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MODOT STUDY RI04-002

IMPLEMENTING THE AASHTO MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE IN MISSOURI

VOLUME I: STUDY FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

Prepared by Applied Research Associates, Inc.
for the Construction and Materials Division
Missouri Department of Transportation

DISCLAIMER

This report has been prepared by Applied Research Associates, Inc. (ARA) under a research project funded by the Missouri Department of Transportation (MoDOT) titled "Implementing the AASHTO Mechanistic-Empirical Pavement Design Guide in Missouri (No. RI04-002)."

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This report was prepared by the following ARA project team members:

Jagannath Mallela (Project Manager)
Leslie Titus-Glover
Harold Von Quintus, P.E.
Michael I. Darter, Ph.D., P.E.
Mark Stanley
Chetana Rao, Ph.D.
Suri Sadasivam, Ph.D

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CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

Nearly 80 percent of State highway agencies (SHAs), including the Missouri Department of Transportation (MoDOT), use some version of the American Association of State Highway and Transportation Officials (AASHTO) *Guide for Design of Pavement Structures*, either the 1972, 1986, or 1993 version. This design procedure is based on the serviceability concept developed at the AASHO Road Test during the late 1950s. Although the 1986 and 1993 editions include some refinements to the original empirical-based design methodology, there are still many limitations, some of which are listed below:

- A single subgrade soil and only an unstabilized, dense granular base were used for the test sections. There are numerous subgrade soils in Missouri, and many other types of bases are in common use, including treated bases and stabilized subgrade soils. In addition, subdrainage was not included as part of the Road Test, but the design feature is used in Missouri.

- Traffic volumes and loading have increased by an order of magnitude since the AASHO Road Test, which means that most current designs are for outside the inference space of the design procedures developed from the original road test data (e.g. 10 million vs. 100 million plus ESALs today).

- As in any accelerated loading facility, the long-term effects of climate and age-related material deterioration are impossible to quantify. Extrapolation to a design life of 30 to 40 years is questionable based on the 2-year Road Test data.

- In earlier AASHTO design procedures, pavement performance is directly related to layer thickness despite the fact that some distresses (such as low temperature cracking and raveling) may be independent of layer thickness.

- The regression equations incorporated in the AASHTO design guides cannot directly accommodate the influence of new materials and ever-evolving design or construction practices. Climatic effects are found in other regions of the country not directly accounted for by the procedure.

- Finally, the primary indicator of pavement performance in earlier AASHTO design procedures is the present serviceability index (PSI), in which the dominant factor is ride quality. Realistically, distresses such as wheel path rutting and cracking may dictate the timing of rehabilitation. The key distresses also must be considered directly to avoid early failures and to obtain optimal design. As the emphasis shifts from building new roadways to maintenance and rehabilitation of existing roadways, the design procedures also must be modified to accommodate these shifts.

- Inability to consider different design features such as full-width AC paving and tied shoulders on PCC pavements.

An empirical design is greatly limited because the inference space is confined to the conditions (materials, loads, and environment) on which the regression equations were based. Accordingly, a 1996 workshop sponsored by the AASHTO Joint Task Force on Pavements (now the Joint Technical Committee on Pavements) initiated an effort to develop an updated pavement design guide based on mechanistic principles that could reasonably and rationally account for various materials, traffic conditions and environments.

Mechanistic or "rational" pavement design adheres to the principles of mechanics that require consideration of the following: the theory used to predict the failure or distress parameter, evaluation of material properties compatible with the theory selected, and determination of the relationship between material properties and failure or some pre-selected performance level (Yoder and Witczak 1975). By using the principles of mechanics as the basis of the design/analysis procedure, an agency can consider a variety of site features and conditions that were not directly included in the experiment from which the design equation was derived. In other words, the agency can consider the effects of different distress types, changes in the design criteria or the condition of the surface that is unacceptable to the user, and changes in material specifications. MoDOT has recognized the advantages and importance of adopting mechanistic principles for the design of new and rehabilitated pavements.

Based on extensive research and development performed under the National Cooperative Highway Research Program (NCHRP) Project 1-37A, a team led by Applied Research Associates, Inc. (ARA) finalized the development of the Mechanistic-Empirical Pavement Design Guide (MEPDG) and accompanying software in 2004 (ARA 2004). This body of work was formally peer reviewed by an independent group under NCHRP Project 1-40A. Simultaneously, the design approach and software were made available to the public through the Transportation Research Board (TRB) for comment and use. The reviews resulted in several comments identifying the strengths and weaknesses of the MEPDG. Several critical comments from these reviews were addressed at the direction of the NCHRP's technical panel under NCHRP project 1-40D, and a revised version of the MEPDG software (version 1.0) was produced and has been posted on TRB's website since 2007. The changes to the design approach and software were documented in NCHRP Research Results Digest 308 (Darter et al. 2007). Subsequently, the MEPDG was discussed and balloted in late 2007 as an AASHTO interim pavement design standard. To facilitate the adoption of the method, an AASHTO Manual of Practice was developed for the MEPDG under NCHRP Project 1-40B (AASHTO 2008).

The MEPDG is built on the premise that each type of pavement failure (distress) it considers is related in a major way to a critical pavement structural response (stress, strain, or deflection). It also assumes that if loadings, materials, and prevailing climate in which the project is located, and their mutual interactions, can be defined as accurately as possible, then knowledge of the critical structural responses would greatly enhance a designer's ability to predict the quantities of distress that will occur on a given pavement over its design life. This will allow a rational adjustment of the section thicknesses and material properties to achieve the desired design life. In recognition of the fact that it is almost impossible to predicate a method solely on a mechanics-based approach (partly due to practical considerations associated with choosing a mechanics model in terms of input sophistication required and computational time and the fact

that pavement inputs are inherently uncertain in nature), the MEPDG relates the mechanics-based pavement responses or response derivatives (e.g., damage) to "real world" pavement distresses through transfer functions. These contain other empirical parameters (e.g., pavement design or materials inputs, climatic factors, soil parameters) and regression coefficients. The difference between mechanistic-empirical and purely empirical performance models (e.g., the AASHO Road Test derived pavement design equations) is that the former are considered more robust and easily to extrapolate due to the embedment of engineering mechanics-based response terms in them.

The structural response models used in the MEPDG represent a good balance between practicality and desired accuracy. The MEPDG transfer functions are very powerful and are based on the best available data set in existence today for pavement modeling purposes, drawn primarily from the Long-Term Pavement Performance (LTPP) database. The MEPDG procedure is therefore considered a significant leap ahead in the continuum of pavement development. Some of the benefits of the MEPDG include:

- Improved procedures to perform pavement rehabilitation design with asphalt overlays, concrete overlays, and other non-overlay rehabilitation and restoration actions.
- Improved design reliability by direct consideration of key distresses (rutting, faulting, cracking) and pavement ride quality through direct consideration of the error in prediction, rather than a large traffic multiplier that results in overly conservative design for heavy traffic.
- Increased pavement life by a better selection of pavement materials and design features to reduce key distresses.
- Improved consideration of new materials and design features.
- More cost-effective designs due to far more comprehensive consideration of subgrade support, material properties, climate conditions, and traffic loadings.
- More cost-effective designs (especially rehabilitation designs) based on more effective use of materials, layer thicknesses, and design features.
- Greatly enhanced capability to conduct failure or forensic analyses by explaining observed distresses, as well as the ability to consider special truck loadings.
- Ability to predict the impact of many construction deficiencies.

In addition, the MEPDG provides a tool for evaluating the impact of material properties and some construction practices on pavement performance, aids in implementing performance-related specifications (PRS) or warranties, and provides some tools for use in a comprehensive pavement management system.

1.2 IMPLEMENTING THE MEPDG

Implementation of the first AASHTO Design Guide in the early 1960s was a major paradigm shift in the design process and required SHAs to undertake major efforts over a period of several years. These efforts included training of staff, analysis of truck axle load and other traffic data to establish factors to estimate the equivalent single axle load (ESAL) applications, laboratory testing of materials and establishing correlations with the new material inputs, field testing such

as establishing the initial serviceability index of new construction, and selection and monitoring of field test sections for calibration to local conditions. These efforts paid off for those SHAs in terms of improved and more cost-effective design for several decades.

The MEPDG represents a similar paradigm shift in the design process. The implementation of this system as an agency standard will likewise require significant changes to pavement design procedures, testing procedures and equipment, traffic data inputs, climatic data inputs, performance criteria, and others. As noted earlier, the MEPDG performance models were calibrated and validated primarily using data primarily from the LTPP program, and use of these models will be beneficial to most agencies. However, to maximize the benefits and accuracy of the MEPDG, proper selection of inputs and calibration to an agency's operational policies in terms of pavement management and preservation, material and construction specifications, traffic, and climatic conditions are essential.

The MEPDG computational software system automates the methods and procedures to perform pavement designs using this new design approach. The software has several default inputs related to soils, traffic, climatic, and materials derived from national or global databases. Customization of these default inputs for Missouri-specific conditions will aid in easier and more uniform system-wide implementation of the software. Likewise, the software also allows each agency to customize the calibration coefficients for each performance model based on the outcome of local validation, calibration, and confirmation studies.

1.3 RESEARCH OBJECTIVES AND IMPLEMENTATION ROAD MAP

To facilitate a transition from the current pavement design methodology to a mechanistic-empirical (M-E) approach, it is imperative that agencies begin to assess their needs in terms of the new MEPDG inputs (traffic, materials and environment). Equally important is the enhancement and calibration of performance prediction models with local data. Accordingly, a Road Map was prepared to aid in the implementation of the MEPDG in Missouri. The objective of this Road Map was to identify the steps and activities needed to locally calibrate the distress prediction models to MoDOT policies and materials, and streamline a design process that will enable MoDOT and its consultants to use M-E principles for pavement design. As such, the Road Map was divided into four parts:

1. Familiarization Plan of the new MEPDG Design Process and Features
    a. Review all inputs, select appropriate test protocols for obtaining them in routine design, and select the input levels considered appropriate for each input parameter. This step was performed at the outset at a policy level. Some revisions of the original plan took place based on the findings from a review of performance data and sensitivity analysis (discussed later in this report).
    b. Establish traffic and materials input default values based on best available data.
    c. Select the types of pavements of interest for new or reconstruction design and the types of rehabilitation and maintenance activities used for existing pavements.

   d. Conduct a preliminary sensitivity analysis of the MEPDG software and comparison with the current MoDOT procedures (a cursory verification of the predictions) for the pavement types and rehabilitation strategies selected.
   e. Provide guidance to set up a laboratory and field testing program to support use of the MEPDG.

2. Calibration Plan to Local Conditions, Policies, and Materials
   a. Select the test sections needed for the local calibration process.
   b. Prepare a pavement section sampling and testing plan.
   c. Conduct the laboratory and field testing of the test sections needed for calibration.
   d. Execute the local calibration of the distress prediction models included in the MEPDG and periodically update the results, as well as include additional calibrations of pavement design and rehabilitation strategies that might change over time.
   e. Evaluate the results of the designs using the local calibration coefficients with respect to the different input levels selected.
   f. Conduct a complete sensitivity analysis using the local calibration coefficients and local default values, if different from the national values, to judge the appropriateness of the distress prediction models and design process.

3. Training and Coordination Plans
   a. Develop a coordination plan between the different departments involved in the pavement design and evaluation process, as well as, between the DOT and industry.
   b. Prepare a technology transfer program, including a training program for MoDOT personnel and industry involved in the pavement design process.

4. User's Guide
   a. Develop a user's guide for pavement design.
   b. Customize the software to include the libraries of local materials, soils and traffic, recommended input levels to be used for different conditions and roadways, default input values and tables recommended for use, calibration factors or functions, performance criteria, reliability levels, etc.

Figure I-1 through Figure I-3 present flow charts showing all of the activities and steps included in MoDOT's MEPDG implementation Road Map and how they relate to one another. It should be noted that some of the activities in Figure I-3 fall outside the timeframe of this project and will be discussed in the final chapter of this report.

1.4 SCOPE OF MEPDG IMPLEMENTATION IN MISSOURI

The broad scope of work for this project included (1) developing default data libraries for MEPDG inputs of relevance to Missouri, (2) verifying, validating, and if needed recalibrating relevant MEPDG models for use in Missouri, and (3) developing recommendations for MEPDG implementation in Missouri.

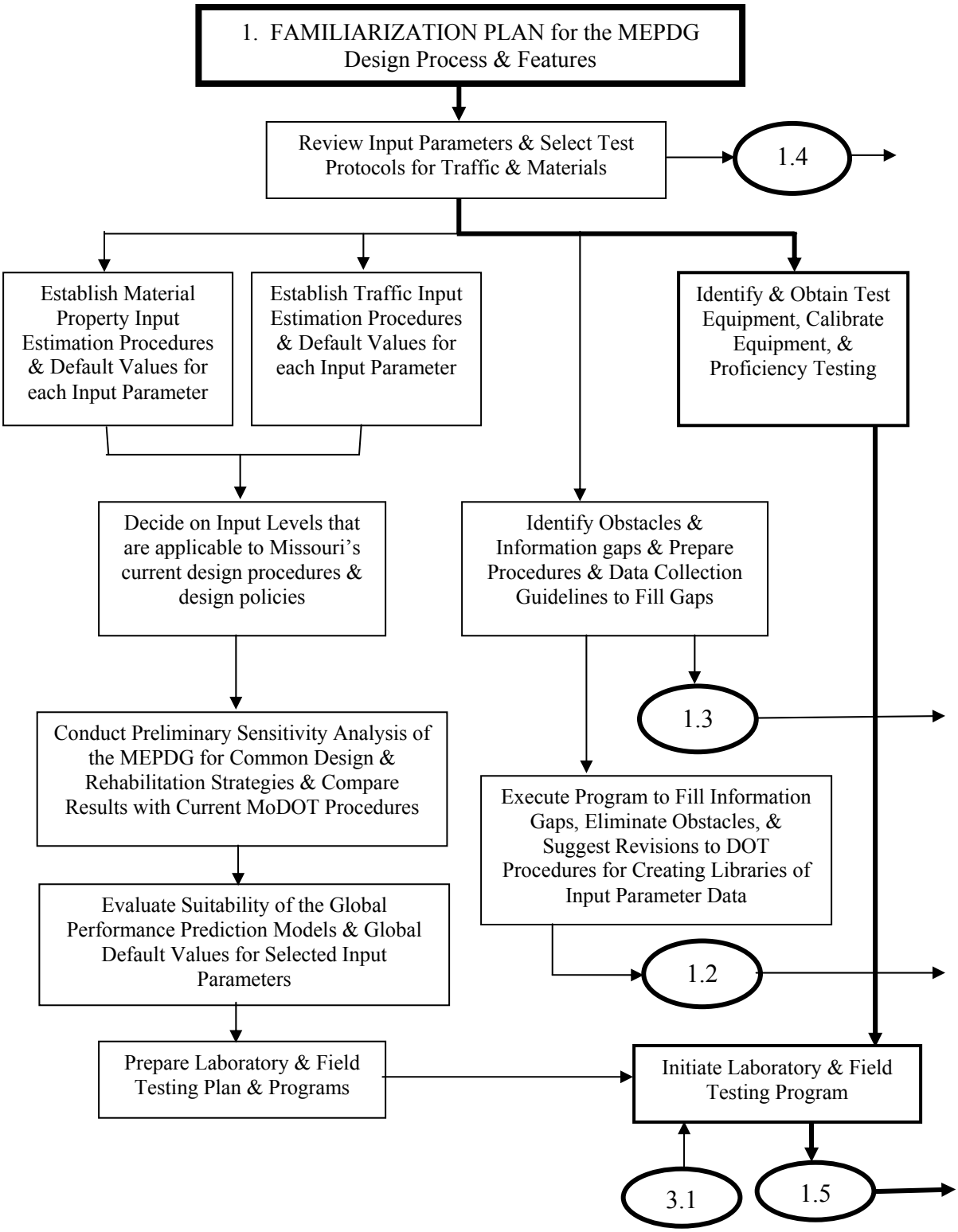


Figure I-1. Flow chart showing all of the familiarization steps and activities considered for implementing the MEPDG in Missouri.

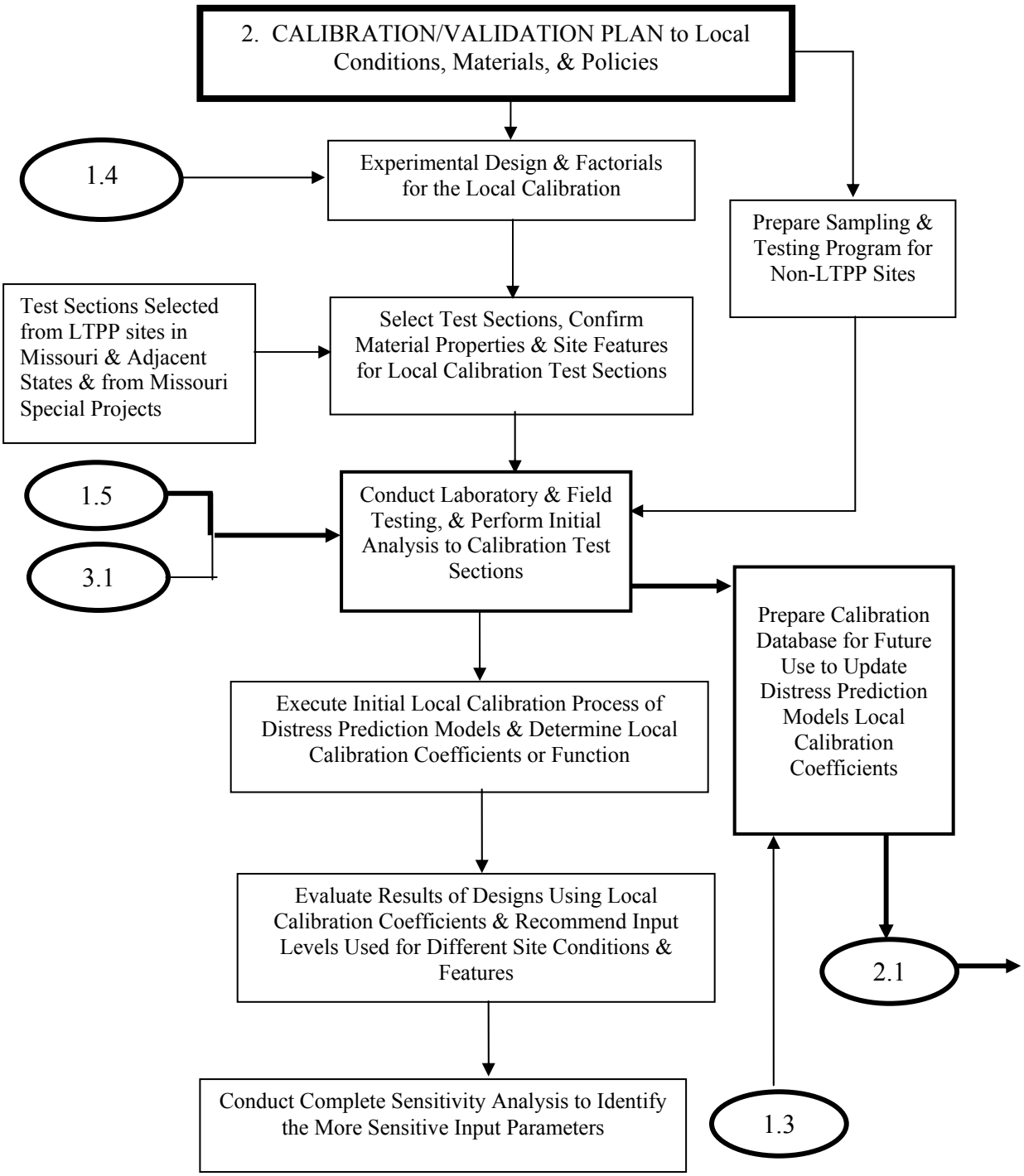


Figure I-2. Flow chart showing all of the calibration steps and activities considered for implementing the MEPDG in Missouri.

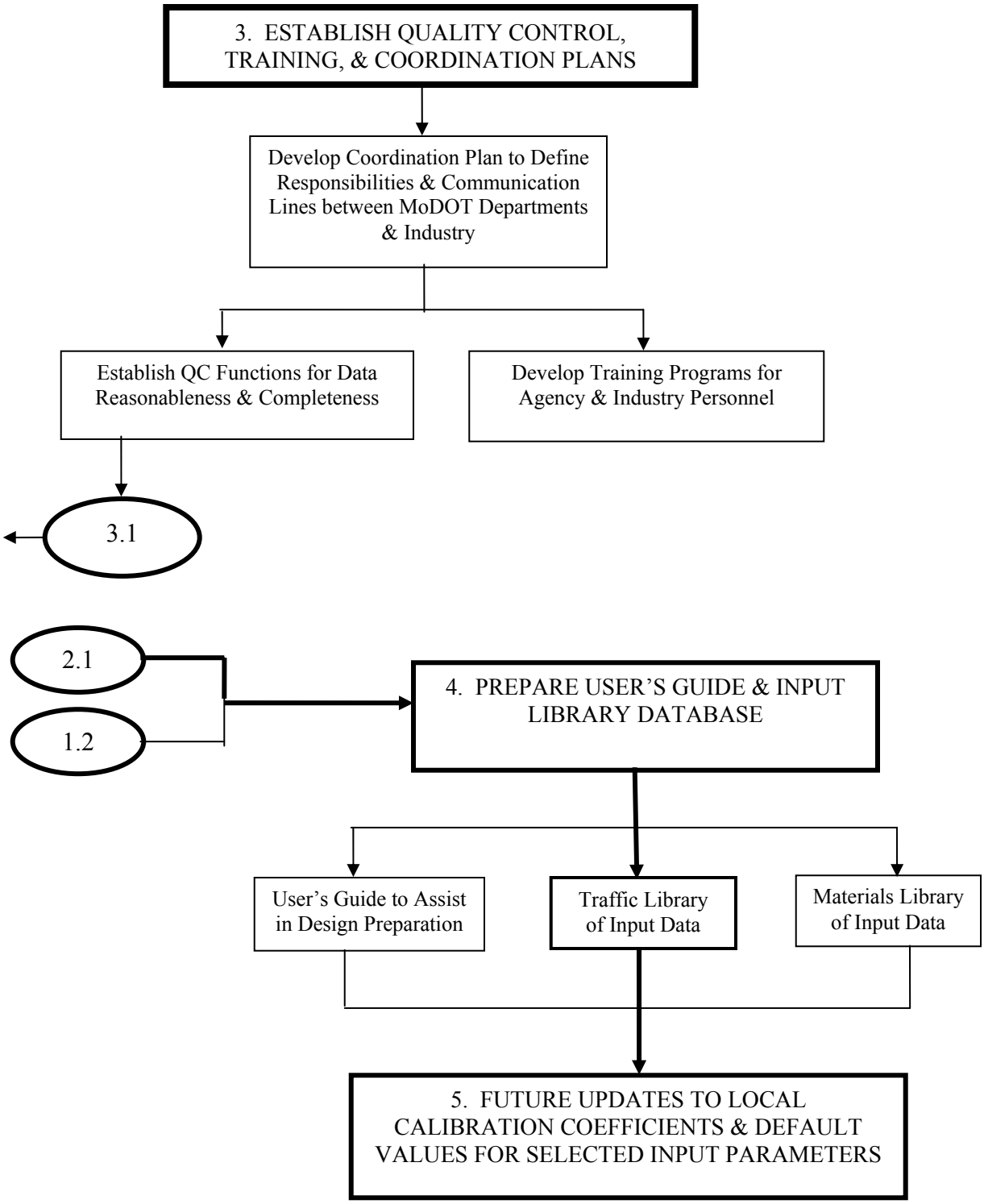


Figure I-3. Flow chart showing the training and future update steps and activities that are suggested to implement the MEPDG in Missouri.

As part of an MEPDG familiarization meeting held at MoDOT's headquarters in Jefferson City in May 2004, the proposed Road Map was discussed. The meeting was held in the context of Step 1 (see Figure I-1) and helped frame the scope of work better and modified the proposed work effort. Key highlights of this meeting are discussed in the following sections.

1.4.1 Overview of Pavement Structures Implemented in Missouri and Pavement Types of Interest for MEPDG Implementation

The Missouri State highway system consists of 71,357 lane-miles of pavement. Pavement types range from low-volume rural collectors to multi-lane, high-volume urban Interstates as follows:

- Total State System—71,357 lane-miles.
    - National Highway System (NHS)—14,273 lane-miles (i.e., 19.3 percent).
    - Remaining Arterials—10,394 lane-miles (i.e., 15.3 percent).
    - Collectors—46,690 lane-miles (i.e., 65.4 percent).

These groups represent different levels of functional importance that require different levels of rehabilitation and maintenance effort. A breakdown of the functional classes according to pavement type is as follows:

- NHS pavement type (14,273 lane-miles).
    - Portland cement concrete (PCC)—39.4 percent.
    - Hot mix asphalt (HMA) overlays of PCC pavements (HMA/PCC)—37.6 percent.
    - Full-depth HMA—23.0 percent.
- Remaining arterials pavement type (10,394 lane-miles).
    - PCC—12.7 percent.
    - HMA/PCC—12.4 percent.
    - Full-depth HMA—74.9 percent.
- Collectors (46,690 lane-miles).
    - PCC—1.2 percent.
    - HMA/PCC—0.4 percent.
    - HMA—98.4 percent.

Design practices are summarized in the following sections.

<u>New HMA Pavements</u>

Full-depth HMA pavements are well represented in all three functional categories; however, historically they have been built on routes of lesser significance. For the purposes of this discussion, HMA pavements with an asphalt treated base, resting directly on the subgrade, are considered full-depth pavements.

Loss of serviceability (present serviceability rating, PSR) for this pavement type can be equally attributable to increases in fatigue and block cracking resulting in excessive levels of roughness.

Missouri has constructed a few full-depth pavements with a Superpave mix design since 1997. The performance of these pavements is difficult to ascertain because of their relatively short lives. However, early trends indicate adequate performance. This pavement type will be the norm for asphalt pavements in the future.

HMA/PCC Pavements

Very few pavements in Missouri were designed originally as composites (HMA/PCC). However, HMA/PCC pavements have become a predominant pavement type on high volume highways through the steady and widespread rehabilitation of aging PCC pavements.

The average service life of HMA/PCC on divided NHS routes, regardless of thickness or traffic (terminal PSR $\geq$ 29) is approximately 10 years. Initial overlays last longer than subsequent overlays. Preliminary evaluations of selected HMA/PCC pavements have shown that thicker HMA overlays have not provided significantly longer service lives than thinner ones, while there is no clear relationship between traffic applications and performance.

Other HMA material types, such as stone matrix asphalt (SMA) and Superpave, have been used to construct HMA/PCC pavements. SMA overlays are used as an alternative to the standard hot mix wearing courses. Their stone-to-stone matrix, blend of superior coarse aggregates, and fiber additives are designed to withstand rutting and wear. SMAs were first placed in Missouri in 1991. Most SMA wearing courses were placed on Type I-B binder mixes, which have since been replaced by SP250 (Superpave) mixes.

Preliminary indications show that in Missouri, SMA overlays have provided roughly 2 more years of service life than conventional HMA overlays. The increase is due to a slowing of the progression of rutting and the advent of reflective cracking when compared to conventional HMA.

Missouri began Superpave construction in 1996 at its LTPP SPS-9 site on U.S. 65 north of Sedalia. The earliest projects were overlays on the NHS. Judging from the modest amount of data available (4 years of data from a handful of projects), Superpave overlays seem to perform better than conventional HMA overlays. The reason for the enhanced performance appears to be the ability to resist reflective cracking more effectively in the early years.

New Jointed Plain Concrete Pavements (JPCP)

Prior to 1994 Missouri had occasionally built some 30-ft JPCP. These pavements tended to form mid-panel cracks. Since 1994, all PCC pavements in Missouri have been built with 14-ft-wide (13-ft wide lanes on non-Interstate routes) driving lane slabs (paved 14-ft. wide or 2 ft. beyond the edge line), joint spacing of 15 ft, doweled joints, and slab thickness ranging from 11 to 14 inches for NHS pavements and remaining arterials. The JPCPs typically are constructed on a Type 5 base or with a 2-ft. daylighted rock base (preferred), which is believed to be very stable and drainable.

Unbonded JPCP (UBOL)

Unbonded JPCP overlays of existing PCC pavements have been built in Missouri since the 1930s on an infrequent basis. Although a dozen or more projects were built before 1990 on the State system, nearly all have been either replaced or overlaid with HMA. The designs varied, and there was never any attempt to standardize the method. They were usually short pavement sections. Some older pavements used gravel for the bond breaker interlayer. A few others were meant to be bonded overlays, but recent coring at two of the sites revealed that the surface preparation was inadequate for bonding.

HMA over Fractured PCC (Rubblized or Break and Seat)

Missouri has four short (~500-ft) rubblized sections located on I-35 in Harrison County and one 0.9 mile section on I-44 (westbound). These were constructed as part of the LTPP SPS-6. They consisted of a pair of 7-in. and a pair of 11-in. asphalt concrete (AC) overlays with one 7-in. and one 11-in. section having longitudinal edge drains installed. All the sections have some longitudinal cracking outside of the wheel paths. Some fatigue cracking has developed in the wheel paths of the sections without edge drains. The performance after 10 years has been acceptable.

Missouri also had four break-and-seat sections constructed at the same LTPP SPS-6 site. They consisted of a pair of 4 inches and 8 inches. AC overlays with one 7-inch and one 11-inch PCC section with longitudinal edge drains installed. All the sections have some longitudinal cracking outside of the wheel paths. Although no fatigue cracking has occurred in the wheel paths, reflective cracking is showing through. The performance after over 10 years has been acceptable, but MoDOT is not considering this option into the future.

Overall, Missouri has had little experience with rubblization and break-and-seat.

Restored JPCP

Full-Depth Repairs

MoDOT has performed countless full-depth repairs, usually preceding the placement of an HMA overlay over an existing PCC pavement. Full-depth repair techniques have been studied in past Missouri research efforts, but no known estimate of extended pavement service life was ever developed.

Diamond Grinding

Diamond grinding restores ride on faulted JPCP. Several pavements were diamond ground and studied for performance. Route 171, north of Joplin, which had developed ¾-in average faulting, was diamond ground in 1986. Fourteen years later, hardly any faulting had returned. The LTPP site on I-35 in Harrison County had two diamond ground test sections. The pavement was 18 years old at the time of restoration. It had some D-cracking at most joints. Faulting was

moderate at around ¼ in. Performance after diamond grinding was adequate for about 4 years before full-depth repairs were required.

Ten miles of I-44 west of Springfield in Greene County were diamond ground in 1996. The pavement was badly faulted and had an extremely poor base and subgrade. Significant pumping and slab cracking was evident throughout the project. Faulting returned within a year.

On the basis of such performance (gathered from a total of 100 centerline miles of diamond ground pavements), it was recommended that diamond grinding in Missouri be limited to pavements with adequate load transfer and support and maximum average faulting around 0.5 in. Expected service lives should be 5 to 10 years depending on the type and condition of the pavement.

Dowel Bar Retrofit

Dowel bar retrofitting restores load transfer across joints and working cracks. Missouri has placed dowel bars in nearly every jointed pavement it has built, so restoring load transfer in this manner never became an issue. Therefore, MoDOT has no experience with retrofitting.

