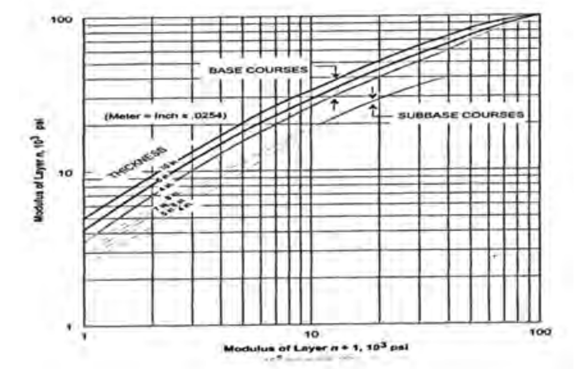

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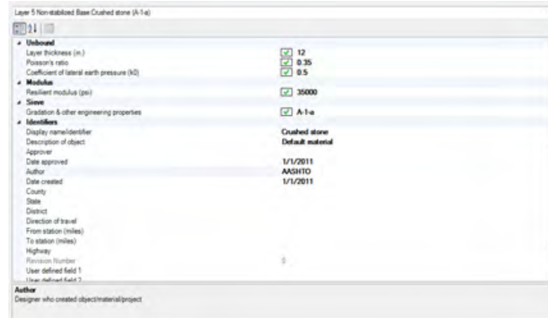
- Subbase Layer



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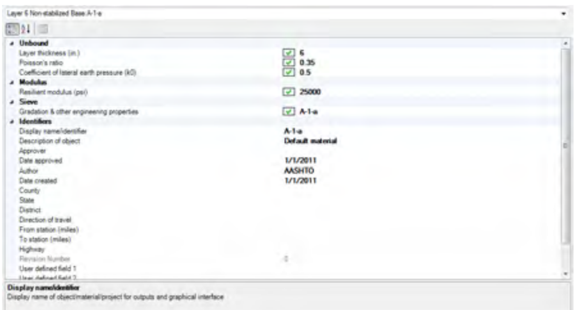
- Select Subgrade Layer



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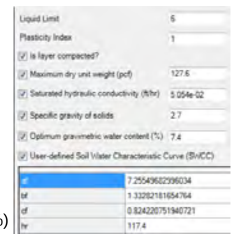
8. Granular Layers Materials Properties
   - Level 3 inputs used for:
     - Liquid limit (L.L)
     - Plasticity Index (P.I)
     - Maximum unit weight (pcf)
     - Saturated hydraulic conductivity (ft/hr)
     - Specific gravity of the soil
     - Optimum gravimetric water content (%)
     - User-defined Soil Water Characteristic Curve (SWCC)
     - Resilient Modulus (Mr)



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9. Distress Models

Distress	Layer	Calibration Coeff.	M-E PDG		AASHTOWare Pavement ME
			National	NYSDOT (Momin, 2012)	NYSDOT (Abdullah, 2015)
Permanent Deformation	HMA	$ß_{r1}$	1	0.436	0.59
		$ß_{r2}$	1	1	1
		$ß_{r3}$	1	1	1
	Base	$ßr_{GB}$	1	2.0654	0.82
	Subgrade	$ßr_{SG}$	1	1.481	0.74
Alligator Cracking	HMA	C_1	1	-0.06883	0.50171
		C_2	1	1.27706	0.22719
Longitudinal Cracking	HMA	C1	7	-1	7
		C2	3.5	2	3.5
		C3	1,000	1,856	1,000
IRI	HMA	C1	40	51.6469	168.709
		C2	0.4	0.000218	-0.0238
		C3	0.008	0.0081	0.00017
		C4	0.015	-0.9351	0.015

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Development of Design Tables

1. Run the design cases
2. Extract the predicted distresses
3. Tabulate the design solutions
4. Design Tables are for the following conditions:
   - Design Reliability 90%
   - Design Life 15 Years (20 years also done)
   - Water Table 10 ft

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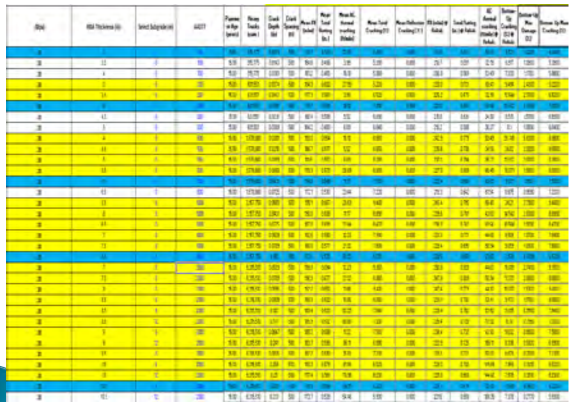

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Example Design Table for one Location

Mr 4 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	4	0
8	250	6	0
16	500	8.5	6
32	1000	10.5	6
64	2000	12.5	6
129	4000	14	6
161	5000	15	6

Mr 5 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	5	0
16	500	7	0
32	1000	9.5	6
64	2000	12	6
129	4000	13.5	6
161	5000	14	6

Mr 6 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	4	0
16	500	6	0
32	1000	8	0
64	2000	11	6
129	4000	13	6
161	5000	13.5	6

Mr 7 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3.5	0
16	500	5.5	0
32	1000	7	0
64	2000	10.5	6
129	4000	12.5	6
161	5000	13	6

Mr 8 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4.5	0
32	1000	6.5	0
64	2000	9	0
129	4000	12	6
161	5000	13	6

Mr 9 KSI			
ESALs (million)	AADTT One Direction	HMA Thickness (in)	Select Subgrade Thickness (in)
2	50	3	0
3	100	3	0
8	250	3	0
16	500	4	0
32	1000	6	0
64	2000	9	0
129	4000	12	0
161	5000	12.5	6

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Development of Design Tables

1. Run the design cases
2. Extract the predicted distresses
3. Tabulate the design solutions
4. Design Tables are for the following conditions:
   - Design Reliability 90%
   - Design Life 15 Years (20 years also done)
   - Water Table 10 ft

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- Comparison of Design Tables for Region 5

AADTT	Buffalo	Dunkirk	Niagara Falls	Buffalo	Dunkirk	Niagara Falls
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
500	7 / 0	7.5 / 0	7 / 0	6 / 0	6.5 / 0	6.5 / 0
1,000	8.5 / 6	9.5 / 6	9 / 6	8 / 0	8.5 / 0	8 / 0
2,000	11 / 6	12 / 6	11 / 6	10 / 0	11 / 6	10 / 6
4,000	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	13 / 6	12 / 6
5,000	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6	12.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	5 / 0	4.5 / 0	4 / 0	4.5 / 0	5 / 0
1,000	7 / 0	7.5 / 0	6.5 / 0	6 / 0	6.5 / 0	6 / 0
2,000	9 / 0	10 / 0	9 / 0	8 / 0	9 / 0	8 / 0
4,000	11.5 / 6	12 / 6	11 / 6	11 / 6	11.5 / 6	11 / 6
5,000	12.5 / 6	13 / 6	11.5 / 6	11.5 / 6	12.5 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
2,000	7.5 / 0	8.5 / 0	7 / 0	6.5 / 0	8 / 0	6.5 / 0
4,000	10 / 6	11 / 6	9.5 / 6	10 / 0	11 / 0	9.5 / 0
5,000	11 / 6	12 / 6	10.5 / 6	10 / 6	11.5 / 6	10 / 6

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- Comparison of Design Tables for Region 6

AADTT	Dansville	Elmira	Wellsville	Dansville	Elmira	Wellsville
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	4 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	6 / 0	4.5 / 0	4.5 / 0	4.5 / 0
500	8 / 0	7.5 / 0	7.5 / 0	6.5 / 0	6.5 / 0	6 / 0
1,000	9.5 / 6	9.5 / 6	9 / 6	9 / 0	8 / 6	8 / 6
2,000	12 / 6	11.5 / 6	11.5 / 6	11 / 6	11 / 6	10.5 / 6
4,000	13.5 / 6	13.5 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6
5,000	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	4 / 0	3 / 0	3.5 / 0	3.5 / 0
500	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0	6 / 0
2,000	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9 / 0	8.5 / 0
4,000	12.5 / 6	12 / 6	11.5 / 6	12 / 6	11.5 / 6	11 / 6
5,000	13 / 6	12.5 / 6	12 / 6	12.5 / 6	12 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	5 / 0
2,000	9 / 0	8 / 0	8 / 0	8 / 0	7.5 / 0	7 / 0
4,000	11.5 / 6	11 / 6	10.5 / 6	11 / 0	10.5 / 0	10 / 0
5,000	12 / 6	11.5 / 6	11 / 6	12 / 6	11.5 / 6	11 / 6

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- Comparison of Design Tables for Region 7

AADTT	Massena	Plattsburgh	Watertown	Massena	Plattsburgh	Watertown
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
250	6 / 0	6 / 0	6 / 0	5 / 0	5 / 0	5 / 0
500	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0
1,000	10 / 6	10 / 6	9.5 / 6	9 / 0	9 / 0	9 / 0
2,000	12 / 6	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6
4,000	14 / 6	14 / 6	13.5 / 6	13 / 6	13 / 6	13 / 6
5,000	14 / 6	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4.5 / 0	4.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
500	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	7 / 0
2,000	10 / 6	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9.5 / 0
4,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0	3.5 / 0
1,000	6 / 0	6 / 0	6 / 0	6 / 0	6 / 0	5 / 0
2,000	9 / 0	9 / 0	8.5 / 0	8 / 0	8 / 0	8 / 0
4,000	11.5 / 6	11.5 / 6	11 / 6	11.5 / 0	11.5 / 0	11 / 0
5,000	12 / 6	12 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6

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- The Design Tables for Downstate New York
  - Comparison of Design Tables for Region 8

AADTT	Montgomery	Poughkeepsie	White Plains	Montgomery	Poughkeepsie	White Plains
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	4 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	6 / 0	6 / 0	5.5 / 0	4.5 / 0	5 / 0	4.5 / 0
500	7.5 / 0	8 / 0	7 / 0	6.5 / 0	7 / 0	6 / 0
1,000	9.5 / 6	10 / 6	9 / 6	9 / 0	9 / 6	8 / 6
2,000	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6	10 / 6
4,000	13.5 / 6	14 / 6	13 / 6	13 / 6	12.5 / 6	12 / 6
5,000	14 / 6	14.5 / 6	13.5 / 6	13.5 / 6	13 / 6	12.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0
500	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
1,000	7.5 / 0	8 / 0	7 / 0	7 / 0	7 / 0	6 / 0
2,000	10.5 / 6	10 / 6	9 / 6	9.5 / 0	9.5 / 6	8 / 6
4,000	12.5 / 6	12.5 / 6	11.5 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	4.5 / 0	4 / 0	5 / 0	4 / 0	4 / 0
1,000	6 / 0	6 / 0	5 / 0	5.5 / 0	6 / 0	5 / 0
2,000	8.5 / 0	9 / 0	7.5 / 0	9.5 / 0	8.5 / 0	7 / 0
4,000	11.5 / 6	11.5 / 6	10 / 6	11 / 0	11 / 0	9.5 / 0
5,000	12 / 6	12 / 6	11 / 6	11.5 / 6	12 / 6	10.5 / 6

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- Comparison of Design Tables for Region 10

AADTT	Farmingdale	Islip	Shirley	Farmingdale	Islip	Shirley
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	5 / 0	5 / 0	5 / 0	4 / 0	4 / 0	4.5 / 0
500	7 / 0	7 / 0	7.5 / 0	5.5 / 0	5.5 / 0	6 / 0
1,000	8.5 / 6	8.5 / 6	9.5 / 6	8.5 / 0	8 / 0	8 / 0
2,000	11 / 6	11 / 6	11.5 / 6	10 / 6	10 / 6	10.5 / 6
4,000	13 / 6	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
5,000	13.5 / 6	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3.5 / 0	4.5 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	6 / 0	4.5 / 0	4.5 / 0	5 / 0
1,000	7 / 0	7 / 0	8 / 0	6 / 0	6 / 0	6 / 0
2,000	9.5 / 6	9 / 6	10.5 / 6	8 / 0	8 / 0	9 / 6
4,000	11.5 / 6	11.5 / 6	12.5 / 6	11 / 6	11 / 6	11.5 / 6
5,000	12 / 6	12 / 6	13.5 / 6	11.5 / 6	11.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	3.5 / 0	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
2,000	7.5 / 0	7.5 / 0	8 / 0	6.5 / 0	6.5 / 0	7 / 0
4,000	10.5 / 6	10.5 / 6	11 / 6	9.5 / 0	9.5 / 0	10 / 0
5,000	11 / 6	11 / 6	11.5 / 6	10.5 / 6	10.5 / 6	11.5 / 6

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- Comparison of Design Tables for Region 11

AADTT	NYC 94728	NYC 94789	NYC 14732	NYC 94728	NYC 94789	NYC 14732
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5 / 0	5 / 0	4.5 / 0	4 / 0	4 / 0
500	8 / 0	6.5 / 0	7 / 0	6.5 / 0	5 / 0	5.5 / 0
1,000	9.5 / 6	8.5 / 6	8.5 / 6	9 / 0	7.5 / 0	8 / 0
2,000	12.5 / 6	11 / 6	11 / 6	11.5 / 6	9.5 / 6	10 / 6
4,000	14 / 6	12.5 / 6	13 / 6	13.5 / 6	12 / 6	12 / 6
5,000	14 / 12	13.5 / 6	13.5 / 6	14 / 6	12.5 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0
1,000	8 / 0	6.5 / 0	6.5 / 0	7 / 0	5.5 / 0	5.5 / 0
2,000	10.5 / 6	8.5 / 6	9 / 6	10 / 6	7.5 / 6	8 / 6
4,000	13 / 6	11 / 6	11.5 / 6	12.5 / 6	10 / 6	10.5 / 6
5,000	13.5 / 6	11.5 / 6	12 / 6	13 / 6	11 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	4.5 / 0	5 / 0	5.5 / 0	4.5 / 0	4.5 / 0
2,000	9 / 0	7 / 0	7.5 / 0	8.5 / 0	6 / 0	6.5 / 0
4,000	12 / 6	9.5 / 6	10 / 6	11.5 / 0	9 / 0	9.5 / 0
5,000	12.5 / 6	10 / 6	11 / 6	12 / 6	10 / 6	10.5 / 6

87

- Comparison of Design Tables for Upstate and Downstate New York
  - At low AADTT, the corresponding design solutions are the same for the Upstate and Downstate regardless the subgrade soil
  - At high AADTT and soft subgrade soil, the corresponding design solutions are thicker for the Upstate part than for the Downstate part of New York State

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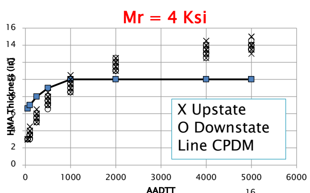
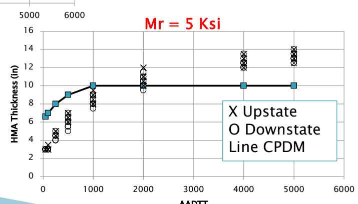
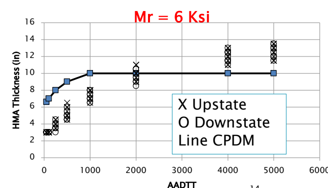
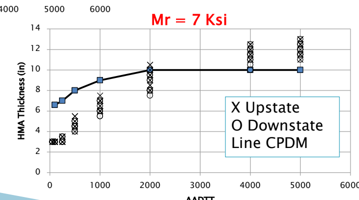
ME Design Tables vs CPDM



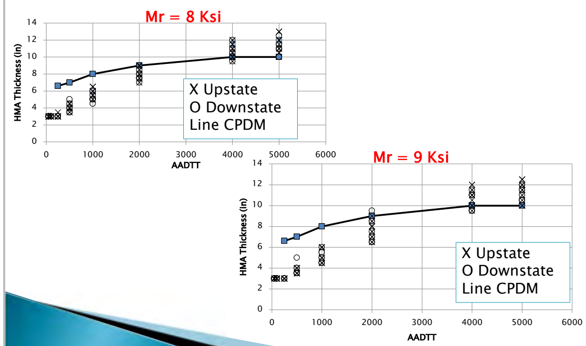


ME Design Tables vs CPDM





ME Design Tables vs CPDM



Conclusions

1. The calibration of the rutting, alligator cracking and IRI models was successful
2. The methodology used to develop simple design tables was successful. The designer needs only AADTT and Mr to design the pavement structure
3. The climates variations have an impact on the design thicknesses; the obtained design tables are different for different locations within the New York State
4. For low traffic volumes, the design solutions are the same throughout the State

92

Conclusions

5. For high truck traffic volumes and soft subgrade soils, the design solutions vary from location to location, even within the same region
6. The design solutions for the Upstate part of New York State ask for thicker asphalt concrete layers that the corresponding design solutions for the Downstate part of the state
7. At low AADTT, the new design tables recommend thinner asphalt concrete layers than those in the CPDM table
8. At high AADTT, the new design tables recommend thicker asphalt concrete layers than those in the CPDM table

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Recommendations

1. NYSDOT should develop a new flexible pavement performance database.
2. The flexible pavement performance models should be recalibrated if the new pavement performance database will be available or any of the distress models change.
3. Additional design tables should be developed for water table depths of less than 10 feet
4. For high AADTT values, a life-cycle cost analysis (LCCA) should be conducted to compare the cost effectiveness of full-depth asphalt pavement designs included in the tables with rigid pavement designs

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ACKNOWLEDGMENTS

We acknowledge the contribution of New York State Department of Transportation engineers in the Traffic, Geotech, Materials, Pavement Management and Research offices. They provided the data and the guidance needed for conducting this work.

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North Carolina – Adventures in Local Calibration

North Carolina
DEPARTMENT OF TRANSPORTATION

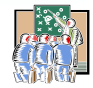
Adventures in Local Calibration

Judith Corley-Lay, PE

November 4, 2014

North Carolina ncdot.gov

Outline of Presentation



- NC preparation history
- Key points in our calibration
- What we wished we had known
- What we wished we had spent more time on
- Next Steps

2

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North Carolina's Preparation History

- Typical Dynamic Moduli for NC mixes (completed 2005)
- Implementation Plan (completed 2007)
- Traffic Data Resources (completed 2011)
- Local Calibration (completed 2011)
- First Production Project – Goldsboro Bypass (2011)
- Improvement of Climate Files (in progress)

3

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Key Points in Our Local Calibration

- Only calibrated flexible pavement. We had just changed our rigid pavement design specs so had no performance data.
- We began local calibration before the production version was available.
- Under significant pressure to implement as quickly as possible.

4

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What we wish we had known…

- Some of the models changed from MEPDG to Darwin-ME. An example was the rutting model which was initially layer by layer.
- Our method of distress surveys made quantifying distresses for calibration more difficult.
- As models are added or improved, recalibration will be required. We needed to set up a calibration database that could be added to and improved for future calibration efforts.

5

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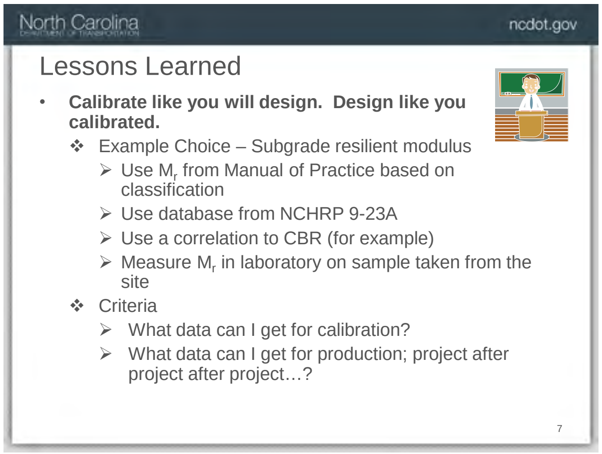
Spend more time on…

- I should have spent time doing queries of our PMS to quantify distress levels for various pavement types and thicknesses.
- Clark should have spent more time in selecting calibration sites. We should have done some extra testing on the sites.
- We should have waited for the production software… it would have saved us time.

6


North Carolina
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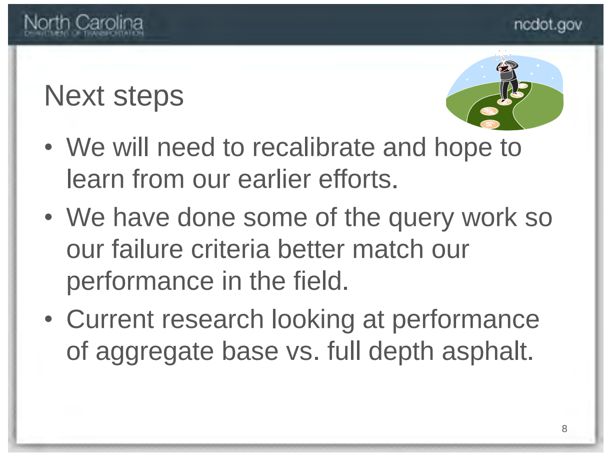
Lessons Learned
Calibrate like you will design. Design like you calibrated.
Example Choice – Subgrade resilient modulus
Use M_r from Manual of Practice based on classification
Use database from NCHRP 9-23A
Use a correlation to CBR (for example)
Measure M_r in laboratory on sample taken from the site
Criteria
What data can I get for calibration?
What data can I get for production; project after project after project…?
7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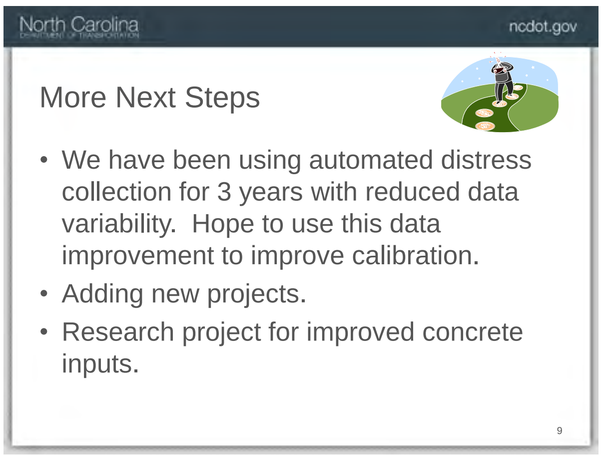
Next steps
We will need to recalibrate and hope to learn from our earlier efforts.
We have done some of the query work so our failure criteria better match our performance in the field.
Current research looking at performance of aggregate base vs. full depth asphalt.
8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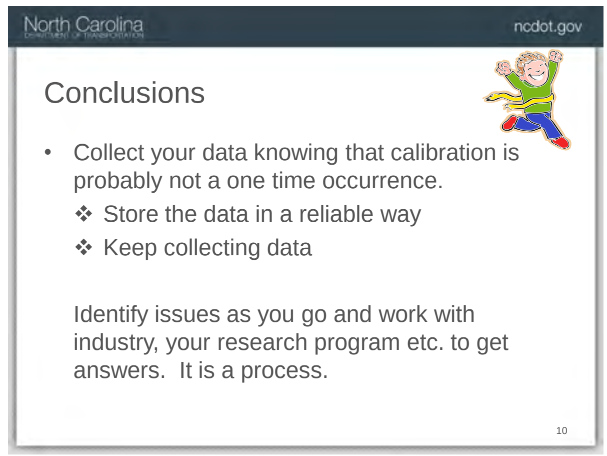
More Next Steps
We have been using automated distress collection for 3 years with reduced data variability. Hope to use this data improvement to improve calibration.
Adding new projects.
Research project for improved concrete inputs.
9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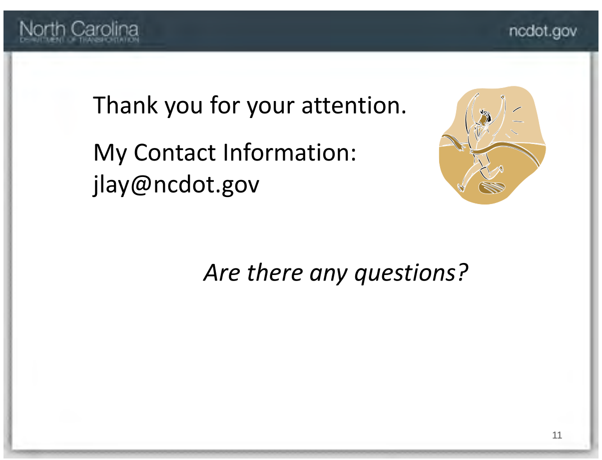
Conclusions
Collect your data knowing that calibration is probably not a one time occurrence.
Store the data in a reliable way
Keep collecting data
Identify issues as you go and work with industry, your research program etc. to get answers. It is a process.
10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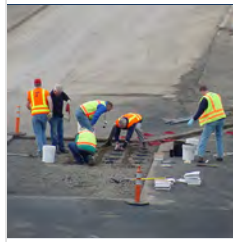
Thank you for your attention.
My Contact Information:
jlay@ncdot.gov
Are there any questions?
11

Oregon DOT History and Experience

ODOT MEPDG

History and Experiences

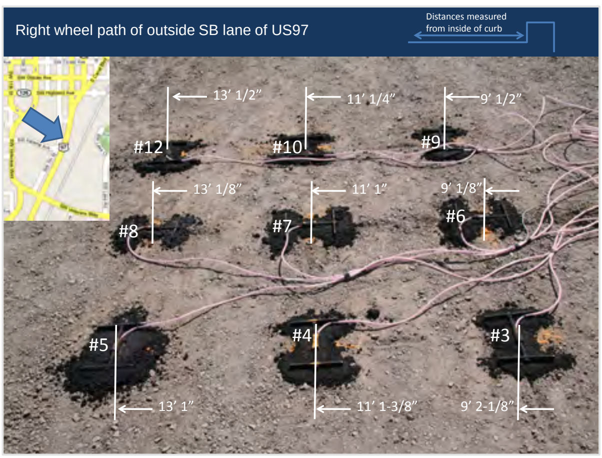
2015 NW MEPDG User Group
Presented by:
Justin Moderie, P.E., G.E., Pavement Design Engineer
ODOT
April 14, 2015



Pre-MEPDG Efforts

- WESLEA / JULEA
- Trended towards thin base and thick AC
- 15" AC
- Installed instrumented sites

Right wheel path of outside SB lane of US97



MEPDG

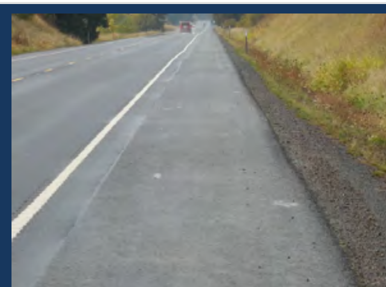
- Traffic study
- Master Curves
- Local calibration

Number Axles/Truck | Axle Configuration | Wheelbase

	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	0.2
Class 8	2.38	0.67	0	0
Class 9	1.13	1.93	0	0
Class 10	1.19	1.09	0.89	0.2
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	0.1

Traffic Study

- Used information from a related traffic study
- No class-specific weight distributions. Used a "virtual" truck.
- Average number of axles per truck
- Average axle spacing
- Hourly truck volume distribution



Dynamic Modulus Master Curves

- Generated with AMPT
- Study by OSU generated the beginning of a library
- Slowly continuing to build
- Including 50% RAP blend and polymer modified (>50% ER)

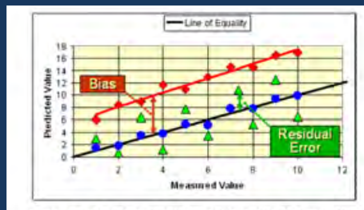
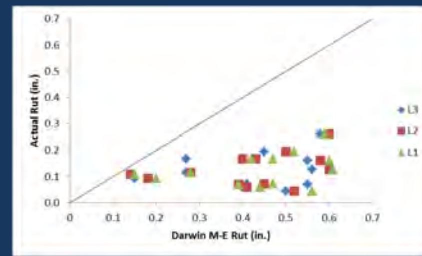


Figure 2-1 The Bias and the Residual Error (Von Quintus 2008a)

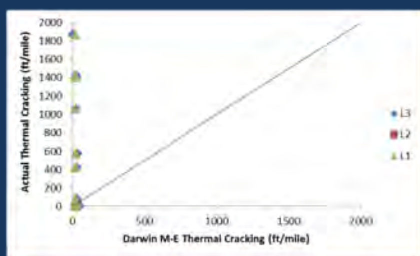
Local Calibration

- Iowa State University – Dr. Chris Williams
- High / Low volume
- Dry Cold / Wet Mild climate
- Asphalt pavements
- Limited CRCP pavements



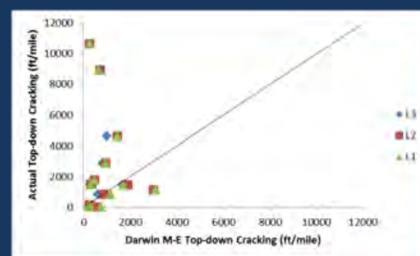
Rutting Disregarded

- Studded tires?
- Subgrade stabilization with cheap aggregate materials



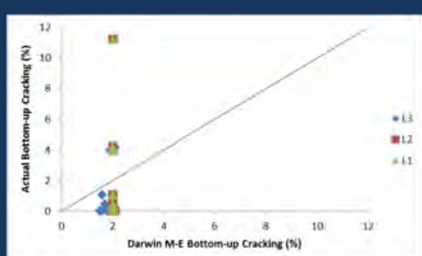
Thermal Cracking

- Has not been an issue since PG binder grades in ~2000



Top Down Cracking

- Questionable model
- Ongoing issue / research at ODOT



Bottom-Up Cracking

- Used calibration coefficient
- Small changes in results from using national calibration
- One set of calibrations for entire state and all traffic levels
- Continue to use on > 40mill 20-year ESALs and as a separate tool

CRCP

- Four sites compared
- Not statistically significant
- Use national calibration
- Results compare well with experience
- Use MEPDG exclusively

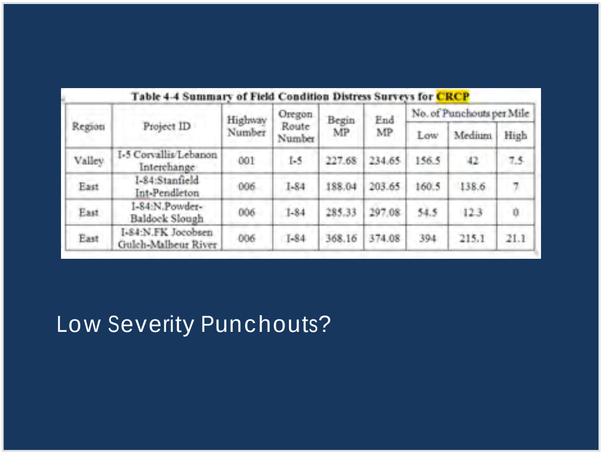
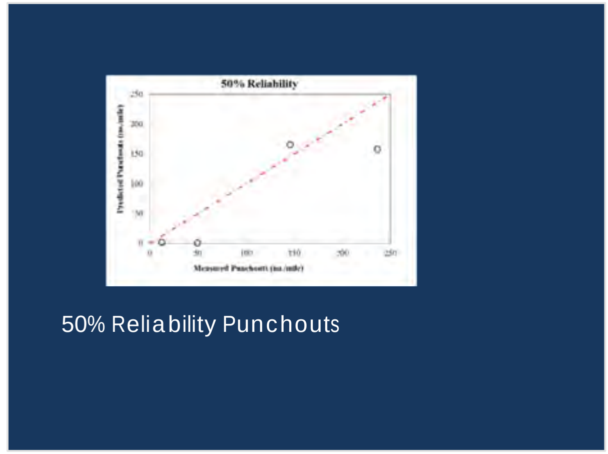


Table 4-4 Summary of Field Condition Distress Surveys for CRCP

Region	Project ID	Highway Number	Oregon Route Number	Begin MP	End MP	No. of Punchouts per Mile		
						Low	Medium	High
Valley	I-5 Corvallis/Lebanon Interchange	001	I-5	227.68	234.65	156.5	42	7.5
East	I-84:Stanfield Int-Pendleton	006	I-84	188.04	203.65	160.5	138.6	7
East	I-84:N.Powder-Baldock Slough	006	I-84	285.33	297.08	54.5	12.3	0
East	I-84:N.FK Jocobsen Gulch-Malheur River	006	I-84	368.16	374.08	394	215.1	21.1



Pennsylvania – MEPDG Implementation

AASHTO MEPDG REGIONAL PEER EXCHANGE MEETING

May 13-14, 2015

Albany, New York

Northeast Region Meeting

- Completed activities under AASHTO service units
  - Hands-on introductory training
  - Implementation Planning meeting with PennDOT personnel
  - Pavement-ME Implementation Plan
  - PA Turnpike participation

Northeast Region Meeting

- Plan to implement in 18 months
- Establish input libraries and defaults
- Verification Process
- Preliminary Design Guide
- Training
- Offer consultant access to software for Department projects under a signed agreement

Northeast Region Meeting

- Pavement Condition Survey Method – Automated
- Conducted introductory training and implementation meeting with PennDOT personnel from across the state.
- Developed Implementation plan from meeting input on pavement types of importance.

Northeast Region Meeting

- Superpave In-situ Stress Strain Investigation (SISSI)
- Rigid Inputs for MEPDG

South Dakota DOT ME Design Guide Materials Properties Experience

ME Design Guide Materials Properties Experience

Gill L. Hedman
Pavement Design Engineer
South Dakota DOT

April 14, 2015

Mechanistic-Empirical Pavement Design: Materials Testing of Resilient and Dynamic Modulus

Study SD 2008-13
Final Report
Prepared by
South Dakota School of Mines and Technology
Rapid City, SD 57701 May 2013

SD 2008-13

The objective of this study was to obtain resilient modulus and dynamic modulus values of construction materials through tests performed with an Asphalt Mixture Performance Tester (AMPT) at the South Dakota School of Mines & Technology (SDSMT). These values were obtained through testing of HMA paving materials and typical soil types within South Dakota to validate resultant data relative to the criteria defined for mechanistic-empirical pavement design processes and ultimate incorporation of the data into a mechanistic-empirical pavement design database.

SD 2008-13 Soils Testing

18 subgrade soils were tested and each subgrade soil sample was subjected to the following laboratory tests: particle size analyses, hydrometer analyses, Atterberg Limits, moisture and density relationships, California Bearing Ratio (CBR) determinations, and resilient modulus tests.

Summary of Subgrade Soil Properties

Material	Classification	% Gravel	% Sand	% Silt	% Clay	PI	MDD, pcf	OMC, %	CBR	Sat. CBR	k_1	k_2	k_3	M_R with σ_3=2psi & σ_d=6psi*
SD34 Lee's Corner	A-7-6, CH sandy fat clay	6.5	25.3	20	46.2	47	89.5	26	13.9	2.33	777.62	0.25	-1.27	8,690
I-90 by Blackhawk	A-4, CL-ML sandy silty clay	0.2	30.6	55.6	13.6	7	124	12	24.7	9.68	1019.6	0.75	-1.5	9,886
SD11/SD42 Minnehaha	A-4, ML silt	0.2	6.6	74.9	18.3	2	111	13	22.9	16.57	723.67	0.57	-1.9	6,787
SD44 E of Scenic	A-7-5, CH fat clay	1.2	11.3	38.8	50.8	73	103	19.5	23.2	3.49	908.71	0.51	-0.47	11,096
SD20 E of Prairie City	A-6, CL lean clay with sand	2.6	19.8	56.1	21.5	11	120.5	11.7	28.1	3.13	1482.63	0.48	-0.51	18,064
US281 Wolsey	A-6, CL SC clayey sand	11.2	49.9	22.1	16.8	15	103	18.5	2.2	2.37	470.2	0.65	-3.42	3,321
SD34 Forestburg	A-2-4, SM silty sand	1.9	74	14.1	10	NP	114	11.5	10.3	6.91	639.28	0.78	-1.6	6,053
US212 Orman Dam	A-4, CL-ML sandy silty clay	11.4	39.1	34.5	15	7	119	11	21.8	7.26	1399.56	0.5	-0.42	17,243
US83 Ft Pierre	A-7-6, CH fat clay	0.5	5.6	39.7	54.2	50	97.5	21.5	22.8	1.87	1865.46	0.34	0.09	14,641
US385 Custer/Hill City	A-2-4, SM silty sand with gravel	33.1	35.6	26.2	5.1	0	120.5	11	18.9	4.49	723.64	0.7	-2.96	5,485
US212 Subgrade (Task 4)	A-4, CL sandy lean clay	1.5	34.9	35.4	28.2	10	128.5	10.5	13.5	1.92	1926.3	0.42	-0.5	22,046
00GZ	A-7-6, CL sandy lean clay	0	32.0	34.0	34.0	24	118	14.8	35.0	1.92	1788.92	0.50	-0.41	22,104
00FI	A-7-6, CH fat clay	0	5.0	37.0	58.0	47	105	20.8	38.0	2.60	2887.67	0.43	-0.13	28,450
00JZ	A-7-6, CH fat clay	0	16.0	40.0	44.0	42	104.7	19.3	24.8	3.00	1510.50	0.46	-0.40	18,827
00XF	A-7-5, CH fat clay	0	15.0	30.0	55.0	40	108.1	18.2	37.0	1.18	1855.64	0.49	-0.49	22,684
6858	A-7-6, CH fat clay	0	10.0	35.0	55.0	47	109.3	17.0	58.0	1.45	1334.09	0.55	-0.13	17,133
6283	A-7-6, CH Fat Clay	0	1.0	34.0	65.0	51	98.5	21.5	36.0	2.50	1968.60	0.42	-0.08	26,438

SD 2008-13 Soils Testing

Two predictive equations were developed to provide a relationship between the resilient modulus and the gradation and CBR of the subgrade soils. Data from laboratory testing of the subgrade soils was utilized, along with multiple variable regressions, to develop the predictive equation. The resilient modulus predictive equation for subgrade soils is:

$$M_r = 10^{0.089 \cdot Ret_{3/8} - 0.063 \cdot G - 0.013 \cdot M + 0.037 \cdot C + 0.035 \cdot CBR + 3.335} \quad (11)$$

where:

M_r = resilient modulus, psi
$Ret_{3/8}$ = percentage retained on 3/8" sieve
S = percentage retained on #200
G = percentage of gravel
M = percentage of silt
C = percentage of clay
CBR = California Bearing Ratio at OMC

The value of the squared correlation coefficient, R^2, from the regression was 0.99 with an adjusted R^2, equal to 0.99. The pertinent statistical data from the regression analysis used to develop the predictive equation is given in Figure 70. The statistical data indicates all variables are significant and that the equation is a good fit to the data. Testing of additional subgrade materials may further validate the developed predictive equation.

SD 2008-13
Location of Subgrade Samples

SD 2008-13
Soils Testing

A predictive equation for higher plastic soils was developed in order to provide a relationship between the resilient modulus, the gradation and CBR of the subgrade soils. Data from laboratory testing of the subgrade soils with a plasticity index (PI) more than 40 was utilized, along with multiple variable regression, to develop the second predictive equation. The resilient modulus predictive equation for subgrade soils with a PI > 40 is:

$$M_r = 10^{0.329 \cdot S - 0.314 \cdot M + 0.332 \cdot C + 0.007 \cdot CBR + 27.948} \quad (12)$$

where:

M_r = resilient modulus, psi
S = percentage of sand
M = percentage of silt
C = percentage of clay
CBR = California Bearing Ratio at OMC

The value of the squared correlation coefficient, R^2, from the regression was 0.90 with an adjusted, R^2, equal to 0.77. The pertinent statistical data from the regression analysis used to develop the predictive equation is given in Figure 71. Testing of additional subgrade materials may further validate the developed predictive equation.

SD 2008-13
Base Testing

6 base materials were subjected to particle size analysis, Atterberg Limits, moisture and density relationships, and resilient modulus tests.