1.4.2 Pavement Sections of Interest for MEPDG Implementation

To ensure that every effort is being made to increase the competition for paving contracts, and that the latest market price is considered when determining pavement type, MoDOT is allowing contractors to bid an alternate pavement design (MoDOT formally implemented this process in 2005). The MEPDG is being considered as a tool for pavement design in this process. The pavement structures of interest to this bidding and pavement type selection approach include new HMA pavements, new JPCP, UBOL, and HMA overlays over rubblized PCC layers. These four pavement options are of primary interest to MoDOT.

Based on this information, the future trends in MoDOT design practices, and other discussions held with MoDOT, a listing of pavement types of interest to MoDOT for MEPDG implementation was developed and presented in Table I-1 along with a summary of all pavement design types (new and rehabilitated) that are considered by the MEPDG. This project deals with only the design types marked with a high level of priority in Table I-1.

1.5 ORGANIZATION OF THE REPORT

This report is organized into two volumes which document the findings of the work conducted under the various tasks of this project:

- Volume I – Summary of Findings, Implementation Plan, and Next Steps.
- Volume II – Validation/Calibration Detailed Results.

Table I-1. Summary of all pavement design types (new and rehabilitated) that are considered by the MEPDG, along with the design types of interest to MoDOT.

Design Types	Description	MEPDG	MoDOT Priority Level
New	Conventional HMA (HMA over unbound aggregate base)	Yes	Moderate
	Full-depth or deep-strength HMA (HMA over asphalt treated base)✓	Yes	High[1]
	Semi-rigid HMA (HMA over chemically treated base)	Yes	Low
	JPCP✓	Yes	High
	Continuously reinforced concrete pavement (CRCP)	Yes	Moderate[2]
Rehabilitation	HMA overlay over existing conventional HMA	Yes	Low
	HMA overlay over exiting full-depth HMA	Yes	Moderate
	HMA over existing PCC (JPCP, jointed reinforced concrete pavement [JRCP], CRCP)✓	Yes	High
	HMA over rubblized PCC✓	Yes	High[3]
	Restored JPCP (existing JPCP subjected to CPR)	Yes	Moderate
	Unbonded JPCP over existing PCC (JPCP, JRCP, CRCP)✓	Yes	High
	Unbonded CRCP over existing PCC (JPCP, JRCP, CRCP)	Yes	Low
	Bonded PCC over existing JPCP	Yes	Low
	Bonded PCC over existing CRCP	Yes	Low
	JPCP over existing HMA surfaced pavement	Yes	Low
	CRCP over existing HMA surfaced pavement	Yes	Low
	JPCP over rubblized PCC	Yes	Low
	CRCP over rubblized PCC	Yes	Low
	Ultrathin whitetopping (Thin JPCP over existing HMA [conventional or full-depth])	No	Moderate[4]

[1]Perpetual HMA (a type of full-depth HMA pavement) is currently under consideration.
[2]CRCP is not a standard PCC design in Missouri.
[3]HMA over rubblized PCC is currently under consideration.
[4]Ultrathin whitetopping is currently under consideration.

Volume I summarizes the entire research effort and documents findings from a review of MEPDG-related literature, an assessment of MoDOT's MEPDG input data needs, MoDOT's laboratory and field testing efforts, sensitivity analysis using MoDOT specific inputs, and model validation and calibration tasks. In addition, Volume I also discusses the steps MoDOT could undertake in the future to fully implement the MEPDG.

Volume II presents a section-by-section comparison of the predicted and measured distresses for MoDOT-specific LTPP and State Pavement Management System sections for HMA pavements and JPCP.

CHAPTER 2. LITERATURE REVIEW AND ASSESSMENT OF MODOT MEPDG-RELATED DATA NEEDS

Even prior to the completion of NCHRP Project 1-37A in 2004, there have been significant efforts by SHAs, transportation organizations, and individual researchers to (1) evaluate the MEPDG design procedure for accuracy and reasonableness, (2) adapt the MEPDG into existing pavement design and evaluation processes, and (3) use the MEDPG for non-structural design type uses such as forensic evaluations and policy decision support. The MEPDG has attracted attention both nationally and internationally.

Several discussion forums, information exchange, regional summits, and training courses have been developed or are in the process of being developed. Some examples include the Federal Highway Administration's (FHWA) Design Guide Implementation Team (DGIT), North Central Region MEPDG User Group (user groups covering other regions of the U.S. are being planned), FHWA Lead States (includes 19 States), and State Pavement Technology Consortium (SPTC) comprising Minnesota, Texas, California, and Washington, Northeast States, Rocky Mountain States. Over two dozen SHAs and some Provinces in Canada have active projects either to evaluate the suitability of MEPDG to their local conditions or to move towards implementing it wholesale. Additionally, AASHTO is preparing to develop commercial grade software to accompany the AASHTO Manual of Practice (AASHTO 2008) by 2011.

A significant body of work is also being sponsored by NCHRP, FHWA, and TRB to (1) improve aspects of the MEPDG which were either found deficient based on initial reviews or which were out of scope of the original study (e.g., reflection cracking, top-down HMA cracking, composite pavements), (2) provide tools to support the MEPDG (e.g., Local Calibration Guide), and (3) implement the MEPDG into existing tools and procedures, such as the Highway Economic Requirements Systems, and Performance Related Specification (PRS) software. Other agencies are pursuing similar goals, with their main focus on implementation support. A summary of some the research projects completed or currently ongoing is presented in Table I-2.

All the aforementioned work has resulted in hundreds of publications exploring various aspects of the MEPDG. Of particular interest to MoDOT's implementation efforts were experiences of agencies and individual researchers related to MEPDG model sensitivity to various input properties and input characterization through laboratory testing or analysis of SHA databases (e.g., traffic climate) to support MEPDG implementation.

To date, sensitivity analysis has been completed and published under contract for Arkansas, California, Florida, Indiana, Iowa, Kansas, Minnesota, Montana, New Jersey, South Dakota, Texas, Utah, Virginia, and Wisconsin. All the sensitivity studies completed and published were performed on nationally calibrated models (NCHRP projects 1-37A, 1-40D). Sensitivity analysis was performed to achieve one or a combination of the following objectives:

Table I-2. Summary of key research studies related to the MEPDG.

Research Objective	Research Title	Agency/Organization
MEPDG review/evaluation	• NCHRP 1-40A – Independent Review of the Recommended Mechanistic-Empirical Design Guide and Software. **Status: Completed.**	NCHRP
Improved performance modeling/design capabilities	• NCHRP 1-42 – Top-Down Fatigue Cracking of Hot Mix Asphalt Layers. Status: Ongoing.	NCHRP
	• NCHRP 9-38 – Endurance Limit of Hot Mix Asphalt Layers to Prevent Fatigue Cracking in Flexible Pavements. **Status: Ongoing.**	NCHRP
	• NCHRP 1-41 – Selection, Calibration, and Validation of a Reflective Cracking Model for Hot Mix Asphalt Overlays. **Status: Ongoing.**	NCHRP
	• NCHRP 9-30A – Rutting Performance Model for HMA Mix & Structural Design. **Status: Ongoing.**	NCHRP
	• SHRP II R21 "Composite Pavement Systems." **Status: Ongoing.**	TRB SHRP II
Development of support tools	• NCHRP 1-39 – Traffic Data Collection, Analysis, and Forecasting for Mechanistic Pavement Design. **Status: Completed.**	NCHRP
	• NCHRP 9-33 – A Mixture Design Manual for Hot Mix Asphalt. **Status: Ongoing.**	NCHRP
Implementation (into new or existing tools)	• NCHRP 9-30(01) – Expand Population of the M-E Database and Conduct Two Pre-Implementation Studies. **Status: Completed.**	NCHRP
	• NCHRP 9-22 – Beta Testing and Validation of HMA Performance Related Specifications. **Status: Ongoing.**	NCHRP
	• Modification of FHWA Highway Performance Data Collection System and Pavement Performance Models **Status: Completed.**	FHWA
	• Adapting Improved Pavement Models To NAPCOM. **Status: Completed.**	FHWA
	• Implementation and Support of New Pavement Equations For Highway Economic Requirements System. **Status: Ongoing.**	FHWA
	• Creation of Reports for Pavements Remaining Service Life Using the Improved Pavement Performance Models Developed for HERS. **Status: Ongoing.**	FHWA
Technology transfer	• FHWA-NHI-131109 Training Course — Analysis of New and Rehabilitated Pavement with M-E Design Guide Software. **Status: Ongoing.**	NHI
	• FHWA-NHI-131064 Training Course — Introduction to Mechanistic Design for New and Rehabilitated Pavements **Status: Ongoing.**	NHI
	• FHWA Design Guide Implementation Team (DGIT) workshops on materials, climate, traffic, local calibration etc. **Status: Ongoing.**	FHWA

- To identify the level of importance for each input parameter.
- To identify the input parameters that can be adjusted during design to satisfy the predetermined pavement performance criteria.
- To check the reasonableness of the model predictions, to identify problems in the software, and to help understand the level of difficulty involved in obtaining the inputs.
- To compare MEPDG with local design approaches.
- To develop design guide implementation plans.

One general observation regarding the findings of the analysis is that seldom have the researchers tried to account for interdependencies between inputs when performing the analysis. This is extremely important to consider for obtaining accurate estimates of model behavior. For example, PCC strength and modulus vary along with other properties such as coefficient of thermal expansion (CTE) and shrinkage. PCC thickness is often linked to dowel diameters in design. HMA air voids, asphalt content and other parameters are interrelated to some degree. Another observation is that interaction effects between materials and climate or between design and materials were not always considered. Such interactions should be based on agency-specific practices and should be factored into sensitivity studies in order to obtain more realistic estimates of model behavior. In this sense, most of the sensitivity analyses reported have very limited applicability and may be misleading.

Determining the hierarchical level at which each input needs to be estimated is in part a policy decision (i.e., it is a function of an agencies operational policy and willingness to devote resources). However, these decisions should also be driven by the expected variability of a given input across a State and the sensitivity of a design method to that particular input. It is in this context that the national literature related to sensitivity of the MEPDG to the various design inputs was deemed useful to the MoDOT study. A summary of the sensitivity of the MEPDG performance models to the procedure's various inputs developed from a review of selected published literature is presented in the following sections, alongside the expected variability of these inputs in Missouri and an assessment of Missouri's current data collection capabilities and future needs.

2.1 TRAFFIC INPUTS

A majority of the literature reviewed focused on the estimation of the axle load spectra, forecasting of traffic volume growth, and seasonal traffic patterns, comparison of estimated load spectra with MEPDG defaults, and sensitivity of the MEPDG models to traffic.

Even though some studies (Timm et al. 2006) showed that the MEPDG traffic axle load and truck class defaults were acceptable, others (Tran and Hall, 2007) showed that there were considerable variations in the data enough to cause substantial deviations in distress and smoothness predictions. Based on the strength of sensitivity studies, a detailed review and preparation of weigh-in-motion (WIM) and automatic vehicle classification (AVC) was considered beneficial in conjunction with the MEPDG implementation efforts (Haider and Harichandran 2007; Al-Yagout et al. 2005; Li et al. 2007). The following salient points were raised in the literature vis-à-vis traffic:

- An analysis of State-wide traffic data (particularly WIM data), after proper quality assurance measures to verify the accuracy of the data are taken, is essential to determine if the MEPDG default inputs are reasonable.
- Duration of traffic data collection had a significant effect on the annual average daily truck traffic (AADTT) data, the hourly truck traffic distribution, and vehicle class distribution.
- Historical traffic data of a minimum of 6 years were necessary to reduce variances in truck volumes.
- Level 1 data are preferred over Level 3 data for critical design situations.
- Models such as rutting in HMA pavements and cracking in JPCP were more sensitive to traffic inputs. Therefore, characterizing traffic at the highest level possible is beneficial for critical design situations.
- MEPDG distress prediction models were found to be very sensitive to overloads present in the load spectra.

Table I-3, Table I-4 and Table I-5 present a detailed summary of all the traffic data/inputs required by the MEPDG along with the recommended input levels for MoDOT to consider using based on the findings of the literature. Based on the strength of the recommendations noted in these tables, the best available traffic data from MoDOT's WIM and AVC sites were analyzed. The findings from this analysis are presented in the next chapter of this report.

2.2 CLIMATE INPUTS

A review of the climatic effects studied showed that the MEPDG distress and smoothness predictions are very sensitive on occasion to climate data (i.e., weather station data) used in modeling (Zaghloul et al. 2006). Other researchers have felt a need to investigate the quality of the weather data and improve it with locally available information (Heitzman 2007). A limited verification of the enhanced integrated climatic model (EICM)—the main climate modeling engine in the MEPDG—showed realistic predictions of moisture contents in unbound materials when compared with field measurements in some studies. Some researchers have found that depth to ground water table (GWT) could have significant impact on flexible distress predictions including top-down cracking, alligator cracking, and rutting.

Missouri has a moderate and humid continental climate; some local and regional variations do exist. The average annual temperature is 50°F in the northwest and about 60°F in the southeast. The heaviest precipitation is in the southeast averaging 48 inches annually. The northwest receives the lightest rainfall, averaging roughly 35 in annually. Snowfall averages 20 inches in the north and 10 inches in the southeast. Due to its location in the interior U.S., Missouri often experiences extremes in temperatures which need to be accounted for in pavement design. However, given these ranges of important key climatic factors, it is not expected that geographic location will influence pavement performance predictions significantly in Missouri.

Table I-3. Summary of traffic volume inputs required for design using the MEPDG.

Parameter (All inputs are mean values)	Significance of Input on Design	Descriptions of Levels 1, 2, & 3 (L1, L2, and L3)	Potential Sources of Data	Recommended Input Level to use for Project Design
AADTT	Major effect	L1: Volumes obtained from AVC or WIM (WIM could be used in place of AVC to obtain truck volume and classification in addition to axle weights) at project site (current or projected historical) L2: (a) Volumes estimated from data at similar sites. (b) Volumes obtained from ATR and estimated % trucks L3: Estimated based on truck volume levels on similar class of highway. (See Note 1)	Extensive past AVC or WIM data collected at many sites around the State as well as traffic volume surveys made over time across the State.	Level 1 to be used for all sites when project performance considered critical (e.g., projects with AADTT higher than 5000) Level 2 or 3 at all other sites
Normalized Truck Volume Classification and Distribution	Moderate effect	L1: Direct from WIM or AVC data measured at the project site or along an adjacent segment without any traffic generators between the measurement and project sites. L2: Estimated from WIM or AVC data measured along roadways in the same region within the same functional classification. L3: State-wide average for the normalized truck volume distributions. These are the state-wide default values determined from an analysis of the WIM and/or AVC data measured over time in Missouri.	Extensive data (W-4 tables) available from previous MoDOT data collection at permanent and temporary WIM and AVC sites, as well as for the LTPP sites. Develop a database of default vehicle volume spectra or distribution for different highway corridors.	Level 1 to be used for all sites when project performance considered critical (e.g., projects with AADTT higher than 5000) Level 2/3 at all other sites, which ever level is available for use in design. The higher input level should be used, however, where and whenever available.
Truck volume monthly adjustment factors	Significant effect on both flexible and rigid pavement design	No levels currently defined.	Historical Missouri WIM and AVC data available. Use historical truck WIM or AVC (volume) data to obtain monthly adjustment factors for different highway corridors and regions as well as site-specific factors Develop a database of default monthly truck traffic volume distribution factors for different highway corridors	Conduct study of Missouri data and use results to derive recommendations.
Hourly truck traffic volume distribution factors	Moderate effect on rigid pavement design	L1: Hourly truck traffic distribution from AVC or WIM at project site L2: Hourly truck traffic distribution estimated from data along similar roadways in the same region L3: State wide Hourly truck traffic volume distribution factors used for class of highway	Historical WIM and AVC Missouri data available. Develop a database of default hourly truck traffic distribution for different highway corridors .	Conduct study of Missouri data and use results to derive recommendations.

AADTT=Average Annual Daily Truck Traffic Two-Directional Base Year

Note 1: Average Annual Truck Traffic; Two-way for the base year. This input parameter is grouped into two types – one for new pavement design projects and the other for rehabilitation and reconstruction projects.

- New Pavement Design Projects. Level 1 input: The input parameter should be estimated from trip generation studies conducted by the traffic department, because no roadway exists for traffic counts to be made. Level 2 and 3 input is the same and come from existing traffic volumes measured on similar roadways (same functional classification) in the same region.
- Rehabilitation/Reconstruction Projects. Level 1 input: The input parameter should be obtained from WIM, AVC, or traffic count data measured along the project site or taken from adjacent roadway segments nearby the project location, as long as there are no traffic generators or deflectors between the data collection site and the project site (intersecting roadways that would change the traffic stream. Level 2 and 3 inputs are the same and come from existing traffic volumes measured on similar roadways (same functional classification) in the same region.

Table I-4. Summary of axle load spectra, lane truck distribution, and directional truck distribution factor inputs required for design using the MEPDG.

Parameter (All inputs are mean values)	**Significance of Input on Design**	**Definition of Levels 1, 2, & 3**	**Potential Sources of Data**	**Recommended Input Level to use for Project Design**
Base year normalized axle load spectra or distributions for single, tandem, tridem, and quadruple axle groups	Moderate effect	L1: Direct WIM measurement on project site or an adjacent segment of roadway.	Extensive data (W-4 tables) available from previous MoDOT data collection at permanent and temporary WIM sites, as well as for the LTPP sites. Develop a database of default axle load spectra for different highway corridors and regions	Level 1 to be used for all sites when project performance considered critical (e.g., projects with AADTT higher than 5000) Level 2/3 at all other sites, which ever level is available for use in design. The higher input level should be used, however, where and whenever available.
		L2: Estimated from WIM data measured along roadways with similar traffic characteristics (similar volume distributions of the major truck types) and in the same functional classification.		
		L3: State wide average normalized axle load distributions used for class of highway. These are the state-wide default values determined from an analysis of the WIM data collected across the state.		
Lane truck distribution (for each design lane)	Minor effect	L1: On site measurements using AVC or WIM data in each lane.	From AVCs of multilane highways if available.	Level 1 to be used for all sites when project performance considered critical (e.g., projects with AADTT higher than 5000) Level 2/3 at all other sites.
		L2: Use of data from other sites with similar truck flow characteristics.		
		L3: Use mean statewide data (as function of number of lanes and ADT)		
Directional truck distribution factor	Minor effect	L1: On site measurements using AVC in each direction.	From AVCs of multilanes and directions if available. Otherwise data likely available from historical records	L1 site specific measurement to be used if site condition appears to have different volumes in each direction. Otherwise use state wide averages.
		L2: Use of data from other sites with similar truck flow characteristics.		
		L3: Use mean statewide data		

Table I-5. Summary of axle load spectra inputs required for design using the MEPDG.

Design Input (All inputs are mean values)	Significance of Input on Design	Definition of Levels 1, 2, & 3	Potential Sources of Data for Missouri	Input Level to use for Project Design
Truck speed	Moderate effect asphalt bound materials only. Slower speeds should be considered for steep upgrades, intersections, and other locations where slow truck traffic exists.	No levels currently defined.	From AVCs with a speed measuring capability. Missouri may have ability to obtain speed data. Otherwise from historical records	Some recommendations exist in MEPDG. Use posted speed for project, unless there are site specific features that reduce the speed of trucks (for example extensive grades on some roadways through mountains.
Truck tire pressure (hot inflated pressure)	Very major effect on flexible pavement design. Moderate effect on rigid pavement design.	No levels currently defined.	MoDOT data or MEPDG defaults	Conduct study and obtain measurements at a few truck weighing stations or enforcement sites. Use median or mean values from these results. If no data becomes available, use 120 psi.
Spacing of tandem, tridem, and quad axles	Minor effect on design.	No levels currently defined.	Extensive data available from previous MoDOT data collection at permanent and temporary WIM sites, as well as for the LTPP sites. Further analysis is required to determine the mean or median values to be used for each axle configuration and to verify the default values currently included in the MEPDG	Use statewide means.
Lateral mean offset of truck dual tires	Very major effect on rigid pavement design. No effect on flexible pavement design.	No levels currently defined.	MoDOT data or MEPDG defaults	Possibly make measurements and use results, or use default values in MEPDG (e.g., 18-in.). Make adjustments for narrowed lane widths.
Standard deviation of lateral offset of truck dual tires	Major effect on both flexible and rigid pavement design.	No levels currently defined.	MoDOT data or MEPDG defaults	Possibly make measurements and use results, or use default values in MEPDG (e.g., 10-in.).
Truck traffic annual growth factor	Major effect on design, especially for long design lives.	No levels currently defined.	Historical WIM and AVC Missouri data available over time.	Provide historical ADT and ADTT data along Missouri highways. Use this data to obtain estimate of growth, but adjust for expected future conditions.

Table I-5. Summary of axle load spectra inputs required for design using the MEPDG, continued.

Design Input (All inputs are mean values)	Significance of Input on Design	Definition of Levels 1, 2, & 3	Potential Sources of Data for Missouri	Input Level to use for Project Design
Mean number axle types per truck for each vehicle class	Moderate effect on design.	L1: Direct WIM measurement on project site (current or historical). L2: Estimated based on WIM measurements on similar roadways within the same region. L3: State wide average used for class of highway.	Historical Missouri WIM data available.	Conduct study of Missouri WIM data and use results to derive recommendations.
Truck axle width, edge to edge	Moderate effect on design rigid pavement design.	No levels currently defined.	MoDOT data or M-E PDG defaults	Conduct study of Missouri data or measurements can be made at some of the enforcement sites and use results to derive recommendations.
Dual tire spacing	Moderate effect on design for flexible pavement design. Minor on rigid design.	No levels currently defined.	MoDOT data or M-E PDG defaults	Use 13 in. without any other data. Measurements can be made at some of the enforcement sites.
Design lane width	Very major effect for rigid pavement.	No levels currently defined.	Designer input for JPCP or CRCP designs. Typically 12 to 14 ft	Use design lane width, normally 12-ft. unless widened lane used, then 14-ft. typical.
Wheel base of truck tractors	Major effect for only JPCP design.	No levels currently defined.	Historical Missouri WIM data	Conduct study of Missouri WIM data and use results to derive recommendations.

Climate-related data from weather stations throughout the U.S. were used in developing site-specific "virtual" weather stations for calibrating the MEPDG performance prediction models. Table I-6 lists the Missouri weather stations for which over 9 years of climate data are available through the MEPDG. Figure I-4 graphically depicts the geographic location of the weather stations listed in Table I-6. Based on the data presented, it appears that the weather stations are a good representation of the climate conditions in Missouri. The need for additional weather stations or higher quality data is ascertained through sensitivity analysis presented in Chapter 4.

GWT depth, a required MEPDG input, is not a part of the weather station data contained in the MEPDG climatic database. Default values will need to be developed as part of implementation based on sensitivity analysis. It was determined at the outset that such data were not easily available from MoDOT records but are available in county soil reports.

Table I-6. Location of MEPDG weather stations in Missouri.

City	State	Location	Latitude	Longitude	Elevation
Cape Girardeau	MO	Cape Girardeau Muni Airport	37.14	-89.34	336
Chesterfield	MO	Spirit of St Louis Airport	38.4	-90.4	488
Columbia	MO	Columbia Regional Airport	38.49	-92.13	881
Jefferson City	MO	Jefferson City Memorial	38.35	-92.1	552
Joplin	MO	Joplin Regional Airport	37.09	-94.3	982
Kansas City	MO	Richard Gebaur	38.5	-94.34	1090
Kansas City	MO	International Airport	39.18	-94.43	1005
Kansas City	MO	Kansas City Downtown Airport	39.07	-94.35	746
Kirksville	MO	Kirksville Regional Airport	40.06	-92.32	961
Poplar Bluff	MO	Poplar Buff Muni Airport	36.46	-90.19	327
Rolla/Vichy	MO	Rolla National Airport	38.08	-91.46	1124
Sedalia	MO	Sedalia Memorial Airport	38.42	-93.11	899
Springfield	MO	Regional Airport	37.14	-93.23	1277
St. Charles	MO	St. Charles Smartt Airport	38.56	-90.26	439
St. Joseph	MO	Rosecrans Memorial Airport	39.46	-94.55	825
St. Louis	MO	International Airport	38.45	-90.22	707
West Plains	MO	West Plains Municipal Airport	36.53	-91.54	1222

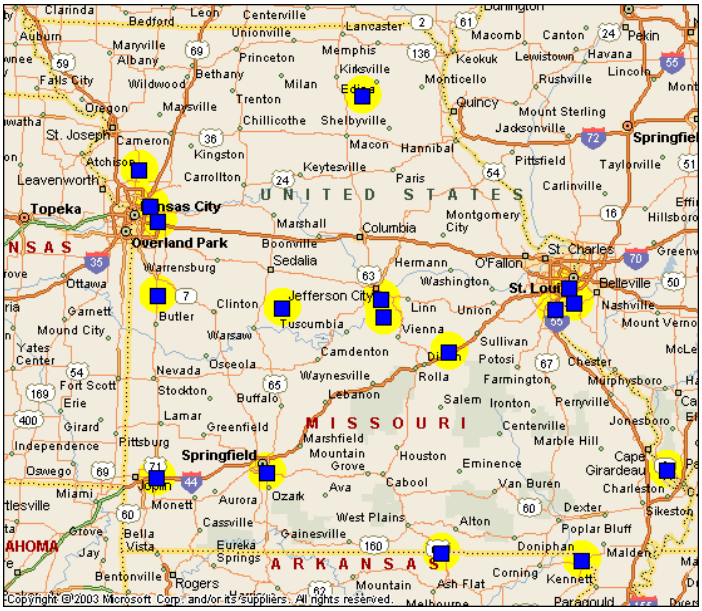


Figure I-4. Missouri map identifying weather stations located in Missouri.

2.3 MATERIAL CHARACTERIZATION FOR NEW PAVEMENT DESIGN

In the MEPDG (ARA 2004), material characterization is required for three purposes: (1) to determine pavement structural responses and response derivatives (e.g., damage), (2) for climatic effects modeling, and (3) for distress predictions (materials inputs are used as empirical factors in some M-E models). Considering the importance of materials inputs and their impact on agency practices, a majority of the MEPDG-related literature to date has focused on materials characterization.

Several types of laboratory and field testing are required to characterize materials for the design of new and rehabilitated HMA and PCC surfaced pavements. The material types of relevance to MoDOT in the MEPDG for new and rehabilitation design fall under six broad categories, as shown in Table I-7. Of these, laboratory characterization of HMA materials has received the most attention in the literature, followed by PCC and unbound materials.

2.3.1 HMA Materials

A majority of the HMA materials-related literature focused on the subject of dynamic modulus |E*| testing. Dynamic modulus is a key material property used in predicting permanent deformation and fatigue cracking of HMA pavement structures. The following key highlights are noted regarding this important material property:

- The MEPDG |E*| predictive equation was reported to provide acceptable estimates of this critical input property at Level 2 (Loulizi et al. 2006; Lundy et al. 2005; Azari et al. 2007). Some researchers found variances in the estimates of |E*| provided by the MEPDG (overestimation) at low loading frequencies or higher temperatures (Azari et al. 2007).

Table I-7. Major material categories.

No.	Material Category	Description	Comments
1	Asphalt materials	Dense graded and open graded Hot Mix AC (HMA)	Commonly used, properties required in MEPDG are generally available.
		Stone matrix asphalt (SMA)	Commonly used, properties required in MEPDG are generally available.
		Polymer modified asphalt (PMA)	New material type for MoDOT, will require laboratory testing to characterize properties for MEPDG.
		Open graded AC stabilized	Seldom used, properties required in MEPDG are generally not available, will require limited laboratory/field testing.
2	PCC materials	Intact PCC	Commonly used, properties required in MEPDG are generally available.
		Fractured PCC---Rubblized	New material type for MoDOT, will require field testing to characterize properties for MEPDG.
3	Chemically stabilized materials	Lime fly ash	Seldom used, will require laboratory testing to characterize properties for MEPDG.
		Lime stabilized/modified soils	Commonly used, will require laboratory testing to characterize properties for MEPDG.
		Open graded cement stabilized materials	Commonly used, will require laboratory testing to characterize properties for MEPDG.
4	Non-stabilized granular base or subbase	Pit runs and large rock	Commonly used, will require field testing to characterize properties for MEPDG.
		Granular material (Type 1 through 5)	Commonly used, will require laboratory testing to characterize properties for MEPDG.
5	Subgrade soil	Clayey sands (A-2-6; A-2-7)	Commonly used, properties required in MEPDG are generally available. Field FWD testing and backcalculation useful.
		Dry-hard, moist stiff, wet/sat-soft low plasticity clays (A-6)	Commonly used, properties required in MEPDG are generally available. Field FWD testing and backcalculation useful.
		Dry-hard, moist stiff, wet/sat-soft high plasticity clays (A-7)	Commonly used, properties required in MEPDG are generally available. Field FWD testing and backcalculation useful.
6	Bedrock	Solid, massive and continuous	Commonly used, will require field testing to characterize properties for MEPDG. Field FWD testing and backcalculation useful.
		Highly fractured, weathered	Commonly used, will require field testing to characterize properties for MEPDG. Field FWD testing and backcalculation useful.

- The Level 2 |E*| predictive equation's accuracy was found to be improved if better characterization of asphalt binder properties was available.
- Conducting Level 1 |E*| testing was recommended for the most critical projects due to the sensitivity of rutting prediction to this parameter.
- For recycled asphalt pavement (RAP) mixes, it was reported that the hierarchical input level used for |E*| and assumed performance grade (PG) binder type were critical in performance predictions.
- Several studies were performed to develop libraries of Level 1 |E*| inputs for typical HMA mixes used by agencies around the country. Some of these studies, which were performed before the AASHTO provisional standards, were finalized for |E*| testing, rendering the data questionable for use with the MEPDG.
- To improve testing accuracy, a test program including four measuring instruments per specimen and two specimens per mix type is recommended for development of |E*| databases using the AASHTO TP 62-03 protocol.

- Based on a laboratory evaluation it was determined that larger aggregates combined with aged materials tend to have high |E*|-values at high temperatures. However, both the |E*|and frequency sweep at constant height (FSCH) tests could not correctly rank the permanent deformation characteristics of the six HMA mixes tested. Test results from the flow number and Hamburg tests were found to correlate fairly well, and both tests were sensitive to the permanent deformation characteristics for the mixtures evaluated. This study points to the deficiency of using only the |E*| test for rutting characterization.

Other HMA material-related studies pointed out some problems with obtaining inputs for rehabilitation design. Alternate means of testing existing pavement layers in the laboratory and correlating those properties with dynamic modulus values were suggested (Loulizi 2006). Correlations were proposed to relate the creep compliance measurements from the indirect tension testing and the bending beam rheometer testing. It was concluded that a mix-specific correlation exists which can be used to predict thermal cracking with approximately the same accuracy (Zofka 2008).

Data required for characterizing the HMA materials at various hierarchical levels (as defined in the MEPDG) and current MoDOT capabilities for testing are presented in Table I-8. The table also includes an assessment of MoDOT's testing needs for each input based on the importance of this parameter determined from literature reviewed including the MEPDG documentation (ARA 2004) after accounting for MoDOT's readiness to commit resources to perform the required testing (as discussed during work scope definition for this project).

Finally, in addition to the HMA mixture and binder properties listed in Table I-8, other HMA mixture characteristics such as surface shortwave absorptivity, thermal conductivity, and heat capacity required for climatic effects modeling also are needed. While it is recognized in the MEPDG that these inputs are extremely important and could have a large impact on the predicted responses and distresses, they have essentially been treated as calibration constants for lack of data to comprehensively determine these values as a function pavement age, materials, and type. Therefore, it is recommended that Level 3 defaults provided by the MEDPG be adopted for MoDOT's use.

2.3.2 PCC Materials

A majority of the PCC materials related literature focused on the CTE testing. PCC mix CTE is a key material property used in predicting transverse cracking and faulting responses of PCC pavement structures. The following key highlights are noted regarding this important material property:

- The CTE was identified as a key input for JPCP design. A comprehensive laboratory test program including 1,800 LTPP test samples conducted by the FHWA revealed that (1) there is no correlation between mean CTE and CTE variability, (2) the results of a single CTE test may not necessarily be representative of the CTE of a mixture due to test variability, and (3) it is important to decrease the test variability by ensuring high testing standards and quality assurance/quality control (QA/QC) in the process of qualifying a mixture (Tanesi et al. 2006).

Table I-8. Testing requirements and corresponding protocols at various hierarchical input levels for HMA materials used in new pavement design.

Materials Category	Measured Property	Recommended Test Protocol	Required for Hierarchical Input Level			Significance of Input to Design	MoDOT's Current Capabilities and Needs
			3	2	1		
Hot-mix asphalt mixture	Dynamic modulus (E*)	AASHTO TP62			X	Moderate	MoDOT has capabilities to perform this test. Develop a database of values for typical mixes based on lab testing for a full range of frequencies and temperatures.
	Tensile strength	AASHTO T322		X	X	High for thermal cracking. However, Missouri does not have a thermal cracking in large parts of the state.	MoDOT has capabilities to perform this test through UMR. Develop a database of values for typical mixes based on lab testing for a full range of frequencies and temperatures.
	Creep Compliance	AASHTO T322		X	X	High for thermal cracking. However, Missouri does not have thermal cracking in large parts of the state.	MoDOT has capabilities to perform this test through UMR. Develop a database of values for typical mixes based on lab testing for a full range of frequencies and temperatures.
	Mixture gradation	AASHTO T27	X	X		High. Mix gradations are used to determine mix stiffness at Levels 2 and 3.	MoDOT routinely tests this input. Large catalog of mix design information available which needs to be cataloged for use in design.
	Mixture volumetrics (as-built) : Effective asphalt content Air voids	AASHTO T308 AASHTO T166 & T209	X	X	X	High. All HMA distresses are affected.	MoDOT routinely tests for effective asphalt by weight. Need to develop a correlation between this and effective asphalt by volume for various mixes. Develop database of as-built air voids from density testing. Set a Level 3 default for air voids.
	Penetration at 77°F	AASHTO T49	X			Low.	MoDOT routinely tests for this value. Use historical data to set Level 3 defaults.
	Unit weight	AASHTO T166	X	X	X	Low.	MoDOT routinely tests for this value. Use historical data to set Level 3 defaults.
	Poisson's ratio		X	X	X	Low.	Use MEPDG predictive model or a typical value at Level 3.

Table I-8. Testing requirements and corresponding protocols at various hierarchical input levels for HMA materials used in new pavement design, continued.

Materials Category	Measured Property	Recommended Test Protocol	Required for Hierarchical Input Level			Significance of Input to Design	MoDOT's Current Capabilities and Needs
			3	2	1	.	
Asphalt binder	Asphalt binder complex shear modulus (G*) and phase angle (δ)	AASHTO T315				High for thermal cracking predictions and Moderate for rutting predictions.	Although MoDOT has capabilities to test for this parameter, it often conducts this testing in specification compliance testing mode and does not perform the full frequency sweep. Need to develop a database of values for typical binders.
	OR						OR
	Conventional binder test data: Penetration	AASHTO T49		X	X		MoDOT has capabilities to perform this test. However, it is not recommended for new designs.
	OR	OR					OR
	Ring and Ball Softening Point	AASHTO T53					MoDOT has capabilities to perform this test. However, it is not recommended for new designs.
	Absolute Viscosity	AASHTO T 202					
	Kinematic Viscosity	AASHTO T201					
	Specific Gravity	AASHTO T228					
	OR	OR					OR
	Brookfield Viscosity	AASHTO T316					MoDOT has capabilities to perform this test. Can be used in lieu of G*-δ data.

- CTE test programs with local aggregates were recommended by some researchers (Tran et al. 2008).
- Coarse aggregate type was recognized as a primary factor affecting CTE by a number of researchers based on AASHTO TP 60 testing (Mallela et al. 2005; Won, 2005; Tanesi 2006; Buch et al. 2008).
- There was conflicting information from literature on the importance of age on CTE testing. Some researchers (Won 2005) state that specimen age at testing has no significant effect, while others found that it does have a significant impact (Buch et al. 2008).
- Some researchers noted the need to improve the CTE test protocol suggested in the MEPDG (AASHTO TP 60) (Won, 2005).

Hall and Beam (2005) found that the unit weights of concrete have a significant effect on rigid pavement performance. Surprisingly few other properties were noted in the literature as being important from a model sensitivity perspective. One study (Kannekanti and Harvey, 2006) showed that the surface shortwave absorptivity of concrete was an important input. However, as stated earlier for HMA materials, MEPDG defaults are recommended for this property and other thermal properties (heat capacity and thermal conductivity) until such a time more information becomes available.

Data required for characterizing the PCC materials at various hierarchical levels (as defined in the MEPDG) and current MoDOT capabilities for testing are presented in Table I-9. The table also includes an assessment of MoDOT's testing needs for each input parameter after accounting for MoDOT's readiness to commit resources to perform the required testing. The importance of input parameters was determined from the review of literature that includes the original MEPDG documentation (ARA 2004).

2.3.3 Unbound Materials

This category covers both granular bases and subgrade materials. One of the primary inputs for unbound materials to the MEPDG procedure is resilient modulus (M_r), the characterization and use of which have has long been debated in the pavement community. M_r is influenced by the imposed stress state as well as soil moisture. While theoretically the MEPDG currently has capabilities to account for both at Level 1, this input level is not recommended for use at this time. To account for the dependence of M_r on stress state requires a computational effort that is considered prohibitive at this time, and the prediction models have not been calibrated or tested at this level.

Regardless, there have been efforts made (Ping and Liu 2007; Titi et al. 2006) to characterize M_r at MEPDG input Level 1 using the protocols recommended by the procedure. These studies have developed predictive relationships to estimate M_r as a function of the applied stress state using a variety of model forms, including the Universal Constitutive Model described in the MEPDG documentation (ARA 2004).

Other literature discussing the default M_r values published in the MEPDG software as a function of soil class was conflicting. For example, one study (Khogali and El Hussein 2007) noted

Table I-9. Testing requirements and corresponding protocols at various hierarchical input levels required for PCC materials used in new pavement design.

Materials Category	Measured Property	Recommended Test Protocol	Hierarchical Input Level			Significance of Input to Design	MoDOT's Current Capabilities and Needs
			3	2	1		
Portland cement concrete mixture	Elastic modulus (chord modulus)	ASTM C469	X[1]		X	High. Affects stress computations and hence cracking predictions in JPCP and punchouts in CRCP.	MoDOT has capabilities to test for all PCC properties. It is recommended that a library of inputs for all these properties be developed from a single batch of material for typical concrete mixes in Missouri (representing typical cements, supplementary cementitious materials, chemical admixtures, aggregate gradations, and water contents). Testing to be performed at various times 7, 14, 28, 90, and 180 days.
	Flexural Strength	AASHTO T97	X[1]		X	High. Affects damage computations and hence predicted cracking in JPCP and punchouts in CRCP.	
	Indirect tensile strength (CRCP only)	AASHTO T198			X	High. Affects punchout development in CRCP.	
	Compressive strength	AASHTO T22	X[1]	X		High. Needed only to estimate other PCC strength and modulus inputs.	
	Coefficient of thermal expansion (CTE)	AASHTO TP60	X			High. Affects all PCC distresses.	
	Shrinkage (Ultimate shrinkage and time to achieve 50% of Ultimate)	AASHTO T160	X	X	X	Moderate. Affects all PCC distresses, especially top-down cracking	
	Unit weight	AASHTO T121	X	X	X	Moderate. Affects faulting in JPCP.	Use historical (Level 3) unit weights based on MoDOT database.
	Poisson's ratio	ASTM C469	X	X	X	Low.	Average values obtained from laboratory testing of modulus and use it as Level 3 default.

problems with adopting the resilient modulus (M_r) values proposed in the software, as these values were generally greater than expected resulting in an underestimation of the required structure. Another more comprehensive evaluation of M_r (Khazanovich et al. 2006) found that the MEPDG default M_r recommendations for various soil types, when interpreted correctly (i.e., after taking into account stress dependency and soil moisture), appear to be reasonable.

While laboratory characterization of M_r appears to be an important part of the MEPDG implementation, interpretation of the test data, and its ultimate use with the MEPDG requires additional work.

For all the attention received by this input in literature, M_r is only one of the input parameters used to characterize unbound materials in the MEDPG and is a required input at all levels (except at Level 2, M_r can be estimated from other commonly tested soil parameters such as the California Bearing Ratio [CBR], the Dynamic Cone Penetration [DCP] based penetration index, AASHTO or Unified Soil Classification [UCS] soil class, etc). In addition, soil gradation, Atterberg limits (plasticity index and liquid limit), soil suction, drainage characteristics, unit weight, and gravimetric moisture content are also needed for modeling climatic effects on M_r.

Table I-10 and Table I-11 lists the various unbound material properties needed at each of the MEPDG hierarchical input levels along with commentary on current MoDOT capabilities for determining these parameters in the laboratory. The table also includes an assessment of MoDOT's testing needs for characterizing unbound materials after accounting for MoDOT's readiness to commit resources to perform the required testing. The importance of input parameters was determined from the review of literature that includes the original MEPDG documentation (ARA 2004).

2.4 MATERIAL CHARACTERIZATION FOR REHABILITATED PAVEMENT DESIGN

Many studies have looked into materials characterization for the MEPDG. Arguably, this is one of the more contentious aspects of the design procedure, but perhaps the most important one for SHAs looking to rehabilitating their aging infrastructure. For MoDOT, however, the application of the MEPDG for rehabilitation designs is primarily to design unbounded PCC overlays of existing PCC pavements and HMA overlays of rubblized PCC pavements. The application of the MEPDG for these rehabilitation types is more straightforward in terms of characterization of in situ material properties, as explained in the following paragraphs.

HMA overlays of rubblized pavements are very similar to new HMA pavement design, with the exception that rubblized layer moduli need to be estimated to run the MEPDG. There have been a number of studies done nationally to characterize the M_r of rubblized layers from which a Level 3 type guidance can be derived for use by MoDOT. In addition, the MoDOT has four LTPP SPS-6 supplemental sections which can be used to verify the defaults established from other national studies.

Table I-10. Testing requirements and corresponding protocols at various hierarchical input levels required for unbound materials used in new pavement design.

Material Category	Measured Property	Recommended Test Protocol	Hierarchical Input Level			Significance of Input to Design	MoDOT's Current Capabilities and Needs
			3	2	1		
Unbound materials	Regression coefficients k_1, k_2, k_3 for the generalized constitutive model[1] that define resilient modulus as a function of stress state[2]	AASHTO T307 or NCHRP 1-28A			X	Very important. However, this input level (Level 1) is not recommended for use at this time.	Done
	Resilient modulus (M_r): M_r at optimum moisture (OMC) and maximum dry density (MDD) or design value	AASHTO T307 or NCHRP 1-28A				Low to moderate depending on pavement thickness. However, low to moderate for typical pavements for which the MEPDG will be used in MoDOT.	MoDOT currently does not have capabilities to test for Mr. However, it is desirable to develop a database of values for typical bases and subgrade soils.
	OR California Bearing Ratio (CBR)	OR AASHTO T193				OR Used to compute M_r. Therefore significance is the same as for M_r	OR MoDOT has ability to test for CBR. Develop database of historical values for use with MEPDG.
	OR R-value	OR AASHTO T190		X		OR N/A	OR N/A
	OR Gradation and Atterberg limit parameters	OR AASHTO T27, T89 & T90				OR Used to compute M_r. Therefore significance is the same as for M_r	OR MoDOT has ability to test for gradation and soil index properties. Develop database of historical values for use with MEPDG.
	OR Dynamic Cone Penetrometer (DCP)	OR ASTM D6951				Used to compute M_r. Therefore significance is the same as for M_r	OR MoDOT routinely tests in place materials using the DCP. Develop database of historical penetration index values for various soils for use with MEPDG.

[1]See section 2.2.5.1 of the M-E PDG Part 2, Chapter 2.
[2]Level 1 inputs for M_r are not recommended for use with the MEPDG at this time.

Table I-11. Testing requirements and corresponding protocols at various hierarchical input levels required for unbound materials used in new pavement design.

Material Category	Measured Property	Recommended Test Protocol	Hierarchical Input Level			Significance of Input to Design	MoDOT's Current Capabilities and Needs
			3	2	1		
	MDD & OMC: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T99 or T180 OR AASHTO T27, T89 & T90		X	X	High. It affects the change of M_r over time.	MoDOT has ability to test for maximum dry density and optimum moisture content..
	Specific gravity: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T100 OR AASHTO T27, T89 & T90		X	X	Low.	MoDOT has ability to test for specific gravity.
	Saturated hydraulic conductivity: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T215 OR AASHTO T27, T89 & T90		X	X	Moderate.	MoDOT has ability to test for saturated hydraulic conductivity.
	Soil water characteristic curve parameters: Direct Testing OR Estimated from MDD, OMC, gradation, and Atterberg limits	Pressure plate, filter paper, and/or Tempe cell testing; AASHTO T99 or T180; AASHTO T100 OR AASHTO T180 or T99 AASHTO T100; AASHTO T27 AASHTO T90 OR AASHTO T27 AASHTO T90	X	X	X	Moderate.	MoDOT has ability to obtain SWCC parameter through the facilities at a local University.

Unbonded PCC overlays of existing PCC pavement performance largely depend on the properties and design features of the overlay and structural condition of the existing pavement. The MEPDG provides the best available guidance to ascertain the damage in the existing layers through visual condition surveys and either nondestructive testing using the falling weight deflectometer (FWD) or limited coring and testing. This guidance can be adopted as MoDOT standard guidance for this design type.

CHAPTER 3. TRAFFIC AND LABORATORY MATERIALS CHARACTERIZATION

3.1 INTRODUCTION

Based on the needs assessment vis-à-vis the inputs required to implement the MEPDG in Missouri, traffic and materials inputs were shortlisted as being of the highest importance. To better define these inputs, MoDOT embarked on three critical tasks:

1. Analysis of the best available WIM data from around the State.
2. Laboratory characterization of typical MoDOT HMA, PCC, dense graded aggregate base materials, and soils.
3. Field testing of in situ pavements, primarily to support the calibration effort.

The objectives of these tasks, which consumed a considerable amount of agency resources and time, were two-fold: (1) to establish libraries of typical values for use in sensitivity studies, local calibration, and routine design, and (2) to compare MoDOT-specific values with global defaults to identify similarities, deviations, and their underlying causes.

This chapter describes the work performed to develop these inputs, the major findings from the data analysis, data storage and planned uses.

3.2 TRAFFIC STUDIES

3.2.1 Overview

The Traffic Monitoring and Analysis group of the Transportation Planning Department at MoDOT is in charge of the traffic data collection, analysis, and reporting in Missouri. This group monitors and reports on traffic data on not just the highway system but also roads and streets. The Traffic Monitoring System (TMS) is used to store and analyze vehicular traffic data. FHWA's Traffic Monitoring Guide (TMG) and the AASHTO Guide for Traffic Data Programs form the basis for MoDOT's TMS. The system is required to collect the following:

- Rural and Urban Interstate, Freeways and Expressways.
- Principal and Minor Arterials.
- Collectors on and off the State systems.

The TMS provides the following traffic information:

- Historical, current (1985-present) and projected traffic volumes.
- Vehicle classifications.
- Truck weights.
- Vehicle miles traveled.
- Travel trends.
- Annual speed monitoring information.
- Various daily, monthly, and annual reports.

For the segments of most interest to this study—the State highway system and other functionally classified roadways—MoDOT's routine data collection plan includes:

- Automatic Traffic Recorders (ATR) located at continuous count sites (operating 24 hours a day, year-round). There are over a 100 permanent counters with approximately 60 percent of them being on the NHS segments. This includes 11 permanent WIM sites (9 of which are part of the LTPP program) and 27 portable WIM counters (of which 7 are a part of the LTPP program) collecting vehicle count, speed, class, and truck weight data. All the WIM sites in Missouri are calibrated once a year.
- The Branson Traveler Roadside Information Program (TRIP) which provides continuous count information from 10 sites in Branson area.
- Portable short-term counts (48-hour counts) collected at over 10,000 locations statewide on a 3-year cycle. AVC data are collected at over 625 of these sites.

Figure I-5 and Figure I-6 present maps showing the locations of Missouri's vehicle classification sites and continuous WIM sites including Pre-Pass sites (used for enforcement purposes). Figure I-7 presents a breakdown of Missouri's continuous ATR traffic data collection program by count, class, and WIM sites.

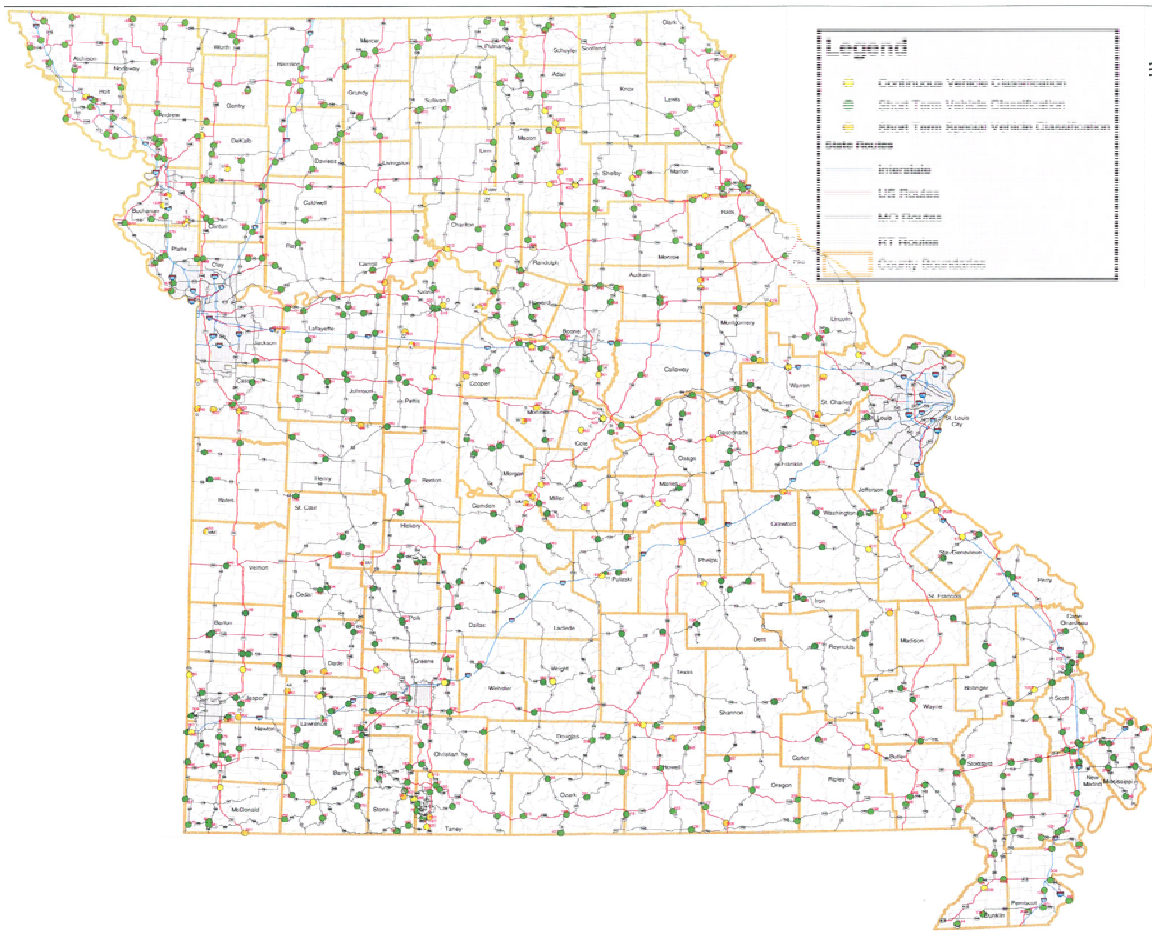


Figure I-5. MoDOT vehicle classification sites.

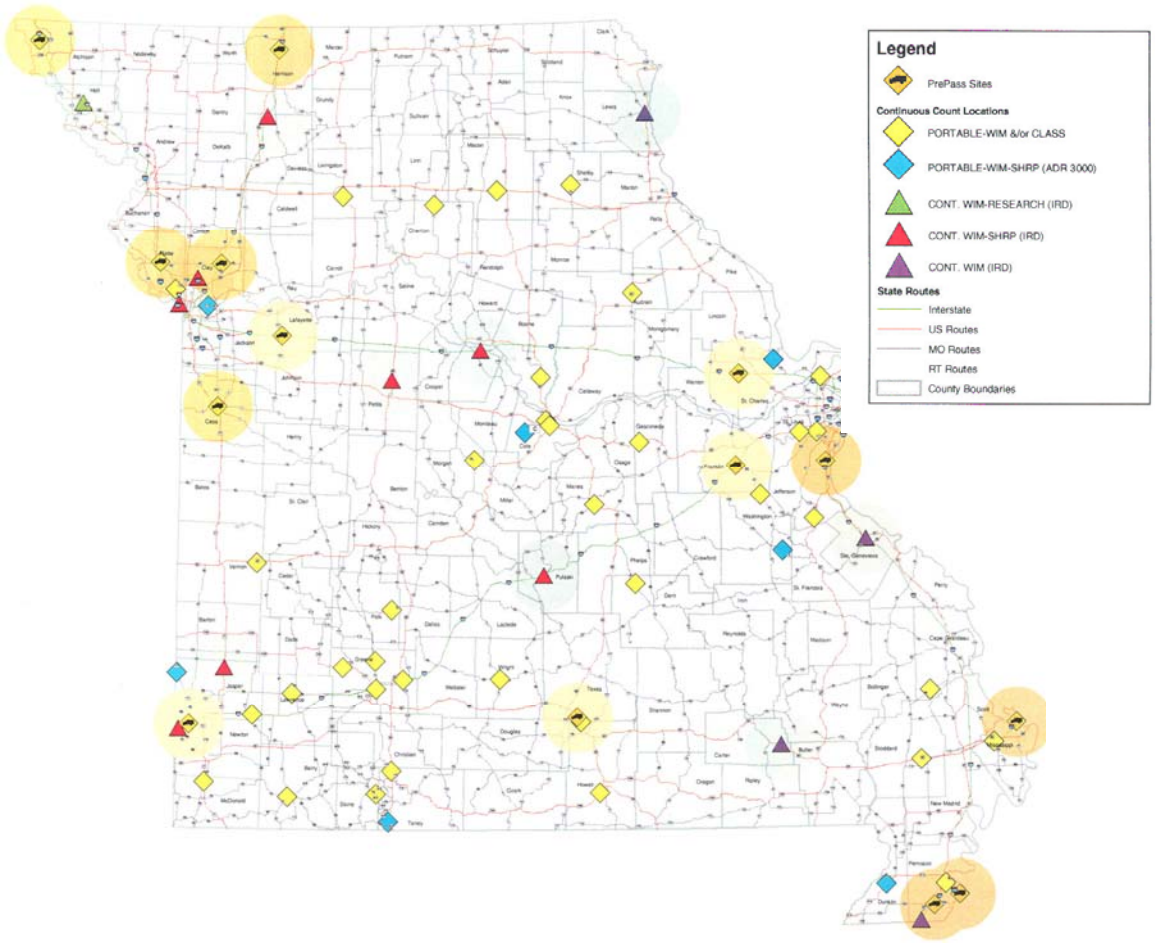


Figure I-6. MoDOT continuous traffic monitoring sites (WIM locations & Pre-Pass sites).

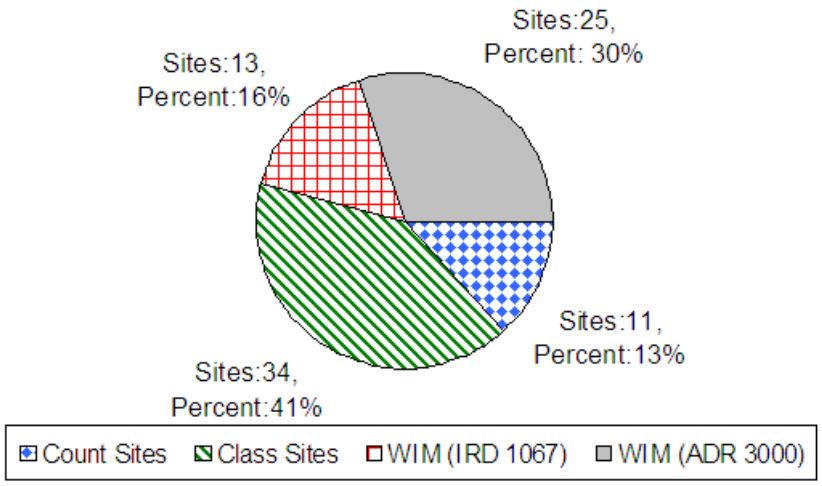


Figure I-7. Breakdown of MoDOT ATR sites by count, class, and WIM type sites.

3.2.2 Dataset Used in the Analysis

For the purposes of this study, only data from the WIM-IRD sites shown in Figure I-6 and Figure I-7 were analyzed (12 of the 13 permanent WIM sites were selected; one of the sites was installed in 2004 and did not have much data). Nine of the sites selected were LTPP sites. The WIM data at these locations were collected using the International Road Dynamics, Inc. IRD 1067 equipment with piezo-loop-piezo sensor configurations. These sites were selected for the following reasons:

- They represent the best possible data available in Missouri for MEPDG analysis purposes (continuous ATR sites with WIM data).
- They adequately represent the highway functional classes over which the MEPDG is planned to be utilized in Missouri.

The sites, the routes, and functional classes they represent are listed in Table I-12. As can be noted from the table, the data analyzed primarily represents rural interstates and U.S. or primary highways.

3.2.3 Data Analysis Objectives and Approach

Raw traffic data were obtained from MoDOT for the period spanning 1996 to 2002 for each of the WIM sites investigated. These data were processed and analyzed to:

- Compare the Missouri-specific traffic data with the traffic defaults in the MEPDG software to either confirm or reject the defaults.
- Develop local traffic input defaults from the processed historic traffic data.
- Produce traffic inputs required by the MEPDG software for routine use.

The data analysis was facilitated by ARA's Advanced Traffic Loading and Analysis System (ATLAS) software. The essential features of ATLAS which were used in this work included:

- A traffic import module which was used to read the W-4 axle weight data.
- A traffic export module to create site-specific MEPDG traffic input files (vehicle classification, axle load distribution, monthly adjustment factors, 24-hour truck counts, and growth rates based on historical data), a reference library database, and ESAL estimates.
- A data analysis module to perform graphical user interface driven quality checks on the historical traffic data, perform data filtering on a site-by-site basis and by truck class.

Table I-12. Identification of MoDOT WIM sites analyzed to develop traffic input defaults.

Site 1D	SHRP/ LTPP ID	WIM data collected in One or both Directions?	Functional Class	ROUTE	Type Count	WIM Lanes	Date Installed	Comment 1	Comment 2
182	N/A	NB	Rural Principal Arterial Interstate (RPA-I)	IS 29	WIM(Res)	both2,4	Sep-99	MOUND CITY	3.2 MI. N/O MO 118 (Research)
182	N/A	SB	Rural Principal Arterial Interstate (RPA-I)	IS 29	WIM(Res)	both2,4	Sep-99	MOUND CITY	3.2 MI. N/O MO 118 (Research)
188	5000	NB	Rural Principal Arterial Interstate (RPA-I)	IS 35	WIM(S)	both2,4	Sep-92	DAVIESS CO	2.0 MI. N/O RT B
188	0600	SB	Rural Principal Arterial Interstate (RPA-I)	IS 35	WIM(S)	both2,4	Sep-92	DAVIESS CO	2.0 MI. N/O RT B
420	4036	NB	Urban Principal Arterial Interstate (UPA-I)	IS 435	WIM(S)	2(NB)	Jul-92	KANSAS CITY	1.0 MI. N/O 108 ST
440	4069	SB	Urban Principal Arterial Interstate (UPA-I)	IS 635	WIM(S)	4(SB)	Oct-92	KANSAS CITY	1.0 MI. N/O KANSAS STATE LINE
441	0900	SB	Rural Principal Arterial Other (RPA-O)	US 65	WIM(S)	4(2)	Jul-95	SEDALIA	3.0 MI. N/O RTS H & HH
500	5473	WB	Rural Principal Arterial Interstate (RPA-I)	IS 70	WIM(S)	both2,4	Jul-92	BOONVILLE	0.5 MI. E/O MO 87
740	5503	SB	Rural Principal Arterial Other (RPA-O)	US 71	WIM(S)	4(SB)	Jul-92	JOPLIN	1.6 MI. S/O RTS H & K
760	7054	EB	Rural Principal Arterial Interstate (RPA-I)	IS 44	WIM(S)	2(EB)	Jul-91	JOPLIN	0.3 MI. W/O SCALES
930	1010	EB	Rural Principal Arterial Interstate (RPA-I)	IS 44	WIM(S)	2(EB)	Oct-92	WAYNESVILLE	1.5 MI. W/O RT H

The following procedure was used to analyze the traffic data:

1. Processed the raw historic traffic data (w-card format) files using the ATLAS software.
2. Analyzed the historic data for each site, including performing QC checks on all the collected data and performing analysis to create the required MEPDG inputs (load spectra by truck class and axle type, vehicle distribution, monthly adjustment factors, hourly distribution factors, etc.).
3. Compared the processed traffic data from each site to the national default values. For example, in this step, the class data from each site was reviewed and assigned a Truck Traffic Classification (TTC) group as defined in the MEPDG. Subsequently, the processed historic traffic data at each site were compared against the national defaults for that group. Similarly, the axle weight data from each step were compared to the national defaults.
4. Performed an analysis of variance in weight distributions by road categories—heavy or medium duty pavements. MoDOT defines a heavy duty pavement as one which receives more than 50,000,000 ESALs accumulated in the design lane of a rigid pavement over a period of 35 years. A medium duty pavement is one which receives less than 50,000,000 ESALs accumulated in the design lane of a rigid pavement over a period of 35 years.
5. Recommended Missouri MEPDG traffic input defaults.
6. Provided recommendations to MoDOT regarding future traffic data collection requirements for MEPDG implementation.

3.2.4 Summary of Findings

The following subsections present key findings of the WIM data analysis. Site-by-site analysis is presented in appendix A.

Data Availability

An important aspect of analyzing WIM data is the adequacy of data. Figure I-8 and Figure I-9 provide a summary of the availability raw data by year and by average number of months within a year for each site analyzed. It can be seen from the figures that adequate WIM data are available at these sites for developing MEPDG inputs, provided the data is good quality.