SD 2008-13
Summary of Base Properties

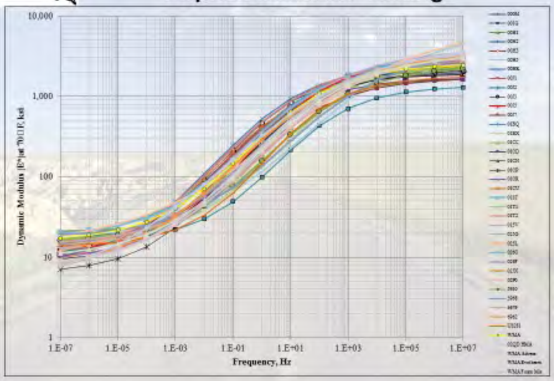
Material	Classifications	% Gravel	% Sand	% Fines	C_u	C_c	PI	MDD pcf	OMC %	k_1	k_2	k_3	M_R value with σ_3=10psi & σ_d=35psi*
RCVBC	A-1-b; SP, poorly-graded sand w/ gravel	23.8	74.4	1.8	6.7	0.8	NP	132	8.7	1013.68	0.68	-0.13	37,164
02KL	A-1-a; GW, well-graded gravel w/ sand	55.5	41.9	2.6	24.0	2.4	NP	133	7.6	628.71	0.65	0.16	27,133
00GR	A-1-a; SW, well-graded sand w/ gravel	30.9	64.9	4.2	16.3	1.0	NP	122.2	12	834.58	0.72	-0.16	31,858
6180	A-1-a; SP, poorly-graded sand w/ gravel	36.5	60.1	4.4	23.1	0.6	NP	124	10.8	534.97	0.68	0.25	26,315
US-281 (Task 4)	A-1-a; GP, poorly-graded gravel w/ sand	54	41	4	32	0.5	-	125	9.0	780.77	0.80	-0.35	28,896
US-212 (Task 4)	A-1-a; SW-SM, well graded sand w silt and gravel	47	47	6	25	1.4	-	126	8.2	1199.49	0.67	-0.37	36,002

SD 2008-13
Asphalt Concrete Testing

36 HMA pavement materials were analyzed in both dynamic modulus and repeated triaxial load tests.

Location of Asphalt Concrete Samples

SD 2008-13 Asphalt Concrete Testing



SD 2008-13 Conclusion and Recommendations

As a result of this project, it is recommended that the South Dakota Department of Transportation continue with the development of a material input parameter database for the Mechanistic-Empirical Pavement Design Guide. This would involve further testing of typical soil and road construction materials in South Dakota for resilient modulus and dynamic modulus, respectively. Additional tests are still recommended for HMAs from the west, subgrade materials from the east, and base materials from the overall of South Dakota. The additional testing and database development will ensure that proper material input values are utilized in future mechanistic-empirical pavement designs. The further testing of typical soil materials for resilient modulus will also allow for continued validation and refinement of the parametric relationships for the resilient modulus that was initially developed for low plasticity soils and high plasticity soils from this project's results.

Finally, it is not recommended that the South Dakota Department of Transportation procure an Asphalt Mixture Performance Tester (AMPT) at this time. The South Dakota School of Mines and Technology is fully capable of completing any required resilient modulus, dynamic modulus, and repeated load triaxial tests for the database development.

South Dakota Department of Transportation Research Project Statement Project SD2014-XX

Research Objectives:

Obtain dynamic modulus and Disk-Shaped Compact Tension values of HMA construction materials through tests performed with the Simple Performance Tester at SDSM&T on HMA paving projects and typical soils types around the state to validate resultant data relative to the criteria defined for M-E pavement design processes, and ultimate incorporation of the data into a AASHTO Pavement ME Design database.

SD2014-XX Research Tasks

1. Perform HMA pavement materials tests with the SPT for dynamic modulus, asphalt binder stability/flow ratio, and frequency sweep at constant height (FS-CH) whereby:
   - Six (6) HMA samples will be submitted each year for a total of 18 samples.
   - Each HMA sample will have 3 testing iterations performed for dynamic modulus, 3 testing iterations performed for stability/flow, and 3 testing iterations performed for FS-CH.
   - All materials samples will be collected and provided by the SDDOT.
2. Perform low temperature cracking tests by using the Disk-Shaped Compact Tension Test ASTM D7313(13) and the Semi Circular Bend Test TP 105 AASHTO Draft Specification on six (6) gyratory HMA samples submitted by SDDOT each construction season for a total of eighteen (18) samples.
3. Perform statistical analyses to evaluate all test results obtained from the HMA pavement materials and prepare the findings for review and approval of SDDOT/TIG members.

Questions?

Thank You!

For more information…

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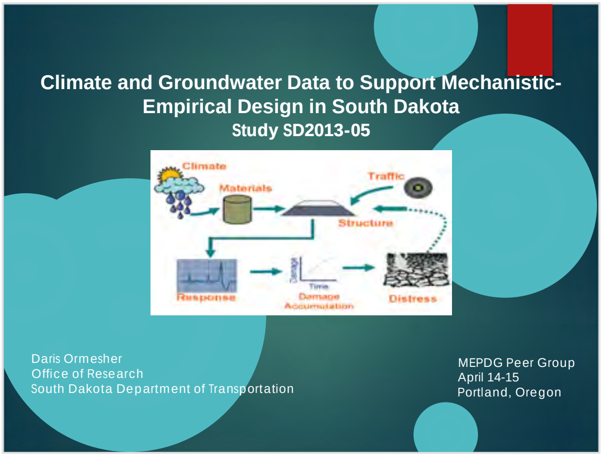
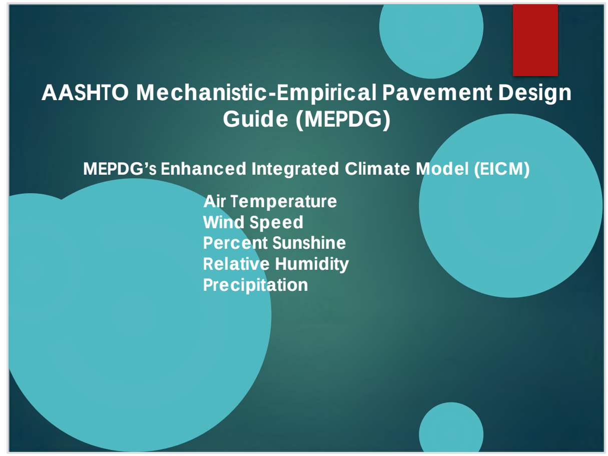
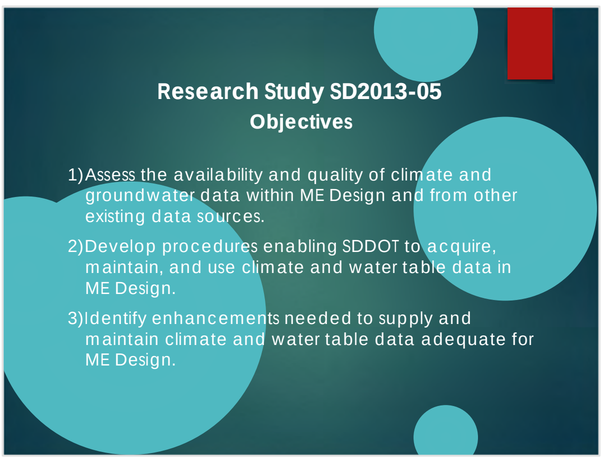
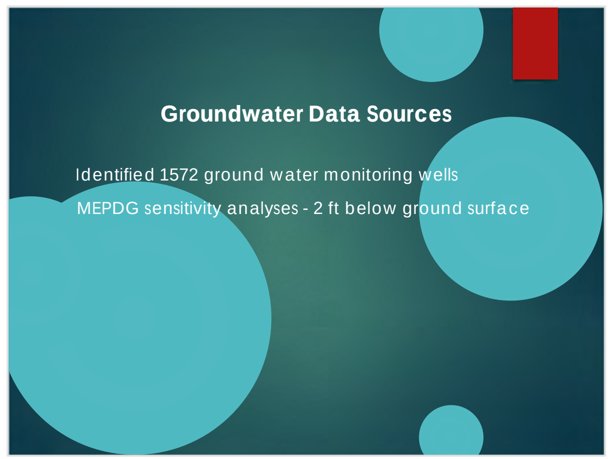
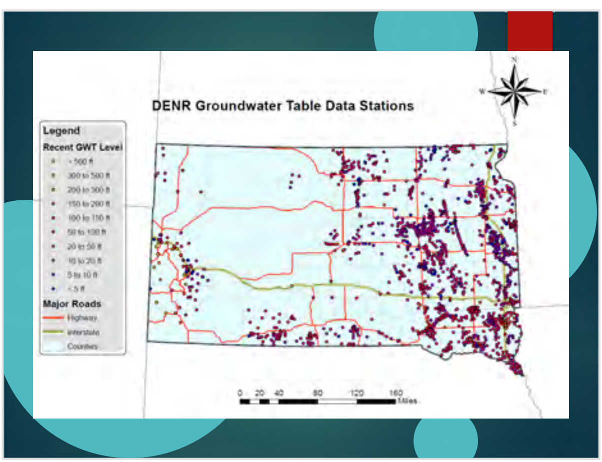
South Dakota DOT – Climate and Groundwater Data to Support Mechanistic-Empirical Design in South Dakota

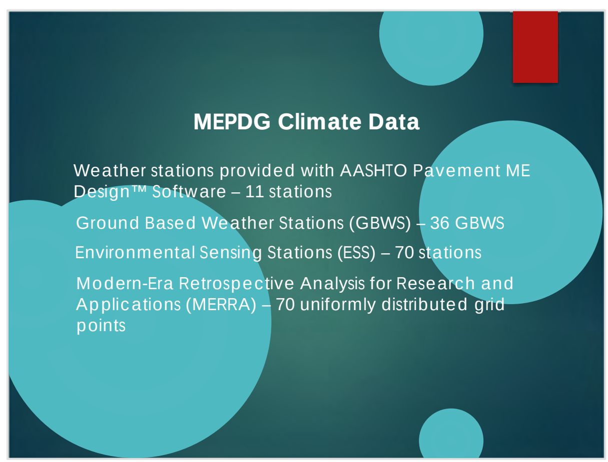


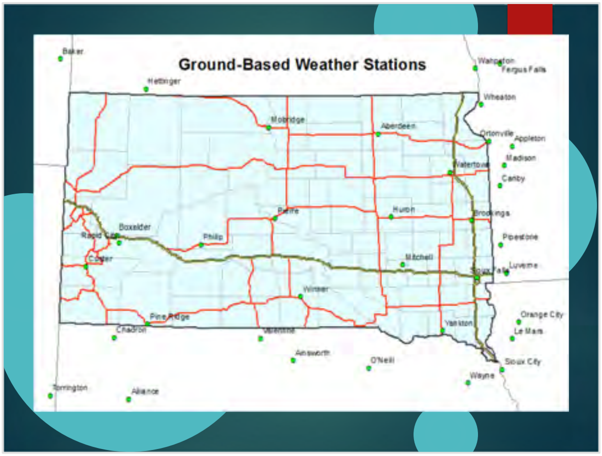


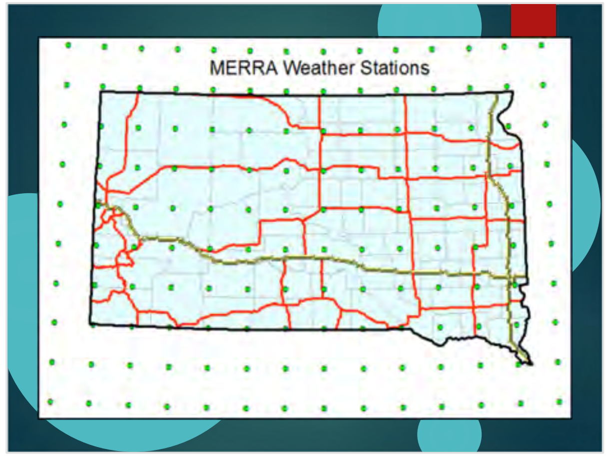


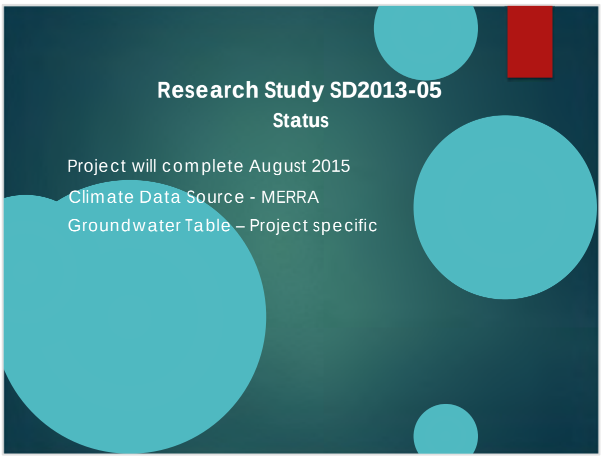







Ground-Based Weather Stations


MERRA Weather Stations


Research Study SD2013-05
Status
Project will complete August 2015
Climate Data Source - MERRA
Groundwater Table – Project specific


"It always rains on tents.
Rainstorms will travel thousands
of miles, against prevailing winds
for the opportunity to rain on a tent."
— Dave Barry

Utah DOT Implementation of the AASHTO Mechanistic-Empirical Design Guide in Utah

Implementation of the AASHTO Mechanistic-Empirical Design Guide in Utah

Steven Anderson & Mike Darter
State Pavement Design Engineer
UDOT & Applied Research Associates, Inc

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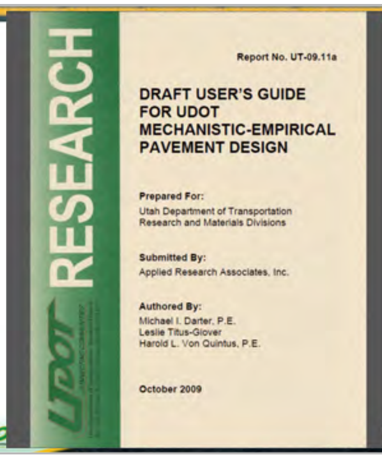
udot.utah.gov

PAVEMENT DESIGN
MANUAL OF INSTRUCTION
January 2014

- Key documents
  - UDOT Manual of Instruction
  - Final Report on Implementation, 2009
  - Recalibration of JPCP with correct CTE
  - Recalibration of HMA rutting model



RESEARCH

Report No. UT-09.11a

DRAFT USER'S GUIDE FOR UDOT MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

Prepared For:
Utah Department of Transportation
Research and Materials Divisions

Submitted By:
Applied Research Associates, Inc.

Authored By:
Michael I. Darter, P.E.
Leslie Titus-Glover
Harold L. Von Quintus, P.E.

October 2009

Rethink Pavement Design

- MEPDG Predicts Performance at a given reliability
- If we don't like the prediction we can change the prediction model or we can change the inputs
- The prediction model is changed through validation and calibration.
- The inputs are refined through testing and experience

Some Specific Advantages: HMA

Old AASHTO 1960-93	New AASHTO ME Design
• Structural design provides only SN, not HMA thickness!	• Directly provides HMA thickness prevent fatigue cracking & rutting
• No connection of asphalt binder grade to performance	• Asphalt binder grade directly related to fatigue cracking, rutting, and low temp cracking
• HMA & base layer coefficients not accurate	• HMA dynamic modulus & creep compliance meas.
• ESALs used for traffic	• Actual axle loads & types
• Climate not considered	• Climate directly considered
• Rehab does not consider reflection cracking	• Rehab directly considers reflection cracking

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Some Specific Advantages: PCC

Old AASHTO 1960-93	New AASHTO ME Design
• Structural design provides only PCC thickness for Serviceability (PSI)	• Directly provides PCC thickness to prevent fatigue cracking, rutting & IRI
• No connection of thickness to joint spacing & load transfer	• Directly connects slab thickness to joint spacing and load transfer
• Base layer benefits not fully considered	• Base layer fully considered through elastic modulus and friction with slab
• ESALs used for traffic	• Actual axle loads & many truck characteristics
• Climate not considered	• Climate directly considered

Utah Testing of Untreated Base Course Resilient Modulus

Utah Resilient Modulus Testing Untreated Base Course (Guthrie, BYU 2013)

UDOT Region	Material	Repetition	Resilient Modulus (ksi)	K_1	K_2	Resilient Modulus COV (%)	Average Resilient Modulus (ksi)
1	Staker Parson, McGuire, Willard	1	27.3	11603	0.458	14.1	24.8
		2	22.1	12302	0.315		
		3	28.4	12053	0.468		
		4	21.6	7306	0.585		
	Stake Parson, Trenton Pit (US-91 Source)	1	31.3	15160	0.391	4.3	32.2
		2	33.2	15176	0.426		
2	Staker Parson, Beck St., Salt Lake City	1	25.8	17314	0.212	8.2	25.5
		2	23.2	15311	0.219		
		3	28.2	5202	0.451		
		4	24.9	6852	0.344		
	Killgore, Parley's Canyon	1	24.9	4396	0.461	4.6	25.7
		2	26.6	4018	0.503		
3	Geneva Rock, Point of the Mountain	1	15.7	7696	0.384	16.8	17.8
		2	20.0	12283	0.258		
	Maesar East Pit, Vernal	1	25.6	14541	0.300	3.5	26.2
		2	26.9	11335	0.466		
4	Staker Parson, Elsinore	1	21.4	12200	0.288	9.4	20.1
		2	18.8	10362	0.315		
	Nielson Construction, Ferron (SR-10 Source)	1	21.5	12107	0.305	3.1	22.0
		2	22.5	14273	0.237		

Materials: Unbound Aggregate Base Course, Mr

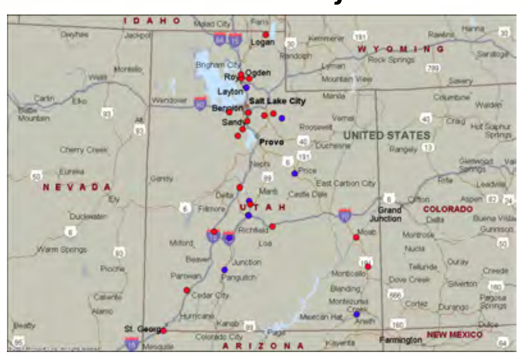
- Resilient moduli from several sources for unbound aggregate base course resilient modulus, Mr showed a range of Mr from 18,000 to 32,000 psi.
- Average = 25,000 psi.
- **Problem:** The Utah flexible and rigid pavements used in calibration (LTPP and non-LTPP) used higher Mr ranging from 25,000 to 40,000 psi.
- Thus, if we now use 25,000 psi in all designs, a thicker HMA pavement will be obtained. PCC pavement not likely affected significantly by this reduction.
- A re-validation is needed to ensure no bias is involved.

Utah Validation of Distress & IRI Prediction Models & Recalibration of Biased Models

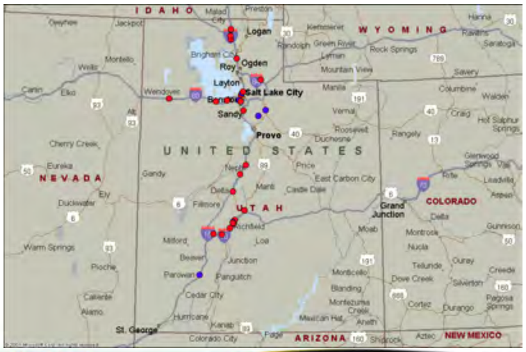
Validation & Calibration of Distress & IRI Models

- Utah Sections: LTPP & Non-LTPP
  - Selected 28 HMA and 23 JPCP
  - Obtained all inputs to run AASHTO ME
  - Run AASHTO ME and examine predictions
  - Obtain all measured performance data
  - Compare predicted & measured performance

28 Utah HMA Projects



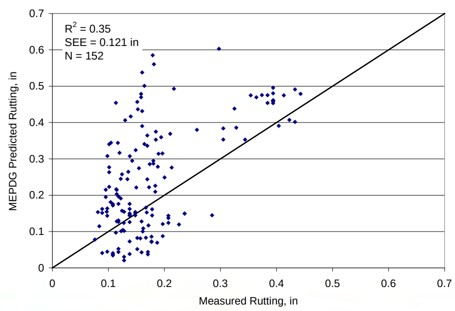
23 Utah JPCP Projects



Summary HMA Utah Validation

- **Alligator cracking**: Valid within small range of cracking. Needs future validation
- **Transverse cracking**: Valid for PG binders/mixes.
- **Rutting**: Over predicted (biased). Required recalibration to match Utah pavement/ materials. Similar accuracy to national.
- **IRI**: Unbiased and similar accuracy to national calibration.

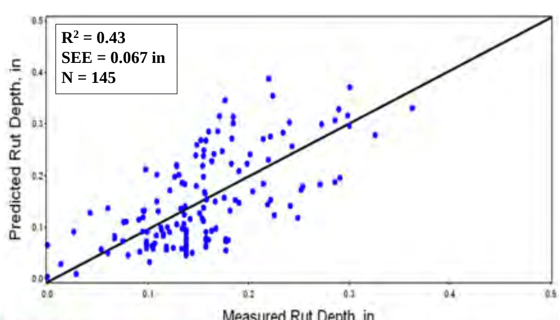
Bias Of National Rut Model In Utah



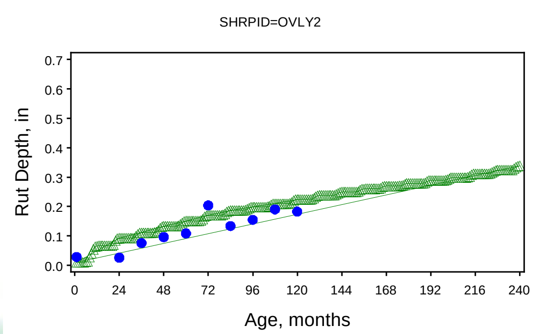
Results for the 2009 Utah, and the 2013 Utah Recalibrations.

- **MEPDG Total Rutting Model= BR1 HMA + BS1*UTBC + BS1*SUBG** (2007 original in software)
- **Local Utah Calibration 2009 = 0.56 HMA + 0.604 UTBC + 0.40 SUBG**
- **Local Utah Calibration 2013 = 0.58 HMA + 0.71 UTBC + 0.28 SUBG**

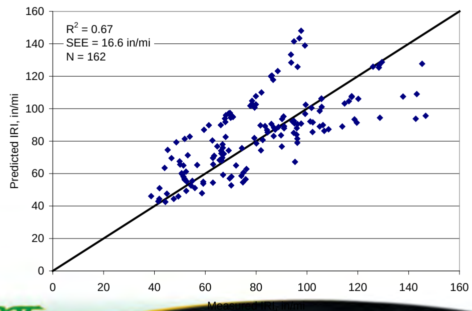
Predicted Vs Measured Rutting, 2013



Section OVLY2 I-15 Millard Co.



Measured Vs Predicted IRI for HMA



National JPCP Re-Calibration 2011
(due to test lab error in CTE testing)

- FHWA provided ARA with "correct" CTE values for all JPCP projects (including UTAH).
- UDOT/U Utah testing provided correct CTE for non-LTPP sections
- ARA re-calibrated the cracking and faulting models using "correct" CTE values. New calibration coefficients were obtained.
- Work done under NCHRP 20-07 project.

Utah CTE Testing

- An MS thesis was done at the U of Utah using UDOT Labs that tested concrete cylinders from 19 aggregate sources around the State.
- Rigby, M. T. and P. Romero, "Coefficient of Thermal Expansion of Portland Cement Concrete in Utah and Mechanistic Empirical Pavement Design Guide Implementation," Technical Report, Utah Department of Transportation, January 2010.
- The CTE values obtained are the "correct CTE".

Utah Concrete CTE Testing

Pit Location	Concrete Supplier	Avg. CTE (x 10-6 in/in/°F)	Primary Aggregate Classification
Moab	LeGrand Johnson	4.42	Granite/Quartzite
Monticello	Sonderegger Inc.	5.33	Quartzite
Cedar City	Sunroc Corp.	4.33	Limestone
Hurricane	Interstate Rock Products	4.27	Volcanic
St. George	Sunroc Corp.	4.63	Limestone
Tooele	Harper Ready Mix	5.96	Quartzite
Pt. of Mountain	Geneva	5.79	Quartzite
Mouth of Big Cottonwood Canyon (Walker Pit)	Binggeli Rock Products	5.24	Quartzite
Heber City	Binggeli Rock Products	6.02	Quartzite
Brigham City 7.8 Bag Mix	JBP	6.08	Quartzite
Highland	Westroc	4.60	Limestone/Dolomite
Vernal	Binggeli Rock Products	5.47	Quartzite
Randlett	Tri-County Concrete	6.08	Quartzite
South Weber	Geneva Rock	6.16	Quartzite
Nephi	Staker Parsons	5.13	Quartzite
Brigham City 6.5 Bag Mix	JBP	6.02	Quartzite
Elsinore	Western Rock	4.64	Volcanic
Nibley	LeGrand Johnson	5.15	Limestone
Fruitland	Cross Roads Concrete	5.93	Quartzite

Revalidation of Utah JPCP 2011
(Using Correct CTE)

- **Transverse fatigue cracking**: Validated NCHRP 20-07 calibration 2011 for Utah pavement/materials with correct CTE.
- **Joint faulting**: Validated for NCHRP 20-07 calibration for Utah pavement/ materials with correct CTE.
- **IRI**: Unbiased and similar accuracy to national calibration with corrected CTE.

National CRACKING Model Coefficients
(NCHRP 20-07, 2011 Correct CTEs)

S-Shaped Curve,
Or Transfer Function

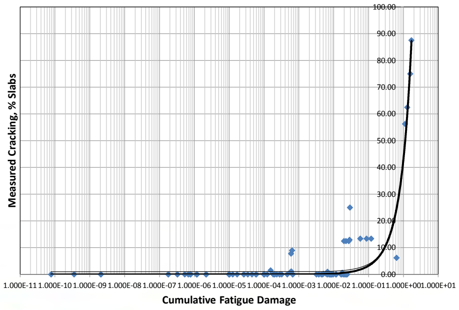
$$CRK = \frac{1}{1 + C_4 (DI_F)^{C_5}}$$

CRK = Percent Slabs Transverse (fatigue) Cracked
DI = Accumulated Fatigue Damage (Miner)

Std. Dev(CRK) = $1.5+(57.08*PCRK)^{0.33}$

C4 = 0.6 & C5 = -2.05 were determined in calibration through regression with field cracking & correct CTE

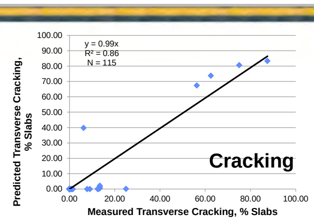
Slab Cracking Vs Fatigue Damage
2011 CTE Recalibration Utah

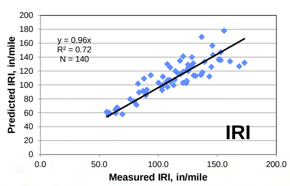


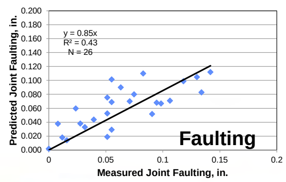
National FAULTING Model Coefficients (NCHRP 20-07, 2011 Corrected CTE)

Model Coefficients	Values
C1	0.51040
C2	0.00838
C3	0.00147
C4	0.008345
C5	5999
C6	0.8404
C7	5.9293
C8	400
R^2	0.5900
SEE	0.0320
N	1184

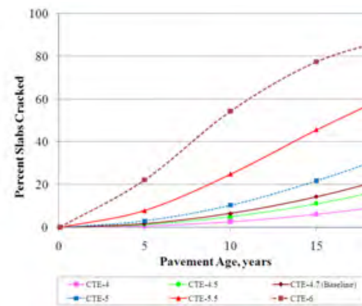
Validation of NCHRP 20-07 Calibration In Utah With Correct CTE Values



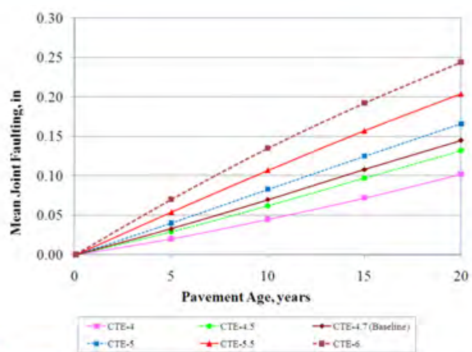




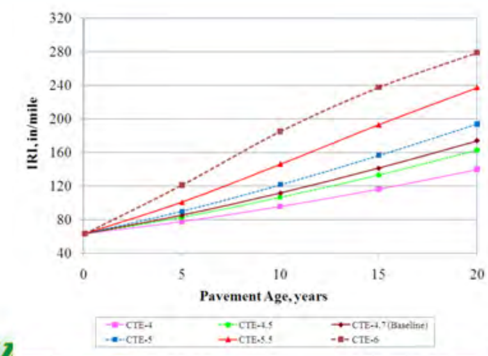
CTE Effect On Trans. Cracking



CTE Effect On Faulting



CTE Effect On IRI



Utah Standard Deviations

Model	Standard Deviation
Transverse Cracking	$1.5+(57.08*PCRK)^{0.33}$
Joint Faulting	$0.0831*(PFLT^{0.3426})+0.00521$

UDOT MEPDG Input Levels

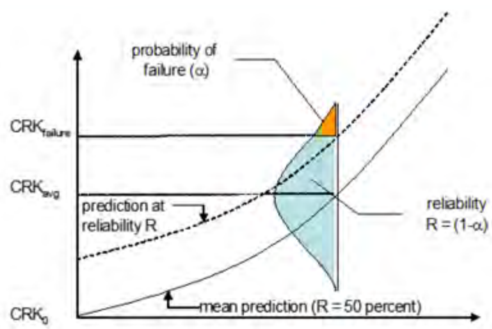
- **Hot mixed asphalt:** Mostly Levels 2 and 3 (with some Level 1 for major projects or unusual materials)
- **Concrete slab:** Same as HMA.
- **Unbound aggregates and soils:** Mean lab tested, Level 3, & FWD testing & backcalculation.
- **Climate:** All Level 1 weather stations.
- **Traffic loadings:** Mostly Level 1, some Level 2 & 3 remote highways.
- **Rehabilitation:** Levels 1(FWD & backcalculation of moduli), plus many Levels 2 and 3.

Implementation Accomplishments

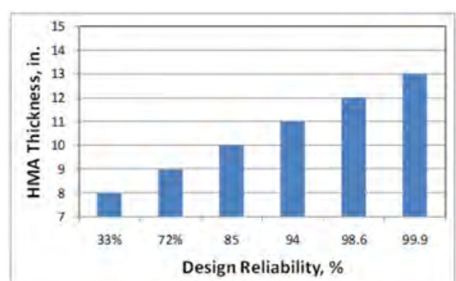
1. **INPUTS:** Recommend UDOT defaults and procedures to obtain proper inputs for the AASHTO ME Design for use in designing asphalt, concrete and rehabilitated pavements.
2. **CALIBRATION:** Verified the National calibration factors and developed new calibration factors for Utah (rutting).
3. **USER'S GUIDE:** Prepared a detailed Manual of Instruction (User's Guide) tailored to UDOT: input procedures, sensitivity, procedures, materials, software, examples.
4. **TRAINING:** Provided hands-on training to UDOT staff and consultants as well as presentations to upper level UDOT. Repeated training many times over the years.

Utah Design Reliability Criteria

Design Reliability Concept



HMA Design Thickness Vs Reliability



Utah Design Reliability Criteria

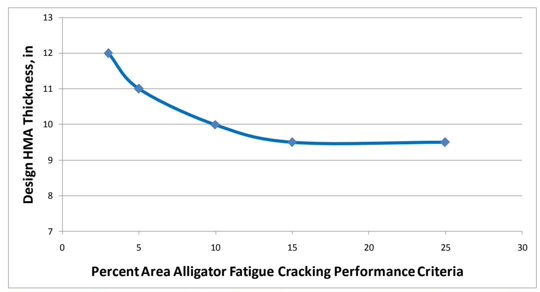
Recommended Level of Reliability	
Functional Classification	Reliability
Interstate Urban and Rural	95
Non Interstate Principal Arterials, Collectors, Local	90

Utah Design Performance Criteria

Utah Performance Criteria for Use in HMA Pavement Design

Pavement Type	Performance Criteria	Maximum Value at End of Design Life at Design Reliability***
HMA pavement and overlays	HMA bottom up fatigue cracking (alligator cracking)	Interstate: 10 percent lane Primary: 15 percent lane Secondary: 25 percent lane
	HMA longitudinal fatigue cracking (top down)**	UDOT does not use in design due to deficiencies in the model. Enter 20,000 ft/mile to avoid triggering reliability.
	Total permanent deformation (rutting of both wheel paths)	Interstate, Primary: 0.75 inch mean
	AC permanent deformation	Interstate, Primary: 0.75 inch mean
	Thermal fracture (transverse cracks)	Interstate: Crack spacing > 70-ft Crack length < 905-ft/mile Primary/Secondary: Crack spacing > 50-ft Crack length < 1,267-ft/mile
	IRI	Interstate/Primary: 170 inch/mile maximum* Secondary: 170 inch/mile maximum*

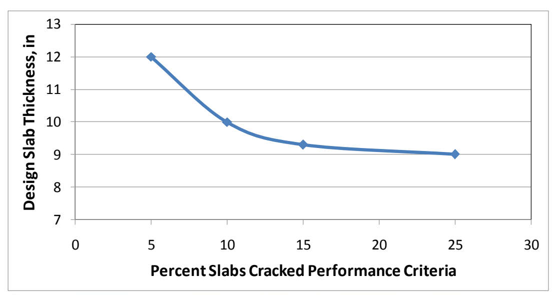
Effect Alligator Cracking Criteria



Utah Performance Criteria for Use in JPCP Design

Pavement Type	Performance Criteria	Maximum Value at End of Design Life at Design Reliability***
JPCP new, CPR, and JPCP overlays	Mean joint faulting	Interstate: 0.15 inch mean all joints Primary: 0.25 inch mean all joints Secondary: 0.25 inch mean all joints
	Percent transverse slab cracking	Interstate: 10 percent Primary: 15 percent Secondary:20 percent
	IRI	Interstate: 170 inch/ mile* Primary/Secondary: 170 inch/mile maximum*

Effect Slab Cracking Criteria



Utah Design Performance Criteria
(At Reliability Level)

Initial IRI Values for New and Rehabilitated Pavement Design			
Pavement Type	IRI – inch/mile		
	Average	Minimum	Maximum
New HMA and HMA/HMA	50	32	106
New JPCP	60	52	116
JPCP subjected to CPR	74	65	85

UDOT

Utah Traffic Data & Files

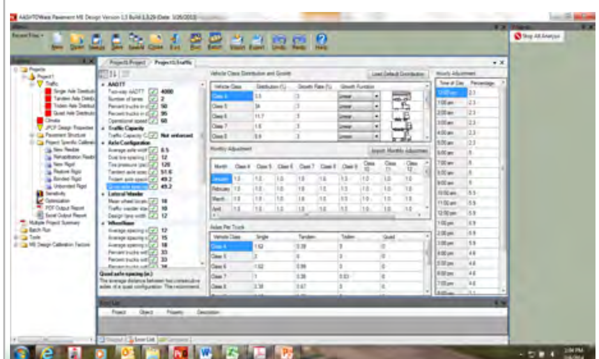
UDOT

Input Procedures: Traffic

- UDOT has sufficient WIM and ATC to generate all needed Level 1 inputs.
  - Truck volume and growth.
  - Vehicle classification distribution.
  - Truck axle load distribution for S, T, T, Q
  - Lane distribution.
  - Truck tractor Wheel base: 2(short), 42(medium), 56(long) percent.
  - Hourly truck distribution
  - Other defaults.

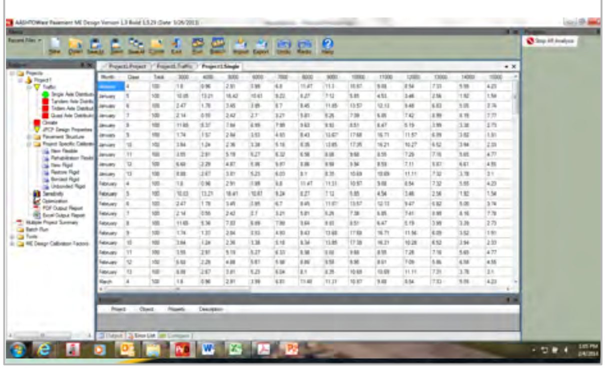
UDOT

Traffic Input Overall Screen Shot



UDOT

Traffic Input Files: Axle Load Distr.



Traffic Input Files List

- Traffic: Initial AADTT, axle configuration, lateral wander, wheel base
- Vehicle class distribution
- Traffic growth
- Axles per truck
- Hourly distribution
- Monthly distribution
- Axle load distribution (single, tandem, tridem, quad)

UDOT

"Old" MEPDG Files

- 10 traffic input files (these are "Old" MEPDG files that UDOT has been generating since 2005 for project design):
  - AxlesPerTruck.txt
  - MonthlyAdjustmentFactor.txt
  - Traffic.txt
  - VehicleClassDistribution.txt
  - HourlyDistribution.txt
  - TrafficGrowth.txt
  - Single.alf
  - Tandem.alf
  - Tridem.alf
  - Quad.alf

Old Axles Per Truck file

,Single,Tandem,Tridem,Quad
Class 4,1.66,0.33,0.01,0
Class 5,2,0,0,0
Class 6,1.17,0.89,0,0
Class 7,1,0.01,0.87,0.13
Class 8,2.22,0.67,0.04,0
Class 9,2.08,1.79,0,0.03
Class 10,1.33,1.18,0.63,0.09
Class 11,4.59,0.08,0.02,0
Class 12,2.14,1.49,0.19,0.06
Class 13,3.94,1.23,0.17,0.26

Old Monthly Adjustment Factor file

Month,Class 4,Class 5,Class 6,Class 7,Class 8,Class 9,Class 10,Class 11,Class 12,Class 13
January,0.87,0.87,0.88,0.47,0.61,0.85,0.47,0.67,2.7,0.65
February,0.88,0.93,0.78,1,0.75,0.93,0.39,1.08,3.01,0.78
March,1.08,1.07,0.87,1.09,1.11,1,0.55,1.97,3.31,0.85
April,1.12,1,0.92,0.4,1.17,0.99,0.52,2.57,2.96,0.93
May,0.97,0.82,0.95,1.29,1.3,1.01,1.42,0.79,0.01,1.03
June,1.32,1.08,1.32,1.84,1.41,1.03,1.45,0.8,0,1.02
July,1.28,1.22,1.39,1.4,1.37,0.95,1.29,0.74,0,0.91
August,1.29,1.06,1.51,1.16,1.25,1.05,1.36,0.71,0,1.01
September,1.11,0.95,1.19,0.99,1.2,0.91,1.06,0.67,0,1.41
October,0.77,0.99,0.6,0.12,0.75,1.15,1.09,0.66,0.01,1.16
November,0.66,1.01,0.73,0.8,0.6,1.11,1.13,0.6,0,1.17
December,0.66,1,0.86,1.44,0.48,1.02,1.27,0.74,0,1.07

Old Traffic file

13506
2
51
60
75
Lines with # signs in them are optional and for user information only.
#Line 1 - Initial two-way AADTT
#Line 2 - Number of lanes in the design direction
#Line 3 - Percent of trucks in the design direction
#Line 4 - Percent of trucks in the design lane
#Line 5 - Operational speed

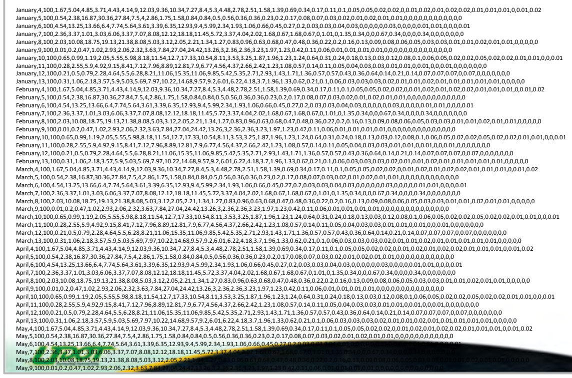
Old Traffic Growth file

0
Linear
5.35
Lines with # signs in them are optional and for user information only.
Line 1 - (0) Composite vehicle class growth - (1) Vehicle-class specific growth
"# Line 2 - Input growth rate (one of the following): No, Linear or Compound"
Line 3 - Growth Rate – Number (%) growth rate

Old Vehicle Class Distribution file

2.54,Class 4
2.54,Class 4
19.57,Class 5
2.18,Class 6
0.02,Class 7
20.26,Class 8
46.69,Class 9
1.43,Class 10
1.49,Class 11
0.37,Class 12
5.44,Class 13
Lines with # signs in them are optional and for user information only.,
#Line 2-11: Percent trucks in Class, Class #(optional)
#The input class number is optional. Minimum requirements for this file is 10 numbers that sum to 100.,

Old Single Axle Distribution file (Tandem, Tridem, Quad same)



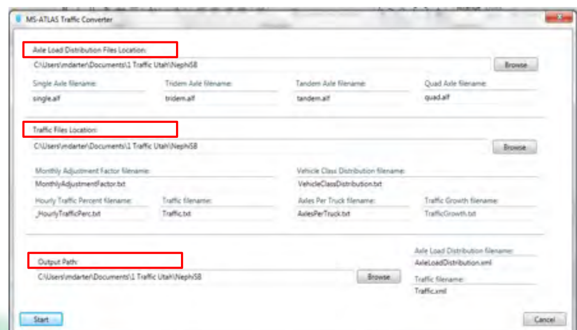
"New" Traffic Files in XML Format: AASHTO ME Software

- **There are only two files now:**
  1. **Traffic** (all traffic data except axle load distribution)
  2. **Axle Load Distribution** (single, tandem, tridem, quad)
- **Input format for all these New files is XML**
- Question? How does the Traffic Bureau get the ten "Old" traffic files into the two "New" XML files for use by designers?