Data Quality

An important aspect of the traffic data analysis includes reviewing the data quality. Given the volume of information that usually needs to be handled in traffic data analysis, data QC becomes a tedious task. The ATLAS software's visual QC interface was used to review year-to-year variations in traffic vehicle class and load spectra at a given site to identify potential outliers. A data quality rating scheme was adopted and applied to shortlist the most suitable dataset for further analysis. The scheme assigns a quality rating on a scale of 1 to 5 (with 1 representing the poorest quality and 5 the best quality of data) to each site analyzed. The quality ratings were based on consistency of multi-year class and load spectra information after filtering out obvious (presumed) outliers. The rating scale is explained below:

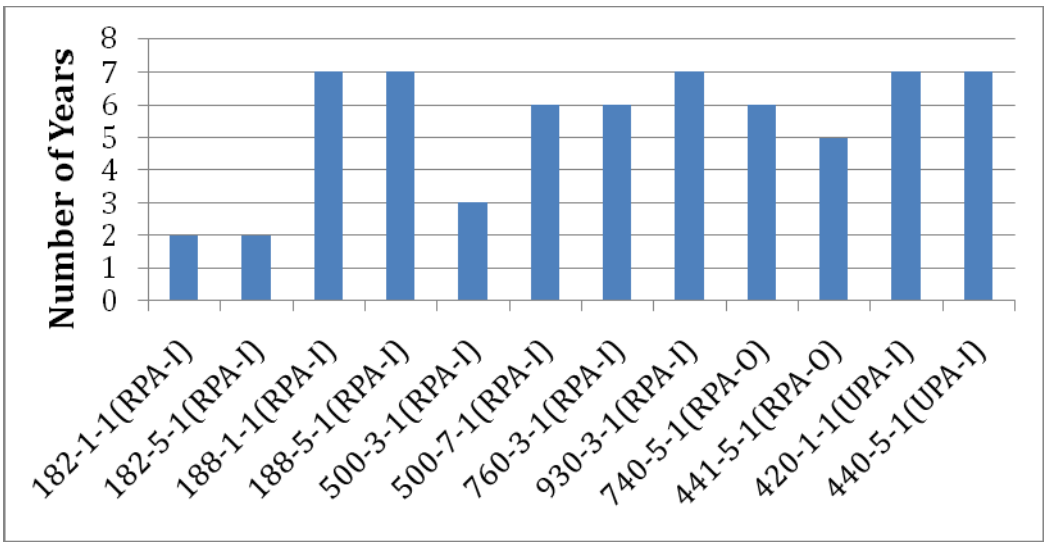


Figure I-8. Number of years of continuous WIM data at each MoDOT site evaluated.

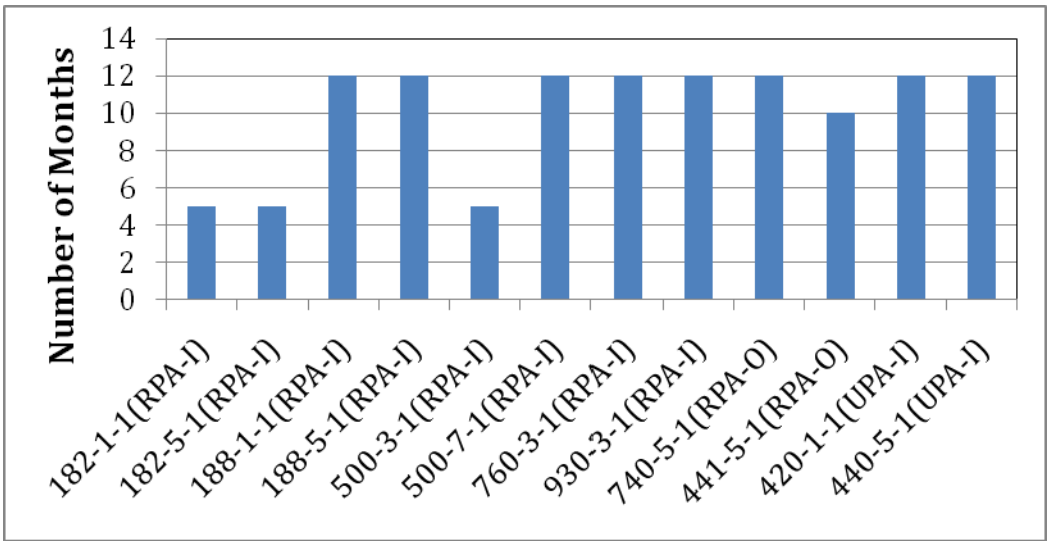


Figure I-9. Average number of months of continuous WIM data at each MoDOT site evaluated.

- Quality rating 5 – Very good; consistent and clean year-to-year trends in class and weight data.
- Quality rating 4 – Good; reasonable, consistent year-to-year trends in class and weight data.
- Quality rating 3 – Average; reasonable trends in class and weight data after considerable filtering of poor quality data.
- Quality rating 2 – Poor; data questionable and contains some misclassifications.
- Quality rating 1 – Very poor; significant amounts of data are questionable with several misclassifications.

Table I-13 presents the quality ratings of the various WIM sites analyzed as part of this study. As can be noted, the sites shortlisted for analysis generally received a rating greater than 3, which is considered adequate to be used for further analysis.

Analysis of Vehicle Class Distribution Data

The MEPDG software provides two methods for entering vehicle classification distribution—site-specific (Level 1) or regional (Level 2) values, or default vehicle class distributions (Level 3) available with the MEPDG software. Selection of the default distributions is facilitated through the provision of 17 sets of class distributions covering a wide range of traffic mixes. The TTC groups were developed based on class data drawn from the LTPP database. Table I-14 presents the percentage of each truck type expected in each TTC group. Guidance for allocating a given traffic mix to one of the TTC groups is provided in Table I-15. As can be noted from this table, the principal factors in selecting a TTC group are the percent buses, single trailer single unit trucks, and multi-trailer trucks.

To compare MoDOT-specific data with national defaults, the vehicle class distributions at each MoDOT site after averaging acceptable yearly distributions were assigned a TTC group based on the criteria presented in Table I-15. The resulting TTC group assignment on a site-by-site basis is presented as follows:

- TTC 1
  - 182-1-1 (RPA-I), 182-5-1 (RPA-I), 188-1-1 (RPA-I), 188-5-1 (RPA-I), 500-3-1 (RPA-I), 500-7-1 (RPA-I), 760-3-1 (RPA-I), 930-3-1 (RPA-I), 740-5-1 (RPA-O)
- TTC 2
  - 441-5-1 (RPA-O), 420-1-1 (UPA-I)
- TTC 6
  - 440-5-1 (UPA-I)

As can be noted, a majority of the sites have a vehicle class distribution that matches the description of TTC 1 followed by TTC 2. This is not surprising, considering that a majority of the WIM sites analyzed were situated on Interstate pavements and the MEPDG notes that TTC groups 1 or 2 are appropriate for such pavement types. Class 9 trucks followed by Class 5 trucks dominate the vehicle types represented in these TTC groups. The one site classified as having a vehicle class distributions matching TTC 6 is located on I-635 in Kansas City on an urban principal arterial. The higher percentage of Class 5 trucks for this TTC group when compared to TTC groups 1 or 2 seems appropriate for this location.

Based on the discussion presented, it appears that the truck distribution patterns analyzed from each of the MoDOT WIM sites evaluated adequately represent the functional class of the pavement on which they are situated. Next, the truck distributions from each of these sites were compared with the MEPDG default distributions after the sites were sorted into their respective TTC groups. Figure I-10 through Figure I-12 present the comparisons of vehicle class distributions for TTC groups 1, 2, and 6. A statistical analysis of the data (not presented here), which consisted of running a paired t-test between a given class distribution and the MEPDG default for the corresponding TTC group, resulted in an acceptance of the null hypothesis (H_0). In other words, there is no statistically significant difference between the MoDOT-specific

Table I-13. Data quality rating and comments on a per site basis (highlighted rows shaded in grey are shortlisted for analysis).

Site ID	Number of Years of Data	Max. Months of Data in a Year	Max. Days of Data in a Year	Quality Rating	Overall comment on Spectra	Spectra category (by tandem axle peaks)
182-1-1	2	5	121	5	Very good. No much variation monthly either	Loaded, consistent monthly & yearly (2000 vs. 2001)
182-5-1	2	5	128	5	Very good.	Loaded, some variation by month
188-1-1	7	12	361	5	Very good (except 98--suspect)	Varies from month to month
188-5-1-	7	12	361	5	Very good using 96, 97, 02 data (98, 99, 00, 01 not good); Great example. for yearly data QC.	Varies from month to month
441-5-1	5	12	333	5	Very good. (exclude 2001--suspect)	Unloaded higher
500-3-1	3	5	112	5	Very good/clean consistent.	Loaded
500-7-1	6	12	365	5	Very good if using only 2001 & 2002 data (98' etc. perhaps bad).	Loaded higher (with fair amount unloaded)
930-3-1	7	12	340	5	Very good. (Used only 00,01,02 data)	Loaded - good, consistent data; not much monthly variation
420-1-1	7	12	352	4	Consistent for 00, 01, 02- Problems 96 to 99; but very light spectra.	Atypical: Lightly Loaded (Differ from all other sites)
470-5-2	1	3	59	4	Good; only 3 winter months data available.	Loaded
760-3-1	6	12	336	4	Good; Excluded 96,99 02--6,7,8,9	Loaded (excld 99' & 01) Not vary too much by months
440-5-1	7	12	362	3	Okay if using only 01 & 02 data (still some variation yearly for some months)	Unloaded
470-5-1	1	10	185	3	Okay after deleting bad months (7,8,10,11); 1-year: Winter/spring good	Loaded higher (with fair amount unloaded)
555-7-1	4	12	358	3	Okay; Heavy and with overloads; varies by month & year	Unloaded, but Heavy!!
658-1-1	5	12	304	3	Okay; (2001 excluded, Aug-96, Nov-98 bad)	Balanced, varies by months
740-5-1	6	12	344	3	Okay (varies a lot yearly; If desired, may use only 2002 data	Balanced, varies by months

Table I-14. Vehicle class distribution for each TTC group in the MEPDG software.

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

Table I-15. Definitions and descriptions for the TTC groups.

Buses in Traffic Stream	Commodities being Transported by Type of Truck		TTC Group No.
	Multi-Trailer	**Single-Trailers and Single-Units**	
Low to none (<2%)	Relatively high amount of multi-trailer trucks (>10%)	Predominantly single-trailer trucks	5
		High percentage of single-trailer trucks, but some single-unit trucks	8
		Mixed truck traffic with a higher percentage of single-trailer trucks	11
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	13
		Predominantly single-unit trucks	16
	Moderate amount of multi-trailer trucks (2-10%)	Predominantly single-trailer trucks	3
		Mixed truck traffic with a higher percentage of single-trailer trucks	7
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	10
		Predominantly single-unit trucks	15
Low to moderate (>2%)	Low to none (<2%)	Predominantly single-trailer trucks	1
		Predominantly single-trailer trucks, but with a low percentage of single-unit trucks	2
		Predominantly single-trailer trucks with a low to moderate amount of single-unit trucks	4
		Mixed truck traffic with a higher percentage of single-trailer trucks	6
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	9
		Mixed truck traffic with a higher percentage of single-unit trucks	12
		Predominantly single-unit trucks	14
Major bus route (>25%)	Low to none (<2%)	Mixed truck traffic with about equal single-unit and single-trailer trucks	17

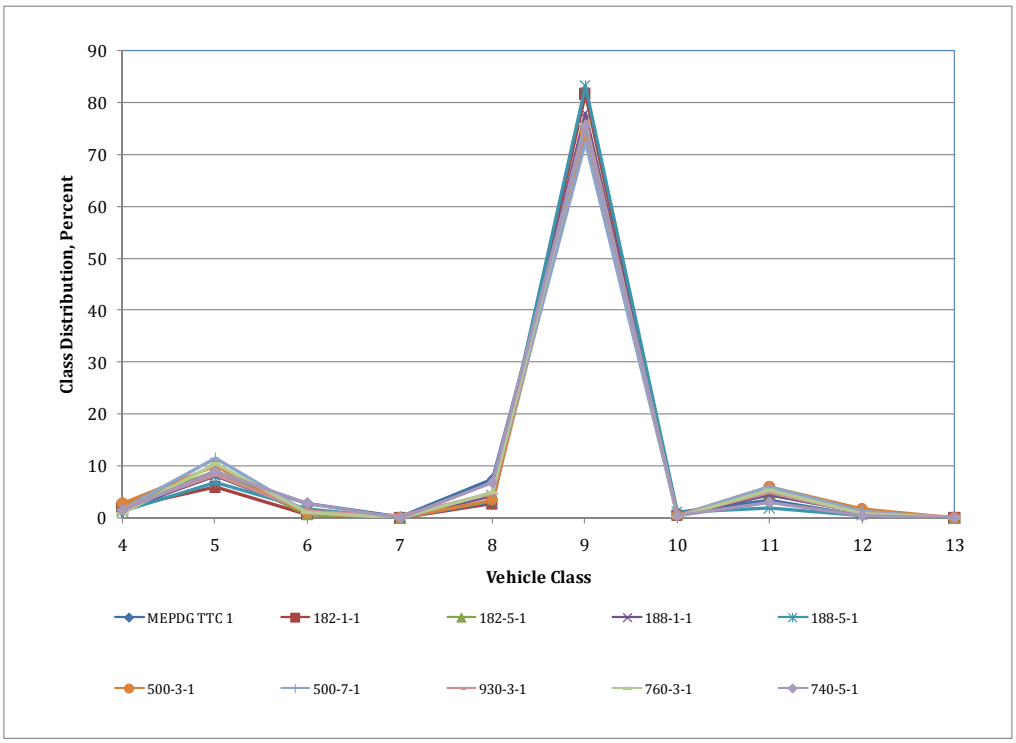


Figure I-10. Comparison of MoDOT site-specific vehicle class distributions classified as TTC 1 with MEPDG TTC 1 default distributions.

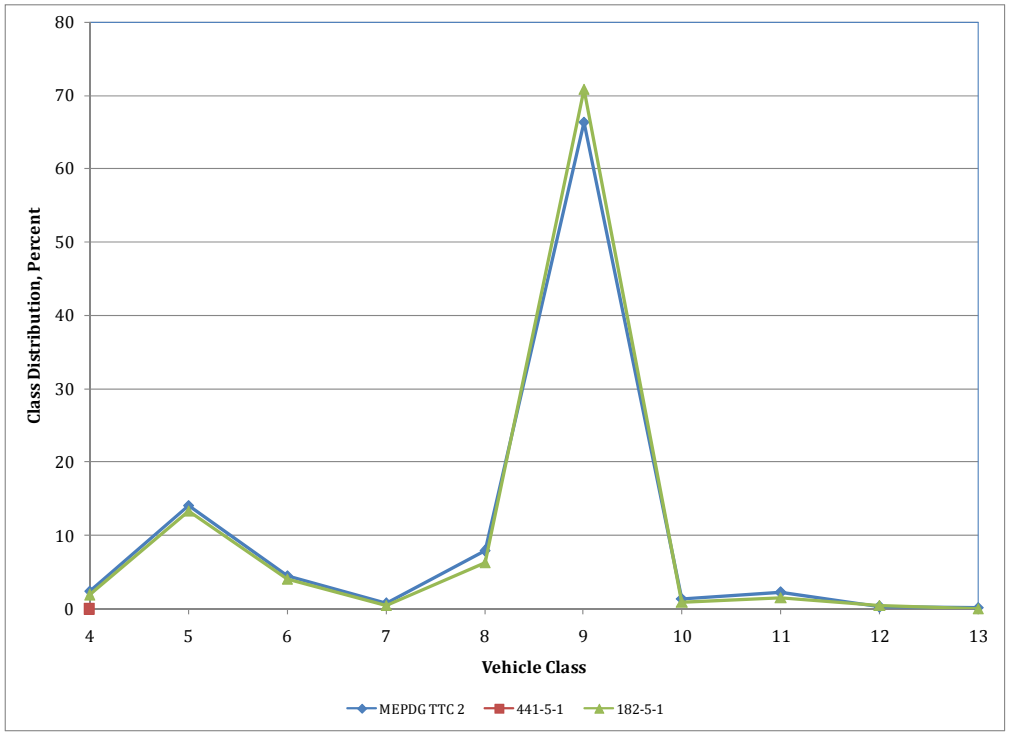


Figure I-11. Comparison of MoDOT site-specific vehicle class distributions classified as TTC 2 with MEPDG TTC 2 default distributions.

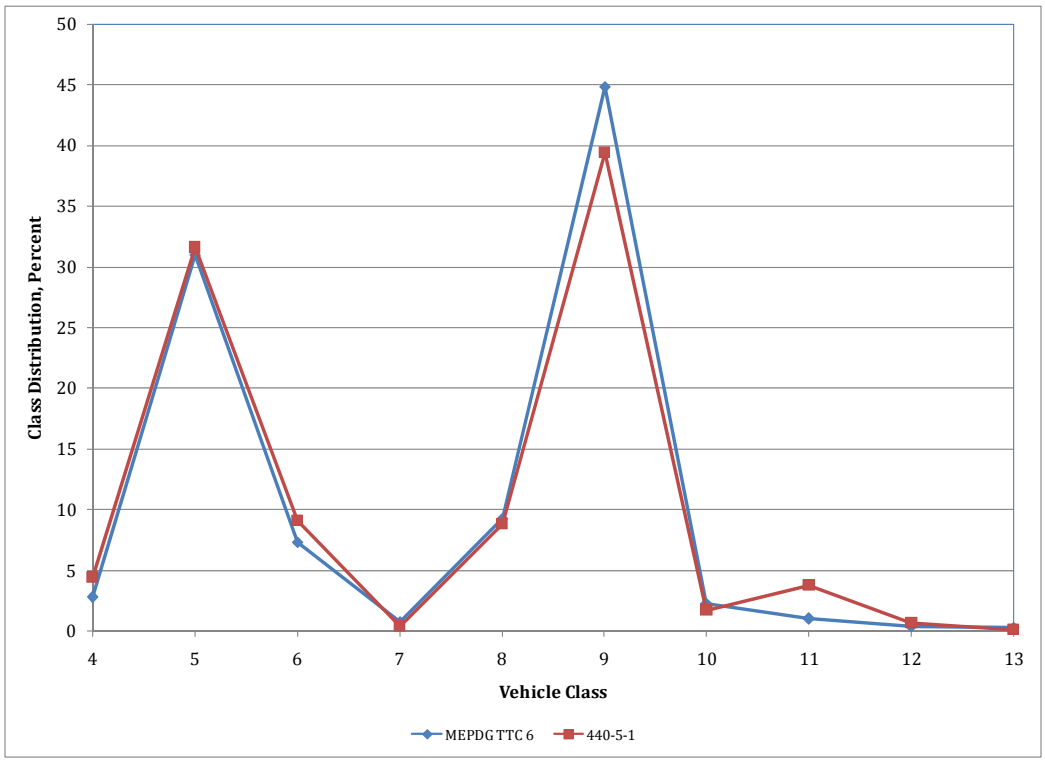


Figure I-12. Comparison of MoDOT site-specific vehicle class distributions classified as TTC 6 with MEPDG TTC 6 default distributions.

vehicle class distributions and the MEPDG default values. This of course means that the MEPDG defaults can be used in design, provided the general characteristics of the traffic data for a given site can be described properly.

Analysis of Hourly Truck Distribution Data

Figure I-13 presents a comparison of the hourly truck distribution data gathered from the various MoDOT sites with the MEPDG default value. A statistical analysis of the data revealed that, in a majority of the cases, there is no significant difference between the MEPDG default values and MoDOT-specific data. Therefore, MEPDG defaults for this input are recommended for use in routing design.

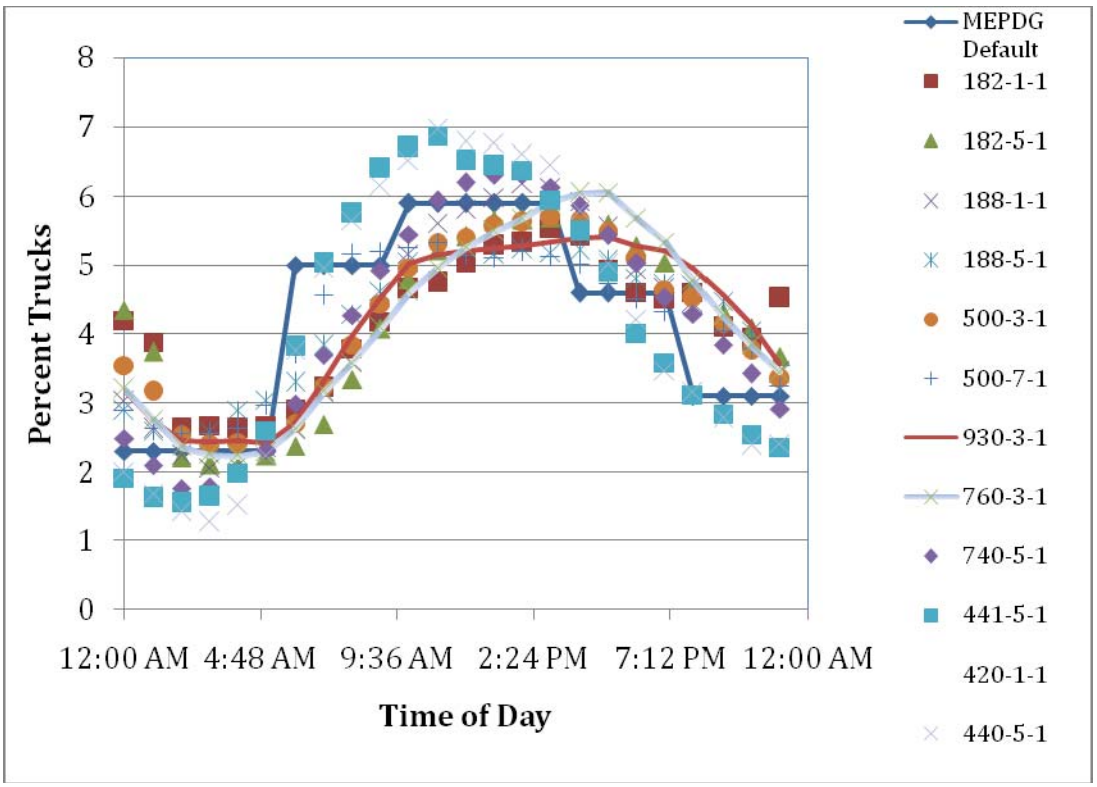
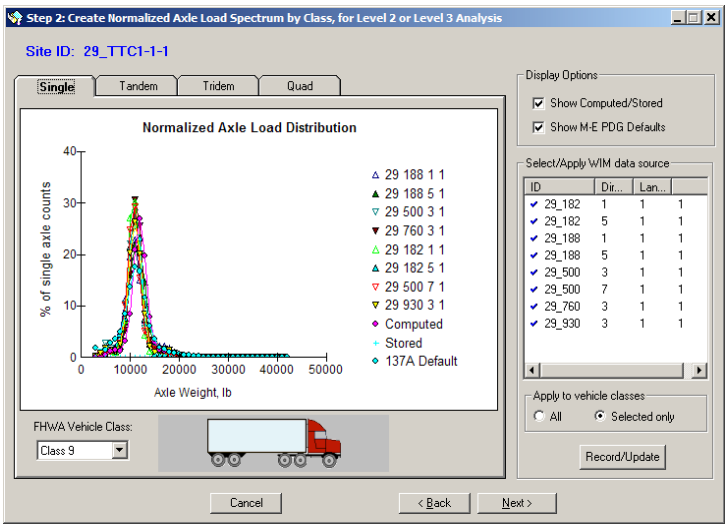


Figure I-13. Comparison of MoDOT site-specific 24-hour truck count data with MEPDG defaults.

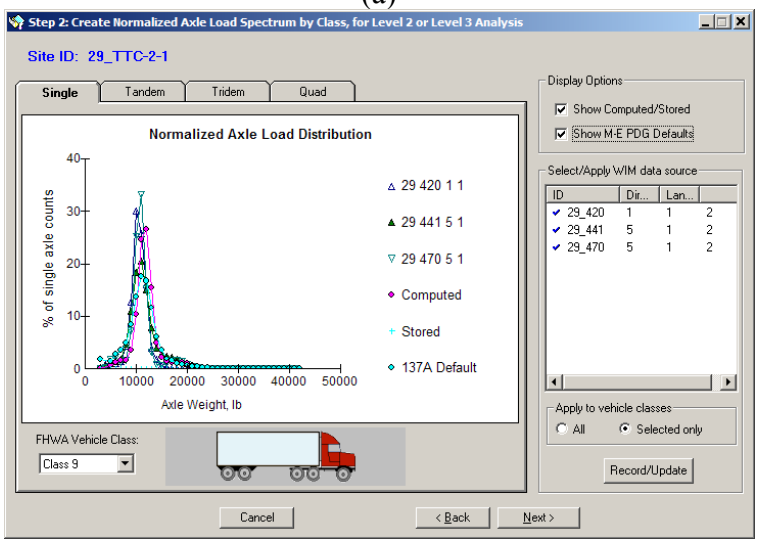
Axle Load Distributions or Spectra

ATLAS screen shots for the single and tandem load distributions for Class 9 trucks for all the selected sites in Missouri along with the MEPDG defaults (noted as 1-37A defaults in the figures) are shown in Figure I-14 and Figure I-15, respectively. Class 9 trucks were chosen for this demonstration because they dominate the truck mix and therefore can be taken as representative for discussion purposes. Figure I-16 and Figure I-17 present similar information for tridem and quad axles for all classes and all TTC groups (all truck classes and TTC groups were combined because there was only a limited amount of data in the MoDOT WIM database for these axle types). The following conclusions can be drawn from the data:

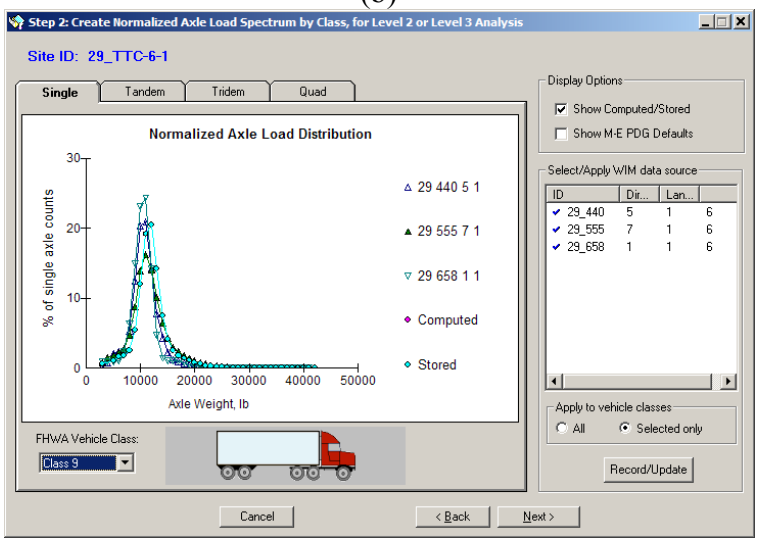
- The shapes of the load spectra followed expected trends (i.e., single peak for single axle load spectra, bi-modal peak for tandem axles, etc.)
- There appears to be some variation in the axle load spectra for single and tandem axles. However, most the loadings where the variations occur are below the legal load limits.
- Overloads do not appear to be a concern, at least for the traffic data set analyzed.
- There appears to be a very limited amount of tridem and quad axle weight data.



(a)

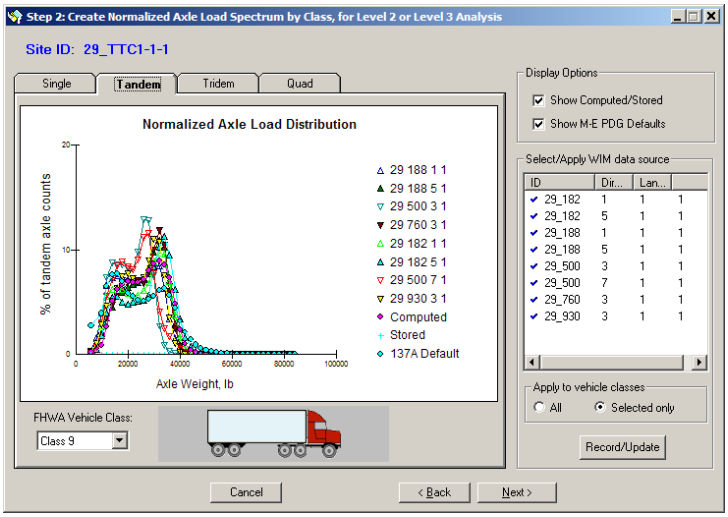


(b)

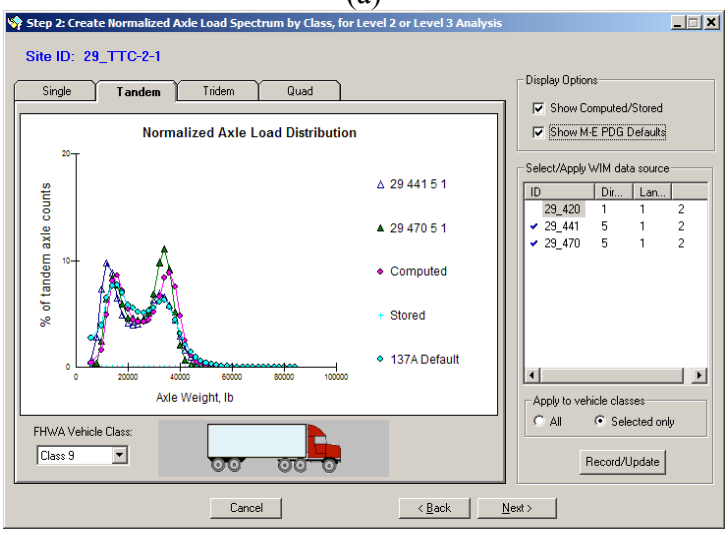


(c)

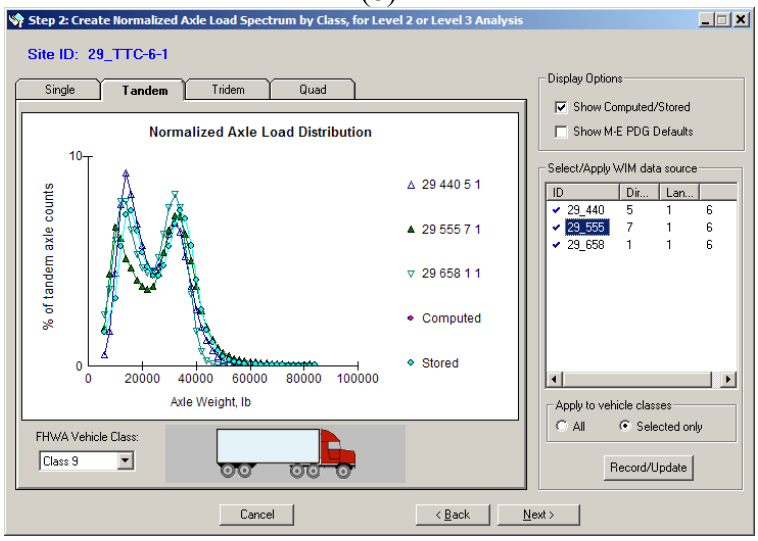
Figure I-14. Single axle load distributions for all analyzed MoDOT WIM sites.



(a)



(b)



(c)

Figure I-15. Tandem axle load distributions for all analyzed MoDOT WIM sites.