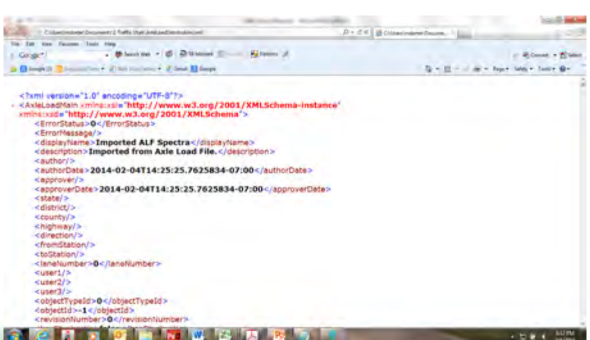
"Traffic Converter" Software

- The **Traffic Converter program** can convert the 10 "old" files into 2 "new" XML format files instantly.
- The **Traffic Converter** was developed by ARA specifically for the Mississippi DOT traffic format of the 10 "old" files. There appears to be very little difference in the Mississippi formats and the Utah formats but some modification was necessary to read old UDOT files (a small change to software may be necessary).

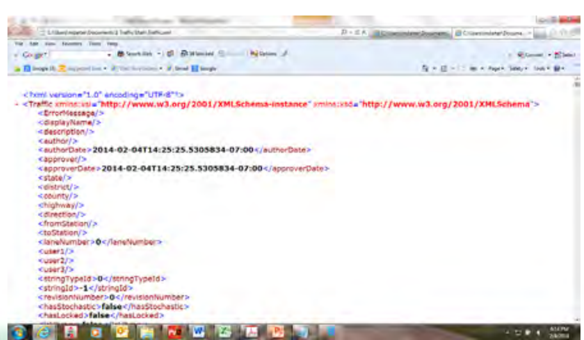
Traffic Converter Program (ARA)



Axle Load Distribution: XML Format



Traffic, XML Format



Other Ways to Enter Traffic Data

- **Monthly Adjustment Factors** can be imported from "old" MEPDG format into the software by Right Clicking on cell in the table the select "Import Monthly Factors in MEPDG (.txt) Format" option.
- **Axle Load Distribution** can be imported from "old" MEPDG file format into the software by Right Clicking "Single Axle Load Distribution" node in the Explorer plane under the Projects tree view, then select "Import ALF File" option. This menu option retrieves axle load distribution factors from a MEPDG (.alf) format to over write any information in the current table.

Other Ways to Enter Traffic Data, Continued

- Note: in order to import all 4 axle load distribution files in .alf format, you have to manually combine all 4 axle load distribution files (.alf) into one file (.alf). Give a file name and save. Make sure to maintain the following sequence: 1) single, 2) tandem, 3) tridem, 4) quad.
- You can also import traffic data in XML format from an agency Oracle database by using the "Get from database" option. You can access this option by right clicking the "Traffic" node in the Explorer pane under the Projects tree view.
- Another time consuming approach is to copy and paste from spreadsheets that include data in the correct format.

How did We Come up with AASHTO ME Design inputs ?

- Used Utah specific values as much as possible
- Utah Calibration Factors and standard deviation were validated, and if biased, new calibration and standard deviations derived.
- Input procedures for each input developed.
- Utah Material Properties (HMA and JPCP).
- Design reliability established.
- Pavement distress and smoothness criteria est.
- Training with pavement type Examples!

AASHTO ME Design Hierarchical Inputs

- **Level 1:** Testing and measuring (lab tests, FWD deflections, WIM).
- **Level 2:** Correlations between standard UDOT tests with ME Design inputs.
- **Level 3:** Defaults obtained from typical local Utah materials and traffic.

Where are we now

- All UDOT pavement designs are done using AASHTO ME.
- Beginning in July 2015 all federally funded local government projects will required to use AASHTO ME.

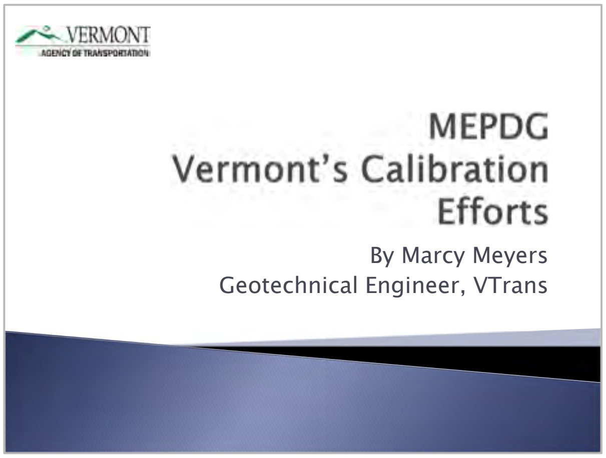
A Few Random Thoughts

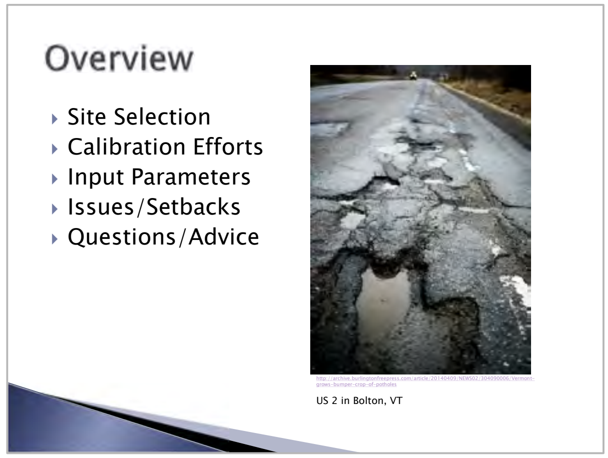
- You cannot just implement and forget, inputs should be a continuous work in progress using feedback from designers (e.g., problem designs).
- Don't throw away your existing knowledge.
- Program predicts average distress, you get to pick the reliability level.

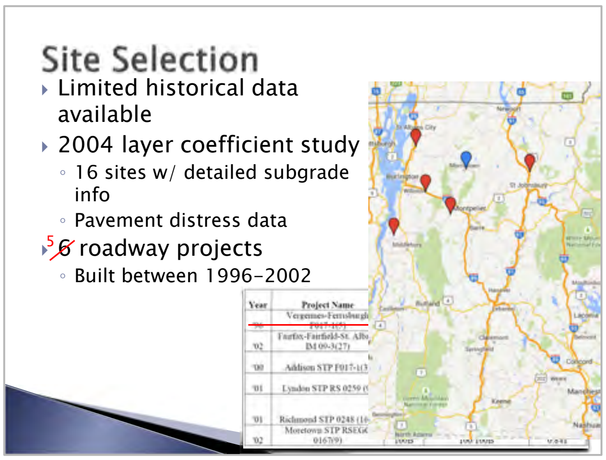
Challenges/Issues/Roadblocks

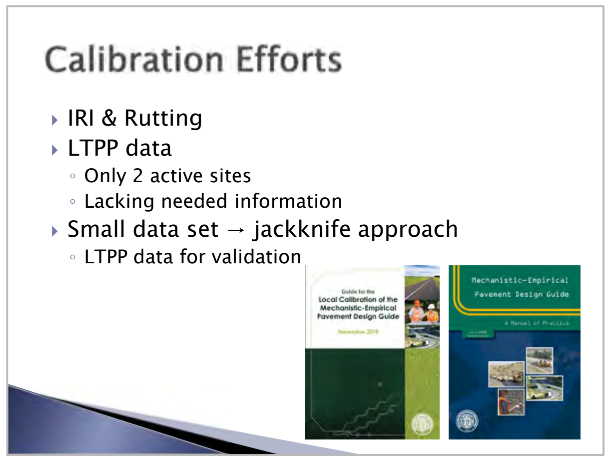
- Training: Many training sessions needed!
- Improvements: Improved ways to obtain inputs, rehab improvements (OL design)

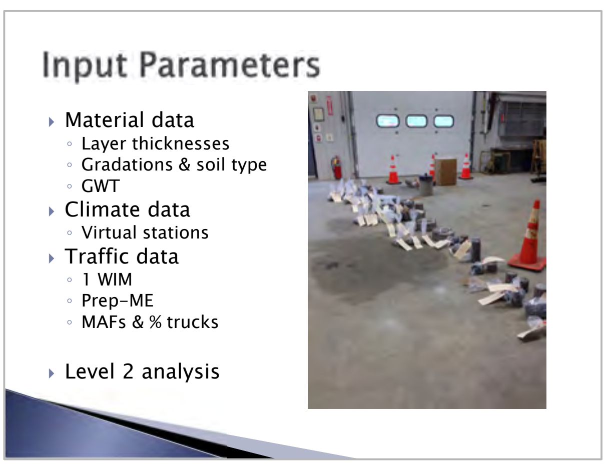
Vermont – MEPDG Vermont's Calibration Efforts

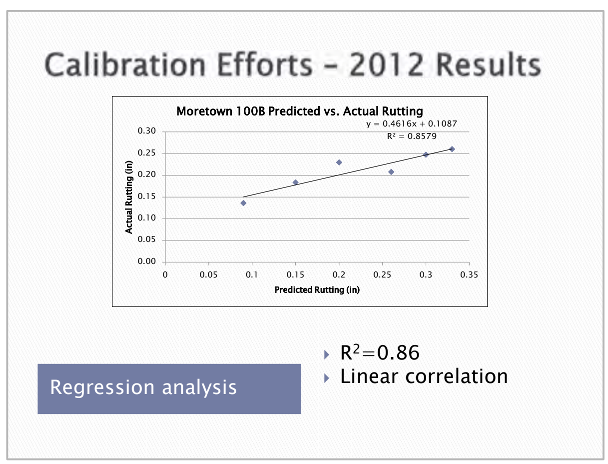




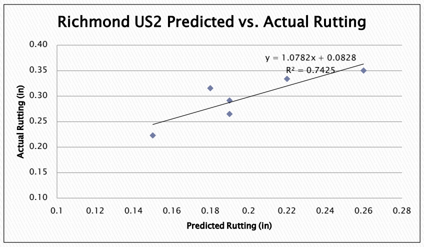








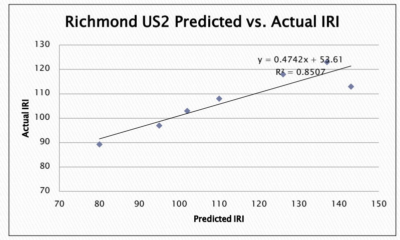
Calibration Efforts – 2012 Results



Regression analysis

- R^2=0.74
- Linear correlation

Calibration Efforts – 2012 Results



Regression analysis

- R^2=0.85
- Linear correlation

Calibration Efforts – 2012 Results

- Rutting R^2 Values:
  - 0.65, 0.82, 0.86, 0.74
- IRI R^2 Values:
  - 0.17, 0.43, 0.85, 0.85

- Measured and predicted values followed same trend of distress over time
- Wasn't enough statistical difference to warrant correction factors at the time
- Variation in collected data led to uncertainty about long term conclusions
- "Collect and re-asses"

Calibration Efforts – 2014-2015

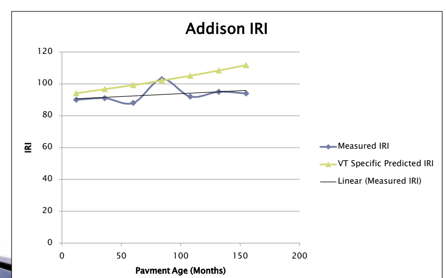
- Monthly adjustment factors
- Site specific % truck distribution
- Addition of one new test site
  - New road construction
  - Lab tested M_R values
  - Installation of WIM
- Step 7

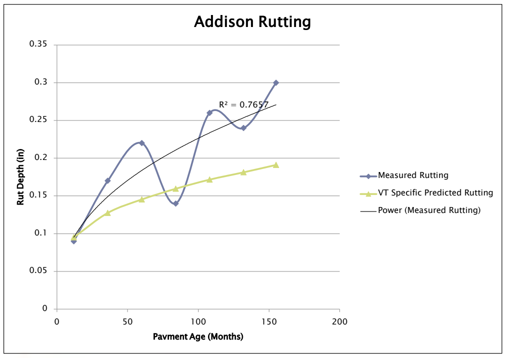
Calibration Efforts – 2014-2015 Preliminary Results

- Rutting R^2 Values:
  - 0.62, 0.83, 0.84, 0.73
  - 0.65, 0.82, 0.86, 0.74
- IRI R^2 Values:
  - 0.13, 0.44, 0.75, 0.62
  - 0.17, 0.43, 0.85, 0.85
- Still awaiting data
  - Inspection dates
  - Traffic MAFs

Issues/Setbacks

- Data quality
  - Field measured values





Issues/Setback

- Staffing
  - Part-time effort
- Getting & formatting the data
  - WIM data
  - Prep-ME
- User's manual
  - Data parameters/analysis

Questions/Advice?

Virginia – Local Calibration

VDOT Local Calibration

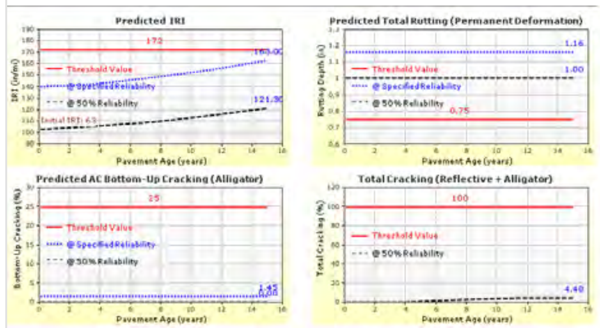
Pre-Calibration Work

- VCTIR research
  - Asphalt
  - Traffic
  - Subgrade
  - Existing Pavement
- VDOT manual for Pavement ME (not published)

Calibration procedure

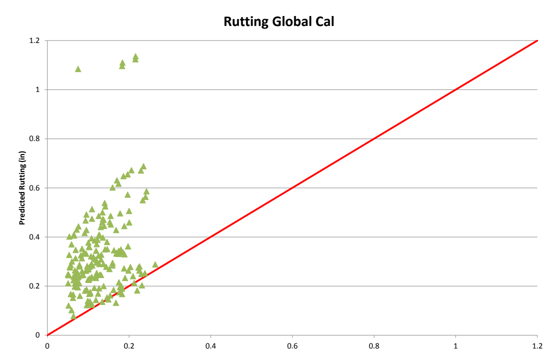
- Measured IRI values that decreased greater than 10% in a given year assumed the pavement had been resurfaced.
- Also compared year of last rehab from PMS records to remove data points on sites that had been resurfaced
- Removed ME distress predictions that were erroneous or missing measured distress points.
- Sites split into calibration/validation sets based on district

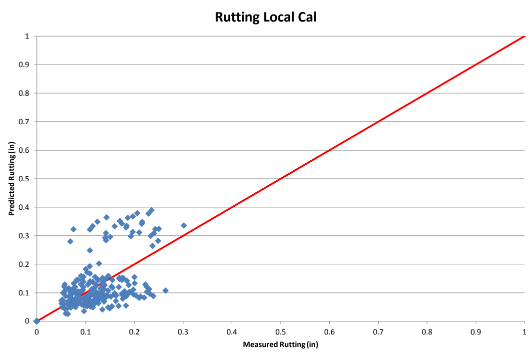
Errors in Prediction



Factorial design

	Asphalt Thickness				
Base Type	5 - 7"	7.1 - 9"	9.1 - 11"	11.1 - 13"	>13"
Graded Aggregate Base	3	8	6	8	5
Cement Stabilized Aggregate Base	2	8	7	2	0
Other	0	1	4	2	0

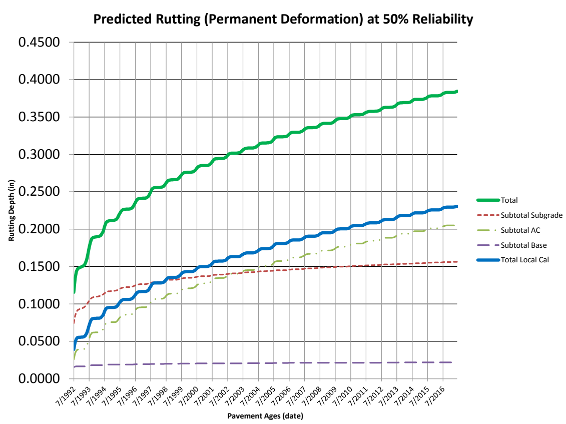


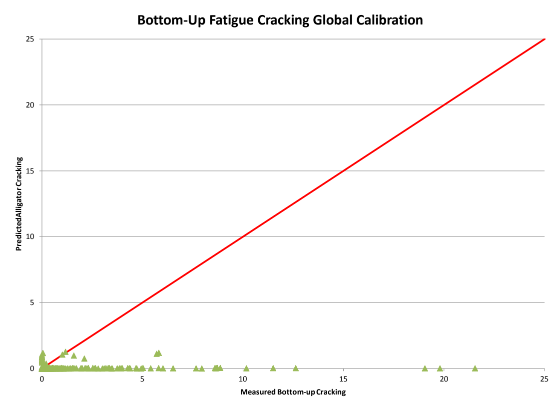


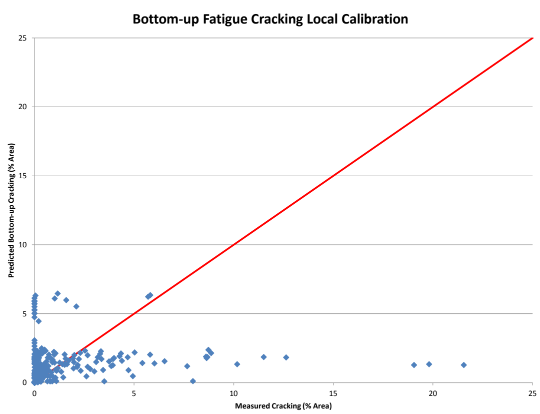
Rutting Calibration Statistics

	Global	Cal	Val	All
N	236	198	38	236
Bias	-0.214	0.000	0.023	0.000
Se,	0.183	0.079	0.033	0.076
R^2, %	16.5	22.2	42.8	23.7
Paired t	0.00	1.00	0.0001	1.00
t-test m	0.017	0.050	0.000	0.050
T-test b	0.000	0.069	0.005	0.069
Se/Sy	3.52	1.50	0.76	1.47
Br1	1.000	0.664	0.664	0.687
Bs1 – fine	1.000	0.151	0.151	0.153
Bs1 – gran	1.000	0.151	0.151	0.153

- No Bias
- Se <0.10
- Slope ~ 1.0
- Intercept ~ 0.0
- R2 and Se/Sy poor



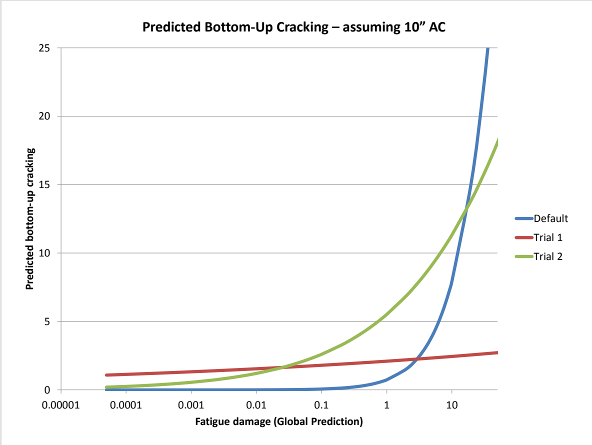


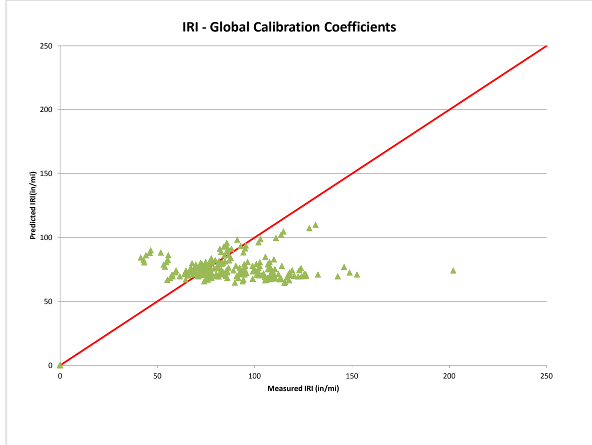


Bottom Up Fatigue Calibration

- Bias removed
- Se < 7%
- slope not equal to 1
- intercept not equal to 0
- R2 is poor

	Global	Cal	Val	All
N	233	195	38	233
Bias, %	1.486	0.000	0.003	0.000
Se, %	3.10	3.52	2.21	3.34
R^2, %	0.51%	3.34%	6.42%	3.04%
Pair t	0.000	1.000	0.9940	1.000
Ttest m	0.000	0.000	0.000	0.000
Ttest b	0.000	0.000	0.000	0.000
Se/Sy	1.005	1.095	0.993	1.085
Bf1	1.000	42.87	42.87	42.87
C1	1.000	0.3190	0.3190	0.3190
C2	1.000	0.3190	0.3190	0.3190





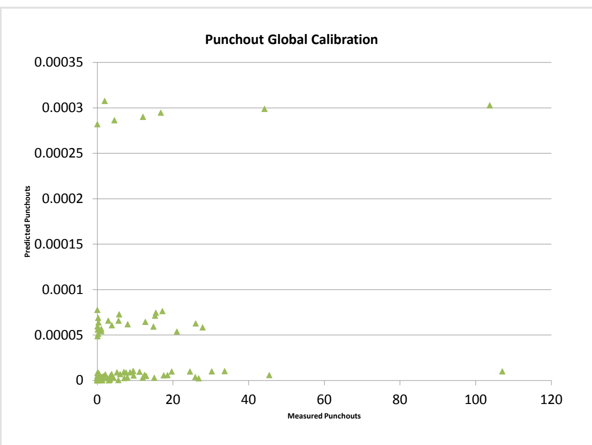
Local Calibration Statistics

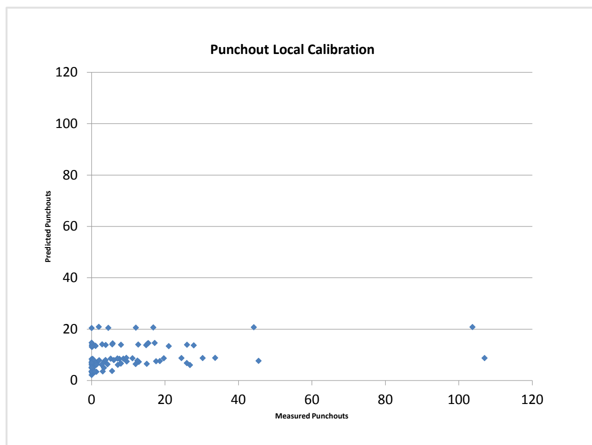
- removed bias through C4
- Slope not equal to 1
- Int not equal to 0
- Se is high
  - Higher with local
- Initial IRI not considered

	Global	Local
N	236	236
Bias	11.641	0.000
Se	23.99	27.51
R^2, %	2.35%	4.91%
Pair T	0.0000	1.0000
Ttest m	0.0000	0.0000
Ttest b	0.0000	0.0000
Se/Sy	1.081	1.239
C1	40	40
C2	0.4	0.4
C3	.008	0.008
C4	.0150	0.0392

PCC Local Calibration

- Identified 22 sites through PMS data
  - Distess data for rigid sections between 07-13
  - Construction data from PMS (built after 1985)
  - 16 CRCP sites





Summary…so far

Positive

- Rutting cal seems good
- Fatigue cracking local cal removes bias in global cal
- Low Se for rut/cracking
- Flex IRI C factors for rutting and cracking seem less significant

Negative

- Small range of measured rut
- Local distress values for rut/cracking < than global targets
- Questionable/missing data
  - Cracking, PO, Initial IRI

Washington DOT MEPDG Experience

MEPDG

- WSDOT's Experience

Jianhua Li

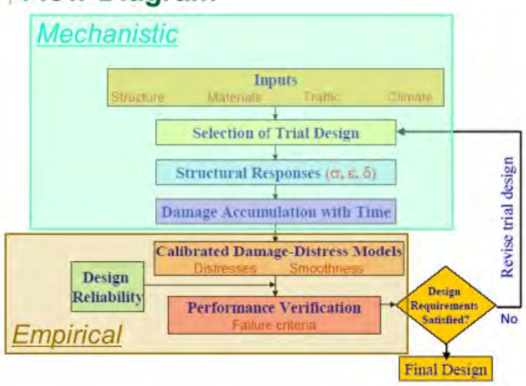
2015 FHWA MEPDG Workshop

WSDOT's Efforts

- **Data preparation**
  - Traffic
  - Materials properties
  - Pavement performance data
- **Calibration and validation statewide**
  - New concrete pavements in 2005 (version 0.6)
  - New flexible pavements in 2008 (version 1.0)

Washington State Department of Transportation 2

Flow Diagram



Pavement Distress Models

Rigid Pavement	Flexible Pavement
IRI	IRI
Transverse cracking	Top down cracking
Faulting	Bottom up cracking
	Thermal cracking
	Rutting – all layer
	Rutting – HMA layer only

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Model Issues

- **Flexible Pavement Models**
  - Top-down cracking
- **Rigid Pavement Models**
  - Transverse cracking only
  - Faulting models – undoweled
- **Rutting models do not consider studded tires**

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Calibration Issues

$$\text{AC Top Down Cracking} = \left(\frac{\boxed{C_4}}{1 + e^{\left(\boxed{C_1} - \boxed{C_2} * \log_{10}(Damage)\right)}}\right) * 10.56$$

Where

C_1, C_2, C_4 *Fixed model coefficients*

$$\text{Damage} = \sum(\Delta DI)_{j,m,l,p,T} = \left(\frac{n}{N_f}\right)_{j,m,l,p,T}$$

$$N_f = 0.00432 \cdot C \cdot \boxed{\beta_{f1}} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \boxed{\beta_{f2}}} \left(\frac{1}{E}\right)^{k_3 \boxed{\beta_{f3}}}$$

$\beta_{f1}, \beta_{f2}, \beta_{f3}$ *Adjustable calibration factors*

$$C = 10^M$$

$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

n	Actual number of axle-load applications within a specific time period
j	Axle-load interval
m	Axle-load type
l	Truck type
p	Month
T	Median temperature of each month.
N_f	Allowable number of axle-load applications
ε	Tensile strain
E	Stiffness of the material
V_a	air voids, %
V_b	effective binder content, %

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Major Findings

- The MEPDG is an advanced analytical tool, but not for everyday design.
- Model issues.
- Calibration is required prior to implementation, and it is a continual process.
- Software cost: $5k/year for single user
  $20~40k/year for a group
- Implementation issues: Developing a user guide
  Preparing design files
  Training engineers
  Checking user feedbacks

Future Work

- Expecting MEPDG upgrades
  - Version 2.0 from AASHTO
  - Software bugs
- Refining the calibration results.
  - Doweled JPCP slabs
  - Superpave HMA
- Testing and calibrating the rehabilitation models
  - HMA overlay on HMA
  - HMA overlay on PCCP

WSDOT will continue to monitor future work related to MEPDG.

WSDOT Pavement Design Table

Design Period ESALs	Layer Thicknesses, ft				
	Flexible Pavement		Rigid Pavement		
	HMA	CSBC Base	PCC Slab	Base Type and Thickness	
< 5,000,000	0.50	0.50	0.67	CSBC only	0.35
5,000,000 to 10,000,000	0.67	0.50	0.75	HMA over CSBC	0.35 + 0.35
10,000,000 to 25,000,000	0.83	0.50	0.83	HMA over CSBC	0.35 + 0.35
25,000,000 to 50,000,000	0.92	0.58	0.92	HMA over CSBC	0.35 + 0.35
50,000,000 to 100,000,000	1.00	0.67	1.00	HMA over CSBC	0.35 + 0.35
100,000,000 to 200,000,000	1.08	0.75	1.08	HMA over CSBC	0.35 + 0.35

WSDOT Pavement Policy (June 2011)

http://www.wsdot.wa.gov/business/materialslab/

Jianhua Li, PhD PE
WSDOT Materials Laboratory
Lijia@wsdot.wa.gov
Phone: 360-709-5564

APPENDIX E. CANADIAN GUIDE

APPENDIX F. PROCESS FOR INCLUDING ADDITIONAL WEATHER STATIONS INTO THE MEPDG (COLORADO DOT)

Implementation of AASHTO Mechanistic-Empirical Pavement Design Guide in Colorado (HAA 00107)

Development of Additional Colorado Weather Stations

Submitted to:

Colorado Department of Transportation
4670 North Holly Street, Unit A
Denver, CO 80216

By
Jagannath Mallela
Leslie-Titus Glover
Biplab B. Bhattacharya

Submitted by:

100 Trade Center Dr., Suite 200
Champaign, IL 61820

September 03, 2013

Introduction

The MEPDG contains 20 Colorado weather stations for use in developing virtual pavement location/site specific climate data for design and analysis. A review of the MEPDG default Colorado weather stations indicated the following:

- There was considerable distance between the weather stations. Increasing the distance between weather stations does negatively impact the accuracy of virtual weather stations created for pavement design.
- Thirteen of the 20 weather stations were located in elevations < 6000 ft. Only one weather station was located in a region with elevation greater than 8500 ft. The remaining weather stations were located in regions with elevation between 6000 and 8500 ft. This implied that higher elevations (very cold and cold climate zones) were under-represented.

Therefore, it was necessary to augment the Colorado weather stations to better characterize and represent Colorado climate conditions.

Augmenting Colorado MEPDG climate data began by identifying weather stations in the state with the data types required for the MEPDG. This was done by CDOT, which identified all significant weather stations in the state. The raw climate data from National Climate Data Center (NCDC) was used in this analysis. Criteria for selecting additional weather stations to augment the MEPDG defaults were as follows:

- Must contain all climate data elements required by the MEPDG (temperature, wind speed, percent cloud cover, precipitation, and humidity).
- Must contain a minimum of 5 years of data.
- Must be located in an unrepresented region/area.
- Must contain good quality data (in terms of both data element magnitude and trends).

Based on the criteria presented above, an additional 22 weather stations were identified for use in developing default weather stations in Colorado.

Climate Data Cleansing and HCD Files Development

The climate data for additional 22 weather stations provided by CDOT was in Excel format. The next step in augmenting the CDOT MEPDG climate data was to conduct a detailed review of all selected weather stations' climate data and transform the data into the form required by the MEPDG (i.e., HCD file format). Transformation of data included cleaning up the raw data, filling gaps in the data, and transforming data into the units of measurement required by the MEPDG. The procedure utilized for data transformation and creation of HCD files is as follows:

1. **Assemble NCDC climate data for weather stations of interest**. The raw NCDC climate data included the variables listed below and was mostly reported on an hourly basis as available:
    a. Time stamp (comprised of Year|Month|Day|Hr presented as a string).
    b. Ambient temperature in degrees F.
    c. Wind speed, in miles per hour.

    d. Percent cloud cover (percentage). Note that this is described as Percent Sunshine in the HCD file, which is 100% - percent cloud clover.
    e. Precipitation, in inches.
    f. Humidity as a percentage.

   Note that for some weather stations, daily rather than hourly estimates of precipitation was reported. The daily precipitation estimates were assigned to a single hour of the day.

2. **Conduct basic QC of raw NCDC climate data**. QC checks were done to ensure that the raw climate data fell within the typical ranges provided in Table 1. Raw data that fell outside the typical range was either removed from the data set or had its value capped at the extreme value of the range.
3. **Transform time stamp to Year|Month|Day|Hr into a unique date/hour**. The NCDC data timestamp was converted into Year, Month, Day, and Hr (time of the day, 00:00 to 23:00). Where the exact hour of the day was not reported (e.g., 10:00 versus 10:15), the report time of the day was rounded to the nearest hour (e.g., 9:57 AM becomes 10:00HRS and 9:57 PM becomes 22:00HRS). The rounded time stamp was further transformed to the HCD timestamp format (e.g., 10:00 becomes 10 while 22:00 becomes 22).
4. **Determine daily precipitation values and convert to hourly precipitation values**. Using the climate data assembled for each weather station, the cumulative precipitation for each day (24-hr period) was determined. This value was then assigned to a single hour of the day (i.e., Year|Month|Day|14).
5. **Determine mean hourly temperature, wind speed, percent sunshine, precipitation, and humidity values**. Using the climate data assembled for each weather station, mean hourly values (i.e., for each combination of Month|Day|Hr) was computed.
6. **Determine earliest reporting date/time**. Determine earliest date/time (e.g., 10:00 January 16, 1957).
7. **Determine latest reporting date/time**. Determine latest date/time (e.g., 16:34 June 26, 2007).
8. **Establish climate file start/end**. This was assigned as follows:
    a. **Start date = the first day of the earliest month of the earliest year**. (e.g., 10:00 January 16, 1957 becomes 00:00 January 1, 1957).
    b. **End date = the last day of the last month of the last year (**16:34 June 26, 2007 becomes 23:00 June 30, 2007).
9. **Generate hourly time stamp for the period between the start and end dates**. Using the start and end dates, an hourly date/time stamp was generated. This was called the baseline timestamp as shown in Table 2.
10. **Using the baseline hourly time stamp established in step 9 as reference, determine all the hours within the start and end dates with and without climate data**. By linking, the NCDC reported climate data and baseline time stamps; all hours within this period with missing climate data was indentified.
11. **Replace missing climate data with mean values**. The missing climate data was replaced using the mean values determined in Step 5.
12. **Check the start and end dates time period to determine if there are still hours with missing data (i.e., hours for which average values are not available).** For this situation, statistical algorithms (splines, interpolation, and extrapolation) was used to determine the best estimates of missing data.
13. **Use the climate data set developed in steps 9 through 12 to develop HCD files**. Each HCD file must contain a unique five-digit code to be identified by MEPDG. The HCD files should follow the file format:

a. Date and time of the record in YYYYMMDDHH format
b. Temperature in ˚F
c. Wind Speed in mile/hr
d. Sunshine in percentage of time exposure
e. Precipitation in inches
f. Relative Humidity in percent

14. **Update MEPDG station.dat file to enable MEPDG to read in new HCD files.** The following information that describes the new climate station must be added to the station.dat file:
    a. A new five-digit station code (must be unique to a climate station and this code should match the HCD file name/code)
    b. Town/City name
    c. State name
    d. Climate station name
    e. Latitude
    f. Longitude
    g. Elevation in feet
    h. Beginning date of the climate data in YYYYMMDD format
    i. Code "C" for complete climate data
    j. End date of the climate data in YYYYMMDD format

Note that the station.dat file should follow exactly the sequence shown above.

15. **Test HCD files using MEPDG interface to determine reasonableness of data entries.** All HCD files were opened by MEPDG interface to flag outliers and erroneous data inputs.
16. **Revise HCD files as needed based on MEPDG outcomes.** Climate data was revised based on outliers or erroneous data. For example, MEPDG will flag warnings if the temperature difference between two adjacent hours is more than 30^0F.
17. **Prepare final files and include in MEPDG database for Colorado.** Additional 22 HCD files were added into MEPDG default HCD folder. The location of default HCD folder is C:\Program Files\AASHTOWare\ME Design\HCD. The default station.dat file was replaced by the updated station.dat file and the file location is C:\Program Files\AASHTOWare\ME Design\Defaults. A summary of climate variables for all 42 Colorado weather stations, including 20 default MEPDG weather stations, are presented in Table 3.

Table 1. Typical climate data ranges used in conducting QA/QC checks.

Climate Variable	Minimum Range	Maximum Range
Temperature, °F	-100	150
Wind speed, mph	0	100
Percent sunshine	0	100
Precipitation	0	10
Relative humidity	0	100

Table 2. Baseline time stamp for MEPDG HCD file development.

Date/Hr	Temp, °F	Wind Speed, mph	Sunshine, percent	Precipitation, in	Humidity, percent
1957010100					
1957010101					
1957010102					
2007123122					
2007123123					

The date and hour have been merged to provide reference date/hr in column 1.

Table 3. Summary of Colorado weather stations.

Station ID	Station	Mean Annual Temp, ^{0}F	Mean Annual Precip, in	No of wet days	Freezing Index, ^{0}F-days	No of Freeze/Thaw cycle
24015	AKRON	50	14.5	140.6	1548.8	121.9
23061	ALAMOSA	42.6	5.9	80.2	4047.6	187.9
93073	ASPEN	41.3	12.5	123.6	3061.3	142.1
03026	BURLINGTON	50.6	13.6	89	1815.3	129.8
93067	CENTENNIAL	50.3	13.2	93.1	1495.2	124
93037	COLORADO SPRINGS	49.7	13.2	98.7	1633.3	130.1
93069	CORTEZ	49.1	8.4	72.7	2159.7	169.5
24046	CRAIG	42.5	11.8	121	3445.4	147.3
03017	DENVER	50.6	13	84.7	1560.9	129.8
93005	DURANGO LA PLATA	47.1	9.1	68.9	2362.2	163.5
23066	GRAND JUNCTION	53.3	7.7	82.1	1244.1	111.7
23067	LA JUNTA	54	9.9	71	1480.4	117
03013	LAMAR	53	12.4	77.2	1947.1	135.9
93009	LEADVILLE	35.1	10.3	125.1	4100.4	162
93010	LIMON	47.3	13.5	106.9	2775.7	179.5
94050	MEEKER	44.1	11.7	105.2	2884.2	148
93013	MONTROSE	50.1	6.9	85.8	1661.6	123.1
93058	PUEBLO	52.7	10.6	77.1	1931	142.2
03016	RIFLE	48.2	9.1	101.2	2010.5	132.8
23070	TRINIDAD	52.5	11.5	68.9	1497.8	129.6
03065	BROOMFIELD	51.5	14.2	61.2	1064.5	96.9
23036	AURORA	50.9	13.6	85.7	914.2	67.3
03038	COPPER MOUNTAIN	33.1	12.7	34.5	2834	64.8
12341	COTTONWOOD PASS	41	12.4	35.7	1466.2	80
12342	DENVER NEXARD	50.5	15.9	54.1	1499.5	138.3
23063	EAGLE CO	42.8	14	108.8	2203.7	103.3
03040	ELBERT CO	45.3	14.3	54	1234	96.6
94015	FORT CARSON	49.8	13.1	117.3	838.7	91.5
94062	FORT COLLINS	48.9	12	122.4	1036.8	82.6
24051	GREELEY	47.8	10.4	85.5	1546	94.8
93007	GUNNISON CO	38.4	6.9	74.9	3200.4	88.6
94025	HAYDEN	42.4	12	140.2	1872.6	64
94076	KREMMLING	39.2	16.1	95.2	3034.8	88.8
03042	LA VETA PASS	38.9	11.6	34.5	1931.3	75.8
12343	STEAMBOAT	33.1	23.4	24	2834	64.8
03039	PAGOSA SPRINGS	32.9	16.5	86.2	2834	63
03069	SAGUACHE	45.2	7.9	36.5	1587.3	91.8
03041	SALIDA	32.9	16.9	21.8	2834	63
12344	GLENWOOD SPRINGS	37.3	16.3	24	2135.4	65.7
03011	TELLURIDE	42.6	22.8	122.5	1338	74.8
12345	WILKERSON PASS	33.3	23.5	24	2787	69
12346	WINTER PARK	35.2	77.2	151.8	2554.1	40

Sensitivity Analysis of Climate Data

Figures 1 through 5 present plots of MEPDG climate data variables across Colorado weather stations. The plots show that mean annual temperature decreases with increase in elevation, freezing index increases with increase in elevation, and number of freeze-thaw cycle decreases with increase in elevation. The trends are reasonable as temperatures in higher elevations are generally lower and stay below freezing for long period.