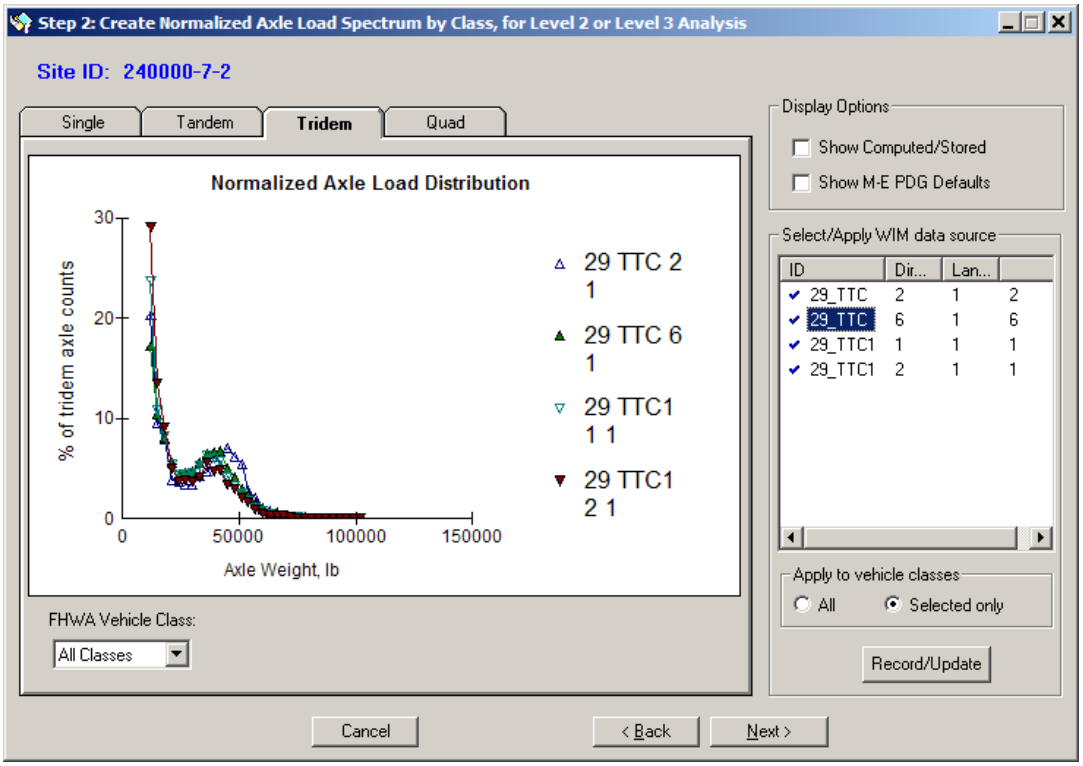


Figure I-16. Tridem axle load distributions for all MoDOT TTC 1 WIM sites.

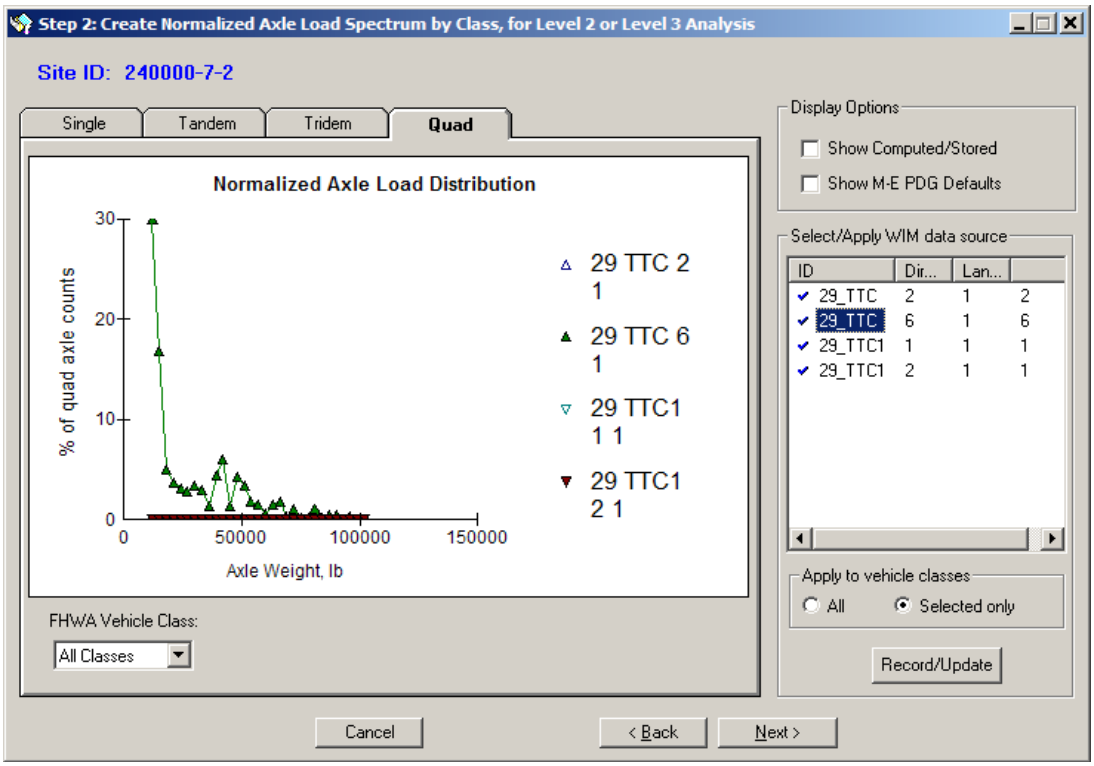


Figure I-17. Tridem axle load distributions for all MoDOT TTC 1 WIM sites.

To better compare the single and tandem axle load spectra with the MEPDG defaults, cumulative load distribution curves, which are a powerful way to represent traffic load frequency data, were prepared for single and tandem axles for all truck classes. The plots were stratified first by TTC group and next by MoDOT's roadway designation—heavy or medium duty pavements. Roadway designation was used as a surrogate for truck weight road groups (TWRGs) to study if load spectra within a given TTC group differ as a function of this variable. A few examples of how the MEPDG load spectra defaults compare with MoDOT's data are presented in this chapter for key axle and truck classes of interest. Class 5 and Class 9 trucks and single and tandem axles were selected for the illustrations since they dominate the data set analyzed.

Figure I-18 through Figure I-23 present a comparison of the cumulative axle load spectra for single axles of Class 5 trucks for the three primary TTC groups and two roadway designations of interest to MoDOT. This particular axle type and truck class combination is considered to be of the highest interest to pavement loadings especially for UPA-I and UPA-O categories but also for other roadway functional classes.

Figure I-24 through Figure I-29 present a comparison of the cumulative axle load spectra for tandem axles of Class 9 trucks for the three primary TTC groups and two roadway designations of interest to MoDOT. Class 9 trucks dominate the truck distributions for all sites in Missouri, and tandem axle of the truck class is very important from a loading standpoint. Hence, this combination of truck class and axle type was considered for analysis.

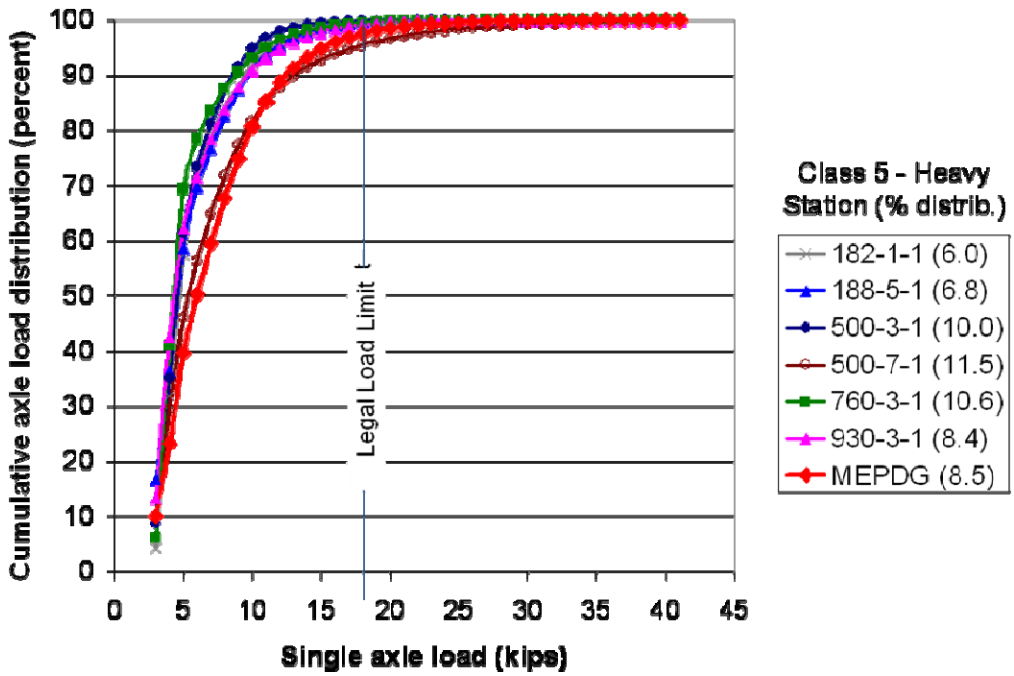


Figure I-18. Cumulative single axle load distribution for Class 5 trucks on MoDOT heavy duty pavements pertaining to TTC 1.

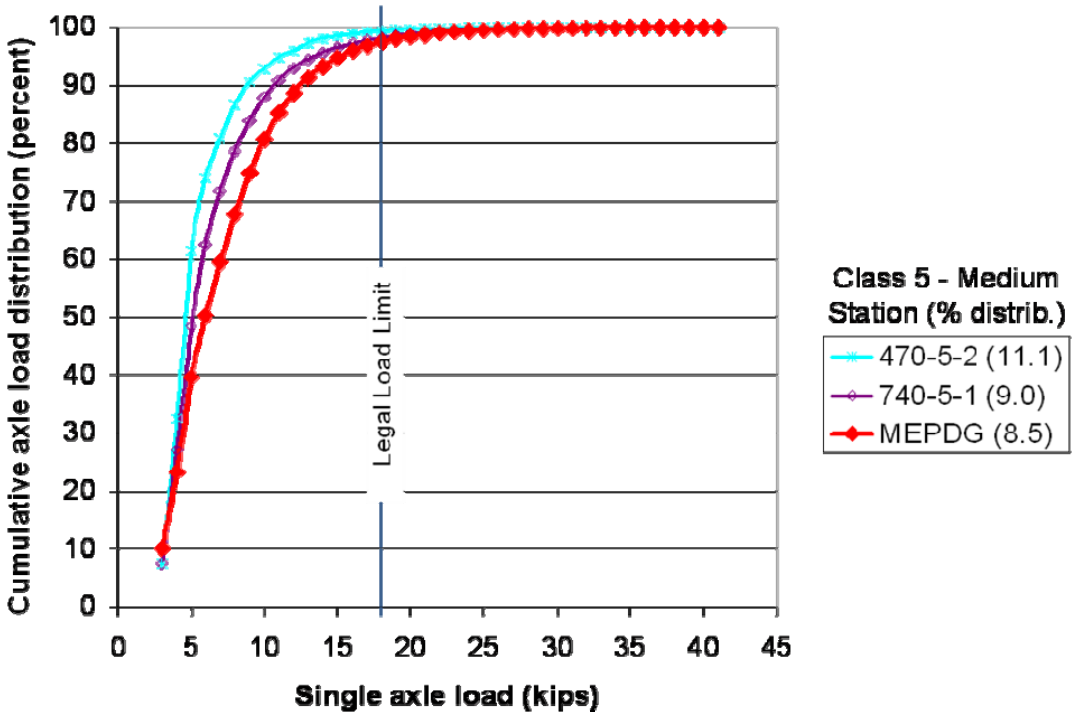


Figure I-19. Cumulative single axle load distribution for Class 5 trucks on MoDOT medium duty pavements pertaining to TTC 1.

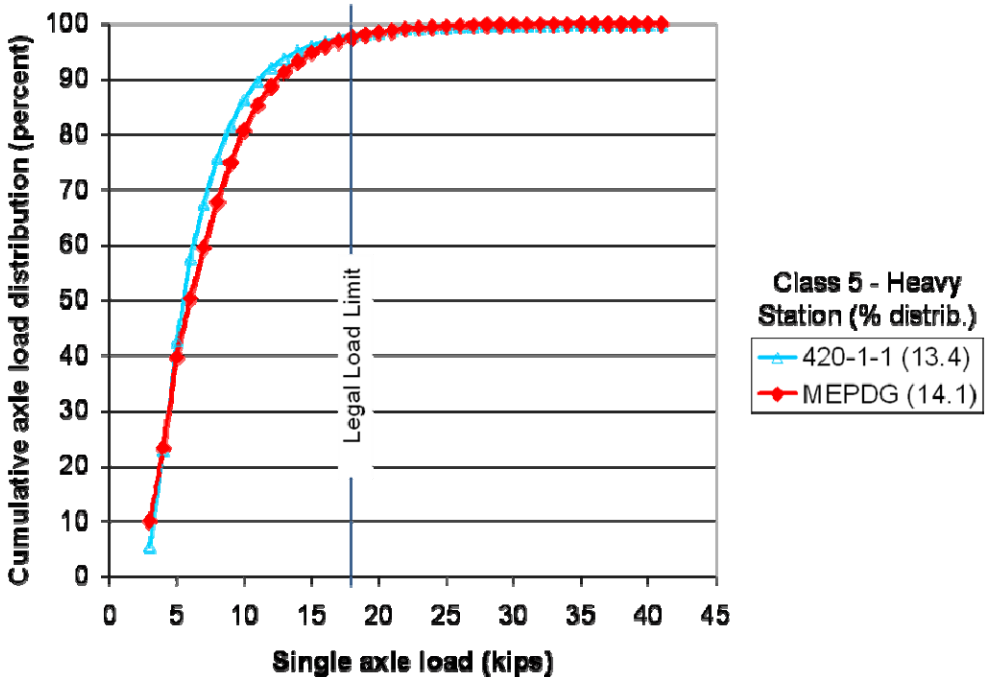


Figure I-20. Cumulative single axle load distribution for Class 5 trucks on MoDOT heavy duty pavements pertaining to TTC 2.

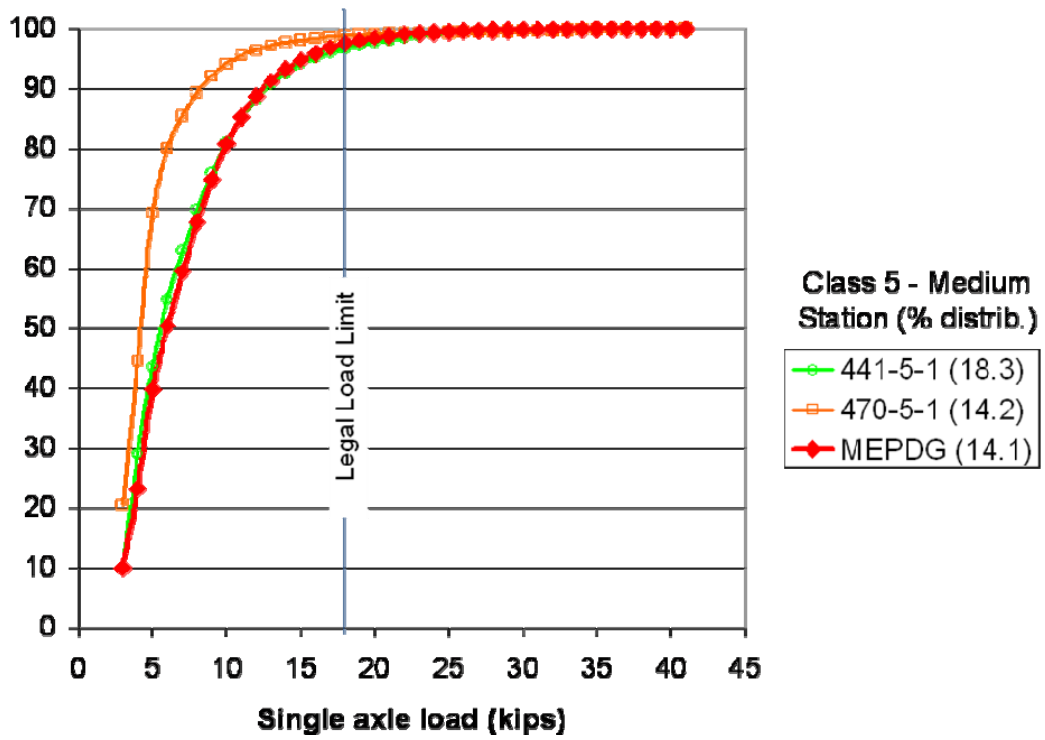


Figure I-21. Cumulative single axle load distribution for Class 5 trucks on MoDOT medium duty pavements pertaining to TTC 2.

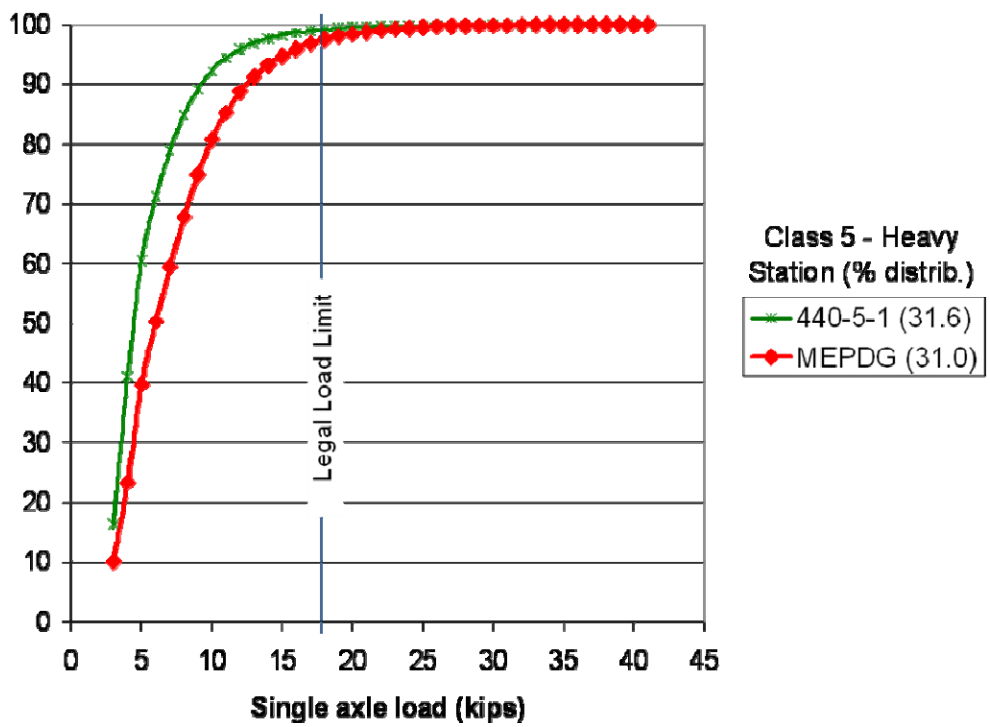


Figure I-22. Cumulative single axle load distribution for Class 5 trucks on MoDOT heavy duty pavements pertaining to TTC 6.

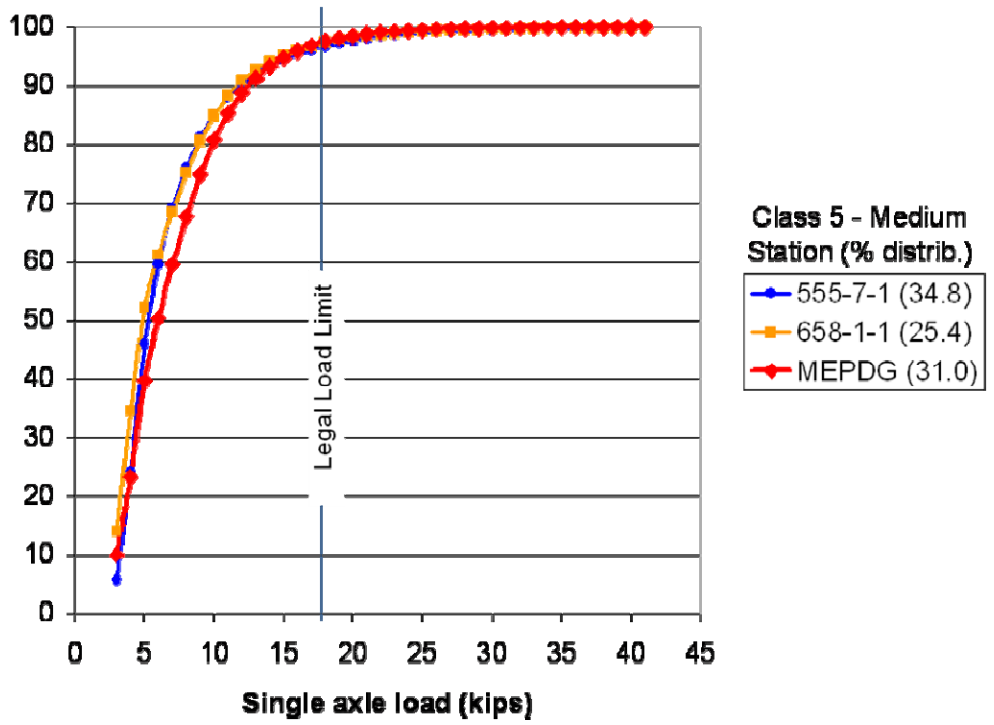


Figure I-23. Cumulative single axle load distribution for Class 5 trucks on MoDOT medium duty pavements pertaining to TTC 6.

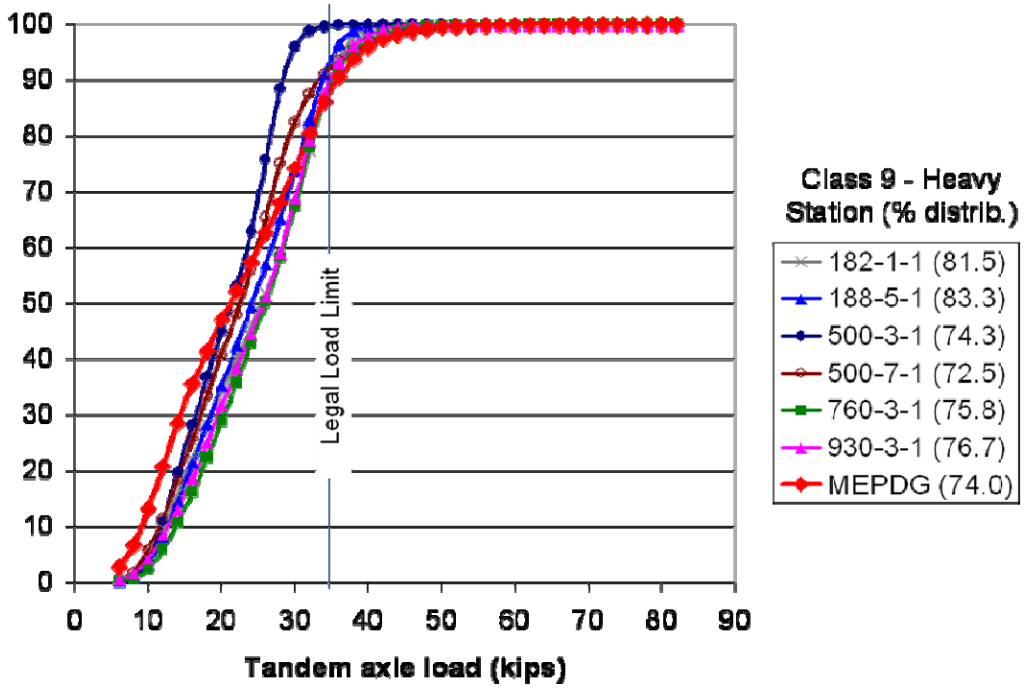


Figure I-24. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT heavy duty pavements pertaining to TTC 1.

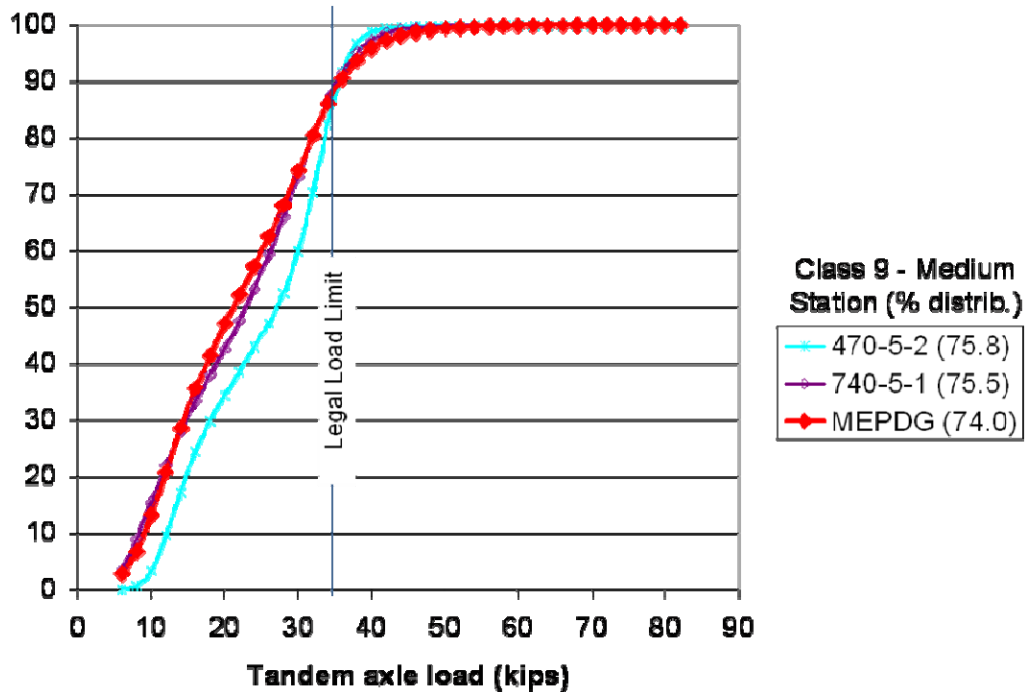


Figure I-25. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT medium duty pavements pertaining to TTC 1.

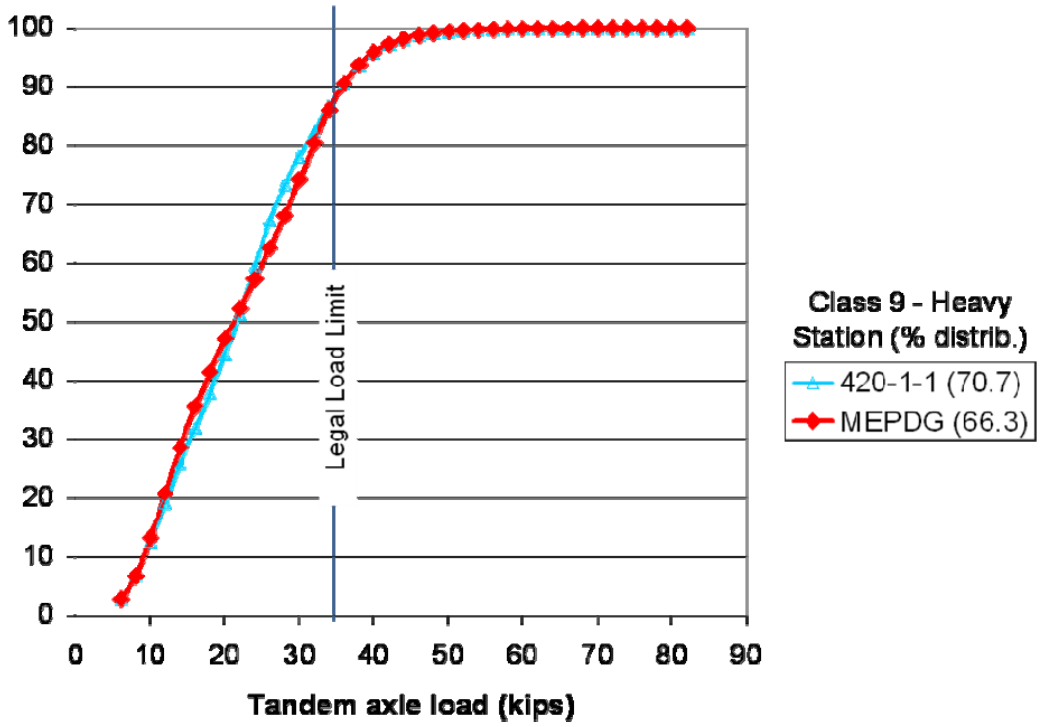


Figure I-26. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT heavy duty pavements pertaining to TTC 2.

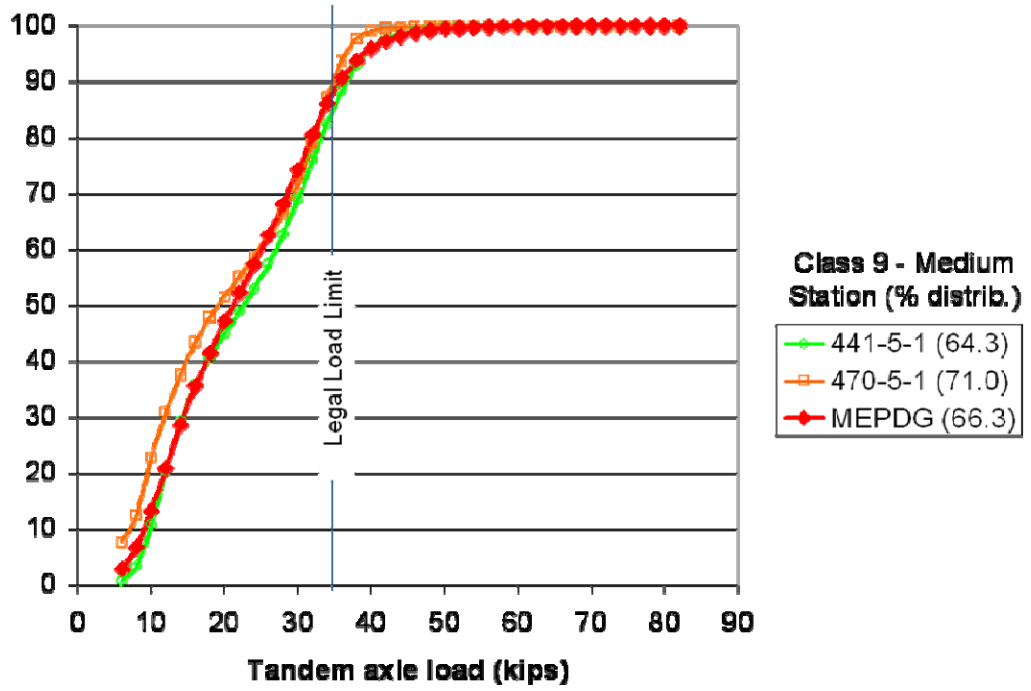


Figure I-27. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT medium duty pavements pertaining to TTC 2.