Figures 6 through 9 present plots of AC alligator cracking, rutting, low temperature thermal cracking, and IRI across Colorado weather stations. The plots show that rutting in general decreases with increase in elevation (i.e. low temperature). The thermal cracking typically depends on low temperature and number of freeze-thaw cycles. Mountains with higher elevations have less number of freeze-thaw cycles as the temperature stays below freezing for long period. The plots are showing similar trend for thermal cracking. The plots also show IRI decreases with increase in elevation. Lower rutting in higher elevation is the primary factor for relatively low IRI in mountains, as rutting contributes significantly to IRI.

Figures 10 through 12 present plots of JPCP transverse cracking, faulting, and IRI across Colorado weather stations. The plots show that transverse cracking in general decreases with increase in elevation (i.e. low temperature). The mean joint faulting typically depends on number of wet days/precipitation and number of freeze-thaw cycles. Mountains with higher elevations have less number of freeze-thaw cycles as the temperature stays below freezing for long period. The plots are showing similar trend for faulting. In addition, weather stations with high number of wet days or precipitation show more faulting as it increases pumping. The plots also show IRI has similar trend as faulting. Lower faulting in higher elevation is the primary factor for relatively low IRI in mountains, as faulting contributes significantly to IRI.

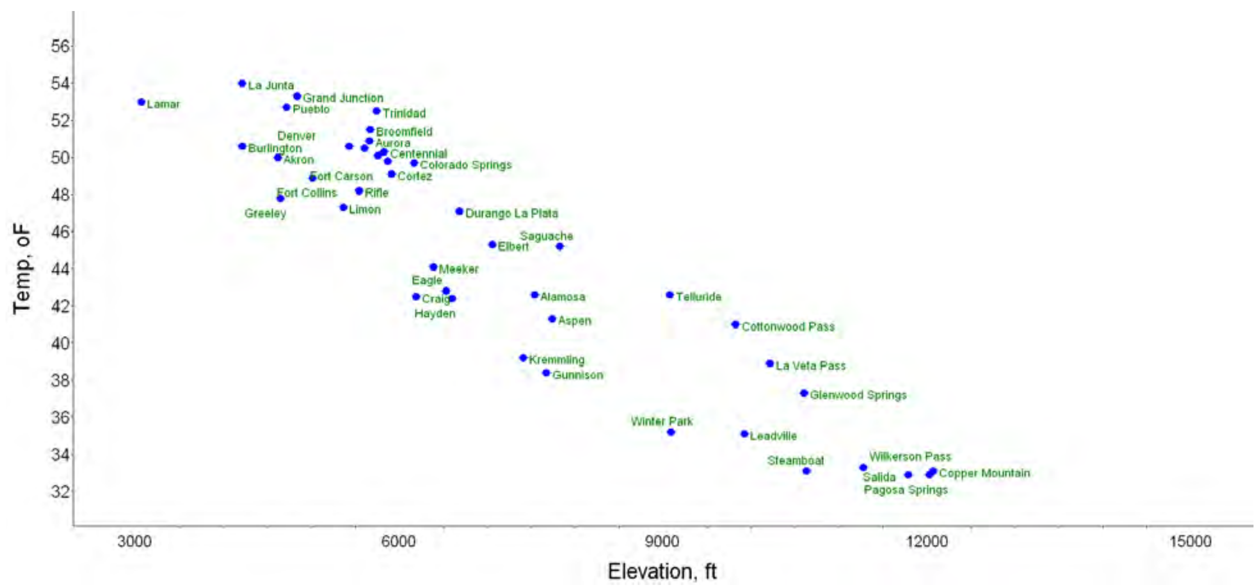

Figure 1. Plot showing change in mean annual temperature data by elevation for different climate stations in Colorado

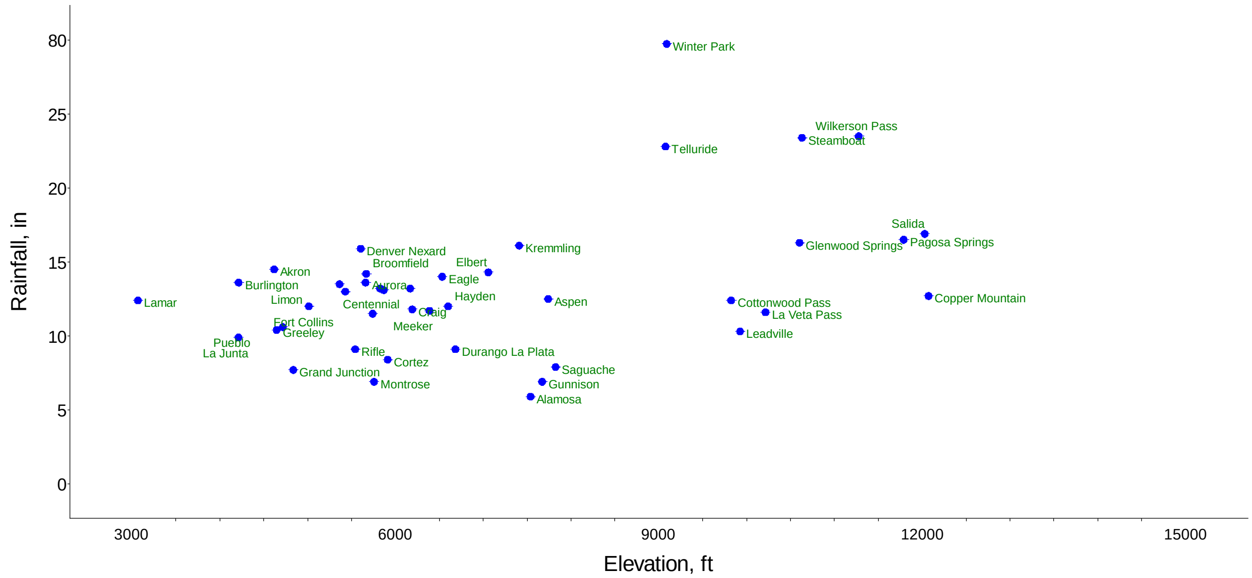

Figure 2. Plot showing change in mean annual precipitation data by elevation for different climate stations in Colorado

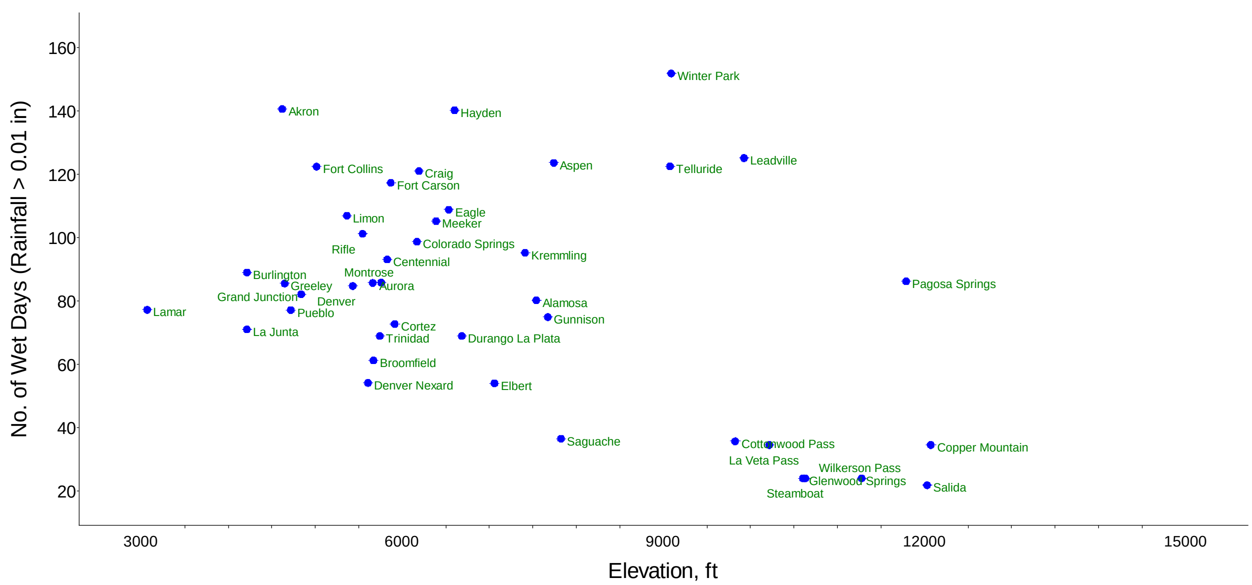

Figure 3. Plot showing change in annual number of wet days data by elevation for different climate stations in Colorado

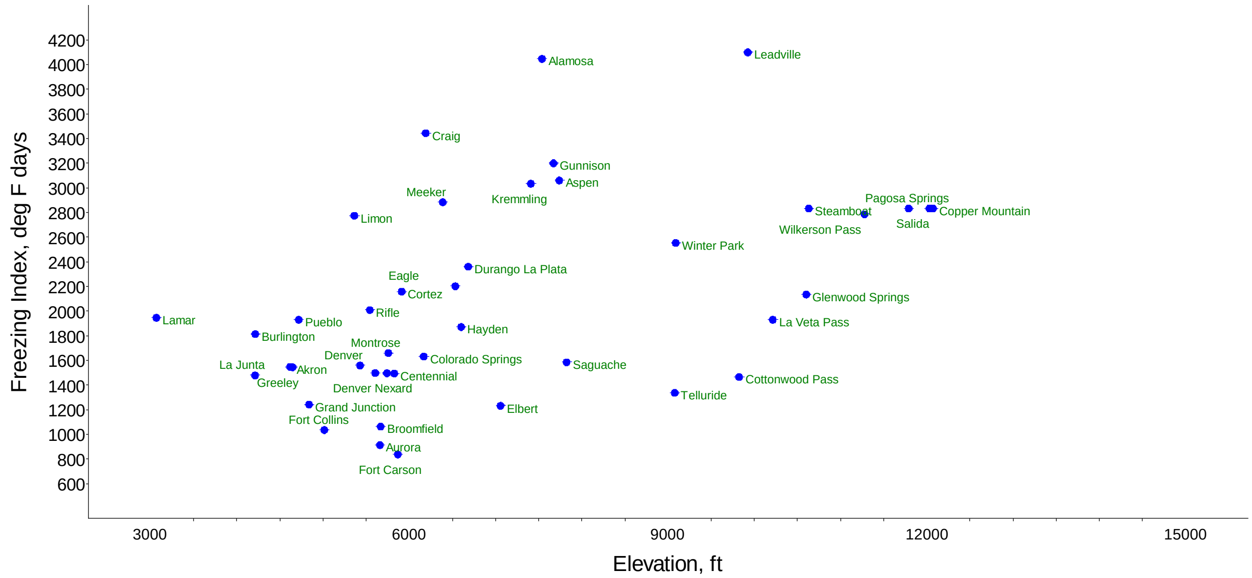

Figure 4. Plot showing change in annual freezing index data by elevation for different climate stations in Colorado

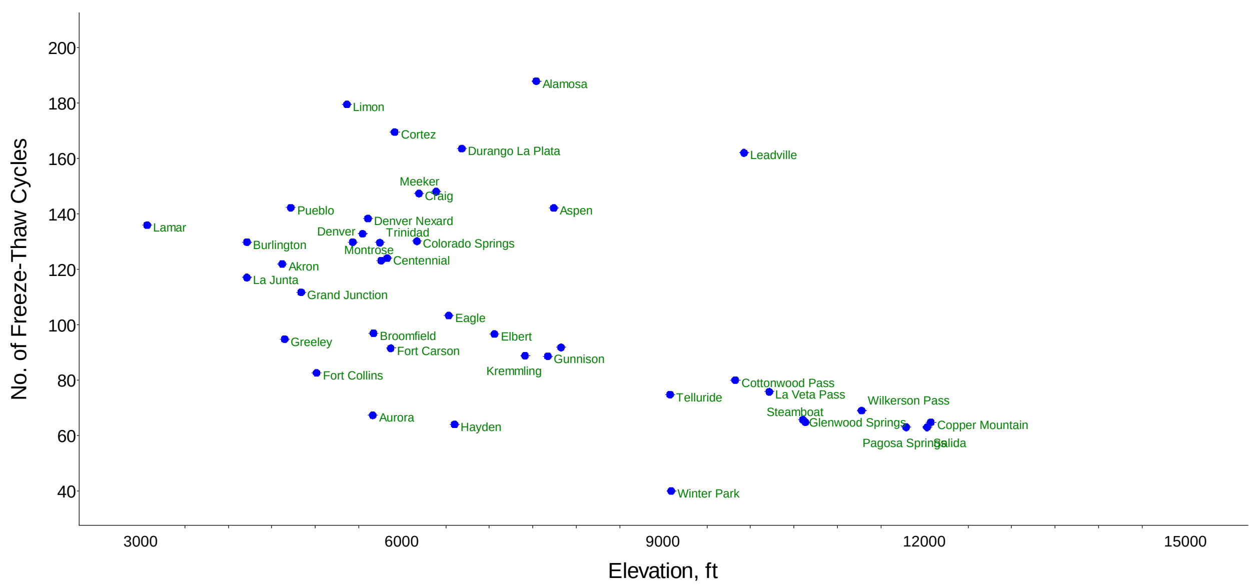

Figure 5. Plot showing change in annual freeze-thaw cycle data by elevation for different climate stations in Colorado

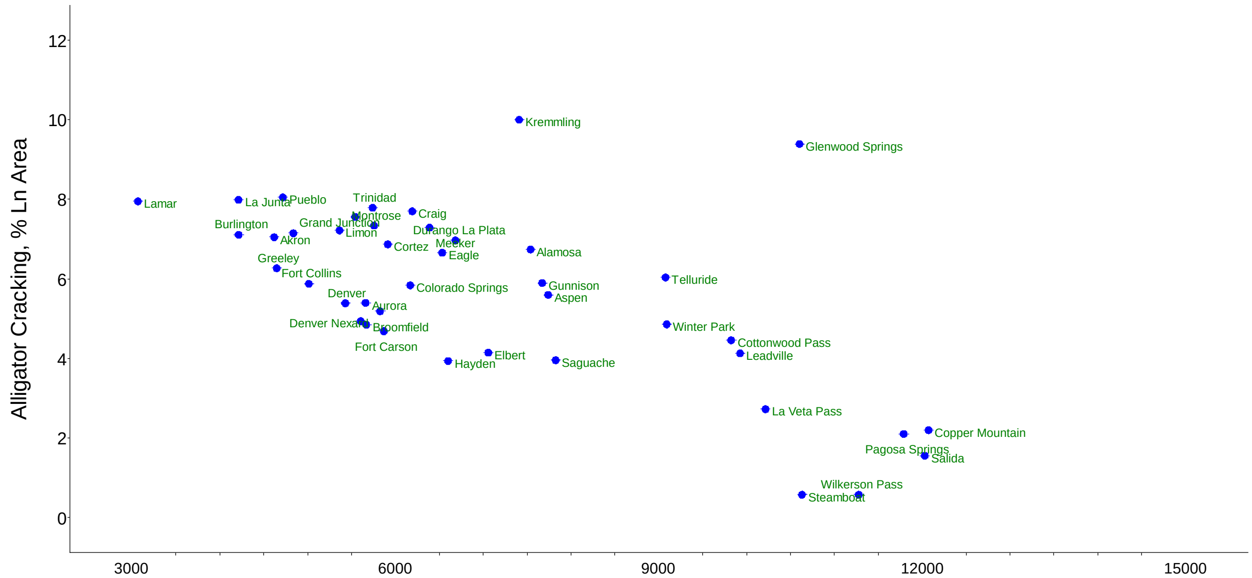

Figure 6. Plot showing change in AC alligator cracking by elevation for different climate stations in Colorado

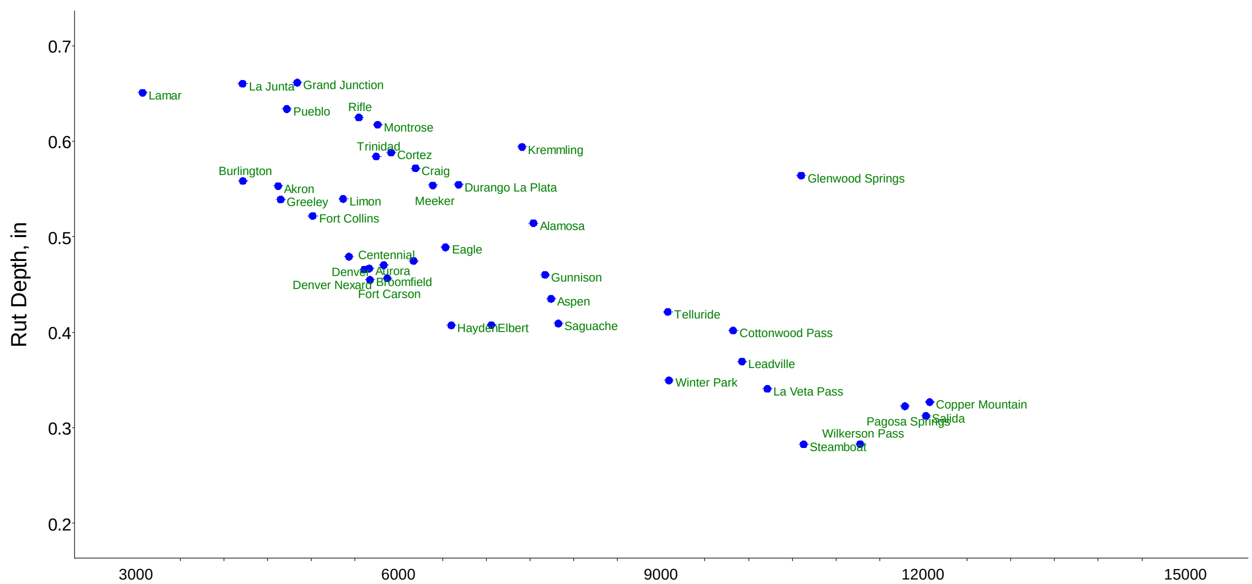

Figure 7. Plot showing change in AC total rutting by elevation for different climate stations in Colorado

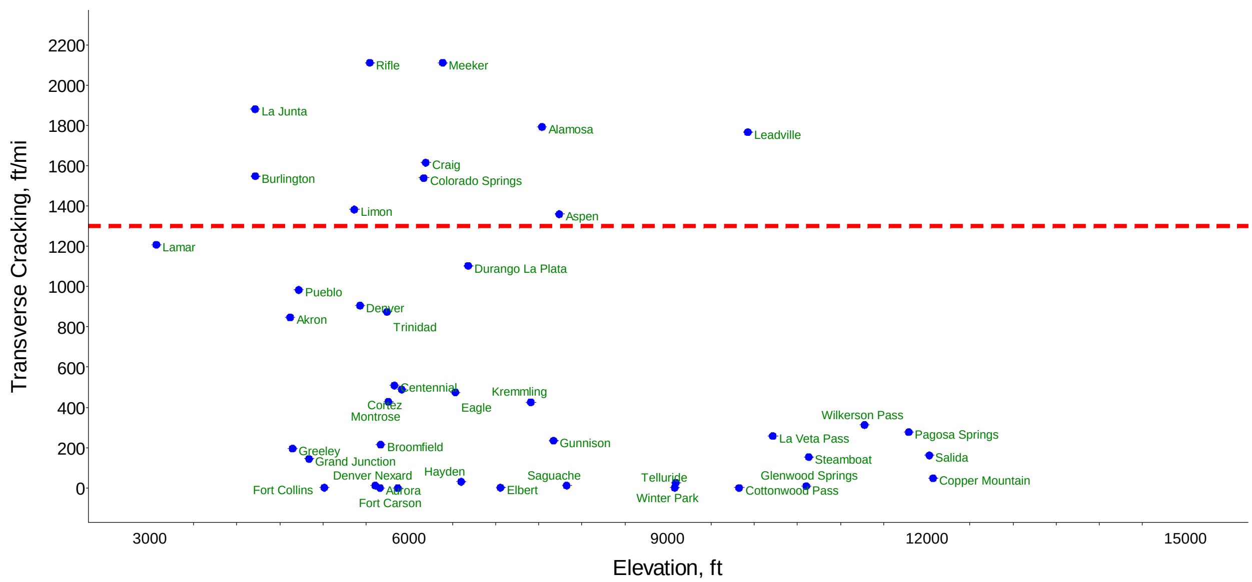

Figure 8. Plot showing change in AC thermal cracking by elevation for different climate stations in Colorado

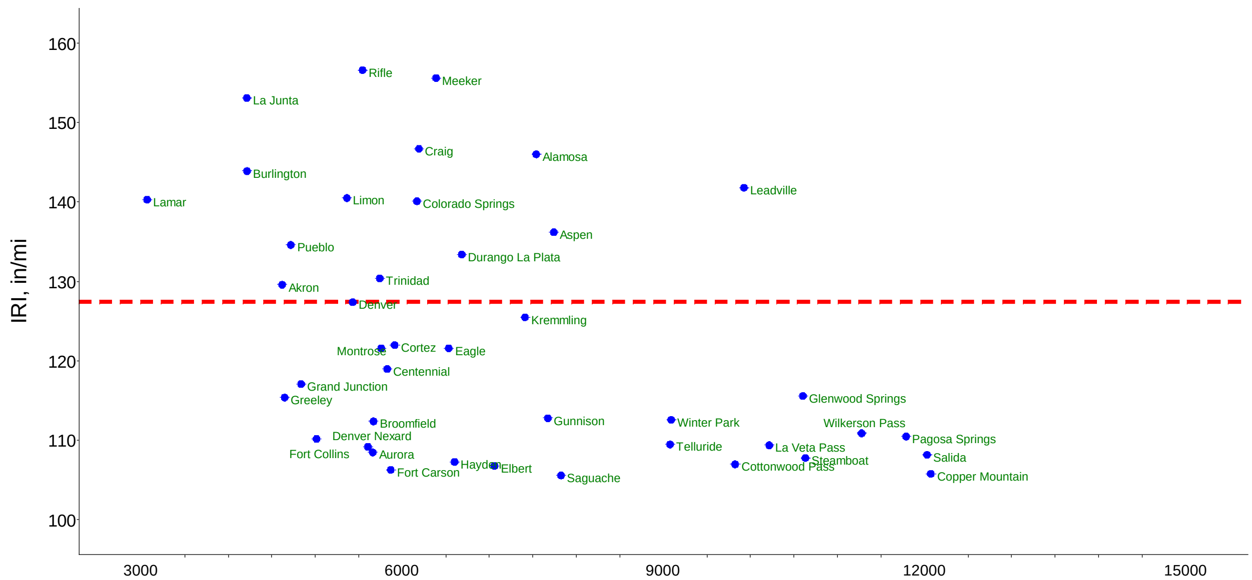

Figure 9. Plot showing change in AC IRI by elevation for different climate stations in Colorado

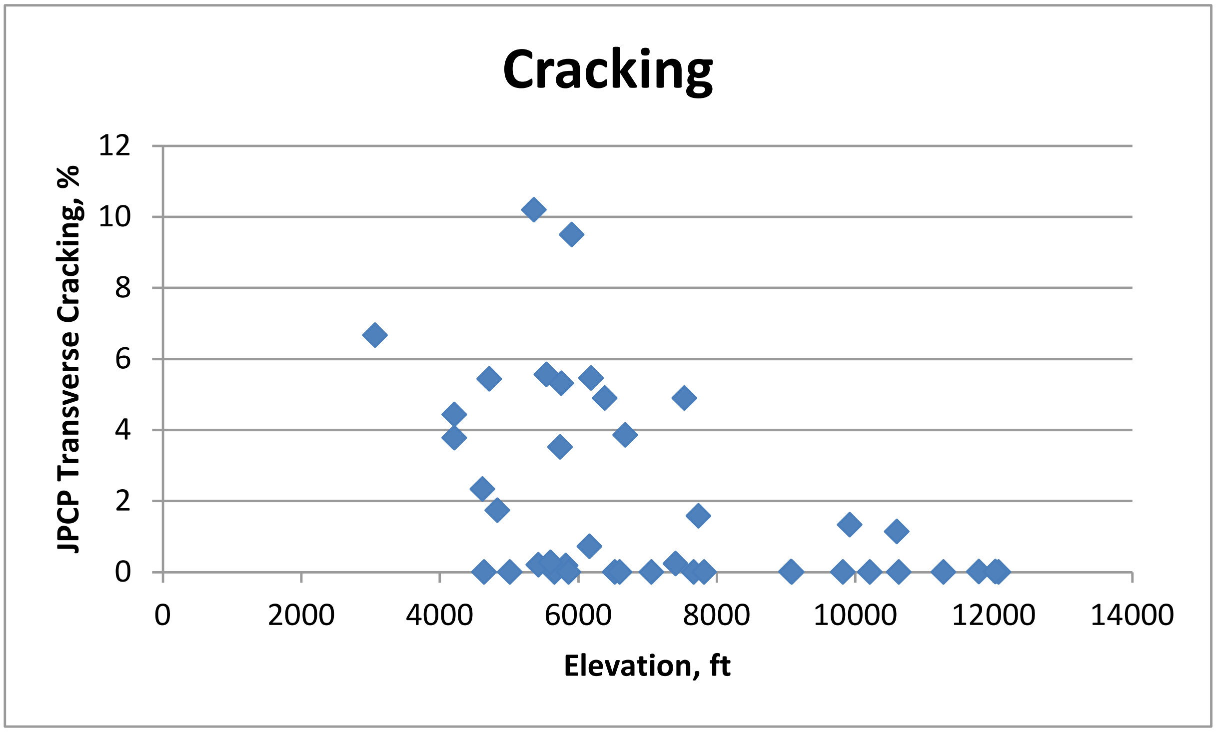

Figure 10. Plot showing change in JPCP Transverse Cracking by elevation for different climate stations in Colorado

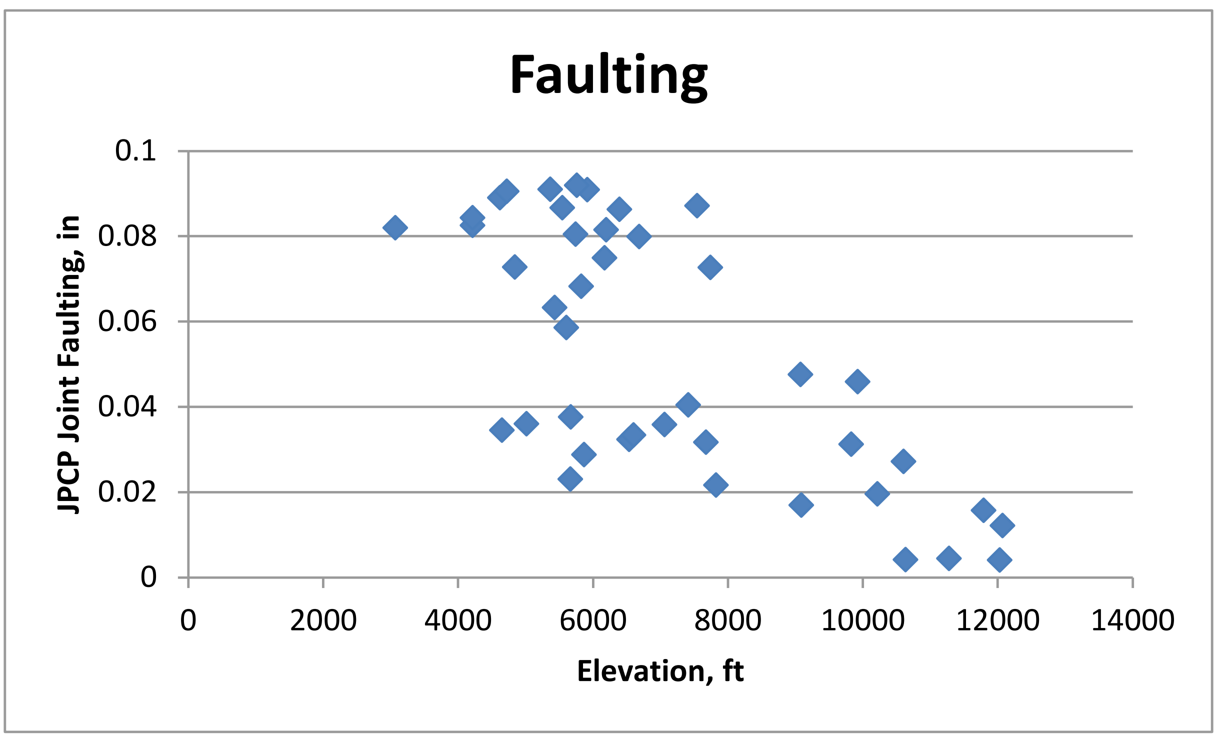

Figure 11. Plot showing change in JPCP Faulting by elevation for different climate stations in Colorado

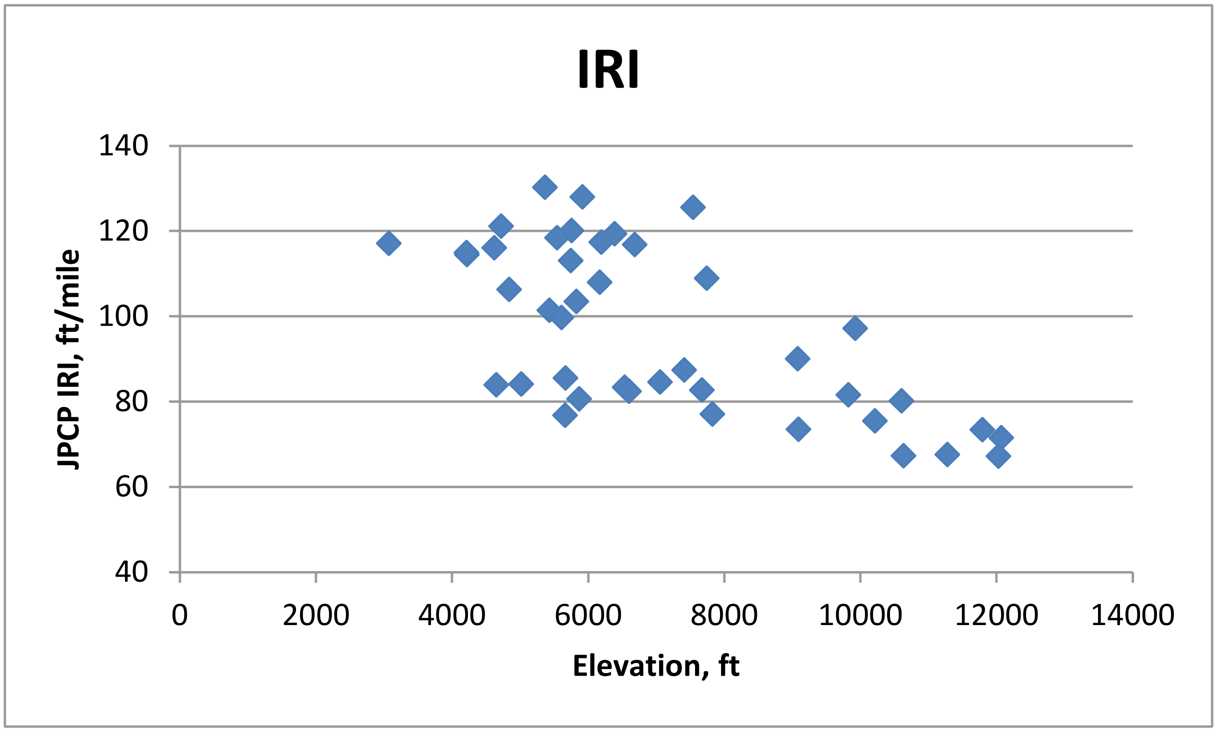

Figure 12. Plot showing change in JPCP IRI by elevation for different climate stations in Colorado

APPENDIX G. PrepME TECHNICAL BULLETIN

RESEARCH PROJECT CAPSULE [12-1PF]

October 2011

TECHNOLOGY TRANSFER PROGRAM

Traffic and Data Preparation for AASHTO MEPDG Analysis and Designs

JUST THE FACTS:

Start Date:
September 1, 2011

Duration:
36 months

End Date:
August 30, 2014

Funding:
SPR

Principal Investigator:
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Louisiana Transportation Research Center
4101 Gourrier Ave
Baton Rouge, LA 70808

Sponsored jointly by the Louisiana Department of Transportation and Development and Louisiana State University

POINTS OF INTEREST:

Problem Addressed / Objective of Research / Mehodology Used Implementation Potential

WWW.LTRC.LSU.EDU

PROBLEM

The Mechanistic Empirical Pavement Design Guide (MEPDG) and the subsequent AASHTO product DARWin-ME are significant advancements in pavement design. However, they are substantially more complex than the 1993 AASHTO Guide, which is currently used in many states. The new systems require significantly more inputs from designers. Among them are many parameters with which today's pavement designers are not familiar. In addition, MEPDG/DARWin-ME provides methodologies for the analysis and performance prediction of different types of flexible and rigid pavements for specific climatic and traffic conditions. However, the models were developed using available Long Term Pavement Performance (LTPP) national wide data sets. These models require local calibration before MEPDG can be used by highway agencies efficiently. Therefore, many data sets need to be pre-processed before their use for MEPDG/DARWin-ME. This process needs to be automated and assisted with software.

Prep-ME was initially designed and made to achieve the purpose of data preparation, data quality control, and database development for MEPDG with a primary module to qualify truck traffic data from weigh-in-motion (WIM) stations. The first version of Prep-ME was tailored for Arkansas Department of Transportation by a research team led by Dr. Kelvin Wang and Dr. Kevin Hall at the University of Arkansas. In mid-2011, through efforts by the Louisiana Transportation Research Center (LTRC) and Federal Highway Administration (FHWA), a pooled-fund project with eight participating federal and state departments of transportation (DOTs) was initiated for a three-year study to make Prep-ME a production worthy program for DARWin-ME for the participating agencies. The pooled-fund contributing agencies are DOTD, FHWA, and state DOTs of Hawaii, Kentucky, Maryland, North Carolina, New Hampshire, and Wisconsin. It is anticipated that more state and private agencies will join the study throughout the course of the three-year period.

OBJECTIVE

The objective of the new version of the Prep-ME software is to assist participating state DOTs the data preparation and improve the management and workflow of the DARWin-ME input data to make the DARWin-ME software more accessible and easier to use. Additionally, Prep-ME can be used as an important tool for calibrating and implementing the DARWin-ME for individual participating agencies. For production use, the existing features in the Prep-ME software need to be enhanced to improve speed, usability, functionality and stability, and compatibility with the DARWin-ME software.

A critical task in the pooled-fund study is to improve the quality control algorithms and their implementations for truck traffic data from WIM stations in state DOTs. An expert sub-group has been formed and is led by North Carolina DOT to advise and guide the implementation of the truck traffic data analysis and quality control. A substantial effort in the study includes the development of database capabilities in Prep-ME that are compatible with the server based SQL data structures that are implemented in DARWin-ME.

METHODOLOGY

For all project objectives to be fully satisfied, the following tasks must be successfully accomplished:

- *Task 1:* Improve the speed of executing the most time consuming numerical engines, including the raw traffic data importing/updating and user's data interpolation functions.
- *Task 2:* Add more features based on a consensus basis of participating states, including the algorithms and their implementations of quality control of WIM truck traffic data.
- *Task 3:* Improve user friendliness/usability of Prep-ME.
- *Task 4:* Test stability and conduct testing of Prep-ME.
- *Task 5:* Report and give documentation of Prep-ME.
- *Task 6:* Conduct training.

IMPLEMENTATION POTENTIAL

The primary final product of the proposed research is the Prep-ME software version 2.0, central database suitable for use with DARWin-ME, and documentation. This research will also provide roadway designers insight into the criticality of specific inputs required in the new pavement design guide, including key truck traffic characteristics. In addition, the participating highway agencies can use the new version of Prep-ME by WIM field data crews to check traffic data quality of WIM equipment in the field.

Learn More

The pooled-fund project solicitation can be found online at: http://www.pooledfund.org/Details/Solicitation/1260

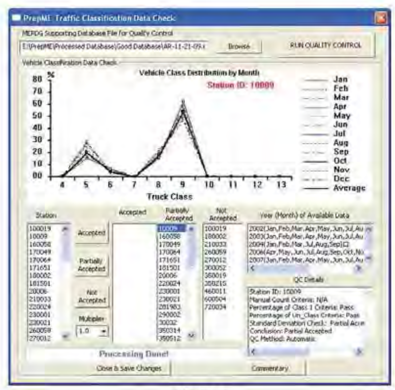

Figure 1
Main screen shot of traffic classification quality control in Prep-ME

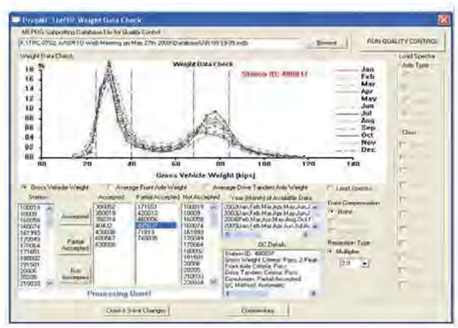

Figure 2
Main screen shot of traffic weight data check in Prep-ME

For more information about LTRC's research program, please visit our Web site at www.ltrc.lsu.edu.

APPENDIX H. SYNTHESIS OF LOCAL CALIBRATION ACTIVITIES

Local Calibration Webinar Series
for AASHTOWare Pavement ME Design

Synthesis of Local Calibration Activities

Task 1 Interim Report

Task Order DTFH61-08-D-00015-T-13001
Indefinite Quantity Contract: DTFH61-08-D-00015

Submitted to:

Federal Highway Administration
Room E73-438, HIPT-20
1200 New Jersey Ave, SE
Washington, DC 20590

Submitted by
Applied Research Associates, Inc.
100, Trade Center Drive, Suite 200
Champaign, IL, 61820

December 31, 2013

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TECHNICAL DOCUMENTATION PAGE

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December 31, 2013</td></tr>
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<tr><td colspan="2">7. Author(s)
Mr. Harold L. Von Quintus, P.E., Mr. Jagannath Mallela, Dr. Suri Sadasivam, P.E., and Dr. Michael I. Darter, P.E.</td><td colspan="2">8. Performing Organization Report No.</td></tr>
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Champaign, Illinois 61820</td><td colspan="2">10. Work Unit No. (TRAIS)</td></tr>
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<tr><td colspan="4">15. Supplementary Notes
The Contracting Officer's Technical Representative (COTR) is Mr. Gary Crawford, P.E.</td></tr>
<tr><td colspan="4">16. Abstract
The objective of this Task Order was to develop and deliver a series of webinars that focus and provide guidance on the process for performing a local calibration of the American Association of State Highway and Transportation Officials (AASHTO) Pavement ME Design software. As part of this objective, a synthesis was prepared to summarize the activities completed and on-going by individual agencies implementing the AASHTO Pavement ME Design software. Specifically, it captures what implementation activities have been planned, as well as the status of current and completed activities by various State agencies. In addition, the information compiled in this document served as a reference to FHWA in planning the local calibration webinars in terms of what other States are doing with regards to implementation and in helping avoid problems experienced by other agencies during their implementation effort.</td></tr>
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Mechanistic-Empirical Pavement Design Guide, ME Design, Transfer Functions, Fatigue Cracking, Rutting, Thermal Cracking, Faulting, Mid-Slab Cracking, International Roughness Index.</td><td colspan="2">18. Distribution Statement
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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EXECUTIVE SUMMARY

This document captures the status of current and completed implementation activities by various State agencies. The information compiled in this report served as a reference document to Federal Highway Administration (FHWA) in planning the local calibration webinars as related to what States are doing relative to implementation and to help avoid problems experienced by other agencies during their implementation effort for those agencies just getting started.[1] The other two objectives of the synthesis report was to: (1) provide a summary of the results from other agencies calibration efforts in terms of the calibration coefficients, and (2) select invited guests from State agencies to participate in the local calibration webinars to provide their perspective on calibrating the Pavement ME Design software.

In summary, many State Department of Transportation (DOT) studies have focused their implementation efforts in two key areas: (1) building input libraries for key material types and traffic loadings, and (2) evaluating the accuracy of using lower hierarchical input levels to produce reasonable predictions of the performance indicators. Numerous studies have focused on hot mix asphalt (HMA) mixtures; specifically, the accuracy of the Witczak dynamic modulus regression equation. Although the results are diverse, most studies have found the Witczak dynamic modulus equation to be reasonable. Use of measured binder test data (i.e. input level 2) greatly improved on the accuracy of dynamic modulus predictions for mixtures with neat asphalt, while significant deviations have been reported for binders with higher performance grades (PG) or modified asphalt. Studies on Portland cement concrete (PCC) mixtures have focused on the coefficient of thermal expansion (CTE) measurement and its significance to rigid pavement performance.

The findings of national level studies, including National Cooperative Highway Research Program (NCHRP) projects 1-40B, 9-30A, and 1-47, are directly applicable to many transportation agencies. More importantly, the lessons learned from various calibration studies are directly applicable for use in determining appropriate design inputs, setting up a sampling matrix to verify, locally calibrate, and validate the Mechanistic Empirical Pavement Design Guide (MEPDG) transfer functions, and selecting design reliability and performance criteria. The following lists some of the more important findings from the literature and projects reviewed.

1. The following local calibration coefficients were found to be significantly different from the global calibration coefficients of the transfer functions from many of the studies reviewed for flexible pavements:
    a. B_{f1} for the fatigue cracking transfer function.
    b. B_{r3} (exponent to the number of load cycles term) and B_{r1} (the intercept term) for the HMA rut depth transfer function.
    c. B_{t3} for the thermal cracking transfer function for agencies located in a warm climate.
    d. C1 or coefficient of the rutting term in the IRI regression equation.

[1] NOTE: Applied Research Associates, Inc. (ARA) recently completed a review of local calibration and implementation activities for the Georgia Department of Transportation as part of their efforts to implement the MEPDG procedure and software. Results from that document are included within this synthesis (Von Quintus, et al., 2013).

2. Selection of design reliability and design criteria or threshold values requires analyses to show how the resulting design depends on these critical inputs. Selecting a high reliability and low design criteria results in unreasonable and costly designs. Reliability and performance criteria should be selected together and not independently.

3. The key findings from the sensitivity analyses conducted under NCHRP project 1-47 can be and should be used by agencies in starting their MEPDG implementation study to develop their individual sampling matrix, select sites for the local calibration, and in evaluating the residual error of the predicted distress values.

4. The local calibration coefficients derived from individual State studies for PCC pavements are reasonably consistent with the global calibration coefficients. However, several important findings or observations were made from some of the local calibration activities for PCC pavements which are summarized below.
    a. More accurate or appropriate design inputs have been established through the local calibration process. For example, the number of months with full friction between the PCC slab and base was improved using local data.
    b. The use of the correct CTE input value for PCC (as measured by AASHTO T336) was found to verify the global calibration coefficients determined under NCHRP project 20-07 in 2010 for several states. This finding makes it possible for an agency to measure the CTE and then use that value directly in design.
    c. Modifications to some the local calibration coefficients were found reduce the standard error of the estimate for the transfer functions which results in a less costly design when using higher reliability values.

5. The following are some consistent findings from flexible pavement local calibration studies:
    a. MEPDG over predicts rutting in the HMA and unbound layers based on using laboratory equivalent resilient modulus values.
    b. Dynamic modulus does not explain the difference in residual error (predicted minus measured distress values) between HMA and polymer modified asphalt (PMA) mixtures for rutting and fatigue cracking.

6. The procedures outlined in the NCHRP projects 1-40B and 9-30A can be used to develop field adjustment factors for fatigue cracking and rutting transfer functions that will remove the bias (consistently over or under predicted distresses as compared to the measured distress values) and reduce the standard error of the estimate.

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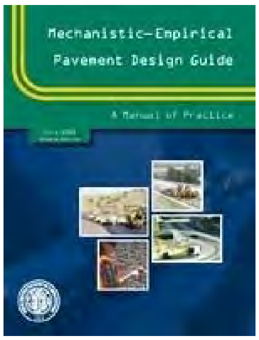
SYNTHESIS OF LOCAL CALIBRATION ACTIVITIES

1. INTRODUCTION

1.1 Background

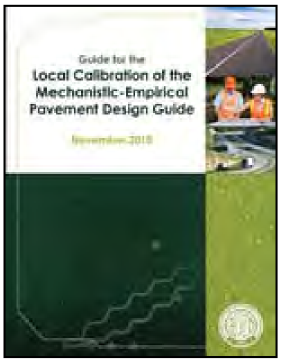
The American Association of State Highway and Transportation Officials (AASHTO) adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) as an interim pavement design standard in 2008. The MEPDG was developed under National Cooperative Highway Research Program (NCHRP) project 1-37A and further modified and calibrated under NCHRP project 1-40D. The MEPDG is founded on fundamental engineering principles and offers several potential benefits over the current AASHTO Design Guide as a tool to effectively design, construct, and manage highway pavements in a cost-effective manner. More importantly, its user-oriented computational software implements an integrated analysis approach for predicting pavement condition over time. In fact, the procedure and associated software is regarded as the most comprehensive and advanced procedure available for pavement analysis and design.



To aid State Department of Transportations (DOTs) and other agencies in MEPDG implementation, the MEPDG Manual of Practice and Local Calibration Guide were prepared under NCHRP project 1-40B. The MEPDG Manual of Practice was balloted and approved by AASHTO in 2008. The Manual of Practice presents information to guide pavement design engineers in making decisions and using the MEPDG for new pavement and rehabilitation design. The 2010 Local Calibration Guide provides guidance in making a decision to recalibrate the MEPDG to local conditions, policies, and materials, and in conducting the local calibration process. The AASHTOWare® Pavement ME Design™ software, shortly referred as Pavement ME Design, was released in April 2011 for production-level pavement designs. A software user-manual was released to provide users with in-depth guidance on the operation and application of the ME Design software.

A few highway agencies in the United States (US) and Canada have already implemented the MEPDG, while many others are investigating the possibility of implementing the MEPDG as their pavement design standard. Several of these agencies have reported that this procedure provides their designers with tools to predict the expected performance of a trial pavement structure over its design life while accounting for design and site-specific variables such as traffic, climate, foundation, materials, pavement layering, thicknesses, and other features (e.g., drainage, load transfer, tied shoulders).

The MEPDG was calibrated using data stored in the Long Term Pavement Performance (LTPP) database, which includes pavement test sections located across North America. However, the developers of the MEPDG recognized the diversity of operational policies, construction and quality assurance specifications, and many other factors affecting pavement performance across North America and built-in adjustments to the prediction models (transfer functions). These adjustments to the prediction models were defined as local calibration coefficients that were set to unity during the global calibration process. The purpose of the local calibration factors was to allow individual agencies to adjust the coefficients of the transfer functions to accurately predict the performance measured on their specific pavements.



Some agencies have already initiated and/or completed the local calibration of the transfer functions, as part of their implementation process, while others are considering local calibration. A document was prepared as part of NCHRP project 1-40B specific to local calibration that was eventually published by AASHTO in 2010—the Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide. Some agencies, however, are of the opinion that the MEPDG Local Calibration Guide is too general.[2]

The AASHTO Task Force on Pavement ME Design, the AASHTO Joint Technical Committee on Pavements, and the Federal Highway Administration (FHWA) have identified a need to provide more detailed guidance and examples in calibrating the MEPDG software—Pavement ME Design—to local conditions, materials, and operational policies of an individual agency. FHWA intends to disseminate this guidance through a series of webinars focused on local calibrationi. Using webinars to disseminate this information has several advantages, including: (1) the material can be delivered to a large audience at different locations at the same time, (2) guidance on local calibration can be delivered in a modular format, which reduces the total time required to deliver each module, and (3) the webinars can be recorded and made available for future on-demand viewing.

FHWA considers implementation of mechanistic-empirical pavement design a critical element to improve the National Highway System. As such, FHWA has developed numerous workshops and other technical material to begin the process of educating and assisting the FHWA field offices, State highway agencies, industry, and others with implementation of the MEPDG and the Pavement ME Design software. Thus, FHWA issued a task order to develop and deliver a series of webinars focusing and providing detailed guidance and examples for performing a local calibration of the MEPDG transfer functions. This task order is a continuation of these activities to provide assistance to the State highway agencies in adopting the MEPDG.

1.2 Task Order Objective

The objective of this Task Order was to develop and deliver a series of webinars that focus and provide guidance on the process for performing a local calibration of the AASHTO Pavement ME Design software. An important outcome of the task order was to prepare a synthesis of the local calibration process and activities that have been completed by individual agencies. This document is the synthesis which served as a reference document to FHWA to summarize what States are doing in regards to implementation and use. The synthesis can be used by agencies just getting started with implementation, because it documents some of the issues and lessons learned by agencies further along in implementation

[2] Based on communications/conversations between project staff and some State agency personnel attending to calibrate the MEPDG transfer functions and/or extracted from various internal documents/memorandums related to calibration.

process. Information used to prepare this document or synthesis was extracted from many agencies, including: Arizona, Colorado, Georgia, Idaho, Indiana, Maine, Minnesota, Mississippi, Missouri, Montana, North Carolina, Ohio, Utah, Virginia, Wisconsin, and Wyoming.

1.3 Scope of Synthesis: Overview of the Literature Review

A literature survey was conducted using bibliographic databases, such as Transportation Research Information Services (TRIS), Research in Progress (RiP), State DOTs Planning and Research websites, and the ASCE research library. The literature search identified documents/projects that are directly or indirectly related to the MEDPG implementation effort such as the laboratory testing and input databases for key materials, studies related to traffic and environment, validation and local calibration of performance models, and deployment. The documents and projects reviewed included both on-going and completed research studies.

Over 200 publications exploring various aspects of the MEPDG have been published to date. The studies collectively provide a vast reservoir of information that is a key to the successful implementation of the MEPDG. The implementation activities have been summarized for this synthesis under the following topics related to inputs and calibration for ease of reference, rather than on an agency basis:

- Material characterization:
  - Asphalt materials
  - Portland Cement Concrete (PCC) materials
  - Chemically stabilized materials
  - Unbound materials (includes environmental effects)
  - Material characterization for rehabilitation
- Climate and environmental effects:
  - Climate data
  - Environmental effects on unbound materials
- Traffic inputs
- Instrumentation and sensitivity analysis:
  - Instrumentation & Accelerated Pavement Testing (APT) studies
  - Sensitivity analysis
- Calibration and deployment of the MEPDG:
  - Calibration and validation
  - Partial or full scale deployment
  - Comparison of results by transfer function

2. MATERIALS CHARACTERIZATION

The implementation activities in the materials characterization area, for the most part, have focused on identifying data needs, as well as, gathering the necessary material properties or inputs required for using the MEPDG.

2.1 Asphalt or Bituminous Materials

Several State DOT including Arizona, Idaho, Colorado, Florida, Georgia, Idaho, Kansas, Minnesota, Mississippi, Missouri, North Carolina, Ohio, Oklahoma, Virginia, and Wisconsin have completed a significant portion of the implementation effort for asphalt materials through research contracts or in-house studies. These activities have focused on one or more of the following objectives:

- Developing an input data library for asphalt materials that represents typical mixtures used for both new/reconstruction and rehabilitation.
- Evaluating the sensitivity of material inputs for reliability assessment and to understand the mixture's relationship to field performance.
- Developing mixture-specific inputs for validation and calibration of performance prediction models used in the MEPDG.
- Measuring the inputs for specialty mixtures, such as stone-matrix asphalt (SMA), cold-recycled and mixtures with high reclaimed or recycled asphalt pavement (RAP) content, that were not included in the original material database used in calibrating the transfer functions and developing the default values for input level 3.
- Developing policy guidance on determining the level of effort required for projects of varying size, cost, and overall importance.

Many studies have included laboratory testing to measure the dynamic modulus of typical mixtures used in their states. A few studies, however, have also included creep compliance, indirect tensile strength, plastic deformation, and fatigue testing. A key issue identified by some agencies is how to enter or use results from these other mixture tests in the Pavement ME Design software for predicting rutting and fatigue cracking. A summary and overview of some completed activities in the area of asphalt materials characterization include:

- FHWA sponsored a project, *Artificial Neural Networks for Asphalt Concrete Dynamic Modulus Prediction (ANNACAP)*, to aid in populating the LTPP database with dynamic modulus data (Kim et al, 2011). The calculated dynamic modulus values are included in the LTPP computed parameter database for asphalt concrete layers with sufficient volumetric and binder data. This computed parameter database was found to be very useful in estimating and demonstrating the in place damage of HMA layers through the deflection basins in accordance with the MEPDG Manual of Practice (Von Quintus et al, 2013). The higher the damage, the greater the amount of fatigue cracking.
- Arizona DOT conducted a comprehensive study of HMA material characterization through a series of projects with Arizona State University (Witczak, 2011). Eleven typical ADOT conventional lab blended hot mix asphalt (HMA) mixtures using five different aggregates were

used in this study. Arizona DOT also developed an *AC (Asphalt Concrete) Binder Characterization Database* that contains properties of six typical AC binders commonly used in Arizona. The agency also developed separate comprehensive databases for dynamic modulus properties, thermal fracture properties, mixture fracture (fatigue) properties and permanent-strains collected from repeated load dynamic tests.

- Colorado DOT sponsored a similar study of HMA materials characterization to support their MEPDG implementation efforts. This study included nine HMA mixtures typically used in Colorado, including both neat asphalt mixtures and polymer modified asphalt (PMA) mixtures. HMA characterization tests include dynamic modulus, repeated shear tests at constant height, creep compliance, indirect tensile strength and volumetric properties. The repeated shear test results were utilized to calibrate the HMA rutting model, while other properties were used in developing materials input libraries.
- Florida DOT started their implementation process through multiple laboratory testing projects.
    - The first project developed a database for referencing available resilient modulus and dynamic modulus values by funding a laboratory testing program for Florida-specific mixtures (Ping and Xiao, 2007). This study found that the dynamic modulus values measured at a loading frequency of 4 Hz was comparable with the resilient modulus obtained from the indirect diametrical test at the same temperature level. By comparing the lab-measured modulus with the predicted values, the study found Witczak's dynamic modulus prediction model comparable for Florida's mixtures used in this study.
    - Florida sponsored a similar study to develop dynamic modulus capabilities for HMA mixtures in compression, torsion, and tension (Birgisson et al, 2004). This study included a new approach to determine creep compliance parameters from a combination of complex modulus and static creep tests. This study also evaluated the relationship between dynamic modulus and the performance of mixtures as defined by rutting and fracture.
- Idaho Transportation Department (ITD), through a research contract with the University of Idaho, developed a material inputs library for HMA (Bayomy et al, 2012). The input library was developed from laboratory test results of 27 Idaho HMA mixtures that included the different binder grades used in Idaho (PG58-28, PG58-34, PG64-28, PG64-34, PG70-28, and PG76-28), varied mix aggregate gradations, and mix volumetric properties. ITD's test program included dynamic modulus, volumetric properties, and Brookfield rotational viscosity and dynamic shear rheometer tests for all of the asphalt binders. The HMA inputs library includes inputs for all MEPDG hierarchical input levels for HMA mixtures and binders typically used in Idaho.
- Illinois DOT conducted dynamic modulus testing for twenty mixtures at 7 and 4 percent air voids (Carpenter, 2007). The test results appeared satisfactory from a structural design standpoint, with measured dynamic modulus values at 20 °C ranging from 1,000,000 psi to 2,000,000 psi.
- The Iowa Highway Research Board (IHRB) funded a study that undertook an experimental plan for characterizing the cold recycled mixtures with foamed asphalt and emulsions. The experimental plan included dynamic modulus, dynamic creep, flow number, and raveling tests (Lee and Kim, 2007; Lee et al, 2009). Iowa DOT also funded a research project to develop the asphalt dynamic modulus master curve directly from falling weight deflectometer (FWD) testing for use in MEPDG flexible pavement analysis and rehabilitation design (Contract Number IHRB-12-06).

- Kansas DOT has sponsored multiple projects related to implementing the MEPDG.
    - One of their first studies was to evaluate if the HMA dynamic modulus could be determined or measured during construction (Gedafa et al, 2009). This study performed a statistical comparison between dynamic modulus measured in the laboratory at different temperatures (40, 70 and 95˚F) to the values backcalculated from FWD measurements and calculated with the Witczak and Hirsh dynamic modulus regression equations. The study concluded that no two approaches provided statistically consistent results (i.e.; some approaches tended to give similar moduli for a certain site, but not for all sites). When compared with the laboratory measured values, the study observed, the Witczak regression equation underestimated the dynamic modulus at low temperatures and overestimated them at high temperatures.
    - Kansas sponsored another study to develop a database of material inputs required by the Pavement ME Design software for HMA mixes (Romanoschi et al, 2009). The study included a large laboratory test program for measuring the dynamic modulus at 7.0 and 4.0 percent air voids, creep compliance, and indirect tensile strengths for a wide range of mixtures. The study found both the Witczak and Hirsch dynamic modulus regression equations underestimated dynamic modulus in comparison to the laboratory measured values.
    - Another Kansas study was completed by Romanoschi et al (2006) to measure and evaluate dynamic modulus, bending stiffness, and fatigue properties of four typical Superpave designed base mixtures. Romanoschi et al found the MEPDG fatigue cracking transfer function over-predicted the fatigue lives for the mixtures with virgin binder and severely under-predicted SBS modified mixtures in comparison to the laboratory-based fatigue test results.
- Maryland State Highway Agency (MDSHA) has assembled a database of material properties primarily involving asphalt binder properties and HMA dynamic modulus. (Schwartz, C.W. and R. Li, 2011). MDSHA is planning to conduct a local calibration of the distress transfer functions through a tiered approach. The first tier of this approach is to verify or confirm the adequacy of the transfer functions using the global calibration coefficients to simulate pavement performance in Maryland. The first tier is just beginning. Other agencies have also used this tiered approach in evaluating and implementing the MEPDDG. Arizona, Colorado, Georgia, Mississippi, and Pennsylvania are agencies using this approach because the results can be digested on an incremental basis by agency personnel unfamiliar with ME-based procedures.
- Clyne and Marasteanu (2004) developed an inventory of the rheological properties of certified asphalt binders used in Minnesota. This study conducted a suite of laboratory tests to evaluate the rheological properties of nine asphalt binders. Marasteanu et al (2003) also conducted laboratory testing on four different HMA mixtures obtained from the Mn/ROAD site to measure input level 1 properties. The study found that the Witczak dynamic modulus regression equation provided higher estimates of dynamic modulus at high temperatures than measured in the laboratory, which is opposite from the Kansas DOT findings.
- Mississippi DOT developed a library of dynamic modulus inputs for typical HMA mixtures. The study included dynamic modulus characterization on 25 mixtures with different combinations of aggregate type, maximum nominal size of aggregates, binder grades, and compaction levels (White et al, 2007). The study also included Asphalt Pavement Analyzer (APA) tests to provide

Mississippi DOT a relative comparison of the mixture's resistance to rutting and potential in-service performance.

- Missouri DOT developed a library of creep compliance and indirect tensile strengths for selected plant-produced surface course mixtures at different air void levels (Richardson and Lushar, 2008). The results from this laboratory test program were used in Missouri's initial local calibration study.
- Nebraska Department of Roads (DOR) developed a database of layer stiffness values (dynamic modulus, creep compliance, and resilient modulus) for various agency-specific mixtures (Im et al, 2010). The study identified some discrepancies between measured and predicted dynamic modulus using the Witczak regression equation, as well as for the default values of creep compliance. At lower temperatures and/or higher loading frequencies (stiffer mixtures), the discrepancies or differences between laboratory-measured and calculated modulus values are mix-dependent.
- The New England Transportation Consortium funded a research project to establish default dynamic modulus for the New England States. In specific, the study investigated whether there was a significant difference between dynamic modulus values for mixtures typically used throughout the region. The study also compared the dynamic modulus of laboratory and plant produced mixes with the MEPDG input level 3 values (Jackson et al, 2011).
- New Jersey DOT sponsored multiple projects related to materials characterization of asphalt mixtures.
  - The first study developed a catalog of dynamic modulus inputs for plant-produced and laboratory-compacted samples for various HMA mixtures (Bennert, 2009). The study evaluated the precision of the Witczak and Hirsch regression equations, and found dynamic modulus values calculated from the Witczak regression equation compared better with the laboratory measured values than those calculated from the Hirsch regression equation. In addition, the precision of the predictions were better for the PG64-22 asphalt binders than for the polymer modified PG76-22 asphalt binders. The precision, however, improved when the actual test data of rolling thin film oven (RTFO) aged asphalt binders were used in lieu of input level 3 default binder properties.
  - Maher and Bennert (2008) compared the laboratory-measured values for Poisson's Ratio of typical HMA mixtures placed in New Jersey with the values estimated with the prediction equation included in the MEPDG. Maher and Bennert found some discrepancies between the measured and predicted values, especially when higher or stiffer asphalt binder grades were used.
- North Carolina DOT also sponsored multiple laboratory studies.
  - The first study was to develop a library of dynamic modulus inputs for commonly used HMA mixtures (Kim et al, 2005). The study included 42 mixtures with varying aggregate sources, aggregate gradations, asphalt sources, asphalt grades, and asphalt contents. The study also evaluated the precision of the Witczak regression equation, and found the predictions compared better at lower temperatures than at higher temperatures. A parametric study was also conducted to study the effects of mixture variables on dynamic modulus. An outcome from this study was an analytical solution developed in accordance with the theory of linear viscoelasticity to estimate the dynamic modulus, phase angle, and Poisson's ratio using results from the indirect tensile test.

- A second study is focusing on determining the MEPDG inputs for warm mix asphalt (WMA), and whether there is a significant difference in comparison to standard HMA mixtures. Within this study, dynamic modulus tests are being performed for stiffness characterization, direct tension cyclic tests for fatigue performance characterization, and triaxial repeated load plastic deformation (TRLPD) tests for rutting characterization. An outcome from this study is to develop recommendations for the MEPDG inputs for WMA mixtures. The date for completion of this study is unknown.

- Ohio DOT developed a database containing mechanical properties of a wide variety of pavement materials utilized in each of the 28 pavement-related research projects conducted within the last two decades (Masada et al, 2004).
- Oklahoma DOT also sponsored multiple studies.
  - One of the study's was to develop an improved procedure of predicting dynamic modulus for Oklahoma's use to minimize the need for performing laboratory testing for each mixture in a pavement structure (Cross et al, 2007). The study includes dynamic modulus testing for 21 mixtures representing different mixture types, regional aggregate sources, binder grades, and mixtures with and without RAP.
  - Another study included rheological tests on asphalt binders commonly used in Oklahoma. The test program also included measuring shear modulus (G*) and phase angle values over a range of temperatures (Hossain et al, 2011). Phase 2 of this study included measurements of dynamic modulus of SMA mixtures. The measured dynamic modulus of the SMA mixtures were compared to values calculated using the MEPDG or Witczak dynamic modulus regression equation (Cross et al, 2011).
  - Oklahoma DOT is currently collecting laboratory and field data vital for HMA mixtures with high RAP contents. The testing plan for this study involves performance testing of high RAP mixtures and rheological properties of the blended (virgin and recovered) binders (Oklahoma DOT contract number: DTRT06-G-0016).
- Lundy et al (2005) conducted dynamic modulus tests on typical asphalt mixtures for the Oregon DOT to evaluate the precision of the Witczak dynamic modulus regression equation. The study concluded the predicted values did not have good correspondence to the measured values.
- South Dakota DOT funded a study to measure the dynamic modulus on typical HMA mixtures produced in South Dakota and subgrade resilient modulus on the more commonly encountered soils in South Dakota (Contract Number: SD2008-10).
- Texas DOT developed a HMA database for use with the MEPDG through a suite of performance tests (Bhasin et al, 2005). This study included several plant and laboratory-produced mixtures. Tests included in the experimental plan were APA, Hamburg, Dynamic Modulus, Flow Time, Flow Number, and Simple Shear at Constant Height for a comparison of test results and an evaluation of rutting resistance for the mixtures included in the study.
- Virginia Tech Transportation Institute (VTRC) has conducted multiple studies in preparing to implement the MEPDG, some of which are summarized below.
  - One of the studies included performing dynamic modulus, creep compliance and tensile strength tests of 11 mixtures (4 base, 4 intermediate, and 3 surface mixes). These mixtures were sampled from different plants across the state. In addition, the resilient modulus test was performed to determine any correlation with the dynamic modulus test. The dynamic moduli calculated using the Witczak regression equation were found to

have reasonable correspondence to the laboratory measured binder properties. The regression equation, however, did not account for some of the differences between the mixtures observed in the laboratory measurements.
    - Virginia DOT has also developed a catalog of properties for the asphalt binders typically used in Virginia. Another objective of this study was to investigate the effect of changes in asphalt-binder properties on the predicted distress levels for trial pavement designs evaluated using the MEPDG (Diefenderfer, S.D., 2011).
    - Addition testing has also been completed on 12 mixtures identified among the most used contract items of VDOT's Maintenance Division (Apeagyei A.K. and S.D. Diefenderfer, 2011).
- Washington DOT developed a database of dynamic modulus values for typical Superpave mixes widely used in the state. Seven mixtures were included in the study (Tashman, L and M.A. Elangovan, 2007).
- Wisconsin DOT has completed three research projects on HMA material characterization. The first project evaluated the stiffness and permanent deformation properties of 12 Wisconsin mixtures using the Asphalt Mixture Performance Tester (AMPT). A database containing dynamic modulus master curve and flow numbers was assembled to support the MEPDG implementation efforts (Bonaquist, 2010). The second project focused on establishing a range of tensile strength and creep compliance properties (Bonaquist, 2011), and the third project focused on creating a database of flow numbers for representative mixtures (Bonaquist, 2012).

As summarized above and extracted from the published documents, most asphalt characterization studies have focused on building libraries of level 1 asphalt material inputs (primarily dynamic modulus) with agency-specific mixtures. Many of these studies have invariably evaluated the Witczak dynamic modulus regression equation for its ability to reasonably predict dynamic modulus values of the agency-specific mixtures. While the calculated dynamic moduli are acceptable for mixtures with neat binders, significant deviations were observed for binders with higher or stiffer PG grades. Use of measured binder test data (i.e. input level 2) in the Witczak regression equation did improve the accuracy of the dynamic modulus predictions.

Fewer studies have focused on material characterization (dynamic modulus) of non-conventional mixtures (such as SMA, mixtures with high RAP content, and cold-recycled mixtures), and measuring the plastic strain and fatigue or fracture properties of HMA mixtures. Similarly, few studies have focused on the characterization of existing asphalt layers for rehabilitation designs and on the effect of mixture type on the predicted distress, but there have been a few exceptions; some of which are listed below.

- The Asphalt Institute sponsored a study to compare the predicted and observed performance between HMA and PMA mixtures. The Asphalt Institute found a significant difference in performance (rutting and fatigue cracking), which was not explained by the MEPDG procedure when using input level 3. Figures 1 and 2 provide a comparison of the predicted and measured distresses for rutting and fatigue cracking, respectively. As shown, the MEPDG over predicted rutting and fatigue cracking for PMA mixtures. This finding is similar to the finding from the Romanoschi, et al., study.

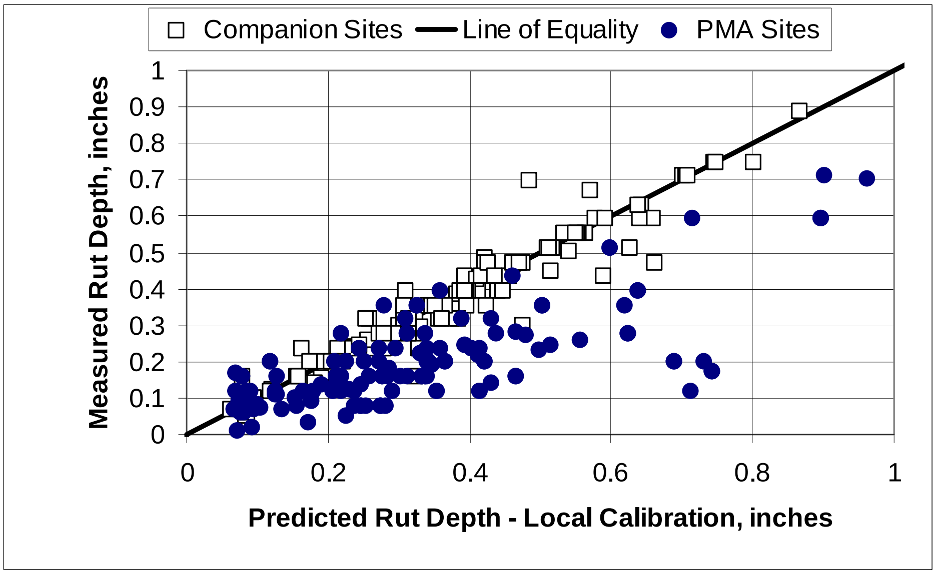


Figure 1. Comparison of Measured and Predicted Rut Depths using the Locally Calibrated Equation for the Companion Sites and those with PMA Mixtures

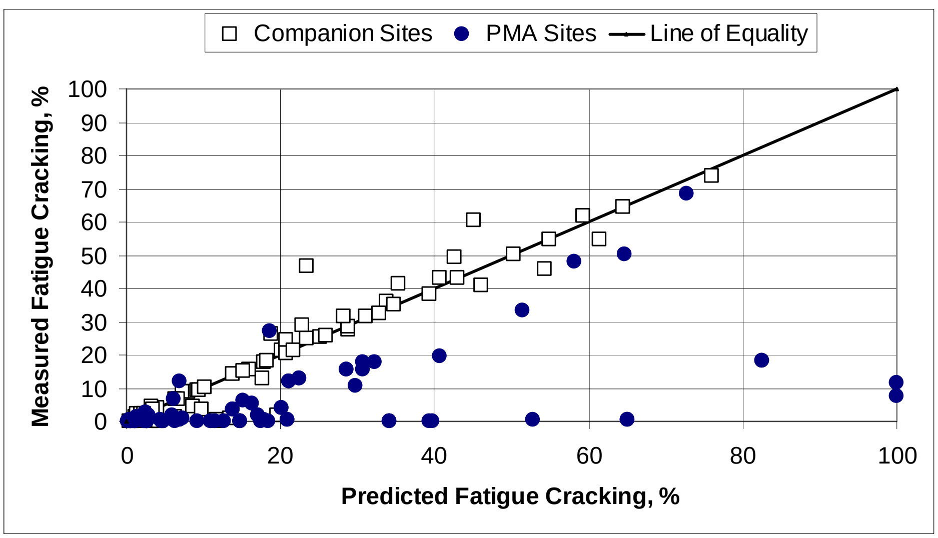


Figure 2 Comparison of the predicted and measured fatigue cracking for the companion sites and those sections with PMA mixtures.

- NCHRP project 9-30A (Von Quintus, et al., 2012) focused on evaluating multiple rut depth transfer functions in comparison to the Kaloush-Witczak transfer function included in the Pavement ME Design software. Repeated load constant height shear and triaxial tests, as well as dynamic modulus tests were performed on a range of dense-graded neat and PMA mixtures with the purpose of relating the laboratory-derived or measured plastic strain constants of the transfer function to field-derived constants. Figures 3 and 4 show the resulting relationships between laboratory and field-derived values that can be used with the Kaloush-Witczak rut depth transfer function in the Pavement ME Design software. Figures 3 and 4 permit an agency to adjust the laboratory-derived repeated load plastic strain constants of different mixtures for use in the MEPDG rut depth prediction methodology – input level 1. The other important finding from NCHRP project 9-30A was the use of repeated load plastic strain tests was significantly more important than dynamic modulus tests. In other words, the plastic strain constants from repeated load triaxial tests explained the differences in rutting of HMA mixtures more accurately, than the use of dynamic modulus; similar to the finding from the Asphalt Institute study and comparison of HMA and PMA mixtures.

2.2 Portland Cement Concrete Materials

Overall, there have been fewer implementation activities pertinent to the characterization of Portland cement concrete (PCC) materials. The PCC material characterization studies completed have been primarily focused on two objectives:

1. Building a library of material properties (strength and modulus) measured on new PCC mixtures.
2. Measuring the thermal properties of PCC with an emphasis on the coefficient of thermal expansion (CTE).

The key activities of State agencies are summarized as follows:

- Arkansas State Highway and Transportation Department (SHTD) developed a catalog of PCC material properties required by the MEPDG, including: CTE, Poisson's ratio, and elastic modulus (Hall and James, 2009). This effort included testing of 24 concrete/cement paste mixtures at ages ranging from 7 to 90 days with various local aggregate types. This study also updated the regression equations of the MEPDG strength gain curve for local conditions.
- Colorado DOT developed a catalog of material properties for 4 PCC mixtures typically used in Colorado. The testing plan included compressive strength, flexural strength, splitting tensile strength, Young's modulus, Poisson's ratio, CTE and shrinkage tests. The results from this teste program were used to build Colorado's input library for standard PCC mixtures.
- Florida DOT developed a catalog of inputs similar to Colorado's using test results from 3 standard PCC mixtures (Ping and Kampmann, 2008). The testing plan included compressive strength, flexural strength, splitting tensile strength, Young's modulus, Poisson's ratio, and CTE. This study also conducted a sensitivity analysis to understand the effects of CTE on PCC behavior for the magnitudes measured on PCC mixtures in Florida. The sensitivity matrix indicated the MEPDG performance predictions are not CTE sensitive to load transfer efficiency, minimally CTE sensitive to faulting, CTE sensitive to bottom-up damage (for thin PCC layers), and extremely CTE sensitive to top-down damage, cracking, and smoothness. The study concluded two out of three pavement performance criteria appeared to be highly susceptible to CTE in Florida JPCP structures.
- Iowa DOT synthesized and analyzed over 20,000 data sets obtained from various sources to determine Iowa-specific inputs for the MEPDG input level 3 for PCC. Most of the data included test results on fresh concrete and for measuring PCC strength properties. The MEPDG level 3 inputs were compared to different statistical parameters from this data set (Wang et al, 2008).

Under a follow-on project, the thermal properties of typical Iowa concrete materials, such as CTE and thermal conductivity were measured and analyzed (Wang et al, 2008).

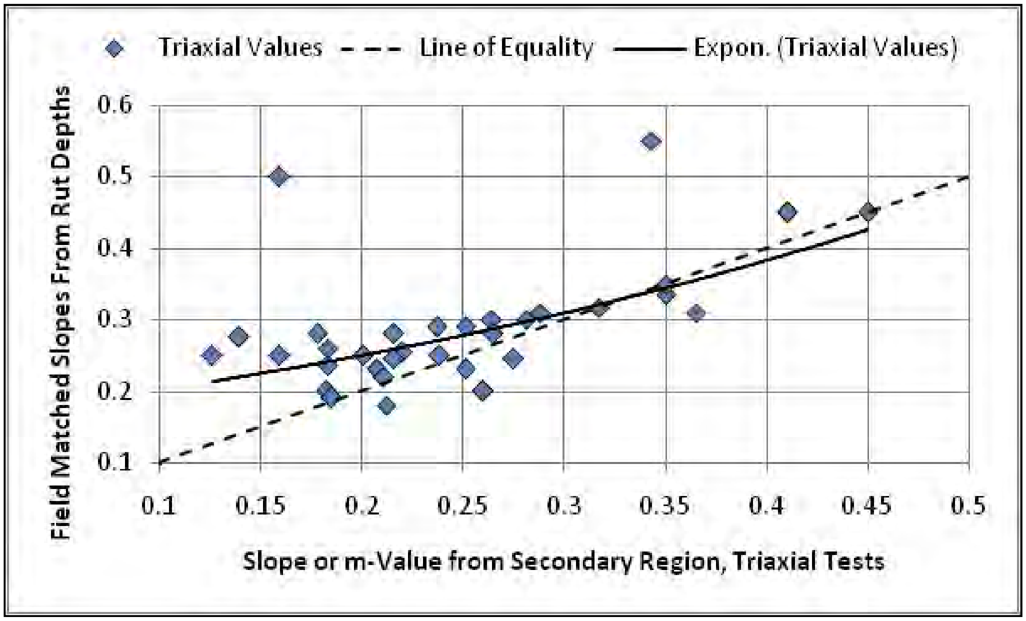


Figure 3. Relationship between Field Matched and Laboratory Measured Slopes or m-Values (N-term exponent)

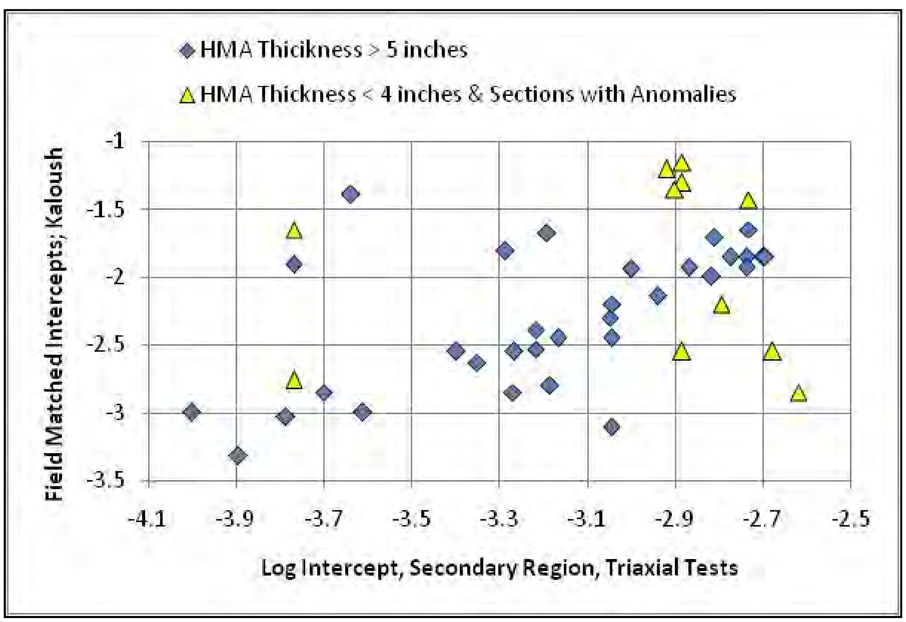


Figure 4. Comparison of Field Matched Intercept and Laboratory-Derived Secondary Region from Repeated Load Triaxial Tests

- Louisiana Transportation Research Center (LTRC) completed a study that measured the CTE of typical PCC mixes used in Louisiana. Three types of aggregates (Kentucky limestone, gravel, and Mexican limestone) were used in this study. CTE was measured on test specimens at various ages (3, 5, 7, 14, 28, 60, and 90 days), with different coarse aggregate proportions (20, 64, and 80 percent of coarse aggregate), and over a range of relative humidity values to identify the factor(s) that has the most critical impact on CTE (Shin, H.C. and Y. Chung, 2011).
- Michigan DOT funded a research study to measure the CTE of typical PCC mixtures made with 8 different sources of coarse aggregates (Buch and Jahangirnejad, 2008). Evaluating the sensitivity of the MEPDG performance predictions, this study found the PCC transverse cracking sensitive to CTE values, slab thickness and joint spacing; and further observed that, from a practical perspective, a combination of thinner slab, longer joint spacing, and higher CTE values could prove detrimental to pavement performance.
- Mississippi DOT developed a library of the MEPDG inputs for PCC materials through a research contract with the University of Mississippi (Al-Ostaz, 2007). The testing plan included a range of concrete mixtures with local aggregate types and cement blends. Extensive testing of all key PCC materials properties was conducted.
- Similarly, Pennsylvania DOT developed a library of PCC inputs for use in the MEPDG through a research contract with the University of Pittsburg (Nassiri. S and J. M. Vandenbossche, 2011).
- Wisconsin DOT developed a database of splitting tensile strength and CTE values for PCC materials. The experimental testing plan included various types of coarse aggregates from 15 sources and various combinations of cementitious materials (Naik et al, 2006). Wisconsin also funded another project that evaluated local aggregates and cementitious materials for fresh concrete, and to measure both the thermal and strength properties (Effinger et al, 2012). This study included fifteen sources of coarse aggregate, two sources of fine aggregate, two sources of ordinary Portland cement, two sources of slag cement, and three sources of fly ash.

While most of the implementation activities relative to PCC mixture characterization have focused on building libraries of PCC material properties, several agencies have measured the CTE values of typical PCC mixtures with local aggregates and understanding the significance of CTE in performance predictions.