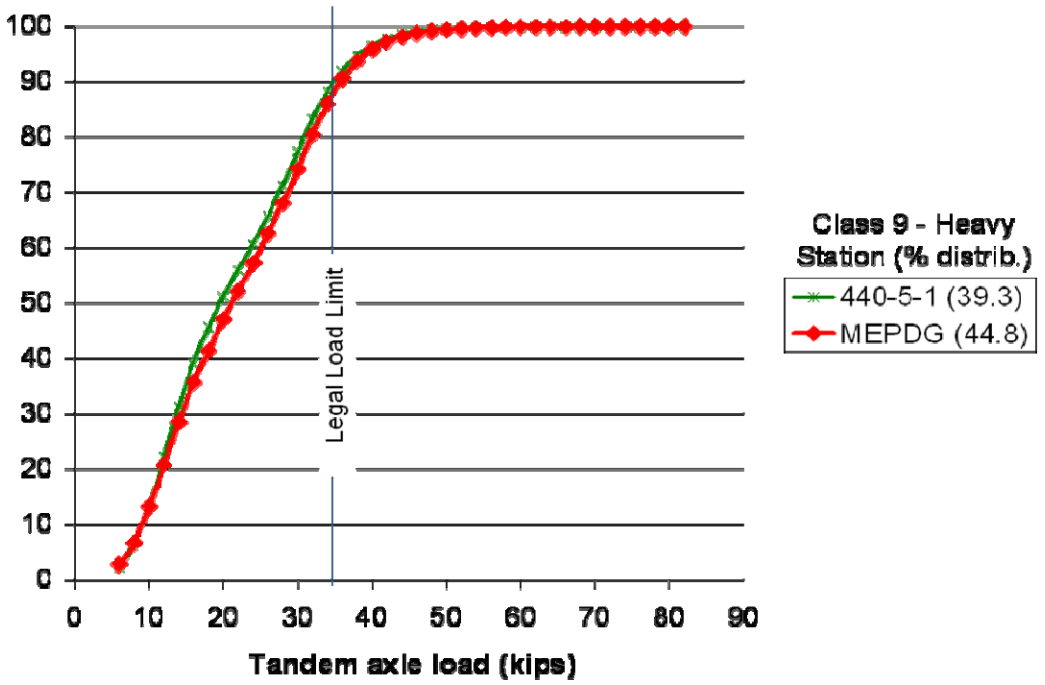


Figure I-28. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT heavy duty pavements pertaining to TTC 6.

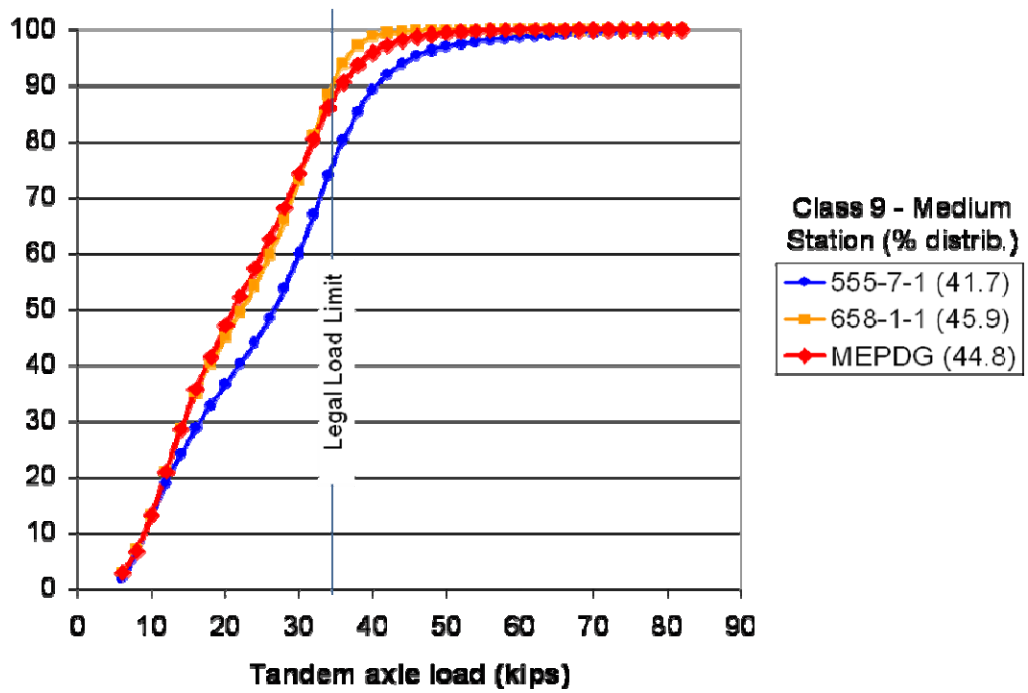


Figure I-29. Cumulative tandem axle load distribution for Class 9 trucks on MoDOT medium duty pavements pertaining to TTC 6.

The following conclusions can be drawn on the basis of the data presented (as well as other load spectra comparisons performed but not reported herein):

- In most cases, the MEPDG default load spectra match the MoDOT-specific load spectra. A paired t-test performed resulted in the acceptance of the null hypothesis (H_0) that the MEPDG default spectra are similar to the MoDOT site-specific spectra for a given truck class and roadway designation (significance level = 0.05). One exception was the tandem axle load spectra for MoDOT site 555-7-1, which showed heavier loadings than the MEPDG default (see Figure I-29). This site was eventually removed from final consideration since it is an ADR 3000 site (portable WIM), the data quality of which were not under the scope of the work performed herein. However, this and other sites representing this TTC group (TTC 6) need to be investigated into the future.
- Within a given TTC group, the axle load spectra do not change with MoDOT's roadway designation (heavy duty vs. medium duty). This finding is as expected.
- The axle load distributions within a truck class were found to be relatively stable from one site to the other, indicating that for all through highways in Missouri (which are a majority of the highways analyzed), the MEPDG defaults can be applied for routine design purposes. A 10 percent maximum site-to-site variance was observed at the legal load levels for the data analyzed (see Figure I-24 through Figure I-29). The impact of this variance is not considered to be significant from a distress prediction standpoint (see discussion later in this section under the heading "Effect of Traffic on Distress Prediction."

Determination of Average Axles per Truck Class

The axles per truck input in the MEPDG represents the mean number of single, tandem, tridem, and quad axles per vehicle class. This input was characterized for all the MoDOT sites investigated and compared with the MEPDG default values. Figure I-30 through Figure I-39 present the axles per truck for Classes 4 through 10, along with the corresponding MEPDG default values. The following observations can be derived from the data presented:

- There is a high degree of consistency in the axles per truck between sites for a given truck class. This is particularly true for Class 5 and Class 9. Expectedly, the highest variance is observed for Class 7; this truck class is not a significant part of the truck mix in Missouri.
- The MEPDG default axles/truck are in good agreement with the Missouri-specific values for all truck classes with the exception of Classes 7 and 13, which form a small portion of the overall truck mix. For Classes 5 and 9, the MEPDG defaults are in excellent agreement with Missouri-specific data.
- Tridems and quad axles appear to be rare in Missouri.

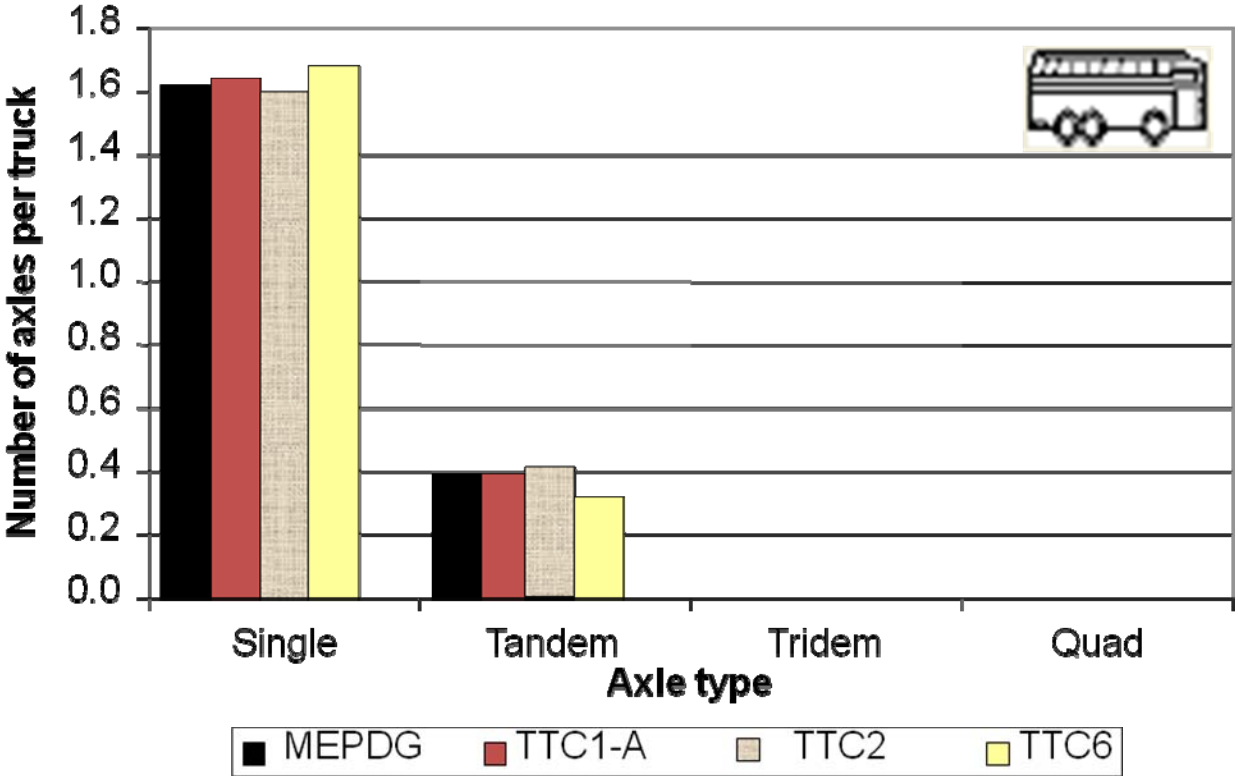


Figure I-30. Missouri-specific axles per truck for Class 4 trucks compared with MEPDG defaults.

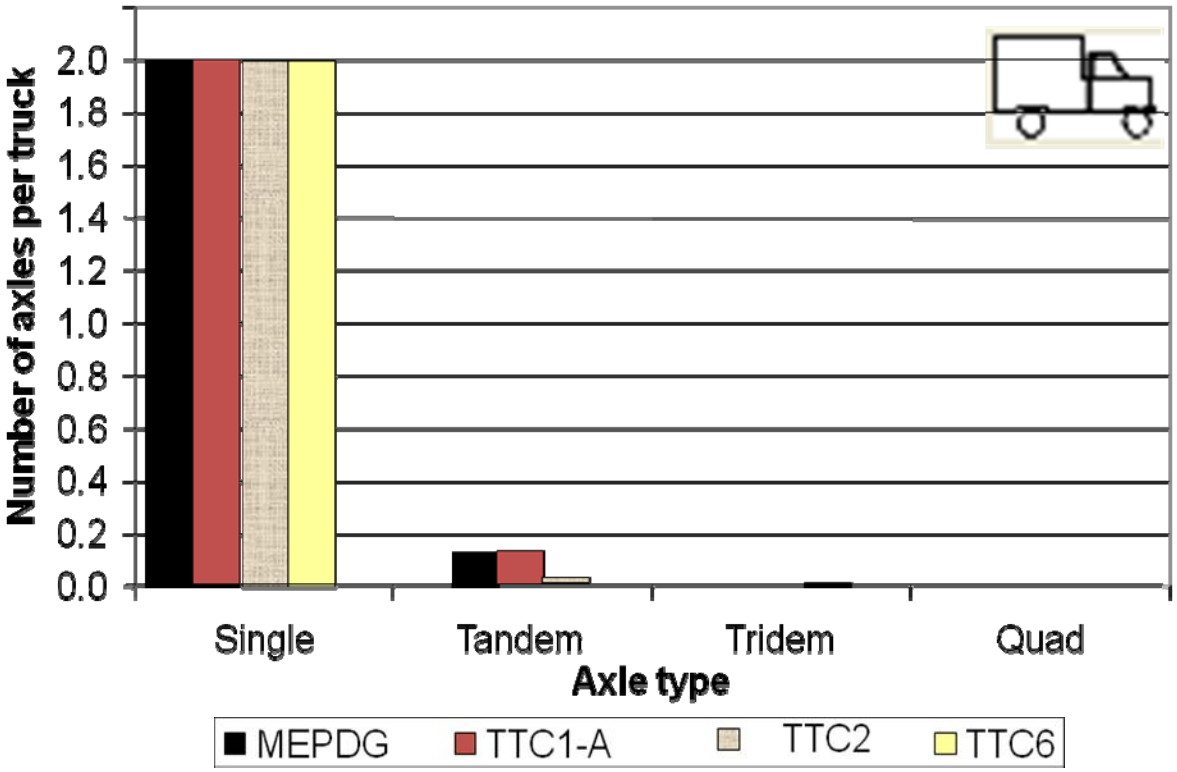


Figure I-31. Missouri-specific axles per truck for Class 5 trucks compared with MEPDG defaults.

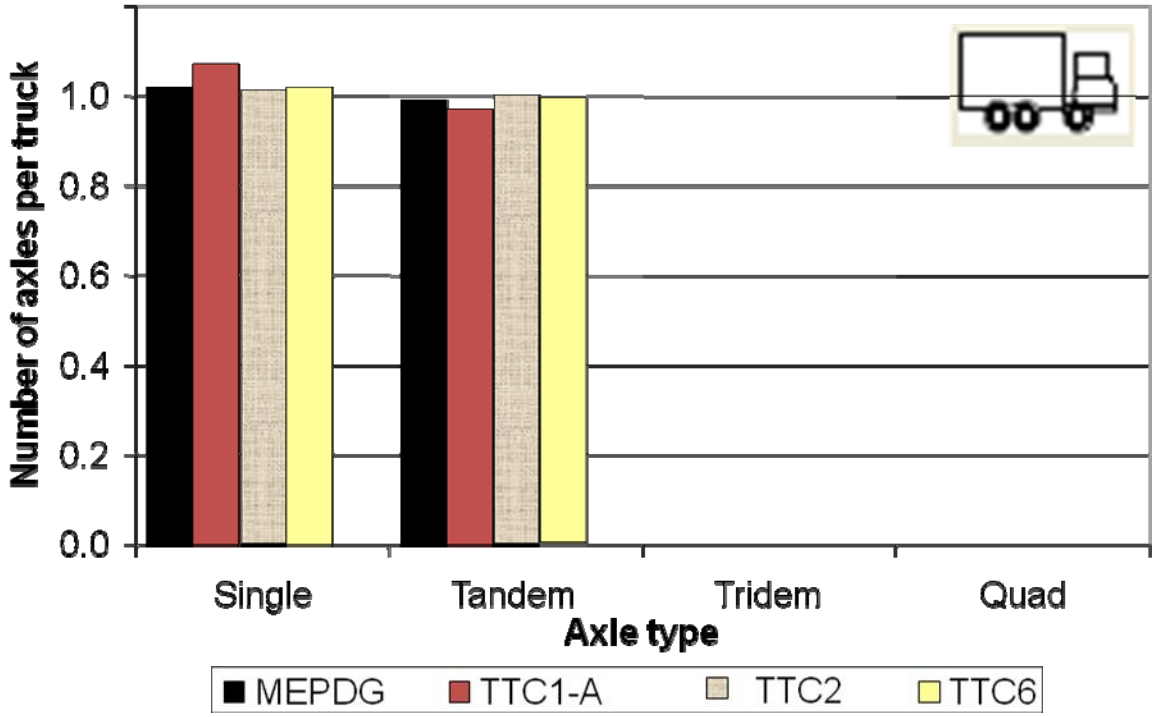


Figure I-32. Missouri-specific axles per truck for Class 6 trucks compared with MEPDG defaults.

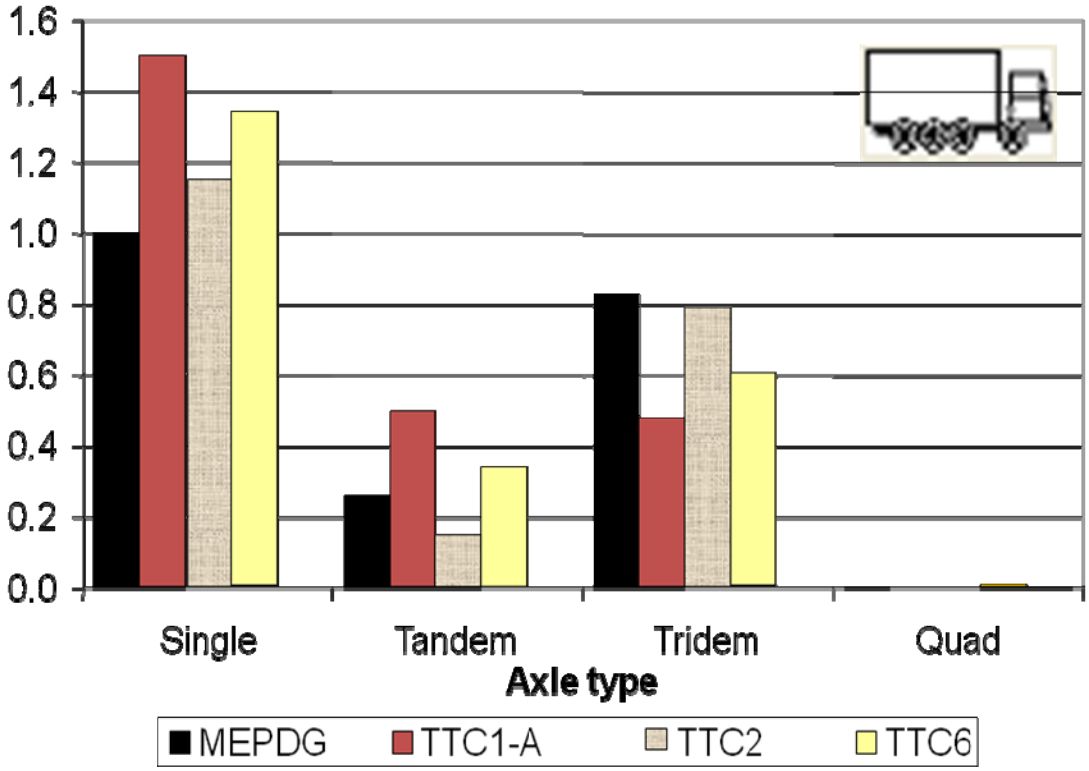


Figure I-33. Missouri-specific axles per truck for Class 7 trucks compared with MEPDG defaults.

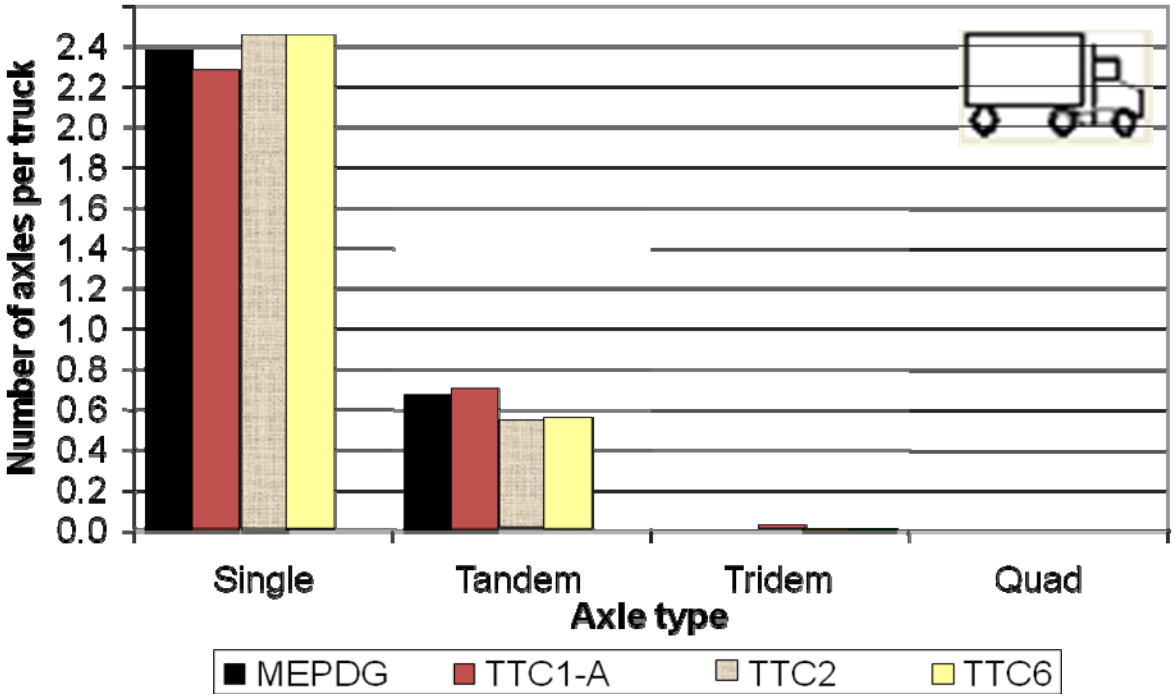


Figure I-34. Missouri-specific axles per truck for Class 8 trucks compared with MEPDG defaults.

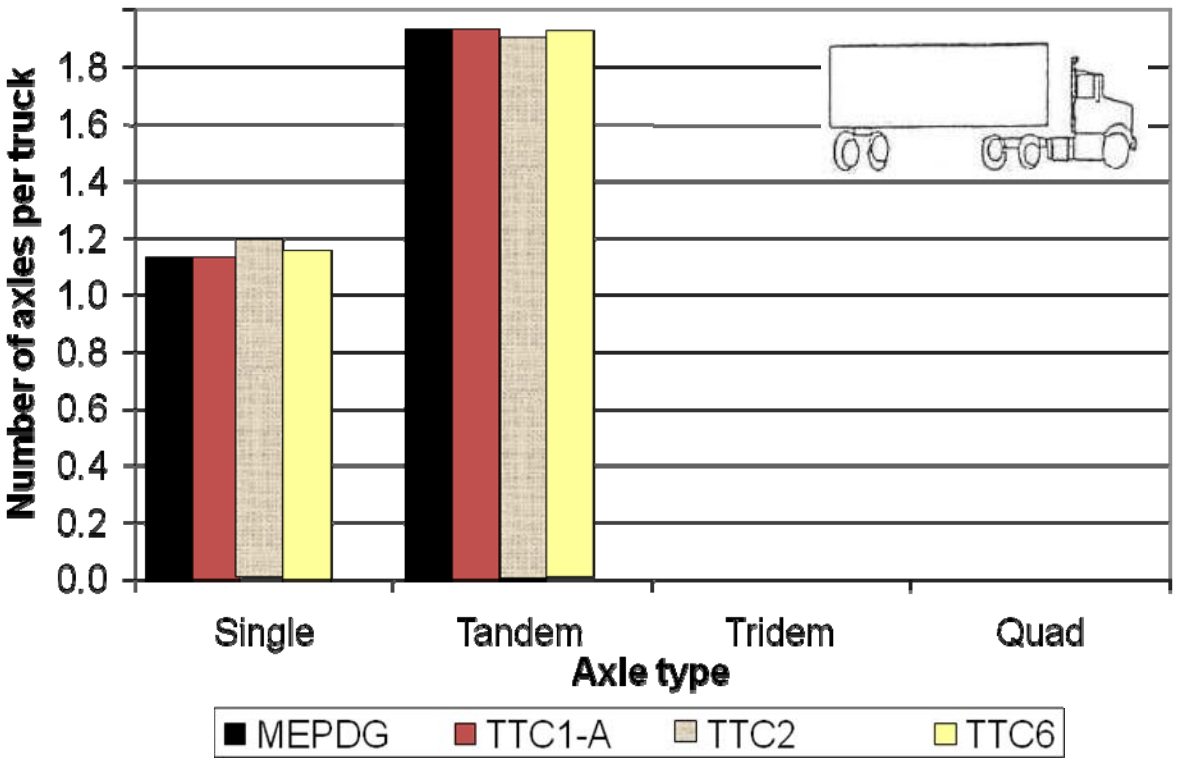


Figure I-35. Missouri-specific axles per truck for Class 9 trucks compared with MEPDG defaults.

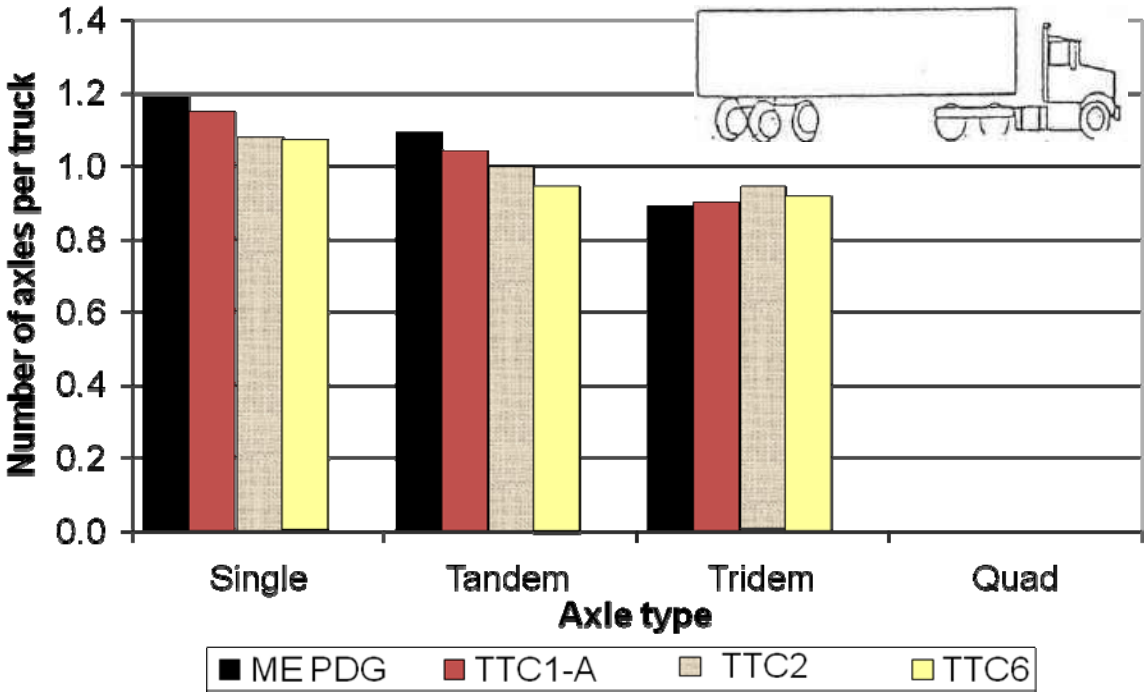


Figure I-36. Missouri-specific axles per truck for Class 10 trucks compared with MEPDG defaults.

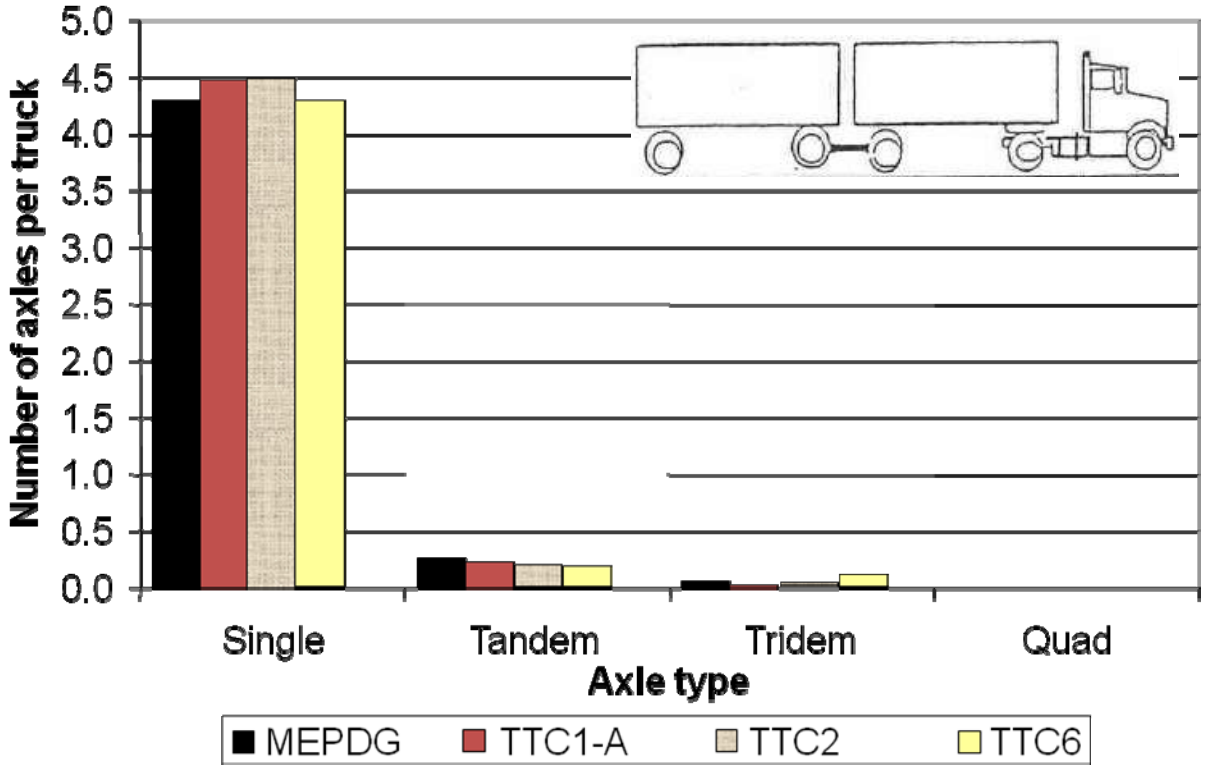


Figure I-37. Missouri-specific axles per truck for Class 11 trucks compared with MEPDG defaults.

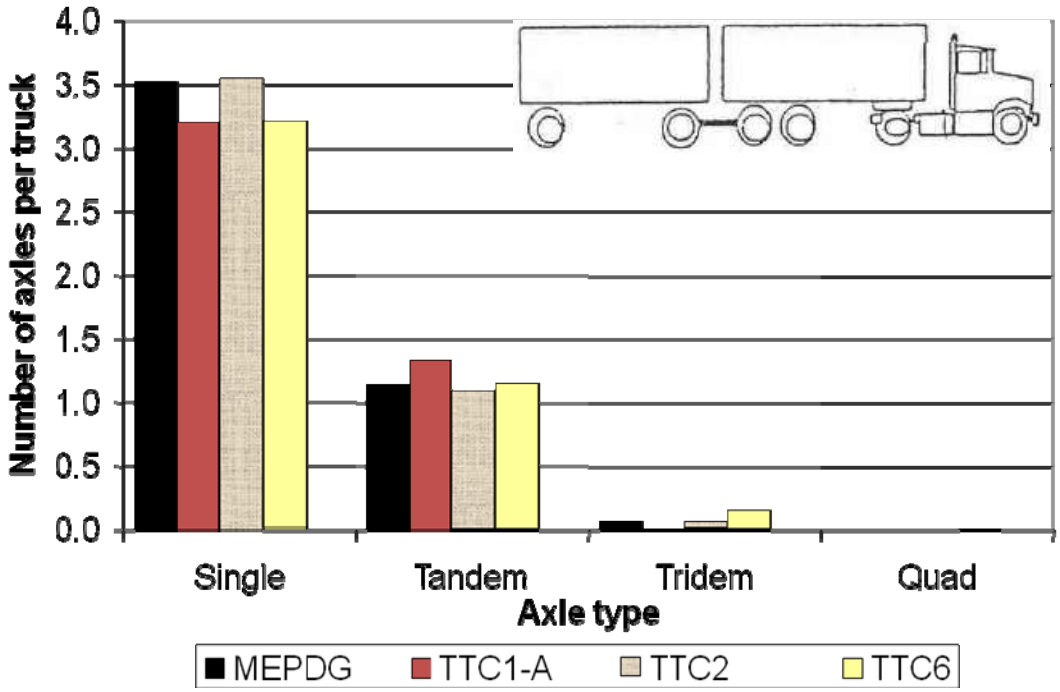


Figure I-38. Missouri-specific axles per truck for Class 12 trucks compared with MEPDG defaults.