2.3 Chemically Stabilized Materials

There have been very few studies for measuring the material properties of chemically stabilized materials for implementing the MEPDG. One reason for this observation could be that the fatigue cracking transfer function for semi-rigid pavements was not calibrated under NCHRP projects 1-37A or 1-40D. The Montana DOT is the only agency to-date that completed a material characterization and local calibration study of the fatigue cracking transfer function of semi-rigid pavements, which was done using version 0.9 of the MEPDG. The following summarizes the findings from that study for use in semi-rigid pavement design.

- For High Strength Cement Aggregate Mixtures (CAM)—(intact cores can be recovered and mixture has cement contents greater than 6 percent, with compressive strengths greater than 1,000 psi:

  - $B_{c1} = 0.85$.
  - $B_{c2} = 1.10$.
- For CAM with moderate levels of cement—intact cores can be recovered and mixture has cement contents greater than 4 percent but less than 6 percent, with compressive strengths generally greater than 300 psi but less than 1,000 psi:
  - $B_{c1} = 0.75$.
  - $B_{c2} = 1.10$.
- For Low Strength CAM—intact cores cannot be recovered and mixture has cement contents generally less than 4 percent, with compressive strengths less than 300 psi:
  - $B_{c1} = 0.65$.
  - $B_{c2} = 1.10$.

Version 0.9 of the MEPDG, however, contained an error in the software where the elastic modulus of the chemically stabilized layer was hard-coded and could not be changed at the time. Thus, any difference in elastic modulus between the chemically stabilized materials had to be considered through the local calibration coefficient, as summarized above. Other on-going studies related to the chemically stabilized layers of semi-rigid pavements are listed below:

- Mississippi DOT funded a study to quantity the effects of compaction and moisture conditions on the strength of chemically stabilized soils (James et al, 2009). The findings were used to optimize pavement structural sections and to provide data to improve construction specifications relative to the MEPDG.
- Although not a part of MEPDG implementation, Oklahoma DOT funded a project to characterize chemically stabilized materials. This study focused on measuring the stiffness and other properties of eight common fine-grained soils (A-4 through A-7-6) stabilized with four different chemical additives (hydrated Lime, cement kiln dust and 2 sources of Class C Fly Ash) in varying amounts (Cerato et al, 2011).
- NCHRP project 4-36 is evaluating different fatigue cracking transfer functions for chemically stabilized materials in comparison to the current transfer function included in the MEPDG. The objective of the project is to recommend a fatigue cracking transfer function for the semi-rigid pavements that can be included in the Pavement ME Design software, if found to be more accurate than the current transfer function. Multiple test sections have been sampled and are being monitored to measure various properties of these layers, in addition to those required as inputs to the MEPDG. The project was recently completed.

2.4 Unbound Materials – Aggregate Base and Embankment Soils

The implementation activities pertinent to the characterization of unbound materials have primarily focused on the following three objectives:

1. Developing a resilient modulus data library for typical granular aggregate base materials and local subgrade soils.
2. Developing a resilient modulus prediction model or constitutive equation based on soil parameters.
3. Using FWD and other non-destructive tests to determine the design resilient modulus.

The key activities of State agencies are summarized as follows:

- Florida DOT developed a database of resilient modulus for local soils (Ping et al, 2003).
- As a part of Idaho's MEPDG research project, default input level 3 resilient modulus values for Idaho unbound materials and subgrade soils were developed (Bayomy et al, 2012). This study analyzed a database of Idaho historical test results collected from the Idaho DOT districts. An outcome from this study was a correlation or regression equation between resilient modulus and R-value for input level 2 of fine grained soils in Idaho.
- Indiana DOT sponsored multiple laboratory studies related to the material characterization of unbound materials and layers.
    - One of the studies was an in-house testing program to assess the resilient and plastic strain behavior of involving 14 cohesive subgrade soils and 5 cohesionless soils commonly encountered in the Indiana (Kim and Siddiki, 2006). The experimental plan included resilient modulus tests, physical property tests, unconfined compressive tests, and Dynamic Cone Penetrometer (DCP) tests. This study recommended the use of regression equations for predicting resilient modulus based on unconfined compressive strengths, and the k_1, k_2 and k_3 coefficients of the resilient modulus constitutive equation based on basic soil properties; such as moisture content, dry density and Atterberg limits.
    - Another Indiana DOT in-house study investigated the relationship between the FWD modulus and laboratory measured resilient modulus of the subgrade soils under four flexible pavement sections (Dai et al, 2010). This study concluded the laboratory resilient modulus of the soil, on average, was 0.48 times the backcalculated elastic modulus value from FWD deflections; slightly higher than the value reported or recommended for use by the MEPDG Manual of Practice (0.35), but similar to the value recommended for use from the Montana DOT implementation study (0.50).
    - Two decades earlier, although not directly related to the MEPDG implementation, Indiana DOT developed a set of resilient modulus data of six soils commonly encountered in Indiana (Lee et al, 1992). This study developed a set of correlations between the resilient modulus and the unconfined compression test results for normal and thawed subgrade conditions. The correlation was based on test results on the specimen sampled from existing subgrades.
- Iowa DOT developed a library of resilient modulus values for typical unbound pavement materials. The resilient modulus values included in the library were determined using the repeated load triaxial resilient modulus test results (Ceylan et al, 2009). This library includes the non-linear, stress-dependent resilient modulus model coefficients values for input level 1. The library also includes the unbound material properties values correlated to resilient modulus for input level 2, and default resilient modulus values for input level 3.
- The Kentucky Transportation Cabinet developed a regression equation which was developed from resilient modulus tests on typical crushed stone aggregate bases (Hopkins et al, 2007). A windows based computer program was developed to make the resilient modulus data and the new regression equation readily available to agency design personnel, and is embedded in the

Kentucky Geotechnical Database. Earlier, the same team developed a similar model or regression equation for predicting the resilient modulus of typical Kentucky soils (Hopkins et al, 2001).

- LTRC conducted field and laboratory tests to develop resilient modulus prediction models for Louisiana's subgrade soils using the DCP, CIMCPT, FWD, Dynaflect, and soil physical property tests (Mohammed et al, 2007). Four soil types and nine overlay rehabilitation pavement projects in Louisiana were selected. LTRC is sponsoring another project to validate the prediction of seasonal variation strengths in the base course and subgrade (Contract Number: 30000425).
- Michigan DOT evaluated whether the processes used by its regions in determining the roadbed resilient modulus and modulus of subgrade reaction are compatible with the new MEPDG requirements (Baladi et al, 2009). Another study determined the resilient moduli of the various granular base and subbase materials by backcalculating the elastic modulus from FWD deflection basins (Baladi et al, 2011).
- Minnesota DOT funded a study to investigate the strength and deformation characteristics of base material produced from various proportions of RAP and aggregate base (Kim and Labuz, 2007). Resilient modulus and the compaction characteristics of these different unbound materials were also evaluated.
- Mississippi DOT developed several predictive models to estimate resilient modulus of typical Mississippi soils from soil index properties (George, 2004). A similar study investigated the viability of using FWD data for deriving resilient modulus through empirical correlations (George et al, 2003). In addition, Mississippi DOT tested 34 subgrade soils, 13 granular base/subbase materials, and 16 stabilized soils for developing their pavement materials library. Mississippi DOT has documented their practical experience, lessons learned, and observations made during the testing and review of the data.
- Missouri DOT developed a library of resilient modulus values for granular base materials and subgrade soils (Richardson et al, 2007). The experimental plan included 27 subgrade soils and five granular base materials commonly found in Missouri. This study also presents regression models to estimate k_1, k_2 and k_3 coefficients from basic or physical soil properties.
- Montana DOT compared over thirty different resilient modulus prediction models available in the literature and evaluated those with laboratory data for two soils sampled in Montana (Mokwa and Akin, 2009). This study discouraged the general use of such models without prior testing and verifying the reliability of the model estimates until additional studies suggest otherwise.
- A New England Transportation Consortium (NETC) study developed regression equations to estimate k_1, k_2 and k_3 coefficients for typical soils encountered in the New England states (Malla and Joshi, 2006). Using the data extracted from the LTPP database, this study developed prediction equations for six AASHTO soil types and was further validated with laboratory measurements.
- The Cold Regions Research and Engineering Laboratory (CRREL) conducted resilient modulus tests on five subgrade soils commonly found in the state of New Hampshire (Janoo et al, 1999).
- New Jersey DOT funded a laboratory program to determine the resilient modulus of typical subgrade soils encountered in New Jersey (Bennert et al, 2000). Laboratory results were used to calibrate a statistical model for predicting the resilient modulus at different moisture contents and stress ratios.

- An Oklahoma DOT study evaluated the effect of post-compaction moisture content on the resilient modulus of selected soil types (Zaman and Khoury, 2007). The findings from this study were used to improve the existing database of resilient modulus and suction values for selected soil types. Another Oklahoma DOT study compiled resilient modulus data for Oklahoma subgrade soils and aggregates (Hossain et al, 2011). This study also conducted statistical analyses of the resilient modulus data to evaluate selective stress-based models for unbound and stabilized subgrade soils, and develop correlations between resilient modulus and other routine soil parameters.
- Another Oklahoma DOT study undertook an experimental plan to investigate engineering properties of chemically stabilized subgrades. The plan included resilient modulus, modulus of elasticity, unconfined compressive strength, moisture susceptibility and three-dimensional swell. Four different types of soils treated with three stabilizers (hydrated lime, class C fly ash and cement kiln dust) were included in this study (Solanki et al, 2009).
- VTRC undertook a program to develop a library of resilient modulus values for Virginia's subgrade soils for use in the MEPDG (Hossain, 2008). More than 100 soil samples from all over Virginia representing every physiographic region were sampled and tested for resilient modulus, soil index properties, standard Proctor, and California Bearing Ratio (CBR) testing. Resilient modulus values and regression coefficients (k-values) of the constitutive equations for resilient modulus were computed for typical Virginia soils. This study observed that only the quick shear test was found to have statistically significant correlations with resilient modulus. Another VTRC study focused on developing a database of resilient modulus values (or k-values) for typical unbound base materials and subgrade soils (Hossain, 2010)
- Wisconsin DOT funded a laboratory testing program to evaluate their physical and compaction properties of commonly found subgrade soils (Titi et al, 2006). This study developed statistical correlations to estimate k_1, k_2 and k_3 coefficients from basic soil properties. Another Wisconsin DOT study undertook an experimental plan to develop a resilient modulus predictive model for typical crushed aggregate base materials encountered in Wisconsin (Eggen and Brittnacher, 2004). The plan included 37 aggregate sources and a wide range of influencing variables, such as physical characteristics, material type, source lithilogy and regional factors, which were all used in the evaluation for their effect on resilient modulus.

Most agencies have undertaken comprehensive laboratory studies to measure resilient modulus properties of typically encountered materials and soils. These studies have then used these experimental results to either build a library of typical values or develop statistical models for estimating resilient modulus from basic soil physical properties.

Some studies have developed empirical models to derive k_1, k_2 and k_3 coefficients for the resilient modulus constitutive model. Although the MEPDG recommends the use of lab measured resilient modulus properties, Khazanovich et al (2006) observed that the MEPDG Manual of Practice does not provide adequate guidance on using the test data in multilayer elastic theory (MLET) analysis. In their 2006 TRB paper, the authors provide a detailed step-by-step guidance on how to determine resilient modulus using the model coefficients by taking stress states into account.

FHWA and various State pavement associations, however, have sponsored numerous studies to determine the in place resilient modulus of aggregate base layers and subgrades using the results from repeated load resilient modulus tests included in the LTPP database and other sources. A few of these include:

- Yau and Von Quintus (2002) developed regression equations to estimate the k1, k2 and k3 coefficients of the universal resilient modulus constitutive equation recommended for use in the MEPDG Manual of Practice. The authors suggested, however, that the regression equations be used with caution because of the poor statistics from the regression analyses.

- Von Quintus and Killingsworth (1998) developed and recommended values for the AASHTO c-factor in relating laboratory-derived resilient modulus for the in place unbound layers in comparison to the field-derived elastic modulus backcalculated from deflection basins. The c-factors were found to be dependent on pavement structure and independent of soil or material type. The variation of the c-factors for each data set or pavement structure, however, was high in some cases – exceeding a coefficient of variation of 30 percent.

As noted above, a few agencies have developed relationships between R-value or CBR and resilient modulus of the soil. The MEPDG relationship between CBR and resilient modulus is shown below:

$$M_r = 2555(CBR)^{0.6}$$

While the CBR test is typically conducted at a range of moisture contents and compactive effort, the design CBR is selected based on the degree of compaction and moisture content expected in the field. In the Pavement ME Design software, when the design CBR is used an input to determine subgrade resilient modulus, the moisture content and density values associated with the input CBR must also be used. If the optimum water content and maximum dry unit weight were entered into the software with the CBR value, the program will significantly underestimate the resilient modulus over time. A few states have observed this fact and have changed or revised their default values for water content and dry density to be consistent with the resilient modulus default values.

The AASHTO MEPDG Interim Manual of Practice provides recommendations of input level 3 default resilient modulus values for use in the ME Design software (see Table 1). Note that the input level 3 resilient modulus values presented in Table 1 represent optimum moisture condition and maximum dry density typically anticipated in the field at the time of construction.

For rehabilitation designs, the resilient modulus of each unbound layer and embankment may be backcalculated from the FWD deflection basin data. A key debate with the use of backcalculated unbound moduli values is related to the relationship between field-derived FWD modulus and laboratory-measured resilient modulus of unbound materials has yet to be resolved. The debate is whether the AASHTO c-factor or ratio (i.e. $C = Mr/E_{FWD}$) between the laboratory-derived resilient modulus and elastic modulus backcalculated from deflection basins is a reality, as well as the accuracy and appropriateness of the ratios that have been reported to date. This debate is significant particularly in the context of rehabilitation design, as the MEPDG requires lab measured resilient moduli at optimum moisture content (at the time of construction) as inputs for unbound layers, while the in-situ condition may not represent the same. Some agencies use the backcalculated elastic modulus value as the input to the resilient modulus to the MEPDG, while most states adjust the backcalculated elastic modulus using the c-factor. Few agencies have investigated this feature and do not adjust or correct the optimum water content and maximum dry unit weight for the unbound layers to be consistent with the in place values.

Table 1. Resilient Modulus for Unbound Materials: MEPDG Input Level 3.

AASHTO Soil Classification	Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi		
	Base/Subbase for Flexible and Rigid Pavements	Embankment & Subgrade for Flexible Pavements	Embankment & Subgrade for Rigid Pavements
A-1-a	40,000	29,500	18,000
A-1-b	38,000	26,500	18,000
A-2-4	32,000	24,500	16,500
A-2-5	28,000	21,500	16,000
A-2-6	26,000	21,000	16,000
A-2-7	24,000	20,500	16,000
A-3	29,000	16,500	16,000
A-4	24,000	16,500	15,000
A-5	20,000	15,500	8,000
A-6	17,000	14,500	14,000
A-7-5	12,000	13,000	10,000
A-7-6	8,000	11,500	13,000

The MEPDG Interim Manual of Practice recommends the use of c-factors presented in Table 2 to adjust FWD backcalculated unbound layer modulus to an equivalent laboratory derived resilient modulus. These factors represent the moisture content and associated dry density of the in-situ materials; therefore, these factors should only be used in conjunction with the in-situ moisture contents and dry densities measured from materials recovered from field sampling such as borings.

Table 2. C-factors Recommended in the MEDPG Manual of Practice to Convert FWD Backcalculated Subgrade Modulus to Laboratory Derived Resilient Modulus.

Layer Type	Location	C-Value or M_r/E_{FWD} Ratio
Aggregate Base/Subbase	Between a Stabilized & HMA Layer	1.43
	Below a PCC Layer	1.32
	Below an HMA Layer	0.62
Subgrade-Embankment	Below a Stabilized Subgrade/Embankment	0.75
	Below an HMA or PCC Layer	0.52
	Below an Unbound Aggregate Base	0.35

However, during global calibration of MEPDG performance models, a c-factor of 0.55 and 0.67 was used for fine-grained and coarse-grained soils. Most agencies have just accepted these values for the c-factors. These factors represent the optimum moisture content and associated maximum dry density of the materials; therefore, these factors should only be used in conjunction with optimum moisture content and maximum dry density of the subgrade material.

An on-going FHWA study is expected to provide more insight into the relationship between field-derived and laboratory-measured modulus values. This project is currently investigating the fundamental principles underlying the observed differences backcalculated layer moduli and laboratory resilient moduli. This study is expected to demonstrate how to use interchangeably laboratory and field derived resilient moduli or provide an explanation on why it is not possible to derive such relationships (Contract Number: DTFH61-09-C-00007).

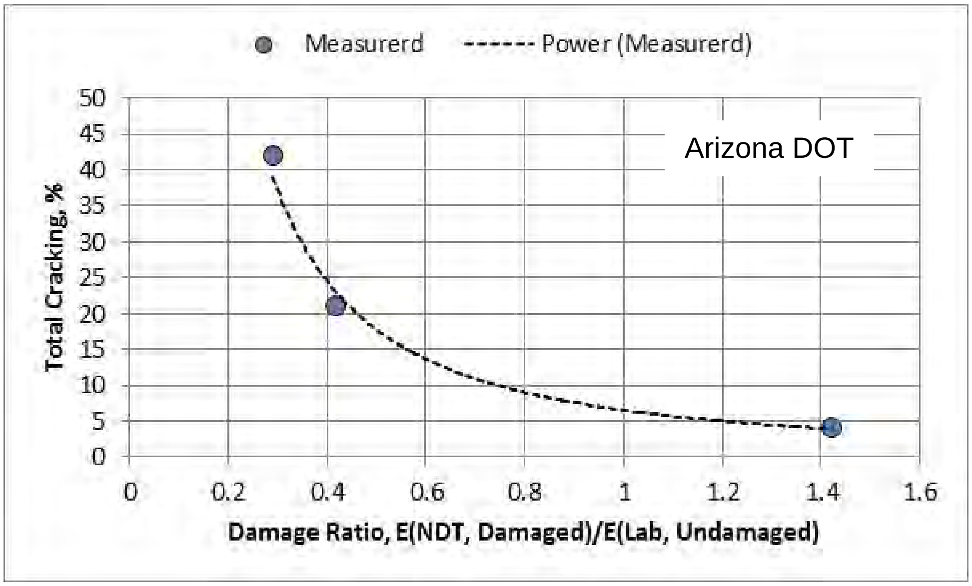
2.5 Material Characterization for Rehabilitation

The implementation efforts in the area of determining the in place damage for rehabilitation have focused on identifying critical material parameters and setting up laboratory programs and field evaluation for rehabilitation design. Texas DOT has developed a framework (Tex-ME) that documents the laboratory and field procedures to be used in material characterization for rehabilitation design; recommendations for Level 1 characterization of existing pavement damage (i.e. the rutting potential of asphalt, granular, and soil layers and the cracking potential of asphalt layers (Zhou et al, 2009)). Other agencies are developing similar protocols for the field investigations and relating surface conditions to the default categories included and defined in the MEPDG Manual of Practice (AASHTO, 2008).

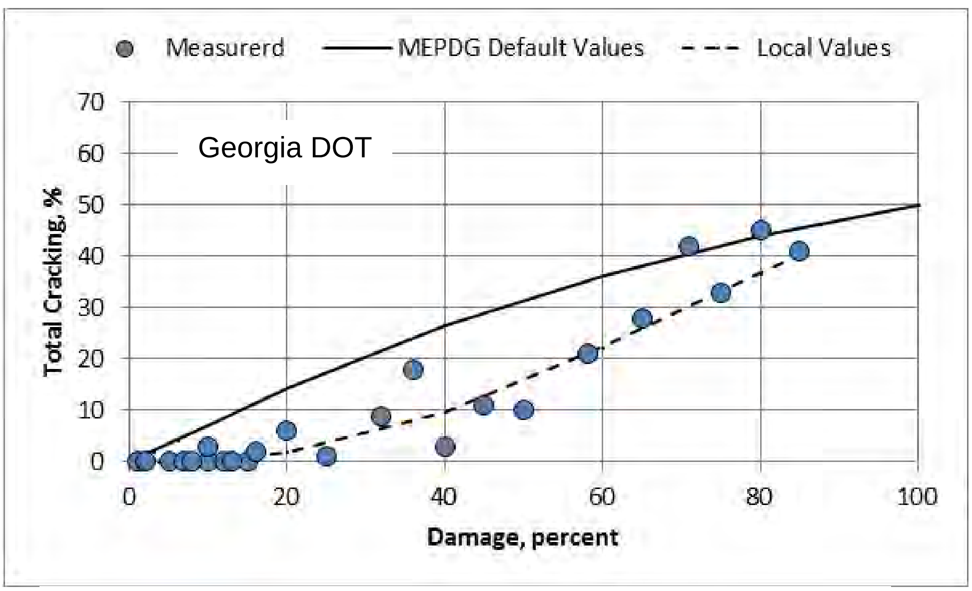
FHWA is currently funding a project to perform backcalculation of all deflection basin data that are stored in the LTPP database for flexible, semi-rigid, rigid, composite, and rehabilitated pavement structures (Contract Number: DTFH61-11-R-00019). The ratio of the backcalculated elastic layer moduli and laboratory measured dynamic moduli can be related to the amount of load related cracking for estimating the fracture coefficients of the fatigue cracking transfer function. In other words, the ratio of the backcalculated and laboratory measured or calculated modulus is the damage index (DI) and used in the fatigue cracking transfer function listed below and defined in the AASHTO Manual of Practice for the MEPDG.

$$FC_{Bottom} = \frac{100}{1 + e^{\left(C_1 C_1^* + C_2 C_2^* \left(Log DI 100\right)\right)}}$$

As an example, GDOT and AZDOT used the in place damage index concept to determine the coefficients of the damage function in relating the moduli ratio (backcalculated to laboratory measured values) to the fatigue cracking predicted through the transfer function. Figure 5 shows some examples of the relationships from Arizona and Georgia. Although the damage index concept has yet to be extensively verified, the procedure is reasonable and provides results that are consistent with destructive sampling and testing.



(a) Arizona DOT: Damage Ratio or Index Related to Total Cracking



(b) Georgia DOT: Damage Ratio or Index Related to Total Cracking

Figure 5. Comparison of the Damage Index from the Backcalculation of Layer Moduli and Total Cracking

3. CLIMATE AND ENVIRONMENTAL EFFECTS

This section of the synthesis on climate and environmental effects is grouped into two major parts: (1) the climate data itself, and (2) the use of climate data on the effects of unbound materials responses.

3.1 Climate Data

Several agencies (Colorado, Florida, Iowa, Minnesota, Mississippi and Wyoming) are either expanding or considering expansion of climatic data sets by adding the coverage period (e.g., 30 to 50 year data sets) and/or coverage area. Mississippi DOT has conducted the more aggressive study in creating weather stations for every county within Mississippi. This project is summarized in the examples listed below. Fewer instances of this database expansion than what is already available in the Pavement ME Design software, however, are documented in the literature. For the most part, the implementation activities published in the literature as related to climate data primarily focused on one or more of the following four objectives:

1. Identifying climatic sub-regions within a state.
2. Assessing the quality of the weather data.
3. Analyzing the impacts on the use of virtual weather stations on performance predictions.
4. Evaluating the validity of climate-related inputs (e.g. thermal conductivity).

Examples of implementation related to the climate data include:

- Mississippi DOT sponsored a study that was completed by the National Center for Asphalt Technology (NCAT). The purpose of the study was to create an expanded weather database that included a weather station within each county in Mississippi.[3] The database is composed of two parts: part 1 is composed of historical data and was used in Mississippi's initial local calibration work; while part 2 is composed of future weather data and will be used for pavement design. A climatologist was used to develop the forecast weather data set 30 years into the future, as well as to fill in the missing data for the historical data set for each county.
- California DOT (Caltrans) evaluated the impact of pavement temperatures and precipitation of seven distinct climatic regions on distress mechanisms for rigid, flexible and composite pavements. Based on the findings, this study summarized the expected effects of climate region on specific pavement distress mechanisms (Harvey et al, 2000).
- Florida DOT's Phase I implementation study assembled a database of key climatic variables (e.g. air temperatures, precipitation, relative humidity, Thornthwaite Moisture Index) and grouped statewide counties into four climatic sub-regions using cluster analysis (Fernando et al, 2007). The missing data were reconciled with interpolation of corresponding hourly records from neighboring stations.
- Minnesota:

[3] Mike Heitzman was the Principal Investigator or project manager on this study to create the expanded weather database. Both the expanded historical data set and the forecast data set were submitted to Mississippi DOT in separate files and then downloaded to the MEPDG software for use in calibration and design, as noted above.

- Johanneck and Khazanovich (2010) evaluated the effect of climate on predicted performance of a composite pavement (HMA over PCC), and the effect of climate file generation. Some inconsistencies in the PCC cracking predictions were observed due to incomplete or questionable records in the MEPDG hourly climatic data.
    - Another study by Johanneck et al (2011) examined the measured and MEPDG modeled temperature distributions in the composite pavement structures at the MnRoad facility. The EICM simulations of the MEPDG produced temperature distributions smaller than the measured distributions when the MEPDG default thermal conductivity value of PCC, k =1.25 BTU / hr-ft-°F, was used. A sensitivity analysis of PCC thermal conductivity indicated that a value of 0.94 BTU / hr-ft-°F resulted in the closest agreement between modeled and measured data (residual error) for a 6-in PCC test section.
- Saha and Bayat (2011) compared the predicted performance of flexible pavements using actual station-based and virtual station-based climatic data. The virtual weather station was created by interpolating climatic data (as a function of geographic distance) from surrounding locations. This study observed that the differences in predicted total rut depth between virtual and actual stations ranged from -20 to 50 percent. Total rut depth and HMA rutting was more sensitive to these differences, while the International Roughness Index (IRI) was less sensitive. These differences were primarily attributed to missing hourly records in some stations and poor quality data.
- Li et al (2010) compared the virtual climate data generated using the MEPDG with the LTPP Automated Weather Station data. This study observed when using a greater number of nearby weather stations provided more accurate results than using the closest weather station. Rather than the distance between the stations, station elevation appeared to have significantly affected the accuracy of the virtual climatic data. Similar to the observations made by Saha and Bayat (2011), this study also concluded that the variations in climatic data appeared to have greater influence on HMA rutting and lesser influence on the predicted IRI.
- Through a research contract with the University of Maryland at College Park, FHWA is currently examining current and emerging needs in climate data collection and engineering indices for use in MEPDG calibration, changes in Superpave binder performance grading, and development of future mechanistic-based infrastructure management including pavement, bridge, and other types of asset management models (Contract Number: DTFH61-11-C-00030).

By evaluating the sensitivity of expected performance to climatic data, the studies documented in the literature reinforce the importance of good-quality hourly climatic records. Missing hourly records and errors in the raw climatic data files are likely to have adverse impact on the precision and reasonableness of the predicted performance. To account for missing records, the interpolation of data from neighboring stations is typically used (e.g. Florida). Such techniques should be used with caution by considering influencing factors; such as elevation differences between the stations.

It should be understood and mentioned that most of the climate studies referenced above were completed prior to a major weather station data clean-up in releasing the latest version of the Pavement ME Design software.

3.2 Environmental Effects of Unbound Materials

The MEPDG uses unsaturated soil behavior concepts (i.e.; relationship between water content and matric suction as defined by the SWCC) to model the effects of moisture on resilient modulus of unbound models. The models used in the MEPDG were largely drawn from the national LTPP database and limited field testing of sections located throughout North America (Zapata, 2009). Zapata, however, suggested a more local or regional calibration may be needed for the EICM, as the national calibration may not be adequate for specific regions of the country. As such, the implementation activities in this area have focused on the following four objectives:

1. Validating the predictions of MEPDG-based Enhanced Integrated Climatic Model (EICM) with actual test data.
2. Examining the validity of EICM-related inputs.
3. Investigating the unbound material response (resilient modulus) to seasonal changes.
4. Analyzing the behavior of unsaturated soils – i.e. the impact of changes in moisture content on measured resilient modulus.

Examples of implementation activities of State agencies related more to the use of climate data are summarized as follows:

- Arkansas SHTD is funding a study to validate the estimations or calculations from the EICM model (Contract Number: TRC-0902). This study is monitoring changes in moisture content and stiffness during wet and dry seasons at selected sites. The measured moisture contents within each season will be compared to the predictions made with the EICM model.
- A Florida DOT study analyzed the effect of drying and wetting cycles on the resilient modulus of different soils to understand the impact of changes in moisture on the effective confining pressure of the material's response (Toros et al, 2008).
- Minnesota DOT funded a study to explore the applications of unsaturated soil behavior concepts in pavement design. This study investigated the effects of soil suction on shear strength and resilient modulus on four soils, each from a different region in Minnesota (Gupta et al, 2007). Results from this study were used to develop models for incorporating soil suction effects on shear strength and resilient modulus measurements of well compacted subgrade soils.
- New York State DOT has funded a research program to model the effects of seasonal variations on layer moduli of unbound (subbase and subgrade) materials. This study covers the climatological and materials conditions of about 90 percent of the geographic area of the state (Contract Number: RF 55505-03-03).
- Ohio DOT investigated resilient modulus and hydraulic conductivity properties of various drainable base materials (Liang, 2007). Liang evaluated the sensitivity of or results from the EICM 3.2 estimations to the material's thermal properties (surface short-wave absorptivity, thermal conductivity, and heat capacity), porosity, and amount of fines (percent passing #200 sieve). The predictions from the EICM were then compared to the environmental field data gathered at the Ohio DOT's ATB 90 project site. The authors concluded the EICM 3.2 exhibited high sensitivity to soil porosity and fines contents for moisture predictions, high to moderate

sensitivity to surface short-wave absorptivity, and low sensitivity to the asphalt thermal conductivity and heat capacity for temperature predictions.

- Oklahoma DOT is currently investigating the effects of soil suction hysteresis on resilient modulus for commonly encountered subgrade soils in Oklahoma through a large laboratory test program (Contract Number: DTRT-06-G-0016). A similar study conducted earlier evaluated the effects of wetting and drying cycles on resilient modulus of eight soils as related to the soil-water characteristics curves (SWCC). Based on the findings from this earlier study, an expanded testing program was recommended to characterize the behavior of soils subjected to cyclical seasonal changes (Zaman and Khoury, 2007).
- As part of NCHRP project 9-23A, a national database of pedologic soil families was developed. This database includes the soil properties that are needed as inputs to the MEPDG. In addition, the database focuses upon the parameters describing the SWCC, but also includes measured soil index properties needed by the EICM in all three hierarchical levels for pavement design (Zapata, 2010). The database is being used in Georgia and Mississippi, to name two, to establish site specific input values.

4. TRAFFIC INPUTS

Arizona, Florida, Georgia, Mississippi, Missouri, Texas, Wyoming, and many other agencies have sponsored traffic studies to develop truck traffic data in support of the MEPDG. Many agencies have sites with axle weight data, but the predominant data is collected with portable weight scales. The accuracy of this data has been analyzed by multiple agencies to determine whether it can be used to develop regional or state-wide averages in developing default normalized axle weight values in support of the MEPDG. The preliminary results from the Arizona, Georgia, and Mississippi studies suggest that the weigh-in-motion (WIM) data collected by these portable scales are insufficient in providing the default or site specific truck traffic inputs. Thus, most of these agencies have implemented plans to collect this data over time for generating more accurate default normalized axle load spectra (NALS). The implementation efforts in this area primarily have focused on the following four objectives:

1. Analyzing WIM data with appropriate quality checks to develop traffic inputs for the MEPDG.
2. Sensitivity analysis of traffic inputs on the MEPDG distress predictions and final pavement design thickness.
3. Applications of statistical models and techniques such as Cluster Analysis in identifying homogenous traffic patterns.
4. Review of current traffic collection infrastructure and practices to meet the requirements of the MEPDG.

The following activities and plans summarize some of the implementation efforts by various State agencies:

- Alabama DOT developed traffic factors and axle load distribution models using WIM data from thirteen sites (Turochy et al, 2005). A sensitivity analysis was performed to determine the impact on the variation in truck factors on the final pavement design thickness. Statistical models, such as some combination of log normal and normal distributions, were developed for axle load spectra. As such, the Alabama DOT has funded a research project to develop the Alabama Traffic Factors for use in ME Pavement Design. As a follow on study, Alabama plans to develop and evaluate the traffic and materials reference library and complete concurrent designs with the Pavement ME Design software. In addition, Alabama plans to perform a local calibration using these libraries in the near future (UTCA Project Number: 12415).
- Arizona DOT evaluated the way traffic data are acquired and compiled to make it compatible with the requirements of the MEPDG (Project Number: SPR 672). This project investigated the existing traffic data collection infrastructures, such as WIM stations, determined their validity and usefulness for use with the MEPDG, and developed a detailed action plan for Arizona to continuously obtain all necessary traffic data and compile that information for effective use in the MEPDG. (Darter, et al 2013)
- Arkansas SHTD developed traffic inputs for initial implementation and a procedure for updating these inputs in the future (Tran and Hall, 2006). Classification and weight data collected at 55 WIM stations were used in this study. Quality control checks were performed to ensure accurate interpretation of the data. A sensitivity analysis was also performed to evaluate the use of default

traffic values in place of statewide vehicle class distribution factors and axle load spectra. Through a follow-up research project, Arkansas SHTD also developed a software program to pre-process raw traffic data, import, check data quality, and generate the required traffic inputs for the MEPDG (Wang et al, 2009).

- California:
    - Caltrans developed traffic inputs and axle load spectra from WIM data collected during 1991 to early 2001 on the State highway network (Lu and Harvey, 2006; Lu et al, 2002). This study also evaluated the possibility of extrapolating available truck traffic data to sites where WIM stations were not installed. Cluster analysis was applied to traffic data to extract influential factors and homogenous traffic patterns to ensure the preservation of useful information during the analysis.
    - Another Caltrans study focused on the truck traffic growth patterns, sensitivity of pavement responses to variation in growth rates, and potential contributing predictors that can be used to predict truck traffic growth rates (Lu et al, 2007). While both linear and compound growth functions can model growth trends, this study observed the linear growth function fitted the data slightly better. This study further recommended that six-year traffic observations should be used, as a minimum, for estimating growth rates to reduce variance in truck volume predictions and their significant effect on pavement design outcomes.
- Colorado DOT completed characterizing LTPP and non-LTPP traffic data for use in the MEPDG. This study used cluster analysis techniques for identifying similarities and dissimilarities among data sources. This study developed a catalog of traffic inputs for use in MEPDG (Mallela et al, 2010).
- Under the MEPDG implementation research project, Idaho developed site-specific and State wide traffic inputs using classification and weight data from 25 WIM sites in Idaho (Bayomy, 2011).
- Indiana DOT developed a Visual Basic program to process WIM data and estimate traffic inputs for the MEPDG.
- Louisiana is evaluating its current traffic characterization techniques for their compatibility with the MEPDG requirements. Louisiana intends to develop traffic load spectra from available truck traffic data sources, update its load equivalency factors, and make recommendations for its future implementation efforts (Contract Number: 736-99-1411).
- Michigan DOT developed a library of site-specific traffic inputs using data from 44 WIM stations (Buch et al, 2009). This study utilized cluster analysis to group sites with similar characteristics and subsequently discriminant analysis to develop regional traffic inputs. Data from all sites were averaged to establish the statewide level 3 inputs. The effects of the developed hierarchical traffic inputs on the predicted performance of rigid and flexible pavements were also investigated.
- Buchanan (2004) reviewed traffic data obtained from LTPP sites located within in the state and developed baseline data for internal use. As a follow on to the initial study, Mississippi DOT developed a software program to automate the processing of raw traffic data and prepare inputs for the MEPDG (Jiang and Saeed, 2007). This study provided additional support in the form of

technical documentation, user’s guide, on-site software installation, and training. Mississippi DOT is currently funding a project to establish procedures for quantifying the effects of changing traffic conditions on pavement performance and to enable traffic personnel in Mississippi to perform the traffic analysis for implementing and using the Pavement ME Design software. This project also involves a software upgrade component so that the traffic analysis tool is compatible with Pavement ME Design software and future Windows platforms (Contract Number: 257).

- Missouri DOT completed the quality analysis of WIM data and preparation of inputs for the MEPDG.
- North Carolina DOT developed a database of traffic data with level 1, 2 and 3 inputs (Stone et al, 2011). This study developed an implementation plan that identified the resources needed for traffic data collection, reviewed existing infrastructure and practices, and identified homogenous groups of traffic patterns for regional inputs and training. Preliminary findings from this study indicate multi-dimensional hierarchical clustering analysis and decision trees are applied in generating regional values of axle load and monthly adjustment factors (Sayyady et al, 2010). This study utilized an MS-Access based quality control procedure to review the WIM volume and weight data. A MEPDG-damage based sensitivity analysis was also conducted to identify sensitive traffic factors.
- Oregon DOT developed state-specific traffic inputs through a traffic characterization study (Elkins and Higgins, 2008). Four WIM stations representing high, moderate, and low average daily truck traffic (ADTT) volumes, were selected to characterize axle weight and spacing spectra on Oregon State highways.
- VTRC developed traffic inputs for Virginia DOT using traffic data from eight interstate and seven primary route WIM stations (Smith and Diefenderfer, 2010). This study evaluated the statistical significance of differences in predicted distresses for flexible and rigid pavements between site-specific and default traffic inputs. This study provided recommendations for using different hierarchical level of traffic inputs for different roadway functional classes. Currently, VTRC is reviewing VDOT’s plan to collect traffic and truck-axle weight data and propose revisions, if needed. The review will assess the data obtained from the Division of Motor Vehicle (DMV) sites and the appropriateness of the truck-weight groups and compile information on truck-travel patterns and characteristics (Contract Number: CSC 1118012-00092722-50012).
- FHWA through the LTPP program has also sponsored multiple studies to improve on the default NALS and other inputs required by the MEPDG procedure. The “best” WIM sites within the program were identified and selected to generate and recommend default axle load distributions to be used in design with the Pavement ME Design software. Recommended default distributions were developed but have yet to be incorporated into the Pavement ME Design software. Those default distributions are being considered for use by the Georgia DOT, as well as other agencies implementing the MEPDG. The limitation of these default distributions is that most of the “best” LTPP WIM stations are located on the interstate system with only a portion located on primary arterials.
- Default values for the truck wheel base were estimated through studies in Arizona, Colorado, Mississippi, Missouri, Utah, other agencies, and a national based LTPP study that was conducted

by Selenzneva, et al (2012). The national mean truck wheel base factors for trucks was determined or estimated from 25 WIM stations. The default values recommended for use are:

- Short = 17%;
- Medium = 22%;
- Long = 61%.

In addition, new axle load distributions for all types of axles and vehicle classes were established in the same study based on many years of WIM measurements (Selenzneva et al, 2012).

In summary, most studies have focused on building traffic input libraries and NALS. Other related efforts include: developing customized software programs to derive MEDPG inputs from WIM data, and evaluating the impact of using MEDPG defaults (input level 3) in place of agency-specific or site-specific (input level 1) traffic data. Some agencies (e.g. Arizona, California, Colorado, Michigan, and North Carolina), have utilized statistical techniques such as cluster analysis to identify homogenous groups of traffic patterns for developing regional traffic inputs. As noted above, Georgia has also sponsored traffic studies to determine the truck traffic inputs to the MEPDG. More importantly, LTPP has sponsored a study to default globally NALS defaults that can be used for design with the MEPDG.

5. INSTRUMENTATION AND SENSITIVITY ANALYSIS

5.1 Instrumentation & Accelerated Pavement Testing Studies

Several State agencies have funded pavement instrumentation studies or accelerated pavement testing studies to conduct full-scale investigation of pavement responses to climate, traffic and changes in material properties. Data collected from these instrumentation studies have been sponsored to assist in the implementation and local calibration process of the response models and MEPDG transfer functions. Some examples of these studies include:

- Kansas DOT constructed five new pavement sections and used four pavement test sections on the Kansas perpetual pavement project on US-75 as test sections to conduct field verification of Superpave mixtures. Both field and laboratory tests were conducted for material characterization (Gedafa et al, 2009). Kansas DOT has continued collecting pavement distress data even after the project was completed, and intends to utilize both material characterization and distress data for local calibration.
- In the North East, Maine DOT is funding a pavement instrumentation study to obtain in-place data necessary for adopting the MEPDG. The New Hampshire DOT is also constructing a fully instrumented pavement section to collect data for local calibration of the MEPDG.
- Oklahoma DOT is funding a similar study that focuses on monitoring and modeling of test sections along I-35 to facilitate collection of MEPDG related data in an accelerated manner for local calibration (Contract Number: SP&R 2200).
- Pennsylvania DOT sponsored a multi-year project called Superpave In-Situ Stress/Strain Investigation (SISSI) encompassing eight different instrumented full-depth HMA pavement sections located across the state (Solaimanian et al, 2006). This project focused on pavement instrumentation, response measurement to vehicle loading and environment, distress evaluation and data collection for the MEPDG transfer function validation. The data collected from these sites is also being used towards the calibration and validation of the MEPDG transfer functions and other models.

5.2 Sensitivity Analysis

State highway agencies and other organizations have conducted or are currently funding several studies to assess the relative sensitivity of the MEPDG performance predictions to various inputs. Some examples of these studies include:

- NCHRP project 1-47 completed extensive sensitivity studies of the MEPDG performance predictions to variability of input parameter values. Global sensitivity analyses were performed for five pavement types under five climate conditions and three traffic levels. Design inputs evaluated in the analyses included traffic volume, layer thicknesses, material properties (e.g., stiffness, strength, HMA and PCC mixture characteristics, subgrade type), groundwater depth, geometric parameters (e.g., lane width), and others. This study found that, for all pavement types and distresses, the sensitivities of the design inputs for the bound surface layers were consistently the highest. (Schwartz, 2011). Results from NCHRP project 1-47 are being used by multiple agencies within their implementation process to select sites for local calibration and in evaluating the residual error of the predicted distress values.

- Earlier, Schwartz (2007) conducted a study for Maryland SHA that compared flexible pavement designs and performance between the empirical 1993 AASHTO pavement design guide and the MEPDG, and performed a sensitivity analysis of various input parameters.
- For Arkansas SHTD, Hall et al (2006) analyzed the sensitivity of the MEPDG performance predictions to various design inputs.
- Kannekanti and Harvey (2006) developed a sample catalog of simple design tables (catalog) for rigid pavement design based on the Pavement ME Design software that are being used for design by Caltrans.
- For Georgia DOT, Watson et al (2009) compared the design results of the AASHTO 1972 Guide with the MEPDG distress predictions at different hierarchical input levels.
- For Michigan DOT, Buch et al (2008) conducted a sensitivity analysis of rigid and flexible pavement models. For rigid pavements, the results showed the effect of PCC slab thickness and edge support on performance were significant among design variables while CTE, flexural, base type and subgrade played an important role among material related properties. For flexible pavements, significant variables include HMA layer thickness, HMA mix characteristics, binder grade, base, subbase and subgrade moduli, and base and subbase thickness. Significant interactions were found among several of the variables in affecting all the performance measures.
- Ala et al (2009) conducted a parametric study for Nebraska DOR to identify the parameters that are important and level of sophistication that is needed at the input level.
- Nebraska DOR is currently funding a study that focuses on investigating the impact of heavy truck loading on damage of flexible pavements. This study intends to compare the MEPDG analysis results to the results from the purely mechanistic approach based on the finite element method (FEM).
- Won (2009) conducted a sensitivity analysis of the MEPDG punchout transfer function using project data of 27 continuously reinforced concrete pavement (CRCP) sections obtained from the Texas DOT Rigid Pavement Database.
- Freeman et al (2006) conducted a sensitivity analysis of the MEPDG for Texas DOT to estimate to what degree the input parameters affect the performance of the initial design.

6. LOCAL CALIBRATION AND DEPLOYMENT OF MEPDG

6.1 Validation and Calibration

All transfer functions and regression equations have an error. This error is often termed the standard error of the estimate (S_e) and can be used to establish confidence intervals for the transfer function. In other words, this error explains the scatter of the data around the 1:1 line (or line of equality) between the predicted and observed distress quantities. The standard error of the estimate of a transfer function is an important factor that must be understood and quantified in making a decision on whether to try and increase the precision of a transfer function.

The MPEDG software includes the global and local calibration factors for distress transfer functions and IRI regression equations. These calibration factors are used to make adjustments to the predicted values so that the difference between the measured and predicted values, defined as the residual error, is minimized. The global calibration coefficients included in the Pavement ME Design software were based on hundreds of pavement sections located throughout North America because of the consistency in the monitored data over time and the diversity of test sections. As noted in the Introduction to this document, however, it is impossible to account for all possible factors that affect pavement distress and performance. In fact, the coefficients of the distress specific transfer functions can be dependent on site factors, layer parameters, or more importantly, the operational and management policies of the agency. The developers of the MEPDG procedure recognized this reality and added features to account for local biases in pavement performance.

Table 3 lists the coefficients of the MEPDG transfer functions or distress and IRI prediction models that should be considered for revising the predictions to eliminate model bias for flexible pavements and HMA overlays, while Table 4 provides the same information but for the rigid pavement transfer functions. To further the implementation process and provide guidance on adjusting the global calibration coefficients or in deriving the local calibration coefficients, NCHRP sponsored work under NCHRP project 1-40B to prepare a document that provides guidance to agencies deciding to conduct local calibration. That document or local calibration guide was adopted and published by AASHTO (AASHTO, 2010). Figures 6 and 7 show the different steps suggested for calibrating and validating the MEPDG to local conditions, policies, and materials. A total of eleven steps are defined within the local calibration guide.

A few agencies have already completed a local calibration of the MEPDG transfer functions and regression equations (as an example; Arizona, Colorado, Indiana, Missouri, Montana, and Utah). Most agencies have derived the local calibration coefficients to remove any bias and improve on the transfer function's prediction accuracy (reduction of the standard error of the estimate for the transfer function). The implementation efforts in this area have generally focused on the following objectives:

- Identification of suitable pavement sections involving a wide range of pavement types for transfer function verification. State PMS sections and the LTPP sites located within the state or neighboring states have been selected for verification and local calibration efforts.

- Selection of proper MEPDG inputs for all test sections or roadway segments used in local calibration. This includes material and subgrade properties, design factors, local climatic conditions, and traffic loadings.
- Verification of the MEPDG transfer functions to estimate distress and IRI.
- Recalibration and validation of transfer functions to local conditions, if the globally-derived coefficients of the calibrated transfer functions are statistically biased or inaccurate for an individual agency.

Table 3. Calibration Parameters to be Adjusted for Eliminating Bias and Reducing the Standard Error of the Flexible Pavement Transfer Functions (after NCHRP Project 1-40B, 2008a)

Distress		Eliminate Bias	Reduce Standard Error
Total Rutting	Unbound Materials & HMA Layers	k_1, β_{s1}, or β_{r1}	k_2, k_3, and β_{r2}, β_{r3}
Load Related Cracking	Alligator Cracking	C_2 or k_1	k_2, k_3, and C_1
	Longitudinal Cracking	C_2 or k_1	k_2, k_3, and C_1
	Semi-Rigid Pavements	C_2 or β_{c1}	C_1, C_2, C_4
Non-Load Related Cracking	Transverse Cracking	β_{t3}	β_{t3}
IRI		C_4	C_1, C_2, C_3

Table 4. Calibration Parameters to be Adjusted for Eliminating Bias and Reducing the Standard Error of the Rigid Pavement Transfer Functions (after NCHRP Project 1-40B, 2008a)

Distress		Eliminate Bias	Reduce Standard Error
Faulting		C_1	C_2, C_3, C_4, C_5, C_6, C_7, C_8
Fatigue Cracking		C_1 or C_4	C_2, C_5
CRCP Punchouts	Fatigue	C_1	C_2
	Punchouts	C_3	C_4, C_5
	Crack Widths	C_6	C_6
IRI	JPCP	C_4	C_1
	CRCP	C_4	C_1, C_2

Multiple studies have been completed regarding the use of the MEPDG to confirm the accuracy and precision of the MEPDG transfer functions. The following summarizes some of the activities related to verification and calibration-validation that have been completed or are underway by some State agencies:

- Arizona DOT has been implementing the MEPDG since 2002 through extensive materials testing (Witczak, 2008) and traffic load characterization (Darter, et al 2013). A local calibration effort was conducted using LTPP and PMS data from throughout Arizona (Darter, 2012). The outcome of the local calibration effort resulted in new calibration coefficients for most of the distress transfer functions and IRI regression equations for both flexible and rigid pavement. The NCHRP 20-07 revised calibration coefficients for JPCP cracking, faulting, and IRI using the AASHTO T 336 test for CTE were also verified.

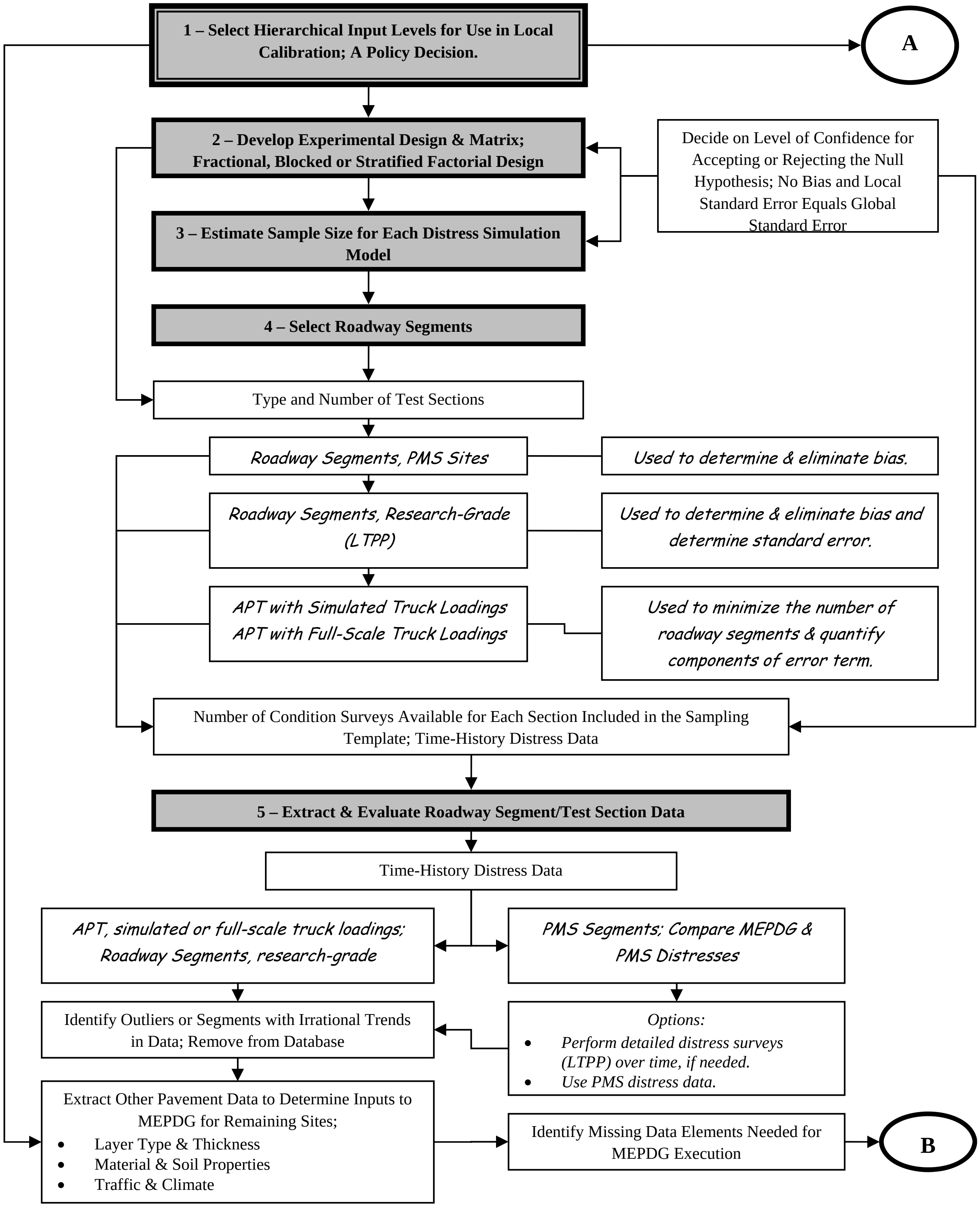


Figure 6. Flow Chart of the Procedure and Steps Suggested for Local Calibration; Steps 1 Through 5 (after NCHRP Project 1-40B, 2008a)

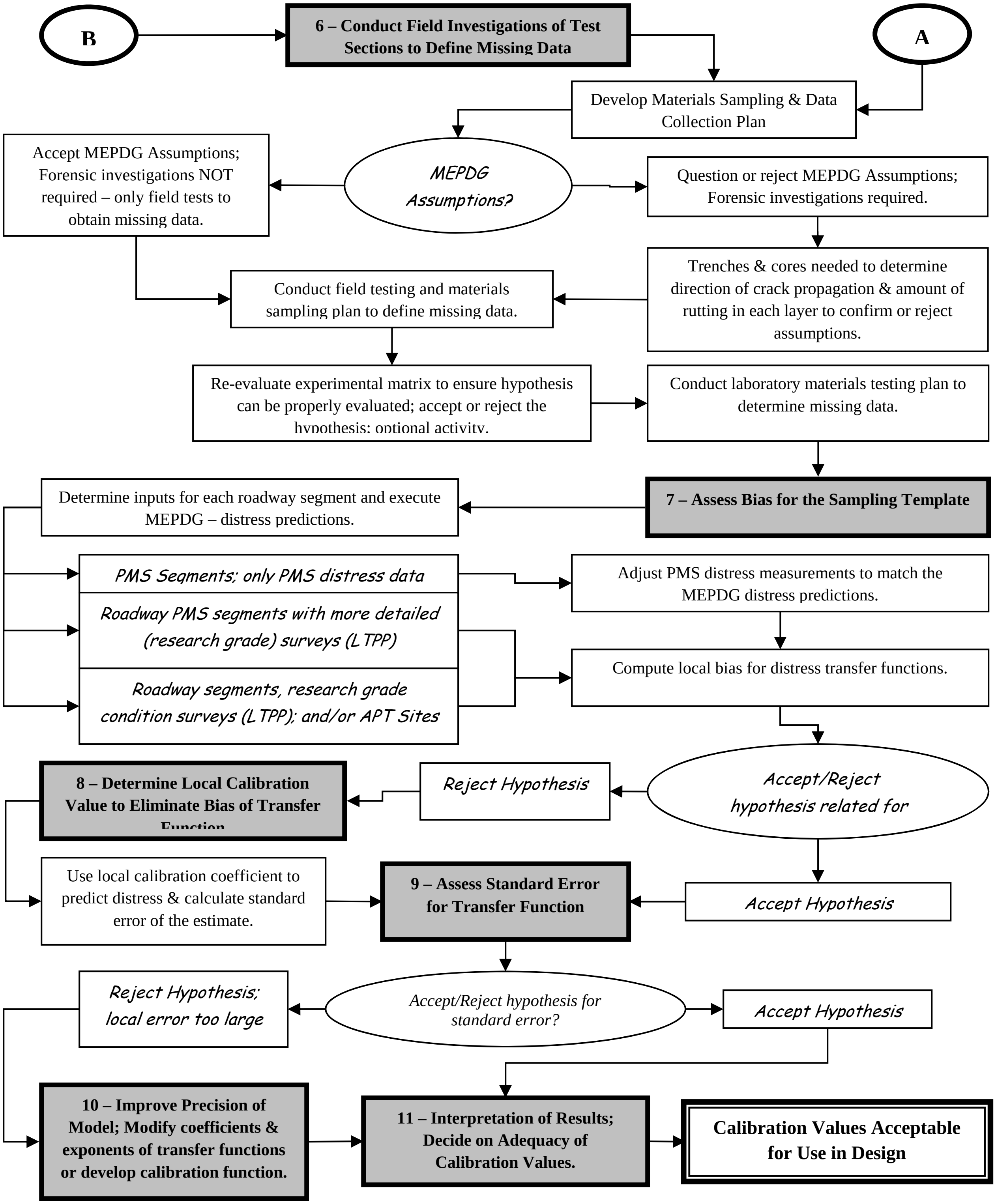


Figure 7. Flow Chart of the Procedure and Steps Suggested for Local Calibration; Steps 6 Through 11 (after NCHRP Project 1-40B, 2008a)

- Arkansas SHTD assessed the suitability of the globally calibrated MEPDG transfer functions for Arkansas conditions. Biased predictions were observed for most of the transfer functions, so a recalibration effort was sponsored. The outcome of the study was local calibration coefficients specific to Arkansas conditions, climates, and materials which provide more accurate distress predictions (Contract Number: TRC1003).
- Colorado DOT has completed their local calibration of the transfer functions for flexible and rigid pavements in pursuing implementation and use of the MEPDG.
- Florida DOT completed a cooperative effort to establish and characterize field test sections for the purpose of compiling a database of materials, geometric, and traffic-related design variables to verify the predictions from the MEPDG program and perform local model calibrations (Oh and Fernando, 2008). From these efforts, researchers and Florida DOT engineers established thickness tables for flexible and rigid pavement designs.
- Idaho ITD issued a contract to the University of Idaho to start their local calibration process.
- The Indiana DOT has implemented the MEPDG and has used the procedure on many projects over the past 4 years. Materials testing and traffic analysis was conducted. Some local verification was also accomplished (Nantung, et at, 2010).
- Through a research contract with Iowa State University (ISU), the Iowa DOT is performing a local calibration study of the MEPDG transfer functions using the Pavement Management Information System (PMIS) data (RiP Project 28337). An earlier study evaluated Iowa's PMIS data retrieved for interstate and primary roads for completeness and definition compatibility with respect to the MEPDG input requirements and outputs. Recommendations were made to update the existing PMIS and add new parameters that are currently unavailable, but are required for MEPDG rehabilitation design (Ceylan at al, 2009).
- Kansas DOT is currently conducting local calibration of the MEPDG transfer functions through a research contract with University of Kansas (Contract Number: RE-0610-01).
- The Kentucky Transportation Cabinet is currently funding a project to evaluate and recalibrate the MEPDG transfer functions for local conditions (Contract Number: P10-396). This study also intends to develop a detailed implementation plan for integration of the MEPDG into the current design process.
- The Midwest Regional University Transportation Center is developing a regional pavement performance database for use in the validation and calibration process of the MEPDG models. This research effort is collecting data from multiple states including: Illinois, Indiana, Iowa, Michigan, Minnesota, Ohio, and Wisconsin. It is expected that the regional database will result in calibration coefficients for more accurate predictions of distress within the Midwest region (Contract Number: 07-01).
- Michigan DOT has a research contract with Michigan State University for the preparation of MEPDG implementation in Michigan. This project includes the HMA characterization (Contract Number: RC-1593), evaluation of rehabilitation designs (Contract Number: RC-1594), and the local calibration and validation of performance models (Contract Number: RC-1595). This project is scheduled to be completed in 2014. Michigan DOT is expecting to begin the final transition to using the Pavement ME Design software upon the completion of this project.
- Minnesota DOT and the Local Road Research Board (LRRB) initiated a study to evaluate the MEPDG transfer functions and appropriateness of the global calibration factors to conditions encountered in Minnesota (Velasquez et al, 2007). This study evaluated the use of default inputs,

identified software deficiencies in version 1.003, and analyzed the results with local conditions for model recalibration. The Minnesota DOT has also conducted a comprehensive evaluation of the MEPDG distress predictions (Yut et al, 2007). The cracking transfer functions were re-calibrated using the design and performance data collected for 65 pavement sections located in Minnesota, Iowa, Wisconsin, and Illinois. A prototype of the catalog of recommended design features for Minnesota low volume PCC pavements was developed using the MEPDG version 0.910. The catalog offers a variety of feasible design alternatives (PCC and base thickness, joint spacing and PCC slab width, edge support type, and dowel diameter) for a given combination of site conditions (traffic, location, and subgrade type).

- The Mississippi DOT conducted a large study for implementing the MEPDG. This study included extensive materials testing (HMA, PCC, unbound), traffic analyses, and expanding the climate database, as well as an initial local calibration effort. This effort was completed in 2013 but an expanded study using results from field or forensic investigations of the non-LTPP segments is underway and will be completed in 2015.
- The Missouri DOT was one of the first agencies to begin a major implementation effort of the MEPDG. Extensive materials testing (HMA, PCC, unbound) and traffic characterization have been accomplished. Local calibration of all flexible and rigid pavement models was then performed and improved model predictions obtained (Mallela, et al, 2009). The calibrated MEPDG has been used on many major projects including design/build projects. Missouri DOT is also planning an expanded local calibration study that will start in 2015.
- Montana DOT was the first agency to develop performance criteria (e.g., ride quality, rutting, fatigue cracking, transverse cracking) of flexible pavements, and used these characteristics in the verification and calibration of the distress prediction models included in the Pavement ME Design software using version 0.9000 (Von Quintus and Moulthrop, 2007). The work conducted within this study included using the Pavement ME Design software to develop local calibration factors in the use of that software for Montana climate, structures, and materials for flexible pavements. This study developed a reference manual that included selection of distress prediction models, traffic characterization and analyses, and a database for calibration of distress prediction models. In addition, a calibration and user's Guide for making future refinements to either the regional or local MDT calibration factors was prepared.
- Through a research contract with North Carolina State University, North Carolina DOT has developed local calibration factors for MEPDG flexible pavement designs (Kim et al, 2011). Earlier, it developed an implementation plan containing detailed recommendations for the steps necessary for the local calibration and validation of the MEPDG procedures.
- Ohio DOT has developed a comprehensive roadmap for the MEPDG implementation that included an assessment of Ohio's needs for the implementation effort, established default values (means and ranges) for those inputs that have adequate data from previous research, and validated nationally calibrated distress and smoothness prediction model using readily available state-specific pavement section data (Mallela et al 2009).
- The Oregon DOT is in the process of implementing the MEPDG for new pavement sections. Forty-four pavement sections throughout Oregon were included for calibration. Oregon initially used some of those sites to verify the global calibration coefficients. Results from the verification study found the MEPDG predicted distresses were significantly different than the measured distresses. Thus, a local calibration of the transfer functions was initiated. Four distress prediction

models (rutting, alligator, longitudinal, and thermal cracking) of the HMA overlays were calibrated for Oregon conditions (Williams and Shaidur).

- Virginia DOT is currently performing local calibration of MEPDG performance models. Virginia started with training its district personnel on the use of the MEPDG. It has been using the MEPDG as a shadow design procedure to AASHTO 1993 until full implementation of MEPDG is completed (Elfino, 2012).
- Vermont DOT is currently pursuing an in-house study evaluating the MEPDG rutting and IRI predictions with observed field data for possible recalibration (Contract / Grant Number: SPR 711).
- Wisconsin DOT initiated two implementation projects. The first generation implementation project included LTPP sections only. Under this effort, Wisconsin developed default data libraries for HMA, concrete, unbound materials and subgrade and traffic. Verification and local calibration of MEPDG performance models using limited LTPP sections were conducted as the next step. Wisconsin DOT then initiated a second MEPDG implementation project in 2009 based on review of a much wider dataset of statewide projects to generate a more robust set of local calibration factors (Wisconsin DOT, 2010).
- Li et al (2009) presented the Washington State DOT's latest efforts on calibrating the flexible and rigid pavement portion of the MEPDG with data obtained from the Washington State Pavement Management System.
- Utah DOT has completed a major local calibration effort to enable the agency to implement the MEPDG in routine or day-to-day design practice (Darter et al, 2009). In this study, the global calibration coefficients were evaluated and found to result in biased predictions. LTPP and other Utah DOT pavement management segments were used to derive the local calibration coefficients appropriate to Utah's local conditions. This study also suggested modifications to some Utah DOT standard procedures and pavement design protocols such as lab testing procedures, equipment, traffic data reporting, software issues and design output interpretation. The NCHRP project 20-07 calibration coefficients for JPCP cracking, faulting, and IRI using the new AASHTO T 336 test for CTE were verified. A comprehensive User's Guide was prepared (Darter, et al, 2009).
- Wyoming DOT is currently pursuing the implementation of the MEPDG. The University of Wyoming is collecting data to be used in their local calibration effort which should be completed in 2014. An outcome from this study includes a design manual/user's guide of recommended procedures for the agency use (Contract Number: RS03(209)).

In summary, Tables 5 and 6 present the national and local calibration coefficients developed by various States for flexible and rigid pavements, respectively. The remainder of this section provides a discussion and comparison of results from the studies completed to date. Results from various calibration studies can be used by agencies just getting started for use in setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions and regression equations.

Table 5. Local Calibration Factors for Flexible Pavements

Distress	Coefficient	National	MO	UT	AZ	CO**	WY**	WI	OH	OR	WA	Midwest	NC
AC Fatigue	Bf1	1	1	1	249.0087	130.3674	1	1	1	1	-3.3	1	
	Bf2	1	1	1	1	1	1	1	1	1	-40	1.2	
	Bf3	1	1	1	1.2334	1.217799	1	1	1	1	20	1.5	
AC Bottom Up Cracking	c1	1	1	1	1	0.07	0.4951	1	1	0.56	1		0.4372
	c2	1	1	1	4.5	2.35	1.469	1	1	0.225	0		0.1505
	c3	6000	6000	6000	6000	6000	6000	6000	6000	6000	~0		
AC Rutting	Br1	1	1.07	0.56	0.69	1.34	1.0896	1.0157	0.51	1.48	0.6		1.0175
	Br2	1	1	1	1	1	1	1	1	1	20.6		1
	Br3	1	1	1	1	1	1	1	1	0.9	8.9		1
Base Rutting; Coarse-Grained Materials/Soils	Bs1	1	0.01	0.604	0.14	0.4	0.9475	0.01	0.32	0			0.7785
Subgrade Rutting; Fine-Grained Materials/Soils	Bs1	1	0.4375	0.4	0.37	0.84	0.6897	0.5731	0.33	0			0.6616
Thermal Fracture	Level 1 K	1.5	0.625	1.5	1.5	7.5	7.5	0.625	1.5				
	Level 2 K	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5				
	Level 3 K	1.5	1.5	1.5	1.5	1.5	7.5	0.3	1.5	10			
IRI	C1 (Rutting)	40	17.7	40	1.2281	35	20.53	18.71	17.6				
	C2 (Fatigue)	0.4	0.975	0.4	0.1175	0.3	0.4094	0.04	1.37				
	C3 (Thermal)	0.008	0.008	0.008	0.008	0.02	0.00179	0.085	0.01				
	C4 (Site Factor)	0.015	0.01	0.015	0.028	0.019	0.015	0.0197	0.066				
Reflection Cracking (AC over AC only)	C	1	1	1	2.55	2.5489	0.75	1	1				
	D	1	1	1	1.23	1.2341	2.2	1	1				

Notes:

** Local calibration coefficients for Colorado and Wyoming are not final estimates and are subject to revisions.

Table 6. Local Calibration Factors for Rigid Pavements

Distress	Coefficients	National	NCHRP 20-07 Task 288[+]	MO[&]	UT[&]	AZ[+]	CO[*+]	WY[*+]	WI[&]	OH[&]
JPCP Transverse or Fatigue Cracking	C1	2	2	2	2	2	2	2	2	2
	C2	1.22	1.22	1.22	1.22	1.22	1.22	1.22	1.22	1.22
	C4	1	0.6	1	0.6	0.19	1	0.6	1	1
	C5	-1.98	-2.05	-1.98	-2.05	-2.067	-1.98	-2.05	-1.98	-1.98
Joint Faulting	C1	1.0184	0.5104	1.0184	1.0184	0.0355	1.0184	0.5104	1.15	1.0184
	C2	0.91656	0.00838	0.91656	0.91656	0.1147	0.91656	0.00838	0.91656	0.91656
	C3	0.0021848	0.00147	0.002185	0.002185	0.00436	0.002185	0.00147	0.004	0.002185
	C4	0.000883739	0.008345	0.000884	0.000884	1.10E-07	0.000884	0.08345	0.000884	0.000884
	C5	250	5999	250	250	20000	250	5999	250	250
	C6	0.4	0.8404	0.4	0.4	2.0389	0.4	0.504	0.4	0.4
	C7	1.83312	5.9293	1.83312	1.83312	0.189	1.83312	5.9293	1.83312	1.83312
	C8	400	400	400	400	400	400	400	400	400
IRI	C1 (Cracks)	0.8203	0.8203	0.82	0.8203	0.6	0.8203	1.7	4.0567	0.82
	C2 (Spall)	0.4417	0.4417	1.17	0.4417	3.48	0.4417	1.32	1.6275	3.7
	C3 (Fault)	1.4929	1.4929	1.43	1.4929	1.22	1.4929	1.8	0.7236	1.711
	C4 (Site Factor)	25.24	25.24	66.8	25.24	45.2	25.24	35	45.2388	5.703

Notes:

[*] Local calibration coefficients for Colorado and Wyoming are not final estimates and may subject to revisions.

[+] National calibration coefficients were adjusted to correct for previous errors in the measurement of CTE under the NCHRP project 20-07 Task 288. Proper CTE input is that measured by AASHTO T336. The NCHRP 20-07 derived calibration coefficients using correct CTE values were found to be adequate for Arizona, Colorado, Utah, and Wyoming.

[&] National calibration coefficients presented in the AASHTO software (not corrected for errors in CTE measurements) were used in the local calibration for Missouri, Ohio and Wisconsin. Utah was originally calibrated using the National coefficients but later the NCHRP 20-07 coefficients were found to be unbiased and are now used as CTE inputs from the AASHTO T336 test can be used directly.

6.2 Comparison of Local Calibration Coefficients of Transfer Functions

Rut Depth Transfer Function

The MEPDG has been found to over predict the rutting in the unbound layers in several States. The over prediction of rut depths in the unbound layers was confirmed for projects where forensic investigations were conducted to measure the rutting in the unbound and HMA layers. In fact, the rutting predicted in the subgrade was greater than the measured rutting at the surface for more than just a few of the test sections. As such, local adjustment or calibration values were determined for the unbound layers and are summarized in Table 7 for the different projects. These values were determined by limiting the rutting in the unbound layers to the values reported from the forensic investigations. For the most part, Table 7 for the fine and coarse-grained materials/soils are within the same range of values summarized in Table 5.

Table 7. Summary of Local Calibration Values for the Rut Depth Transfer Function

<table>
<tr><th rowspan="2" colspan="2">Project Identification</th><th colspan="2">Unbound Materials/Soils, β_{s1}</th><th colspan="3">HMA Calibration Values</th></tr>
<tr><th>Fine-Grained</th><th>Coarse-Grained</th><th>β_{r1}</th><th>β_{r3}</th><th>β_{r2}</th></tr>
<tr><td rowspan="2" colspan="2">NCHRP Projects 9-30 & 1-40B; Verification Studies, Version 0.900 of the MEPDG.</td><td>0.30</td><td>0.30</td><td colspan="3">Values dependent on volumetric properties of HMA; the values below represent the overall range.</td></tr>
<tr><td colspan="2">Insufficient information to determine effect of varying soil types.</td><td>6.9 to 10.8</td><td>0.65 to 0.90</td><td>0.90 to 1.10</td></tr>
<tr><td rowspan="2" colspan="2">Montana DOT; Based on version 0.900 of the MEPDG</td><td rowspan="2">0.30</td><td rowspan="2">0.30</td><td colspan="3">Values dependent on the volumetric properties of HMA; the values below represent overall averages.</td></tr>
<tr><td>7.0</td><td>0.70</td><td>1.13</td></tr>
<tr><td colspan="2">Kansas DOT; PM Segments; HMA Overlay Projects; All Mixtures (Version 1.0)</td><td>0.50</td><td>0.50</td><td>1.5</td><td>0.95</td><td>1.00</td></tr>
<tr><td rowspan="3">Kansas PM Segments; New Construction</td><td>Convent.</td><td rowspan="3">0.50</td><td rowspan="3">0.50</td><td>1.5</td><td>0.90</td><td>1.00</td></tr>
<tr><td>Superpave</td><td>1.5</td><td>1.20</td><td>1.00</td></tr>
<tr><td>PMA</td><td>2.5</td><td>1.15</td><td>1.00</td></tr>
<tr><td rowspan="2" colspan="2">LTPP SPS-1 & SPS-5 Projects built in accordance with specification; conventional HMA mixtures (Version 1.0).</td><td rowspan="2">0.50</td><td rowspan="2">0.50</td><td colspan="2">Value dependent on the air void & asphalt content</td><td>1.00</td></tr>
<tr><td>1.25 to 1.60</td><td>0.90 to 1.15</td><td>1.00</td></tr>
<tr><td rowspan="2" colspan="2">LTPP SPS-1 Projects with anomalies or construction difficulties, unbound layers.</td><td colspan="2">Values dependent on density and moisture content; values below represent the range found.</td><td rowspan="2">---</td><td rowspan="2">---</td><td rowspan="2">---</td></tr>
<tr><td>0.50 to 1.25</td><td>0.50 to 3.0</td></tr>
<tr><td colspan="2">NCHRP Project 9-30A; based on extensive laboratory tests; Mixture Specific</td><td>0.3 to 0.75</td><td>0.3 to 0.75</td><td>1.0 to 20.0</td><td>0.5 to 1.0</td><td>1.0</td></tr>
</table>

Most of the unbound material local calibration values are less than 1.0 (the global value), with the exception for the condition where construction anomalies occurred; high water contents and low densities (refer to Table 7). The unbound material local calibration values for the Montana and northwest sections (located in states adjacent to Montana) were found to be lower than the mid-west sections—probably because most of these have heavier truck traffic and thicker HMA layers. The unbound layers in the northwest sites were also found to have lower water contents in the subgrade soils over time.

HMA mixture specific factors, documented under NCHRP project 1-40B, were used to modify or adjust the MEPDG global calibration factors where sufficient data was available. These projects included all test sections located in Montana and selected SPS-1 and SPS-5 projects. Mixture specific calibration parameters were not used with the Kansas pavement management segments for demonstrating use of the local calibration guide, because insufficient mixture data were unavailable for those segments. As summarized in Table 7, a significant difference exists between the local calibration values for the Montana and Kansas examples, especially for β_{r1} and β_{r3}. The values for the local calibration coefficient for the exponent of the number of load repetitions terms, B_{r3}, summarized in Table 7 are slightly different than those values summarized in Table 5. The similarity between the two tables, however, is that all studies to date found non-unity for at least one of the local calibration coefficients for the HMA rutting transfer function.

Table 8 summarizes the diagnostic statistics (bias and standard error) for each of the facilities and types of experiments that were used in NCHRP projects 9-30 and 1-40B, while Figure 8 shows a comparison of the predicted and measured rut depths for all data sets and projects. No systematic difference in the standard error, bias, and other statistics was found between the different experiments. This suggests that the MEPDG rut depth transfer function and model adequately account for many different factors; including HMA volumetric properties, HMA layer thickness, truck loading condition, and climate. Specifically, the MEPDG rut depth prediction model with the NCHRP 1-40B mixture adjustments adequately accounted for differences between the asphalt grades and HMA mixtures – fine to coarse graded aggregate blends from well compacted to poorly compacted mixtures. These data include results from different wheel loads, test temperatures, and mixtures.

More importantly, previous studies have found significant differences between the test results from APT simulated truck traffic, APT full-scale truck traffic, and actual roadway sections with mixed truck traffic. A question that has been continually raised by industry is how to combine the results from different APT experiments – simulated truck loads to full-scale truck loads. These significant differences have been adequately accounted for or normalized through the MEPDG rut depth prediction model.

In summary, the prediction model is believed to be a reasonable simulation of the experiments and is reasonably accurate in predicting rutting over a diverse range of site conditions, wheel or truck loads, design features, volumetric properties, and HMA mixtures. Using local calibration values that are dependent on the HMA volumetric properties provide a closer estimate to the measured values. Thus, it is concluded that the majority of the error (difference between the measured and predicted HMA rut depths) is a result of distress measurement error and not a lack-of-fit modeling error.

Table 8. Summary of the Bias and Standard Error for the Rut Depth Transfer Function from Independent Data Sets

Facility/Project Identification		No. of Points	Bias, in.	Standard Error, in.	R^2 Term	s_e/s_y
Global Calibration Statistics		*334*		*0.107*	*0.577*	*0.818*
FHWA ALF – Simulated Loading APT		28	+0.053	0.1830	0.769	0.57
Florida ALF – Simulated Loading APT		40	+0.085	0.0945	0.524	0.58
GTI ALF – Simulated Loading APT		8	+0.146	0.1643	Poor	1.99
NCAT Test Track; Full Scale APT, Round 1		28	-0.001	0.0377	0.857	0.41
NCAT Test Track; Full Scale APT, Round 2		24	+0.062	0.0434	0.978	0.41
WesTrack; Full Scale APT		76	+0.142	0.0844	0.900	0.34
MnRoads Test Facility – Mixed Traffic		60	+0.038	0.0652	0.791	0.42
SPS-1 Projects; General; Mixed Traffic		108	-0.0178	0.1339	0.673	0.61
HMA Overlay Experiments; Mixed Traffic		46	+0.062	0.0426	0.673	0.31
Montana DOT	New Construction, Conventional	67	+0.0069	0.0536	0.888	0.342
	New Construction, Semi-Rigid	18	-0.0103	0.0457	0.664	0.662
	HMA Overlays	50	+0.0126	0.0520	0.873	0.359
Northwest, Adjacent to Montana; Project Sites	Conventional	72	+0.0108	0.0539	0.763	0.418
	Semi-Rigid	32	-0.0023	0.0472	0.866	0.384
	HMA Overlays	75	+0.0059	0.0501	0.704	0.499
Kansas DOT	PM Segments; Full-Depth Projects	77	+0.0249	0.00397	0.650	0.522
	PM Segments; HMA Overlays	35	+0.0278	0.0725	Poor	0.841
Mid-West; Project Sites	LTPP SPS-1 Sites	122	+0.028	0.134	0.676	0.640
	LTPP SPS-5 Sites	158	-0.031	0.0642	0.357	0.768

Fatigue Cracking Transfer Function for Flexible Pavements and HMA Overlays – Alligator Cracking or Bottom-Up Fatigue Cracking

The HMA mixture adjustment factors documented under NCHRP Project 1-40B were used to modify the MEPDG global calibration parameters where sufficient data was available. The projects where the mixture adjustment factors were and were not used was the same as for rutting. Table 9 lists the overall average values or range of values that were determined for the different data sets. As summarized, the local calibration values between the Montana and Kansas sites are different. The β_{f1} values for the Kansas sites are a lot lower. Potential reasons for the lower values from the Kansas sites could be: the types of cracks exhibited between the two projects are different (the reported area fatigue cracking for the Kansas site initiated at the surface), the higher coefficient for the Montana sites is a result of pavement preservation activities, or a combination of both of these reasons.

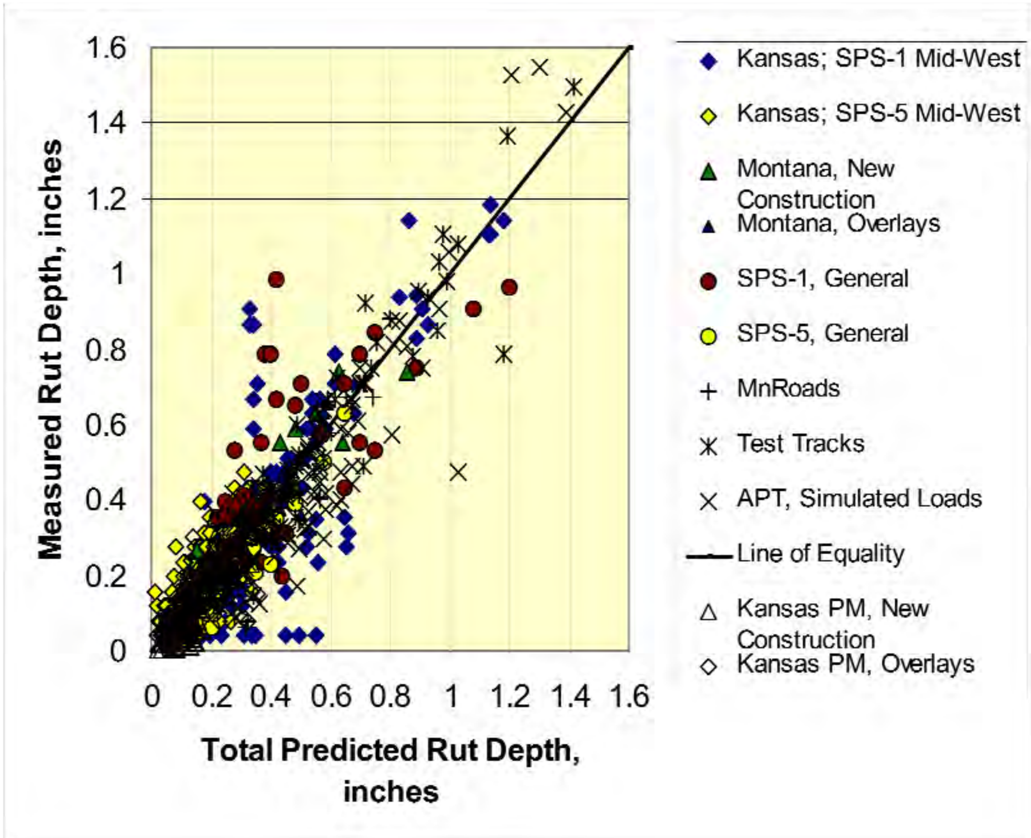


Figure 8. Comparison of Measured and Predicted Rut Depths for Different Data Sources

Unlike for rutting, the local calibration coefficients for fatigue cracking are highly variable between the different studies summarized in Tables 5 and 9, with the largest variability occurring for the fatigue cracking coefficient of B_{f1}.