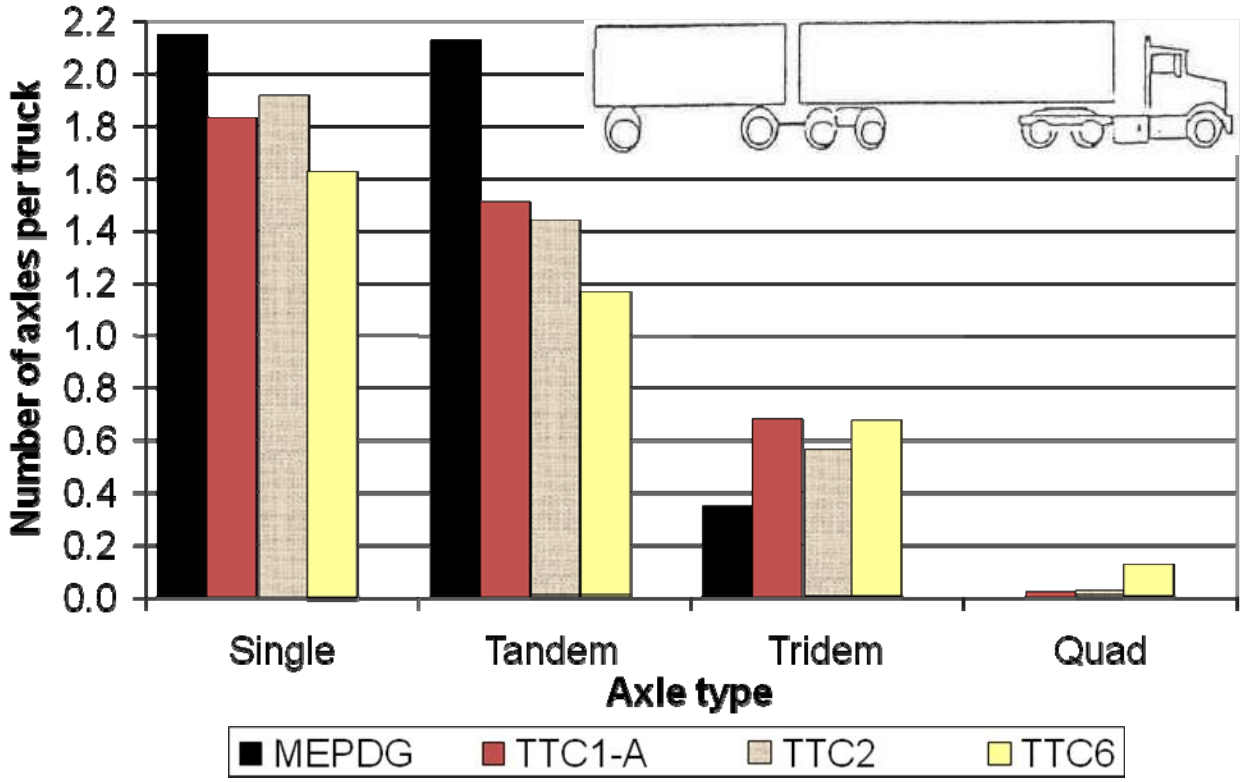


Figure I-39. Missouri-specific axles per truck for Class 13 trucks compared with MEPDG defaults.

Monthly Adjustment Factors

Figure I-40 presents the monthly adjustment factors for truck traffic collected from the various MoDOT sites along with the MEPDG default value.

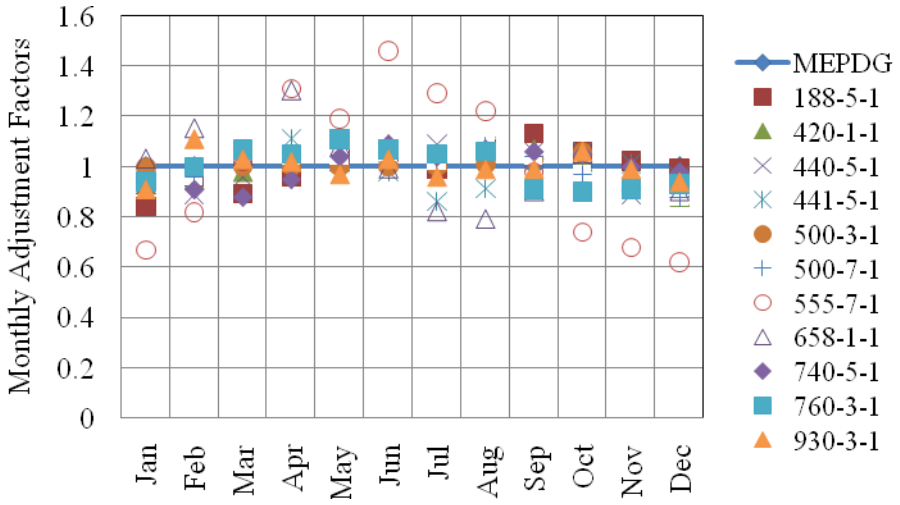


Figure I-40. Monthly adjustment factors for truck traffic at MoDOT sites.

Effect of Traffic on Distress Prediction

Impact of Traffic Characteristics of MoDOT's Designated Heavy and Medium Routes on MEPDG Distress Prediction

A sensitivity analysis was conducted to investigate the effect of traffic characteristics on MoDOT designated heavy and medium duty pavements. Two sites were selected based on their traffic volumes: site ID: 188-5-1 (Heavy duty) and site ID: 740-5-1 (Medium duty). The traffic at these two sites is predominantly comprised of single trailer trucks (Class 9) with lower percentage of multiple trailers. Both these sites fall under the TTC-1 truck classification group (see Table I-15 for a description).

Two new pavement sections, HMA and JPCP, were selected for the sensitivity analysis and are presented in Figure I-41. The HMA pavement sections consisted of 8 inches of HMA on 4 inches of crushed stone dense-graded aggregate base and A-2-6 subgrade. The JPCP section consisted of 8 or 9 inches of PCC on 4 inches of A-1-a aggregate base and A-6 subgrade. The HMA pavement section was evaluated for alligator cracking, longitudinal cracking, HMA rutting, total rutting, and smoothness. The JPCP section was evaluated for bottom-up and top-down damage, fatigue cracking (transverse cracking), faulting and smoothness (IRI). The analysis was conducted for both 8-in. and 9-in. of JPCP.

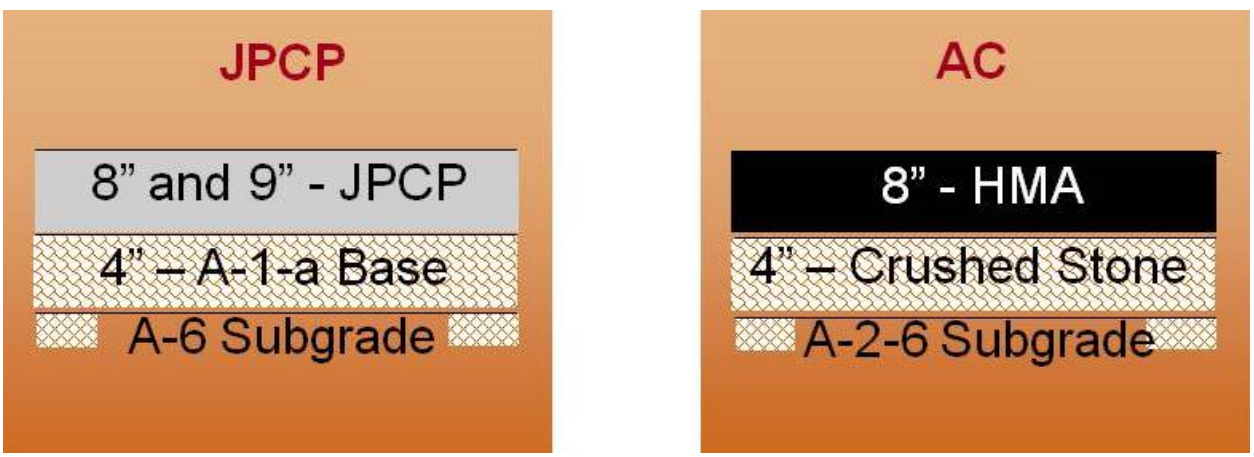


Figure I-41. Pavement sections for sensitivity analysis of traffic characteristics on MoDOT designated roadway categories.

The traffic characteristics selected for the sensitivity analysis were the vehicle class distributions, the axle load distributions, the monthly adjustment factors and the hourly truck distributions. The vehicle class and the axle load distributions are shown in figures Figure I-10 and Figure I-18, respectively. The monthly adjustment factors and the hourly truck distributions are shown in figures Figure I-40 and Figure I-13, respectively. Site-specific values for these traffic inputs of the two sites, 188-5-1 and 740-5-1, were used in the analysis. However, to enable comparisons, the same two-way AADTT value (1541) and growth rate of (4 percent—compound) were used. Other characteristics including the climate and material properties of structural layers were unchanged.

Figure I-42 and Figure I-43 present the predictions of slab cracking for JPCP sections with a PCC thickness of 8-in. and 9-in., respectively. The differences in predicted cracking at the end of design period between the heavy-duty and medium-duty pavements were about 8% and 4% for 8-in. and 9-in. pavements, respectively. Figure I-44 through Figure I-47 present the comparison of bottom-up and top-down fatigue damage for JPCP sections of 8-in. and 9-in., respectively. Figure I-48 through Figure I-51 present the comparison of faulting and IRI for JPCP sections with a PCC thickness of 8-in. and 9-in., respectively. The difference in faulting and smoothness between the heavy-duty and medium-duty pavements was 5 % for both JPCP sections.

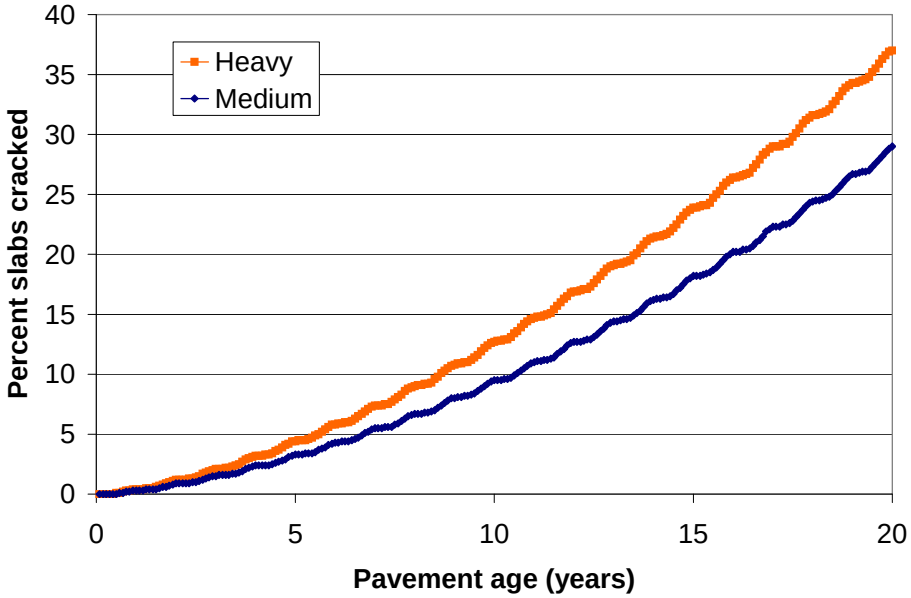


Figure I-42. Comparison of predicted slab cracking for 8-inch JPCP sections.

Figure I-43. Comparison of predicted slab cracking for 9-inch JPCP sections.

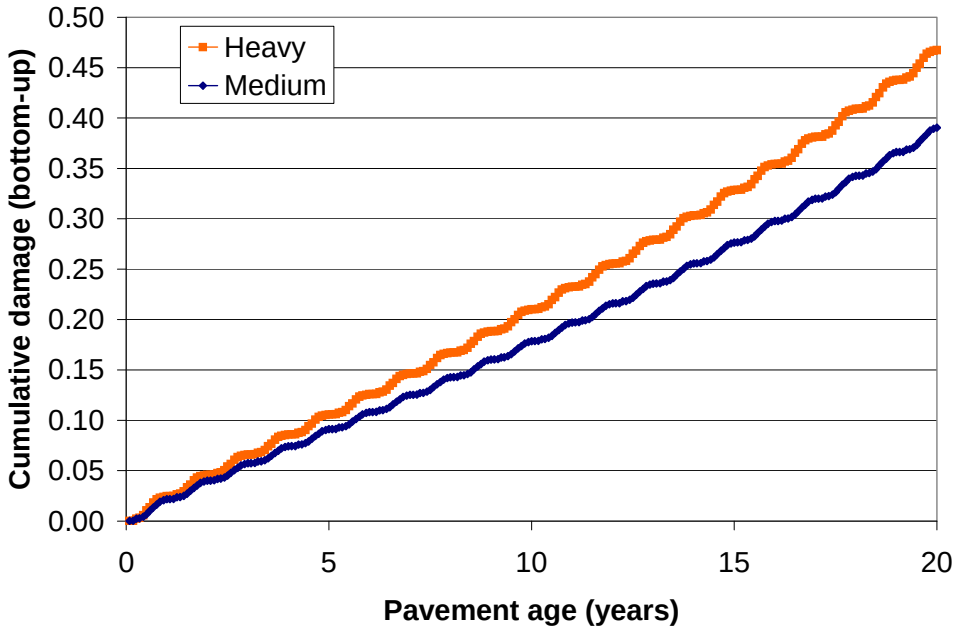


Figure I-44. Comparison of predicted bottom-up fatigue damage for 8-inch JPCP sections.

Figure I-45. Comparison of predicted bottom-up fatigue damage for 9-inch JPCP sections.

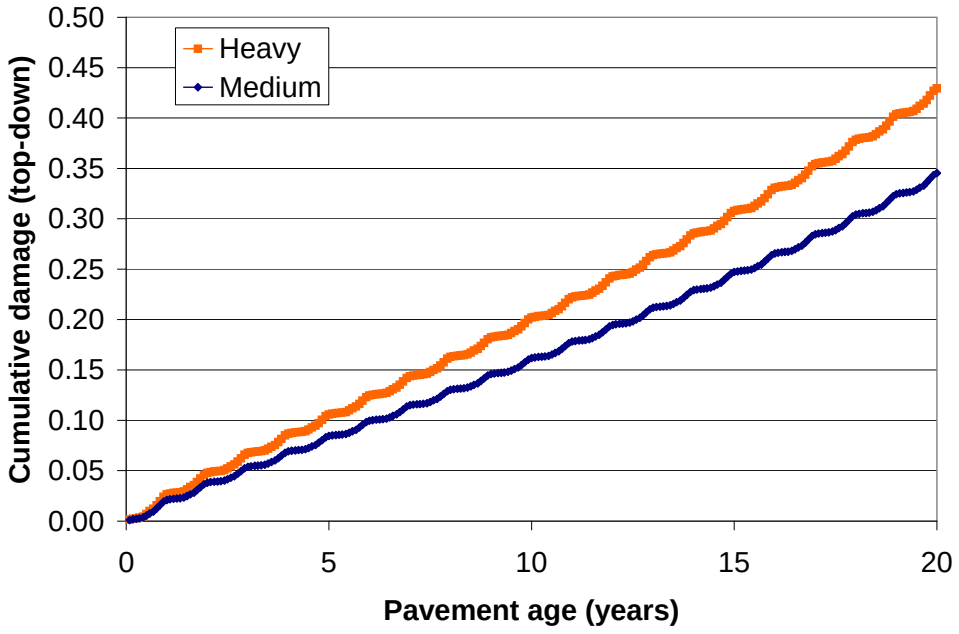


Figure I-46. Comparison of predicted top-down fatigue damage for 8-inch JPCP sections.

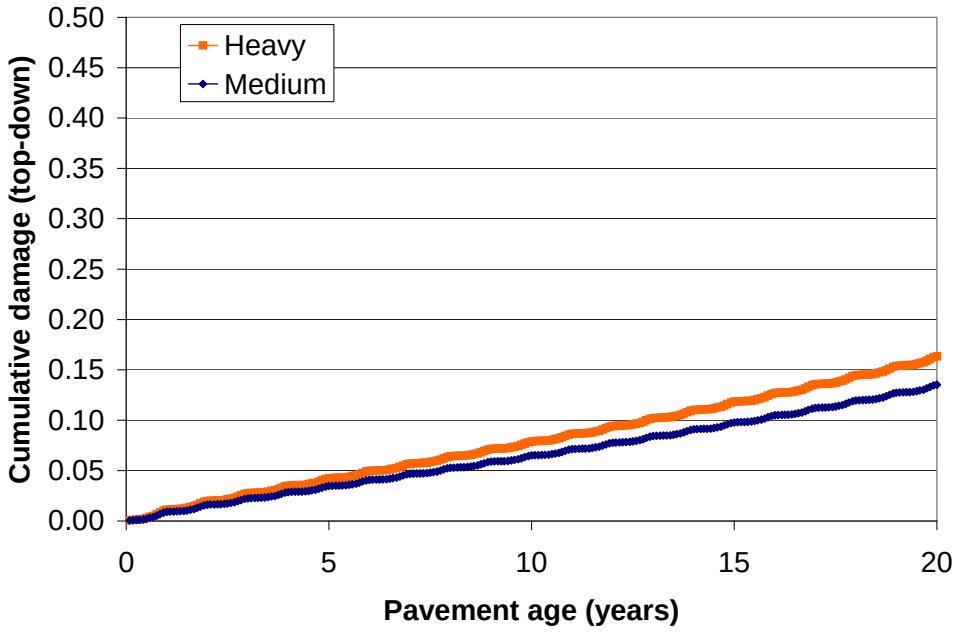


Figure I-47. Comparison of predicted top-down fatigue damage for 9-inch JPCP sections.

Figure I-48. Comparison of predicted faulting for 8-inch JPCP sections.

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Figure I-49. Comparison of predicted faulting for 9-inch JPCP sections.

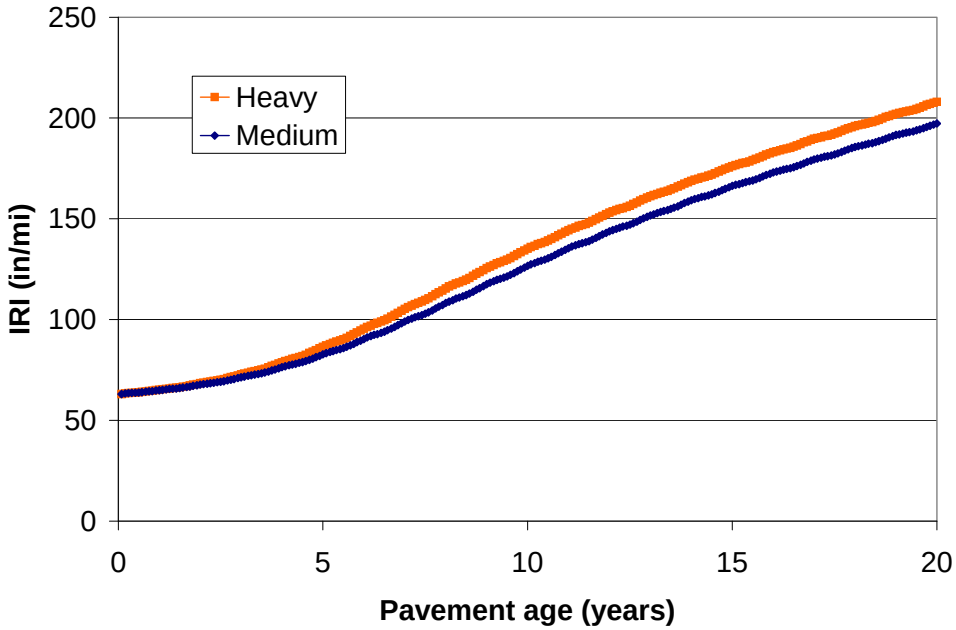


Figure I-50. Comparison of predicted IRI for 8-inch JPCP sections.

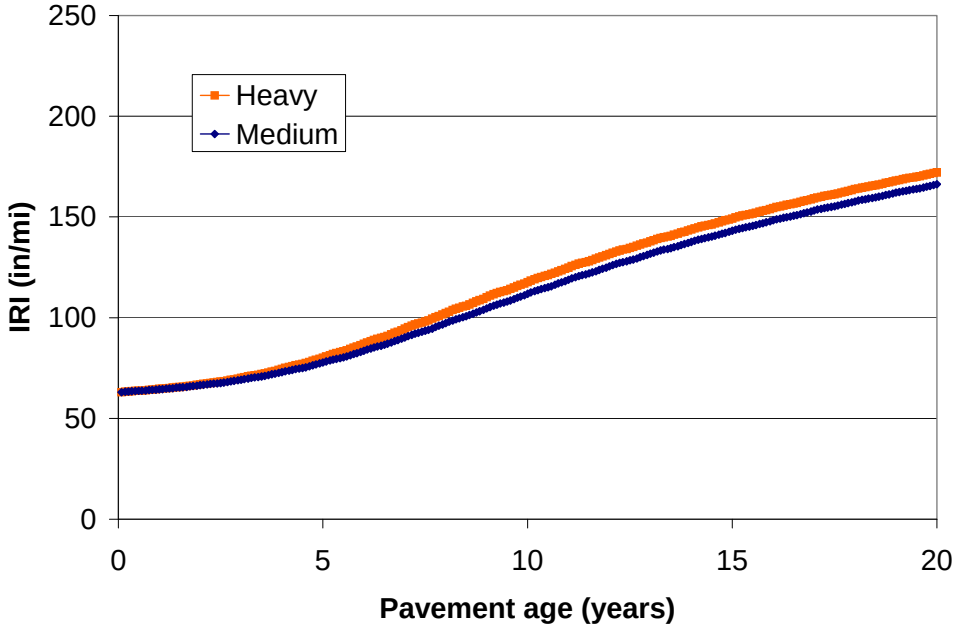


Figure I-51. Comparison of predicted IRI for 9-inch JPCP sections.

It is evident from the figures that the traffic characteristics of the two selected sites did not have significant impact on the key distresses of JPCP sections as the differences in predicted distresses between the heavy and medium duty pavements were generally below 10 percent. The traffic characteristics have only minimal impact on predicted distresses. However, this impact appeared to be more pronounced in the thinner section (8-inch) than the thicker one (9-inch).

Figure I-52 through Figure I-56 present a comparison of predicted distresses of the HMA section as a function of the traffic characteristics at the two selected sites. Figure I-52 and Figure I-53 present the predicted amount of alligator and longitudinal cracking, respectively. Figure I-54 through Figure I-56 present the predicted amount of HMA rutting, total rutting and smoothness of the HMA pavement, respectively.

Figure I-52. Comparison of predicted alligator cracking for heavy and medium-duty HMA sections.

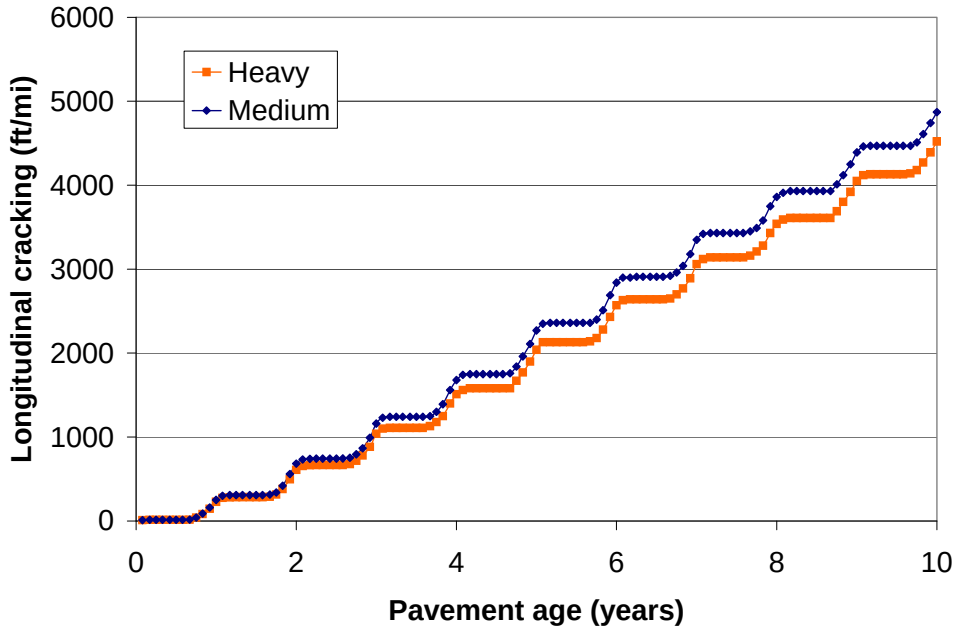


Figure I-53. Comparison of predicted longitudinal cracking for heavy and medium-duty HMA sections.

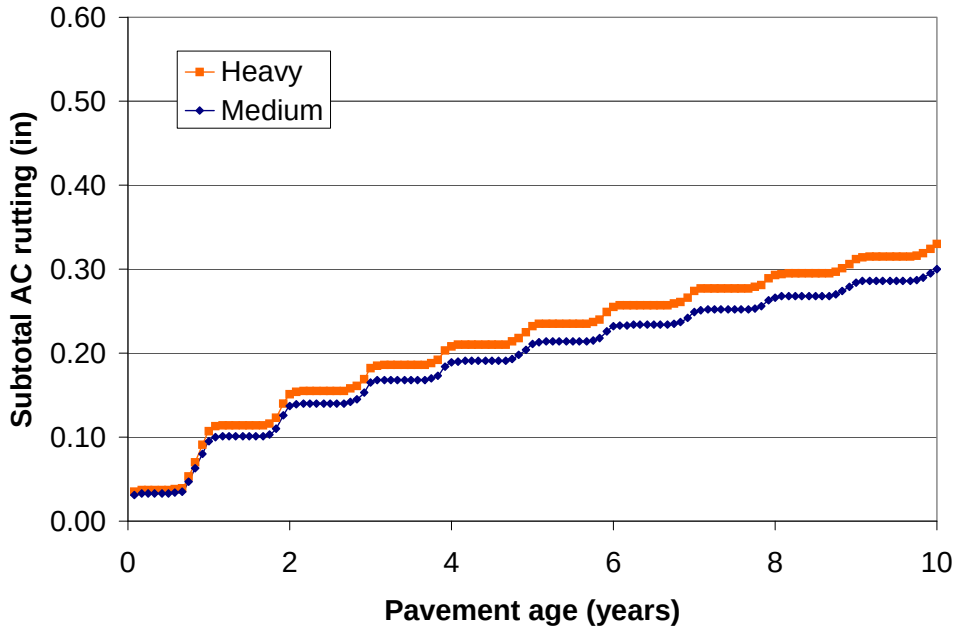


Figure I-54. Comparison of predicted subgrade rutting for heavy and medium-duty HMA sections.

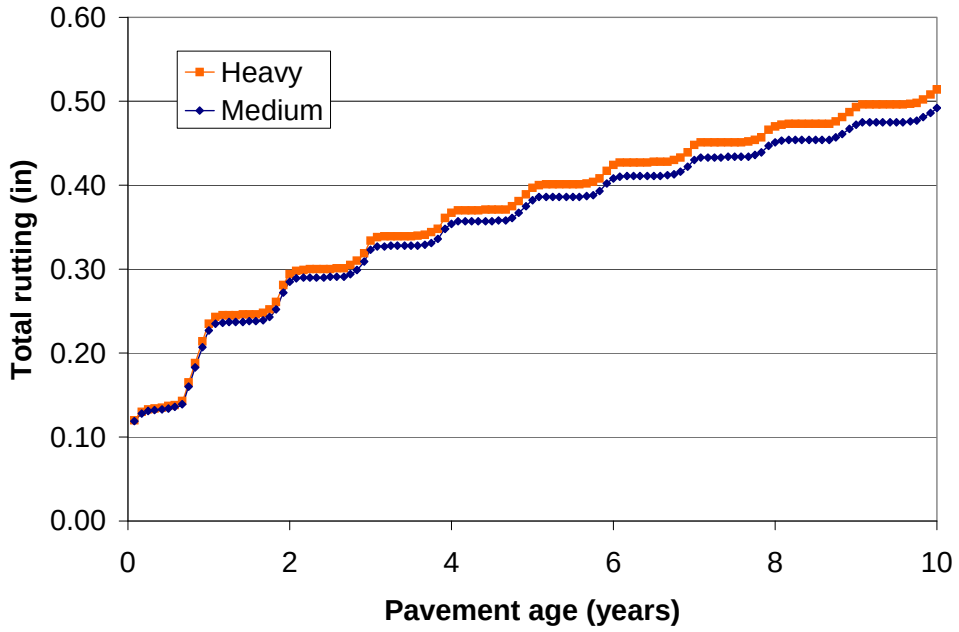


Figure I-55. Comparison of predicted total rutting for heavy and medium-duty HMA sections.

Figure I-56. Comparison of predicted IRI for heavy and medium-duty HMA sections.

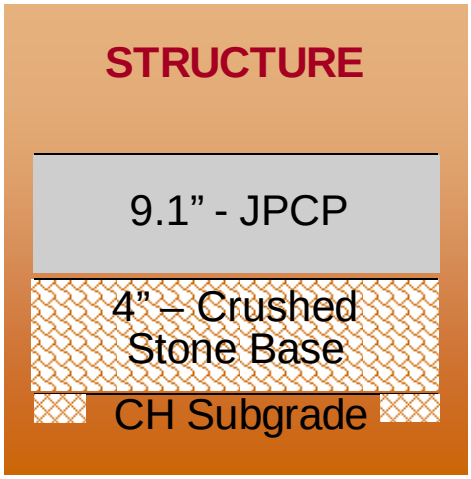
Figure I-52 through Figure I-56 indicate that the traffic characteristics of the two selected sites did not have significant effect on the predicted HMA cracking, rutting and IRI. There was no appreciable difference in the predicted cracking between the heavy and medium duty pavements. The predicted rutting was within the range of 0.25-in. The traffic characteristics appeared to have minimal effect on predicted rutting. Thus, it can concluded from this analysis that the traffic characteristics collected at the two sites, while each representing MoDOT's heavy and medium duty pavements, did not have any significant impact on the predicted performance of JPCP and HMA pavements.

Based on the strength of the analysis presented it can be concluded that the TTC group does not need to be further subdivided to match MoDOT's route designations.

Sensitivity of Predicted Distress/Smoothness to MEPDG Default and Site-by-Site Truck Class and Weight Information

Another analysis was conducted to study (1) how sensitive the MEPDG distress and smoothness predictions are to the axle load spectrum data obtained for each individual MoDOT WIM site and (2) how the distresses/smoothness predicted using MEPDG's default (level 3) vehicle class distribution and axle load spectra compare with those predicted using site specific class and weight data (level 1) all other things being equal. A total of 13 WIM sites were selected for this analysis. As discussed earlier, out of these, 8 sites are in the TTC-1 group, 3 in TTC-2 group and the remaining 2 in the TTC-6 group. For comparison purposes, a two-way AADTT value of 1541 and a compound truck traffic growth rate of 4 percent were assumed for all the sites. The pavement performance predicted using the site-specific traffic inputs were compared with that of the MEPDG level 3 traffic inputs.

The analysis was conducted using a JPCP section, as shown in Figure I-57. All other inputs including climate, material and layer properties remained unchanged. Figure I-57 also provides a brief summary of the pavement structure inputs used in the analysis. The performance measures included the percent slabs cracked, the bottom-up and top-down fatigue damage, the faulting and the IRI.