Table 10 summarizes the diagnostic statistics (bias, standard error, and RMSE) for each of the facilities and types of experiments for area fatigue cracking. Figure 9 shows a comparison of the predicted and measured fatigue cracking for all data sets from those sections with fatigue cracking—assumed to be bottom-up cracking. In summary, it is believed that the area fatigue cracking transfer function provides a reasonable estimate of fatigue cracking.

It should be noted that the test sections with longitudinal cracking in the wheel paths measured for the MnRoads and FHWA APT experiments are included in

Table 10. The length of longitudinal cracking was converted to an area basis – assuming that a longitudinal crack affects the mixture response within 6 inches either side of the crack. As summarized, the correlation between the predicted and measured values is very poor.

The experiments or facilities with the greatest s_e/s_y and greater bias are those with longitudinal cracking. This suggests that the area fatigue cracking prediction model for bottom-up cracking does not accurately predict the occurrence of these longitudinal cracks that initiate at the surface of the HMA layers. Even excluding those sites, with longitudinal cracking, the fatigue cracking transfer function has much less precision for the Kansas sections (including both the pavement management segments and SPS-1 and SPS-5 projects), than for the Montana and northwest project sites. The major difference between these two data sets is that many of the Kansas and mid-west sites have low levels of fatigue cracking and many of those sites with higher levels of fatigue cracking have very high asphalt viscosities. This observation suggests a bias in the HMA mixture properties between the two data sets, as well as a difference between pavement preservation strategies.

Table 9. Summary of Local Calibration Values for the Area Fatigue Cracking Transfer Function

<table>
<tr><th colspan="2">Project Identification</th><th>β_{f1}</th><th>β_{f2}</th><th>β_{f3}</th><th>C_2</th></tr>
<tr><td colspan="2" rowspan="2">NCHRP Projects 9-30 & 1-40B; Verification Studies, Version 0.900 of the MEPDG</td><td colspan="4">Values dependent on the volumetric properties.</td></tr>
<tr><td>0.75 to 10.0</td><td>1.00</td><td>0.70 to 1.35</td><td>1.0 to 3.0</td></tr>
<tr><td colspan="2" rowspan="2">Montana DOT; Based on version 0.900 of the MEPDG, with pavement preservation treatments</td><td colspan="4">Values dependent on the volumetric properties.</td></tr>
<tr><td>13.21</td><td>1.00</td><td>1.25</td><td>1.00</td></tr>
<tr><td colspan="2" rowspan="2">Northwest Sites; Located in States Adjacent to Montana, without pavement preservation treatments</td><td colspan="4">Values dependent on the volumetric properties.</td></tr>
<tr><td>1.0 to 5.0</td><td>1.00</td><td>1.00</td><td>1.0 to 3.0</td></tr>
<tr><td colspan="2">Kansas DOT; PM Segments; HMA Overlay Projects; All HMA Mixtures</td><td>0.05</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td rowspan="3">Kansas DOT; PM Segments; New Construction</td><td>Conventional HMA Mixes</td><td>0.05</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td>PMA</td><td>0.005</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td>Superpave</td><td>0.0005</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td rowspan="3">Mid-West Sites</td><td>LTPP SPS-1 Projects built in accordance with specifications</td><td>0.005</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td>LTPP SPS-1 Projects with anomalies or production difficulties</td><td>1.00</td><td>1.00</td><td>1.00</td><td>1.0 to 4.0</td></tr>
<tr><td>LTPP SPS-5 Projects; Debonding between HMA Overlay and Existing Surface</td><td>0.005</td><td>1.00</td><td>1.00</td><td>1.0 to 4.0</td></tr>
</table>

The MEPDG fatigue cracking transfer function was found to be a reasonable estimate of the measured magnitudes over a diverse range of mixtures and structures. The standard error for the area fatigue cracking prediction model was found to be relatively large (less precision) but reasonable for a distress that exhibits high variability measurements. These errors consist of both measurement and lack-of-fit modeling errors. It is believed that the measurement errors are the greater of the two. The lack-of-fit error can be explained by a potential loss of bond between the HMA overlay and existing surface and/or production difficulties identified in the construction reports where the asphalt was severely hardened during production.

The Arizona fatigue cracking local calibration study was conducted using substantial data from the State PMS database, research sections, and LTPP. The results produced a significantly different calibration relationship than the global based relationship: C2 equals unity from the global calibration while the Arizona C2 value was found to be 4.5 from their local calibration study (refer to Table 5).

Table 10. Summary of the Bias and Standard Error for the Area Fatigue Cracking Transfer Function from Independent Data Sets

Facility/Project Identification		**No. of Points**	**Bias, %**	**Standard Error, %**	**R^2 Term**	s_e/s_y
Global Calibration Statistics		*405*	---	*5.01*	*0.275*	*0.815*
FHWA APT, Simulated Truck Loading		28	+5.85	8.30	Poor	1.42
WesTrack APT, Full Scale Truck Loading		58	+0.70	9.40	0.893	0.35
NCAT, Full Scale Truck Loading		24	-1.96	4.726	0.998	0.338
MnRoads, Mixed Truck Traffic		60	+2.20	4.75	Poor	3.10
Roadway Sections, Mixed Truck Traffic		100	-0.98	6.938	0.999	0.53
Montana DOT	New Construction	58	+1.11	2.34	0.573	0.401
	Semi-Rigid Pavements	16	0.00	0.000	---	---
	HMA Overlays	50	-0.02	8.17	0.913	0.318
Northwest Sites, Adjacent to Montana	New Construction	76	+0.15	2.45	0.900	0.315
	Semi-Rigid Pavements	51	+0.51	1.51	0.234	0.532
	HMA Overlays	76	+0.67	7.67	0.877	0.318
Kansas DOT	PM Segments; Full-Depth Projects	177	+0.383	2.154	0.322	1.399
	PM Segments; HMA Overlay Projects	35	+1.272	1.441	Poor	0.806
Mid-West Project Sites	LTPP SPS-1 Sections	122	+1.363	1.433	0.360	0.885
	LTPP SPS-5 Sections	158	-1.150	4.900	0.683	0.890

Fatigue Cracking Transfer Function for Semi-Rigid Pavements

Sites were selected in Montana and adjacent States to calibrate the semi-rigid transfer function for fatigue cracking. Unfortunately most of these sites exhibited little load related cracking, as reported in the LTPP database. Only one of the Montana sections had any fatigue cracking recorded on the semi-rigid pavement structures.

The MEPDG-Version 0.900 was used to predict the fatigue cracking of this pavement design strategy by varying the local calibration parameters. These local calibration coefficients were found to be mixture quality dependent, as expected. Based on the data available for regional calibration refinement, the following are the local calibration values for use in predicting the fatigue cracking of semi-rigid pavements (based on version 0.900 of the MEPDG).

- For High Strength CAM Mixtures (intact cores can be recovered and mixture has cement contents greater than 6 percent, with compressive strengths generally greater than 1,000 psi):
    - $B_{c1} = 0.85$.
    - $B_{c2} = 1.10$.
- For CAM Mixtures with moderate levels of cement (intact cores can be recovered and mixture has cement contents greater than 4 percent but less than 6 percent, with compressive strengths generally greater than 300 psi but less than 1,000 psi):
    - $B_{c1} = 0.75$.
    - $B_{c2} = 1.10$.
- For Low Strength CAM Mixtures (intact cores cannot be recovered and mixture has cement contents generally less than 4 percent, with compressive strengths less than 300 psi):
    - $B_{c1} = 0.65$.
    - $B_{c2} = 1.10$.

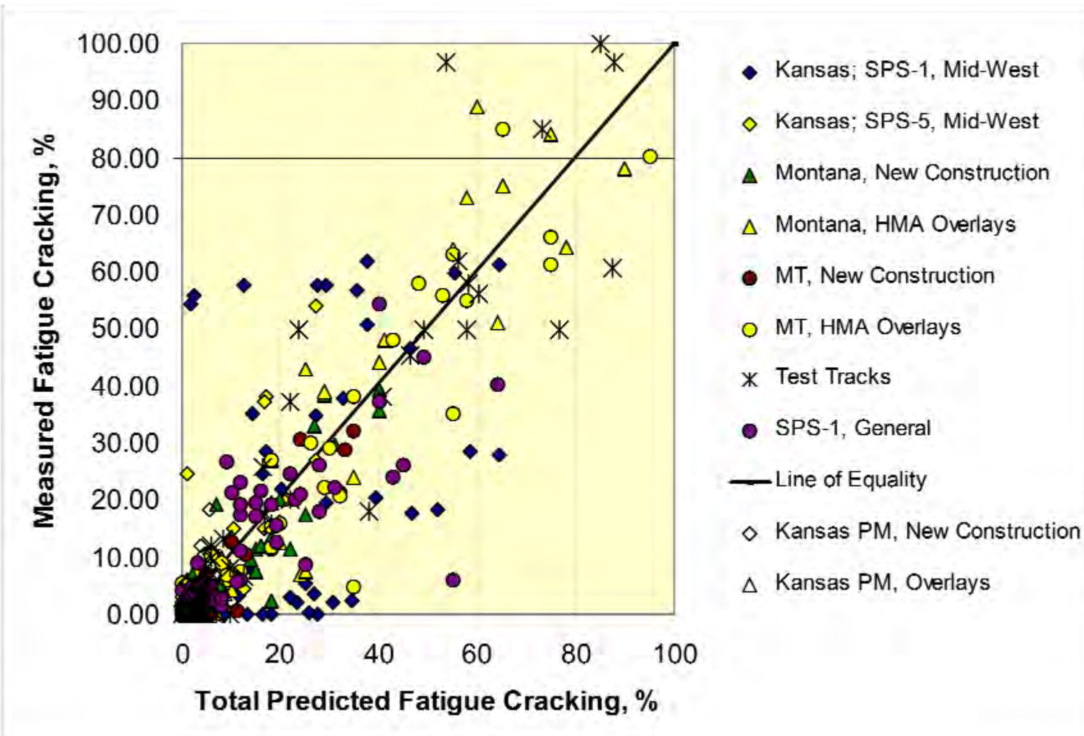


Figure 9. Comparison of Measured and Predicted Fatigue Cracking for all Fatigue Cracking Data Combined, Excluding the Sections with Longitudinal Cracking

Fatigue Cracking Transfer Function for Longitudinal Cracking or Top-Down Cracking

Longitudinal cracking in the wheel paths were calculated for the different test sections and found to be inaccurate for test sections located in Montana and Kansas, as well as those built in adjacent States. The longitudinal cracking predicted for the test sections was found to be significantly greater for some test sections and significantly lower for others. In fact, significant lengths of longitudinal cracks were predicted for those sections that have yet to exhibit this type of cracking.

No consistent trend in the predictions could be identified to reduce the bias (improvement in accuracy) of this transfer function. It is believed that there is a significant lack-of-fit modeling error for the occurrence of longitudinal cracks, because the bias (residual error) is considered too large for use in structural design. As such, the top-down fatigue cracking model is not recommended for use in making design decisions until it is further refined based on work completed under NCHRP project 1-42A.

In summary, no consistent result could be identified to adjust the local calibration factors to improve on the accuracy of the transfer function. It is believed that the transfer function is not using the critical response parameter and material properties that affect the occurrence of cracks that initiate at the HMA surface. This difference is considered a lack-of-fit modeling error. It is believed by the author that more than just some of this type of cracking reported or documented in the LTPP database and in other experiments is a result of construction deficiencies (such as inadequate bond between HMA layers and longitudinal segregation), rather than any HMA mixture property.

Transverse (Low Temperature) Cracking Transfer Function

The MEPDG transverse cracking transfer function was used to calculate the length of thermal cracks for all test sections. In general, the MEPDG over-predicted the length of transverse cracks for all test sections where pavement preservation treatments had been used, even for those sections where indirect tensile strength and creep compliance tests were performed on the HMA. An average local calibration factor of 0.25 was determined from the test sections located in Montana, with pavement preservation treatments applied within a short time period after construction. Conversely, it significantly under predicted the transverse cracking of most sections without any pavement preservation treatment. The local calibration factor was found to be agency dependent for the test sections, without any pavement preservation treatment.

Table 11 lists the overall range of values that were determined from the local calibration effort for the transverse cracking transfer function, while Table 12 summarizes the diagnostic statistics for the different projects or sets of data. These values generally cover the same range of values summarized in Table 5, but were found to be highly mixture and/or production specific. Figure 10 compares the predicted and measured values of transverse cracking. The MEPDG has a maximum length of thermal cracking of 2,200 ft./mi. Obviously, some of the sites have exhibited much greater lengths of transverse cracking (refer to Figure 9). This limit in the software can result in a relatively large bias, which cannot be eliminated.

The MEPDG transfer function with the local calibration factor was found to be reasonable for predicting transverse cracks in HMA pavements and overlays. However, the standard error is relatively large. In summary, it is believed that there are both lack-of-fit and measurement errors in terms of predicting the crack growth with time.

Smoothness or IRI Regression Equation for Flexible Pavements

The MEPDG regression equation for predicting smoothness or increasing roughness was developed from a regression analysis of hundreds of test sections included in the LTPP program. This prediction model is not based on mechanistic principles so it can only be revised using regression-based procedures. Table 13 summarizes the diagnostic statistics (bias, standard error, and RMSE) for each of the facilities and types of experiments for IRI. Figure 11 compares the measured and predicted IRI values for all sites. In summary, the IRI regression equation has been found to be adequate, both in terms of accuracy and precision, for all conditions.

Table 11. Summary of the Local Calibration Values for the Thermal Cracking Transfer Function

Project Identification		β_{t1}	β_{t2}	β_{t3}
Montana DOT; application of pavement preservation treatments.		---	---	0.25
Northwest Sites, located in states adjacent to Montana, but without pavement preservation treatments; appears to be agency dependent.		---	---	1.0 to 5.0
Kansas PM Segments; Full-Depth Projects	PMA	---	---	2.0
	Conventional	---	---	2.0
	Superpave	---	---	3.5
Kansas PMS Segments; HMA Overlay Projects	PMA	---	---	2.0
	Conventional	---	---	7.5
	Superpave	---	---	7.5
LTPP Projects; HMA produced in accordance with specifications	Conventional	---	---	Dependent on Asphalt Content & Air Voids
LTPP Projects; Severely aged asphalt	Conventional	---	---	7.5 to 20.0

Table 12. Summary of the Bias and Standard Error for the Thermal Cracking Transfer Function from Independent Data Sets

Facility/Project Identification	**No. of Points**	**Bias, in./mi.**	**Standard Error, in./mi.**	**R^2 Term**	**S_e/S_y**
Global Calibration Statistics	*28*	---	---	*0.064*	---
Montana DOT; New Construction & Overlay Projects	110	-26.5	353.1	0.763	0.634
Kansas PM Segments; Full-Depth Pavement Projects	177	-59.4	313.6	0.595	0.829
Kansas PM Segments; HMA Overlay Projects	35	-43.7	410.2	0.736	1.136
LTPP SPS-1 Projects	122	+23.53	287.4	0.696	0.583
LTPP SPS-5 Projects	158	-100.1	606.6	0.639	0.979

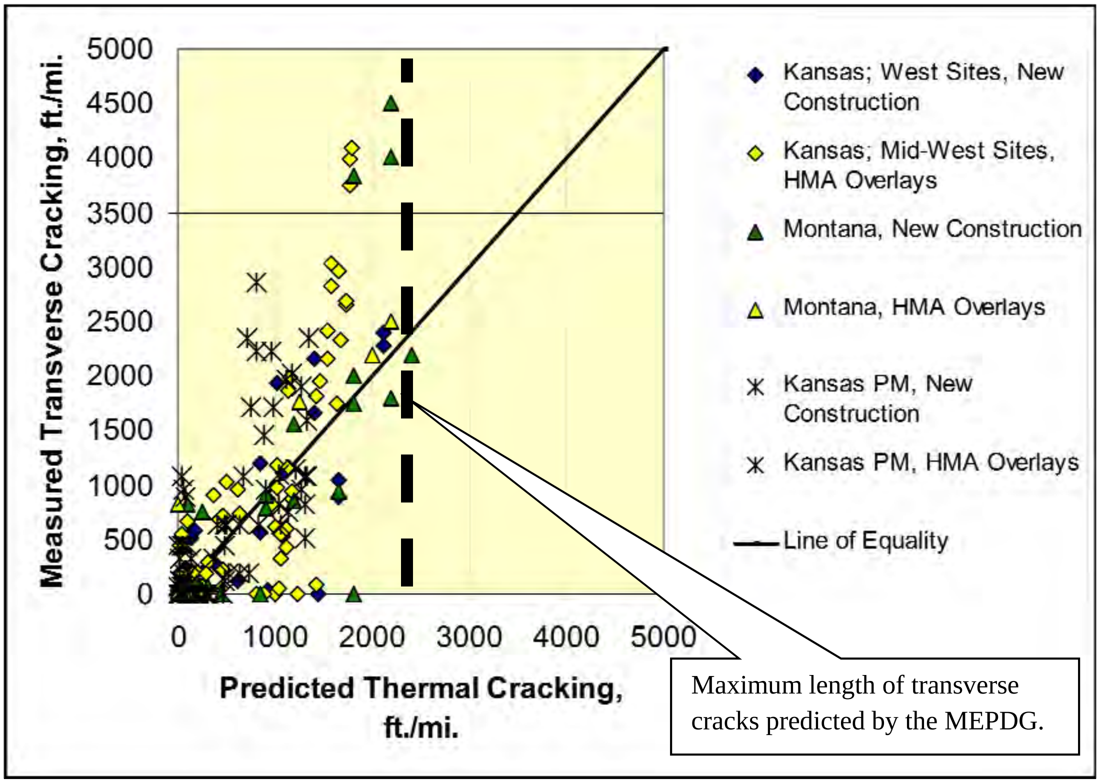


Figure 10. Comparison of Measured and Predicted Thermal (Transverse) Cracking

Table 13. Summary of the Bias and Standard Error for the IRI from Independent Data Sets

Facility/Project Identification		**No. of Points**	**Bias, in./mi.**	**Standard Error, in./mi.**	**R^2 Term**	**s_e/s_y**
Global Calibration Statistics		*1926*		*18.9*	*0.560*	
Montana DOT	New Construction	110	+0.27	6.08	0.887	0.417
	HMA Overlays	120	-2.65	6.91	0.892	0.352
Kansas PM Segments; Full-Depth Pavement Projects		177	-2.87	15.0	0.703	0.632
Kansas PM Segments; HMA Overlay Projects		35	+0.38	14.3	0.402	0.646
Mid-West Sites	LTPP SPS-1 Projects	122	+2.804	14.348	0.668	0.631
	LTPP SPS-5 Projects	158	+4.900	14.077	0.121	0.773

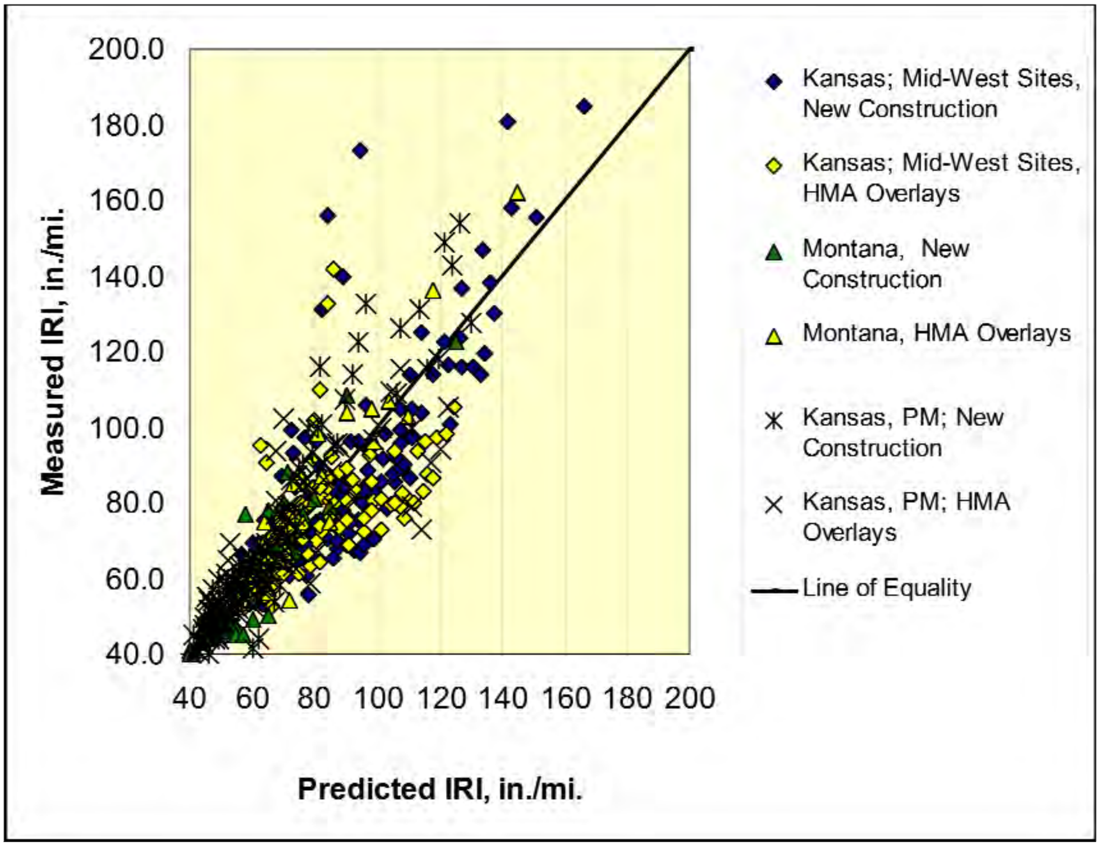


Figure 11. Comparison of Measured and Predicted IRI from Different Data Sources

Transverse Fatigue Cracking of JPCP

The local calibration of transverse fatigue cracking of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona calibration utilized the correct CTE values as measured by AASHTO T 336 testing and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. Findings showed the following results:

- The slab to base (for Lean Concrete Bases or LCB) friction value should be set at 0 months. The specifications used to construct these bases (e.g., smooth LCB surface, two coatings of wax based curing compounds) resulted in immediate breaking of the bond and reduction in interfacial friction. This resulted in rapid cracking of many of these sections. When the MEPDG was set at 0 months friction, their cracking was accurately predicted.

- The local calibration coefficients tested were those developed under NCHRP project 20-07 for the correct CTE values of the PCC. The Arizona calibration verified these coefficients and made only small changes in the recommended values. A plot of the calibration curve is given in Figure 12.

Other studies performed in Utah, Colorado, and Wyoming (using a combination of LTPP sections from surrounding States) found that the NCHRP 20-07 calibration coefficients along with correct CTE values produce unbiased predictions and was adopted.

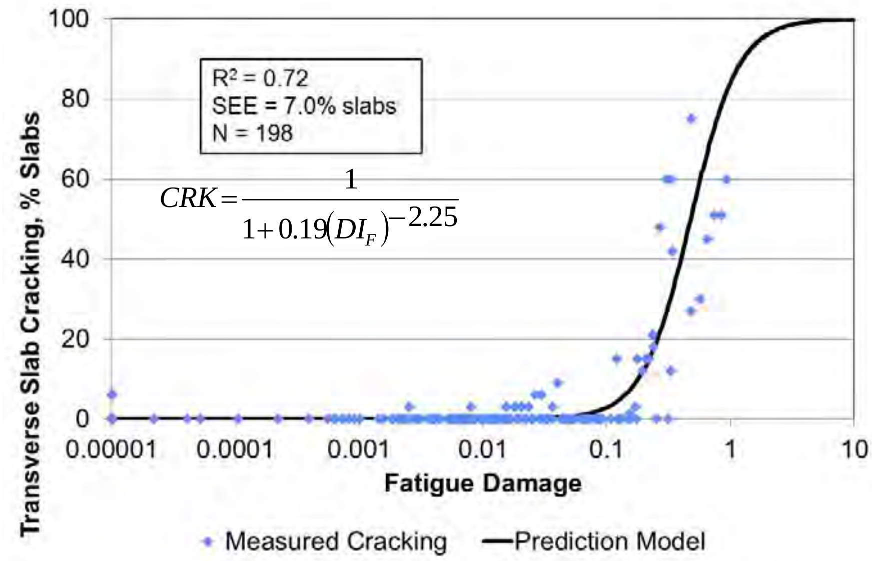


Figure 12. Arizona Recalibration of Transverse (Fatigue) Cracking Model using Correct CTE Values (all LCB sections have zero months friction)

Transverse Joint Faulting of JPCP

The local calibration of transverse joint faulting of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona calibration utilized the correct CTE values as measured by AASHTO T336 testing and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. Findings showed the following results:

- The local calibration coefficients tested were those developed under NCHRP project 20-07 for the correct CTE values of the PCC. The Arizona calibration verified these coefficients but made some changes to further minimize the prediction error.

Other studies performed in Utah, Colorado, and Wyoming (using a combination of LTPP sections from surrounding States) found that the NCHRP 20-07 calibration coefficients along with correct CTE values produce unbiased predictions and was adopted.

IRI of JPCP

The local calibration of IRI of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona local calibration utilized the revised transverse cracking and joint faulting coefficients and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. The IRI revised model coefficients are shown in Table 14 and resulted in a significant reduction of prediction error as shown. The final Arizona predicted versus measured IRI is shown in Figure 13. This local calibration of IRI for Arizona clearly shows the benefits obtained by reducing the standard error of prediction, which will impact design reliability and construction costs. Similar results were obtained for other states including Utah, Colorado, and Wyoming. The Wyoming local calibration study used a combination of LTPP sections from surrounding States.

Table 14. Summary of Changes in the IRI Calibration Coefficients for Arizona Local Calibration

Model Coefficients	Global Calibration Coefficient	Arizona Local Calibration Coefficient
J1 (Cracking)	0.8203	0.60
J2 (Spalling)	0.4417	3.48
J3 (Faulting)	0.4929	1.22
J4 (Site Factor)	25.24	45.2
Standard Error of Prediction	24.6 in/mile	9.8 in/mile

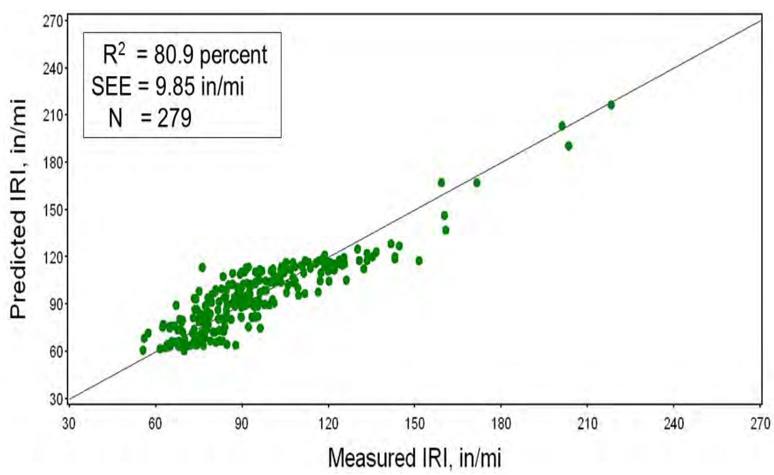


Figure 13. Predicted IRI (using recalibrated coefficients) versus Measured IRI for Arizona

Punchouts and IRI of CRCP

Arizona made a comparison of the performance of two CRCP sections over 20 years to the MEPDG predictions. Results indicated that the MEPDG global calibration predicted well for these sections. No other local calibration study of CRCP punchouts or IRI model has been accomplished to date. The calibration coefficients obtained from the NCHRP project 20-07 study using the correct CTE values produced an unbiased punchout prediction model.

Summary

A summary of the significant findings from the calibration refinement studies are listed below, as they relate to the MEPDG transfer functions and their predictive capability.

- The local calibration for a State is critically important and can be used to adjust the predicted performance indicators to minimize the over and under prediction bias and reduce prediction error.
- Some of the local calibration parameters are dependent on material properties and material types. Accounting for this effect reduced the standard error or increased the precision of the transfer function.
- The error component that has the greatest effect or impact on the standard error or precision of the transfer function and model is the measurement error of the performance indicator. This error can be observed from plots of cracking, rutting, or faulting over time where significant variations occur for the same section from year to year. Some of these measurement errors are large and until they are reduced, reducing the lack-of-fit modeling error component (lack of model ability to accurately characterize the distress) will have a small effect on the overall standard error of the estimate term.
- The other item found to have a significant effect on the standard error and bias of a transfer function (primarily load related cracking) is construction anomalies (errors in mixture properties or thickness). If not properly identified, there can be a large bias between the predicted and measured values.
- With adequate local calibration, the following transfer functions are considered appropriate for use in day-to-day designs for flexible pavements.

    a. Rut depth transfer function.
    b. Area fatigue cracking (bottom initiated) transfer function.
        i. The fatigue cracking transfer function for semi-rigid pavements needs additional confirmation work to support the local calibration values using the latest version of the MEPDG.
    c. Thermal cracking transfer function for cold climates. The transverse cracking transfer function does not have the capability to predict transverse cracks caused by other mechanisms, other than low temperatures. These other mechanisms include: severe aging or hardening, asphalt absorption or shrinkage, and other long term conditions that can occur in hotter climates.
    d. Smoothness or IRI regression equation.

- With adequate local calibration, the following transfer functions are considered appropriate for use in day-to-day designs for rigid pavements.
    a. Transverse (fatigue) cracking of JPCP.
    b. Transverse joint faulting of JPCP.
    c. Smoothness or IRI of JPCP.
    d. Edge punchouts of CRCP.
    e. Smoothness or IRI of CRCP.

- No consistent trend in the prediction of LCWP could be identified within these studies to reduce the bias and standard error, and improve on the accuracy and precision of this transfer function. The LCWP prediction model is considered inaccurate for the projects and test sections used in the calibration refinement study. It is believed that there is a significant lack-of-fit modeling error (theoretical assumptions) for the occurrence of LCWP.

6.3 Partial or Full Scale Deployment

Few State agencies have either completed the calibration effort for some or all aspects of the MEPDG. These agencies have either developed design catalogues (e.g. California, Florida and Washington State) or User Guides (e.g. Missouri and Indiana) that provides guidance on input selection and design procedures. Examples include:

- The MEPDG is fully implemented in Indiana since January 1, 2009. Guidance for using MEPDG in routine pavement design is published in the INDOT 2013 Design Manual Chapter 52. (INDOT, 2013). INDOT has documented the cost savings with efficient pavement designs resulting from the implementation of MEPDG (Nantung, 2010).
- Missouri DOT has completed the full-scale research study enabling the agency to facilitate a transition from the current pavement design methodology to MEPDG. This effort resulted in development of a User's Guide with default data libraries for MEPDG inputs of relevance to local materials, soils and traffic, recommended input levels to be used for different conditions and roadways, performance criteria, reliability levels and calibration factors. This effort also conducted the verification, validation and recalibration of relevant MEPDG models for use in Missouri and provided recommendations for the MEPDG deployment in Missouri (Mallela et al, 2009).
- Utah DOT has completed a major local calibration effort from 2004 to 2009 and along with extensive staff training has allowed the agency to utilize the MEPDG for pavement design. A comprehensive User's Guide was developed that detailed the selection of inputs, design performance and reliability criteria, and procedures to achieve passing designs. The major calibration adjustment was to eliminate the over prediction of rutting. One aspect of implementation that was discussed was selection of design reliability since the MEPDG generally resulted in thinner HMA and JPCP designs at the same level of reliability.

6.4 Design Reliability and Performance Criteria

Selection of design reliability and design performance criteria was studied by Arizona. A sensitivity analysis was conducted that showed how the resulting design depends on these critical inputs. Selection of too high of reliability and/or performance criteria resulted in unreasonable and costly designs. It was recommended that the selection of reliability and performance criteria should be done together and not independently. These factors can affect the design as much as any other inputs and need to be more fully considered in the implementation effort.

7. SUMMARY

This synthesis report intends to capture the status of current and completed implementation activities by various State agencies. The information compiled in this report will serve as a reference document to FHWA to see what other States are doing with regards to implementation and help prevent avoidable problems experienced by other agencies during their implementation effort. The other intent of the synthesis report is to provide a summary of the results from other agencies calibration efforts in planning a sampling matrix and experimental factorial, if a local calibration is believed to be required by an individual agency.

Most State DOT studies have focused on building data libraries for key material types, and evaluate the ability of lower hierarchical input levels to produce reasonable predictions for the agency-specific material types. Numerous studies have focused on HMA mixtures; the evaluation of Level 3 Witczak dynamic modulus model indicates that the model predictions appeared to be acceptable for mixtures with conventional binder, significant deviations were observed for binders with higher PG grades. Use of measured binder test data (i.e. input level 2) in Witczak model has greatly improved the accuracy of dynamic modulus predictions. Studies on PCC mixtures have particularly emphasized on the CTE measurement and its significance in rigid pavement performance. Although the results are diverse, most studies have concluded the Witczak dynamic modulus regression equation is reasonable.

The findings of national level studies, including NCHRP projects 1-40B, 9-30A, and 1-47, are directly applicable to other highway agencies. More importantly, the lessons learned from various calibration studies are directly applicable for use in setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions. The following lists some of the more important findings from the literature and projects reviewed under this Task.

1. Selection of design reliability and design performance criteria requires sensitivity analyses to show how the resulting design depends on these critical inputs. Selection of too high of reliability and performance criteria will result in unreasonable and costly designs. Selection of reliability and performance criteria should be done together and not independently.

2. The key findings of the sensitivity analyses conducted under NCHRP project 1-47 will be used in the MEPDG implementation study to select sites for the local calibration and in evaluating the residual error of the predicted distress values.

3. The local calibration factors determined from State calibration studies for PCC pavements are reasonably consistent with the global coefficients. However, several important advantages were obtained through State local calibration for PCC pavements.
    a. More accurate design inputs were established through the local calibration process. For example, the estimate of the number of months of full friction between the slab and base was improved using local data.
    b. The use of the correct CTE input for the PCC (as measured by AASHTO T336) was found to verify the national coefficients determined under NCHRP project 20-07 in 2010

for several states. This makes it possible for the State to test the CTE of PCC and then use directly the value in design.

c. Modifications to some the calibration coefficients were found to be valuable in reducing the standard error of prediction which is used directly in reliability design.

4. The following are some consistent findings from flexible pavement calibration studies:
    a. MEPDG over predicts rutting in the unbound layers based on using laboratory equivalent resilient modulus values.
    b. Dynamic modulus does not explain the different in rutting between HMA and PMA mixtures.

5. The following local calibration coefficients were found to be significantly different between many of the studies reviewed:
    a. B_{f1} for the fatigue cracking transfer function.
    b. B_{r3} (exponent to the number of load cycles term) and B_{r1} (the intercept term) for the HMA rut depth transfer function.
    c. C1 or coefficient of the rutting term in the IRI regression equation.

6. The procedures outlined in the NCHRP projects 1-40B and 9-30A can be used to develop field adjustment factors for fatigue cracking and rutting models.

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2. Correlating M-E PDG with Vermont Conditions, Contract/Grant No. SPR 711, Sponsored by Vermont Agency of Transportation, Montpelier, VT. http://rip.trb.org/browse/dproject.asp?n=22922
3. Development of a Traffic Data Input System in Arizona for the Mechanistic Empirical Pavement Design Guide , Contract/Grant No. 672, Sponsored by Arizona Department of Transportation, Phoenix, AZ. http://rip.trb.org/browse/dproject.asp?n=20051
4. Implementation of MEPDG for Asphalt Pavements with RAP, Contract number: DTRT06-G-0016, Sponsored by Oklahoma Department of Transportation, Oklahoma City, OK. http://rip.trb.org/browse/dproject.asp?n=24907
5. Implementation of the Mechanistic-Empirical Pavement Design Guide in the Wyoming Department of Transportation, Contract/Grant No. RS03(209), Sponsored by Wyoming Department of Transportation, Cheyenne, WY. http://rip.trb.org/browse/dproject.asp?n=21498
6. Inputs of Portland Cement Concrete Parameters Needed for the Design of New and Rehabilitated Pavements in Mississippi, Contract/Grant No. 177, Sponsored by Mississippi Department of Transportation, Jackson, MS. http://rip.trb.org/browse/dproject.asp?n=10002
7. Instrumented Pavement Construction, Contract/Grant No. SPR Item 2200, Sponsored by Oklahoma Department of Transportation, Oklahoma City, OK. http://rip.trb.org/browse/dproject.asp?n=12549
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9. Modeling & Calibration of SDDOT Mechanistic-Empirical Pavement Design Criteria, Contract No. SD2007-08, Sponsored by South Dakota Department of Transportation, Pierre, SD.

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15. Improved Characterization of Truck Traffic Loading for MDOT Pavement Design, Contract/Grant No. 257, Sponsored by Mississippi Department of Transportation, Jackson, MS. http://rip.trb.org/view/2012/P/1231596

16. Mechanistic-Empirical Pavement Design: Materials Testing of Resilient and Dynamic Modulus, Project No. SD2008-10, Sponsored by South Dakota Department of Transportation, Pierre, SD. http://sddot.com/business/research/projects/ProjectInfo.aspx?ColorTheme=Red&ProjNbr=SD2008-10

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APPENDIX I. AGENCY REPORTS AND RESEARCH PROJECTS

The following provides a summary of completed agency research projects and reports, and on-going research studies.

AASHTO

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- *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide*. 2010. American Association of State Highway and Transportation Officials, Washington, DC. https://bookstore.transportation.org/collection_detail.aspx?ID=99.

Alabama DOT

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- *Traffic and Data Preparation for AASHTO MEPDG Analysis and Design*. **In progress**. Project No. TPF-5(242).
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APPENDIX J. TRAINING COURSES AND WORKSHOPS

The following provides a list of MEPDG and AASHTOWare Pavement ME™ training courses and workshops.

Federal Highway Administration

http://www.fhwa.dot.gov/pavement/dgit/dgitwork.cfm.

- Local Calibration Webinars (2014)
  - Introduction to Local Calibration.
  - Preparing for Local Calibration.
  - Determining the Local Calibration Coefficients.
- AASHTOWare Pavement ME Design Software Webinars (2013)
  - Getting Started with ME Design.
  - Climatic Inputs.
  - Traffic Inputs.
  - Material and Design Inputs for New Pavement Design.
  - Material and Design Inputs for Pavement Rehab with Asphalt Overlays.
  - Material and Design Inputs for Pavement Rehab with Concrete Overlays.
  - New Asphalt Pavement Structures.
  - Asphalt Overlays of Asphalt Pavements.
  - New Concrete Pavement Structures.
  - Unbonded Concrete Overlays.
- Climatic Considerations for Mechanistic-Empirical Pavement Design Workshop (2006).
- Traffic Inputs for MEPDG Workshop (2007).
- Use of Pavement Management System Data to Calibrate Mechanistic-Empirical Pavement Design Workshop (2006).
- Materials Inputs for Mechanistic-Empirical Pavement Design Workshop (2005).
- Mechanistic-Empirical Pavement Design and Construction Methodologies Workshop (2004).
- MEPDG Introductory Workshop (2004).
- Executive Summary for M-E Pavement Design (2004).

NHI Training

- 131109 – Analysis of New and Rehabilitated Pavement Performance with Mechanistic-Empirical Pavement Design Software (Under Development).
- 131064 – Introduction to Mechanistic Design.
- 132040 – Geotechnical Aspects of Pavements.
- 151018 – Application of the Traffic Monitoring Guide.