- Features
  - 20-foot spacing
  - 1.125" dia dowels @ 12 in centers
  - -10 F permanent curl/warp effective gradient
- Concrete mix
  - Compressive Strength = 6900 psi (28-day)
  - Coefficient of Thermal Expansion = 5.3 in/in/deg F
- Crushed stone base
  - Resilient modulus = 24,000 psi
- Subgrade
  - High plasticity clay (CH) subgrade
  - Resilient modulus = 7,825 psi

Figure I-57. JPCP section used for MEPDG traffic sensitivity analysis.

Figure I-58 through Figure I-60 compare the percent slab cracking predicted for various MoDOT sites categorized under TTC groups 1, 2 and 6, respectively. Figure I-61 through Figure I-66 compare the cumulative fatigue damage, bottom-up as well as top-down cracking, for these sites. Figure I-67 through Figure I-72 show the comparison of the faulting and smoothness predictions for these sites.

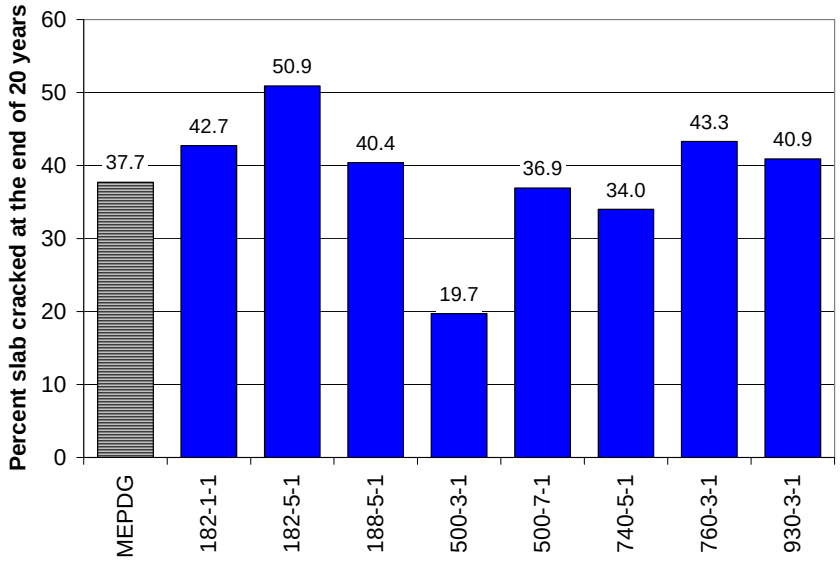


Figure I-58. Comparison of predicted slab cracking using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

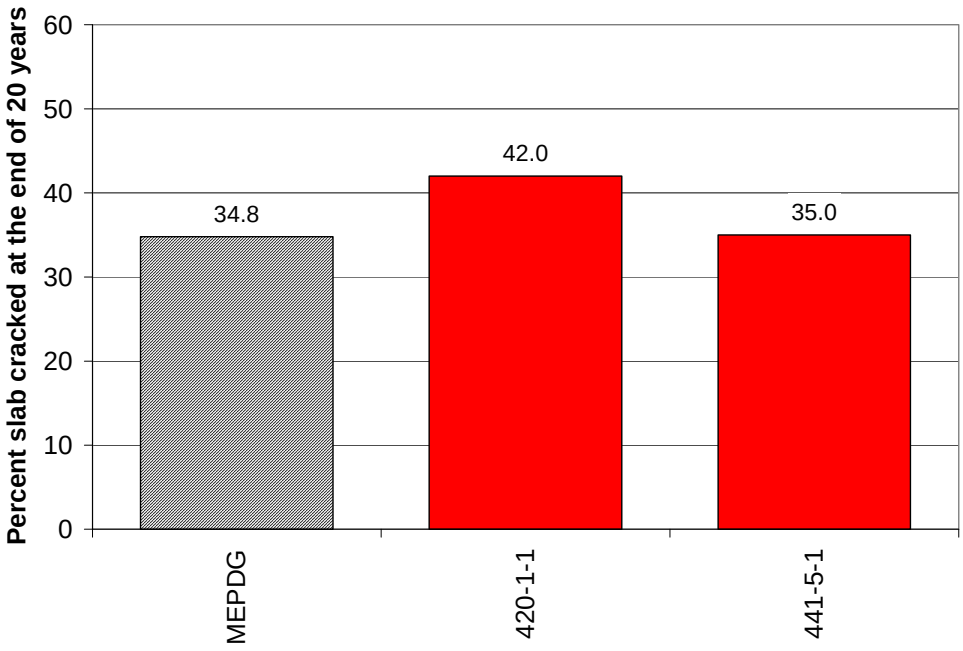


Figure I-59. Comparison of predicted slab cracking using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-2 traffic sites.

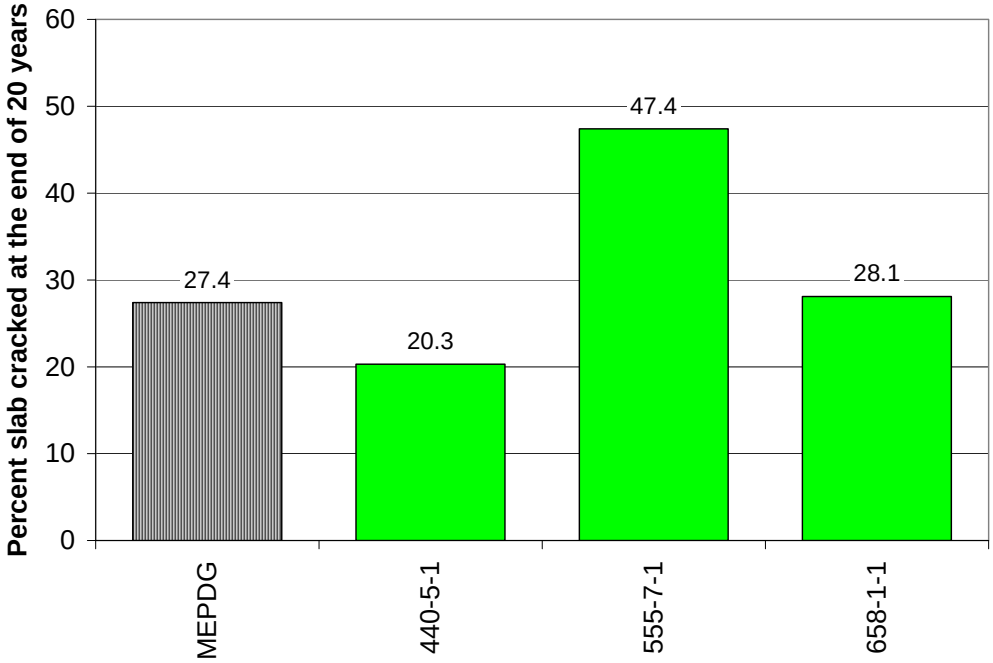


Figure I-60. Comparison of predicted slab cracking using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-6 traffic sites.

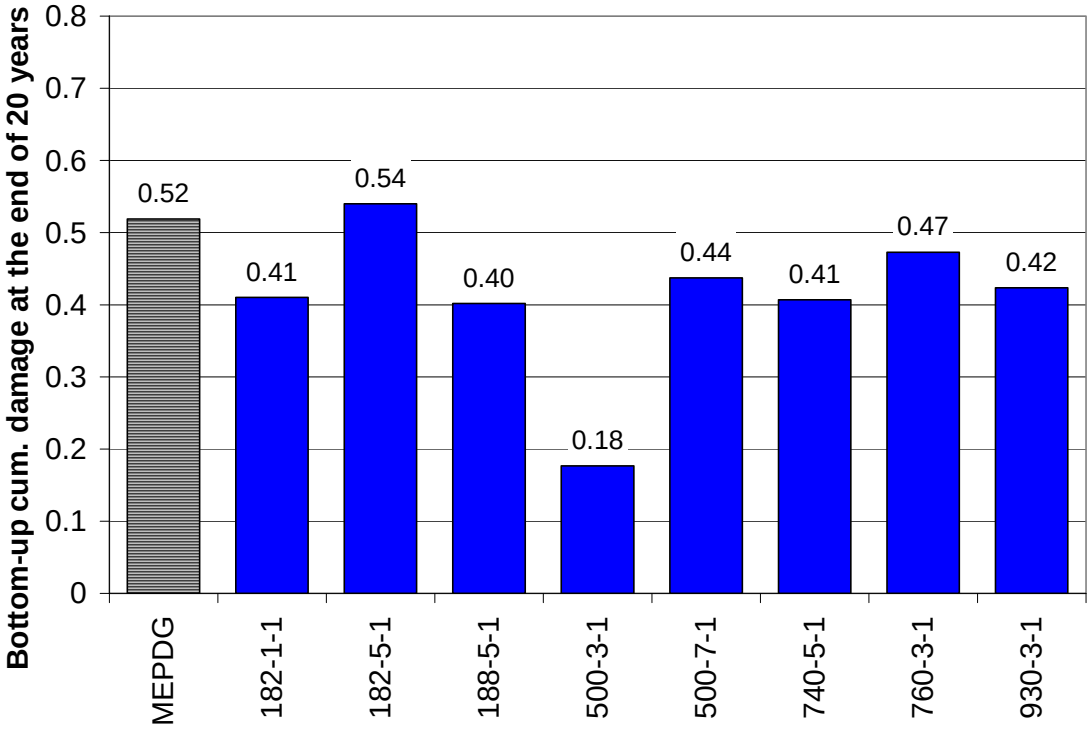


Figure I-61. Comparison of predicted bottom-up damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

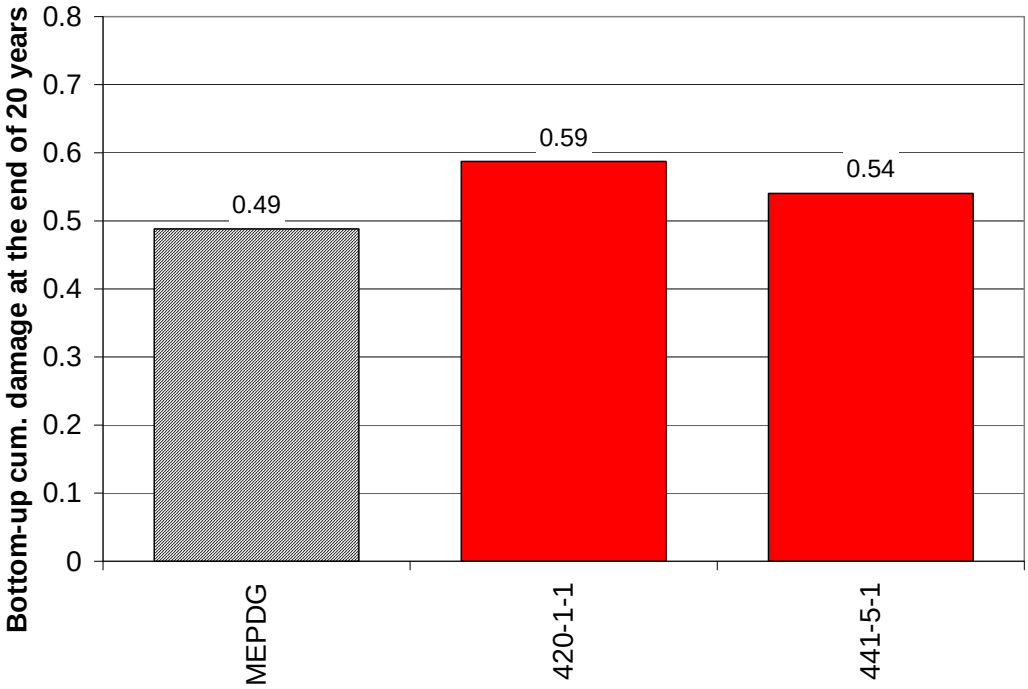


Figure I-62. Comparison of predicted bottom-up damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

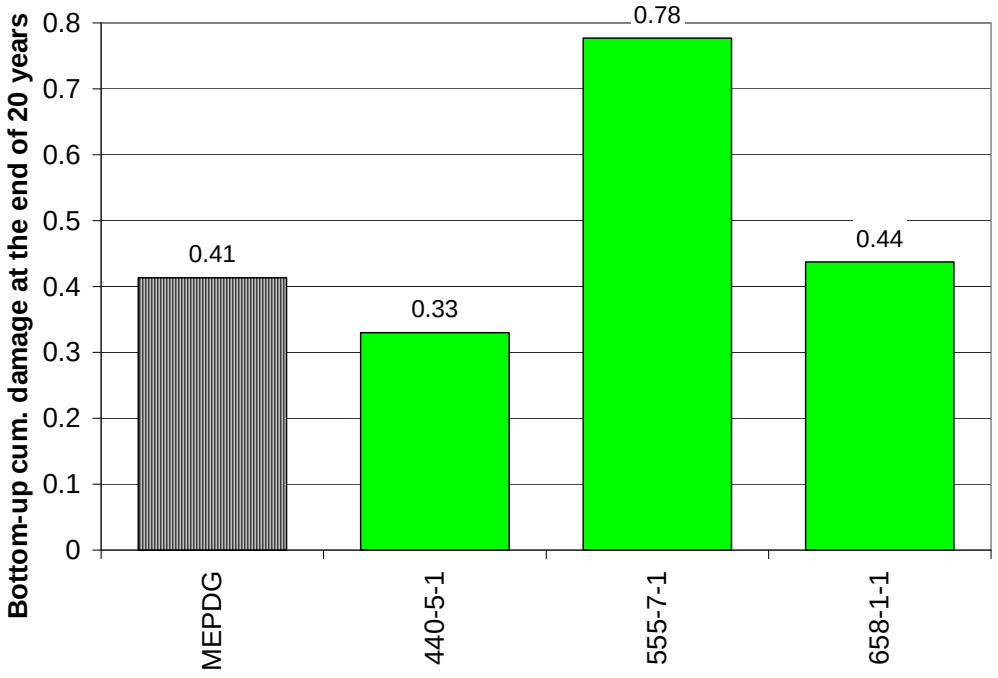


Figure I-63. Comparison of predicted bottom-up damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-6 traffic sites.

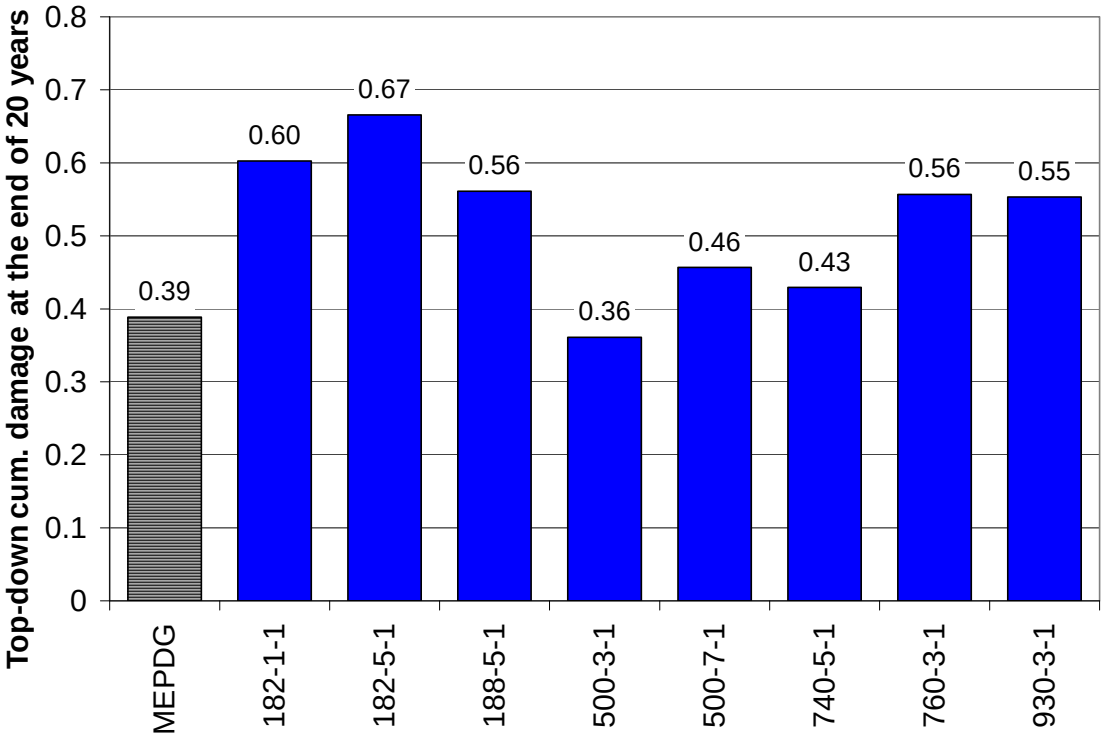


Figure I-64. Comparison of predicted top-down damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

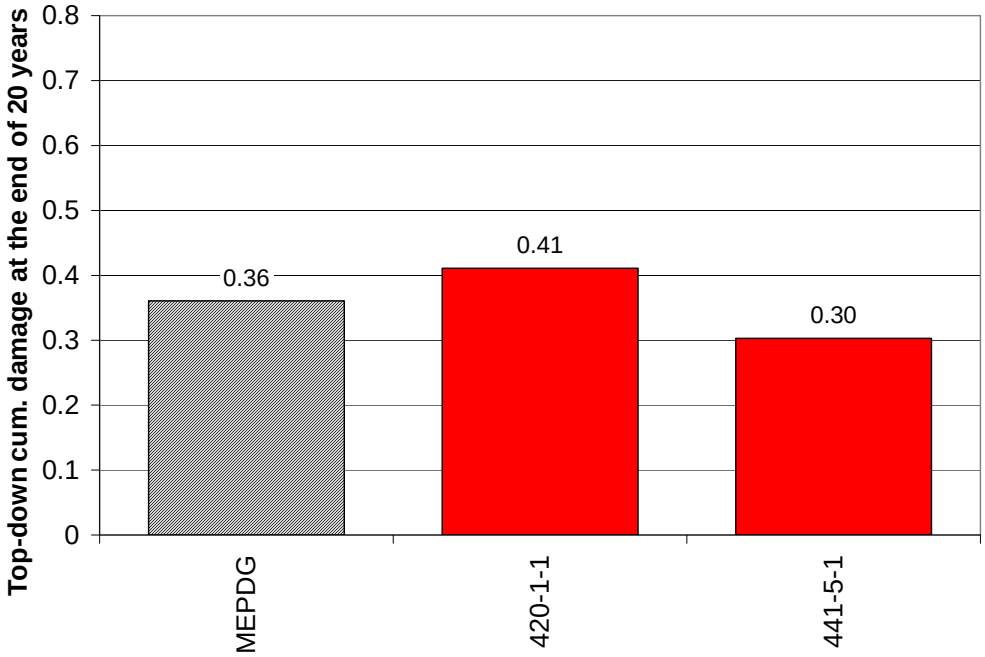


Figure I-65. Comparison of predicted top-down damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-2 traffic sites.

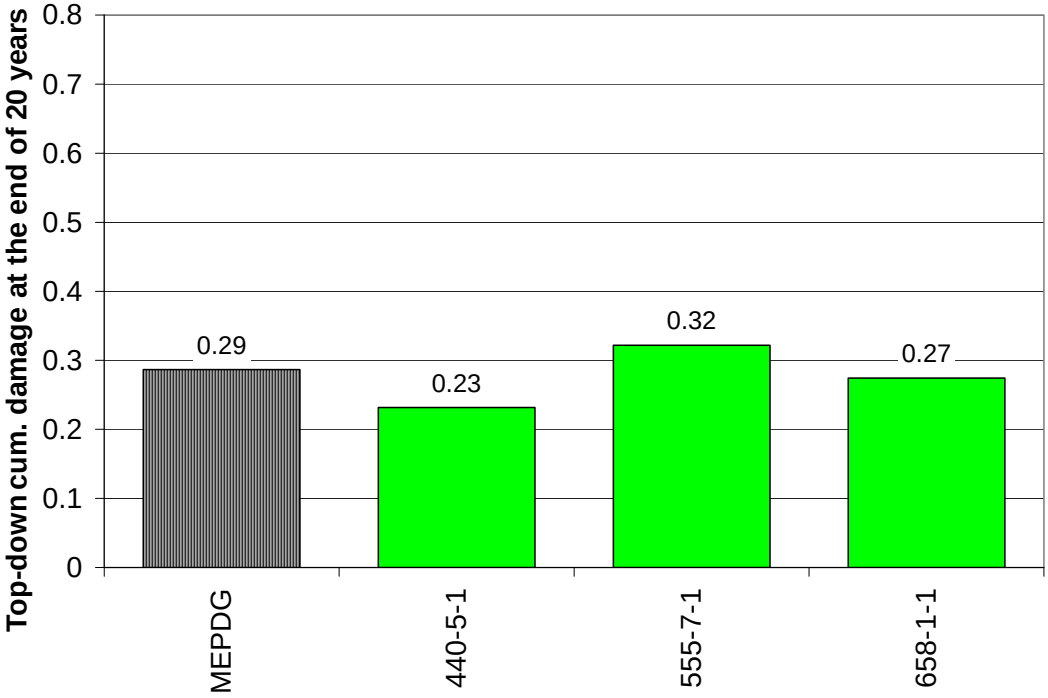


Figure I-66. Comparison of predicted top-down damage using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-6 traffic sites.

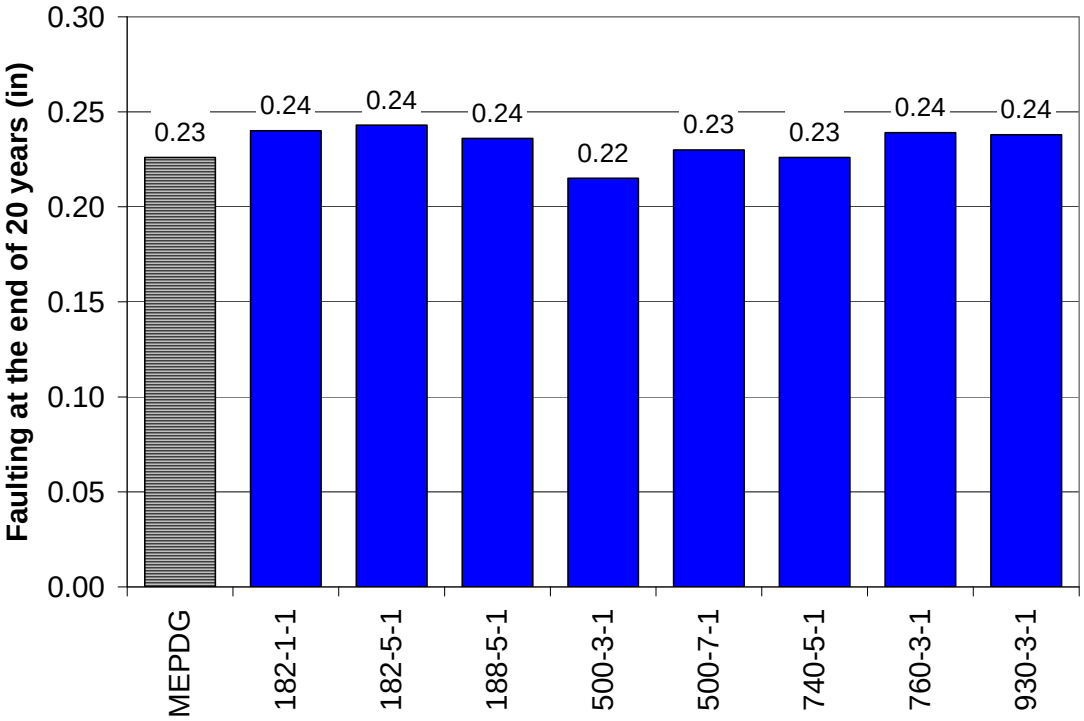


Figure I-67. Comparison of predicted mean joint faulting using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

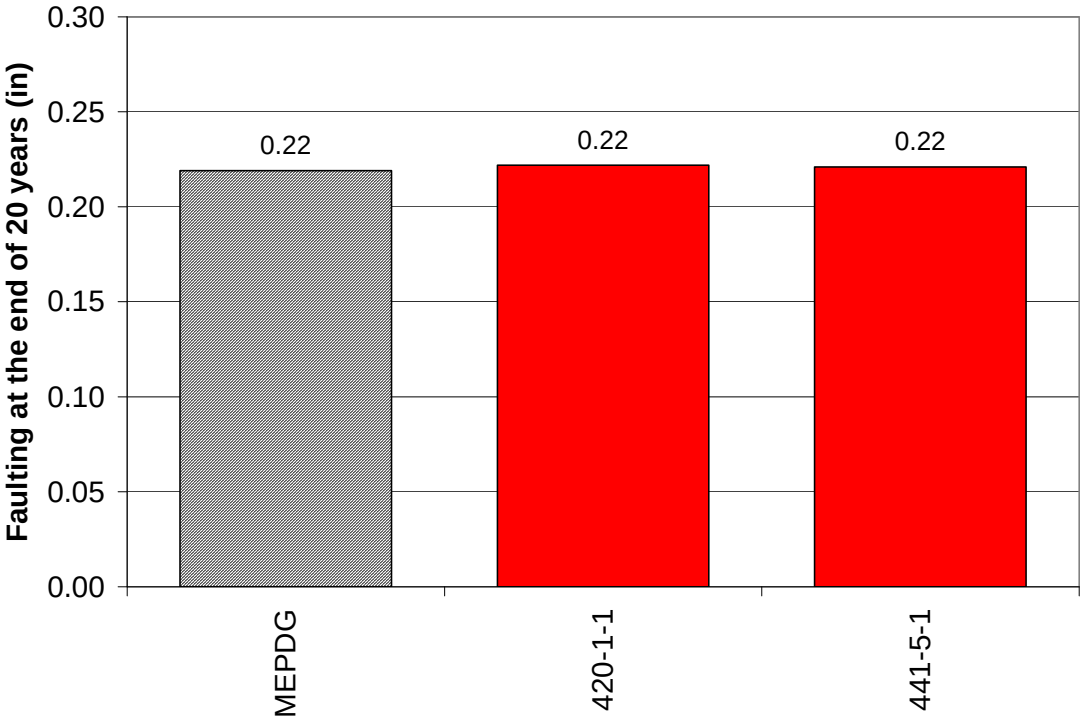


Figure I-68. Comparison of predicted mean joint faulting using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-2 traffic sites.

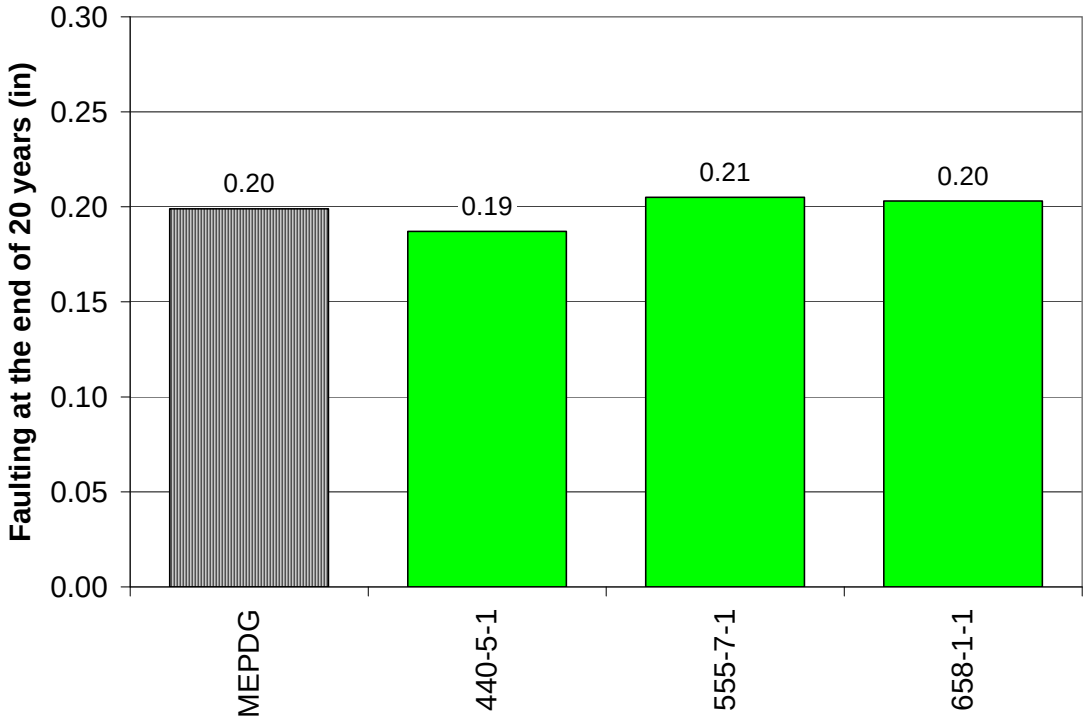


Figure I-69. Comparison of predicted mean joint faulting using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-6 traffic sites.

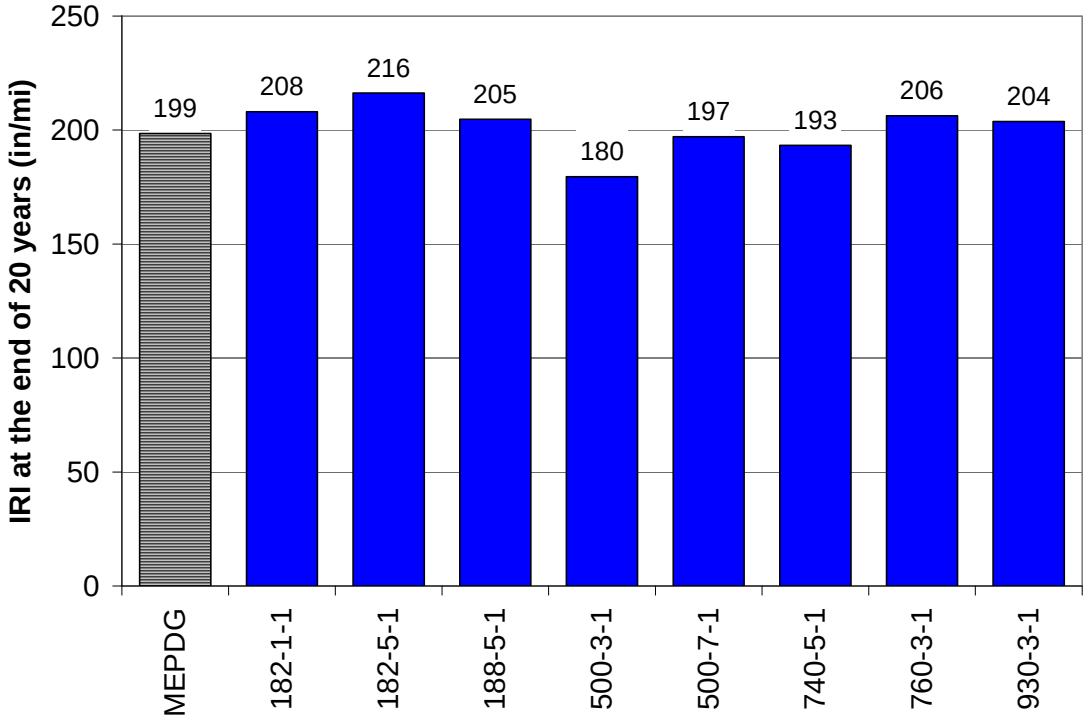


Figure I-70. Comparison of predicted smoothness using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-1 traffic sites.

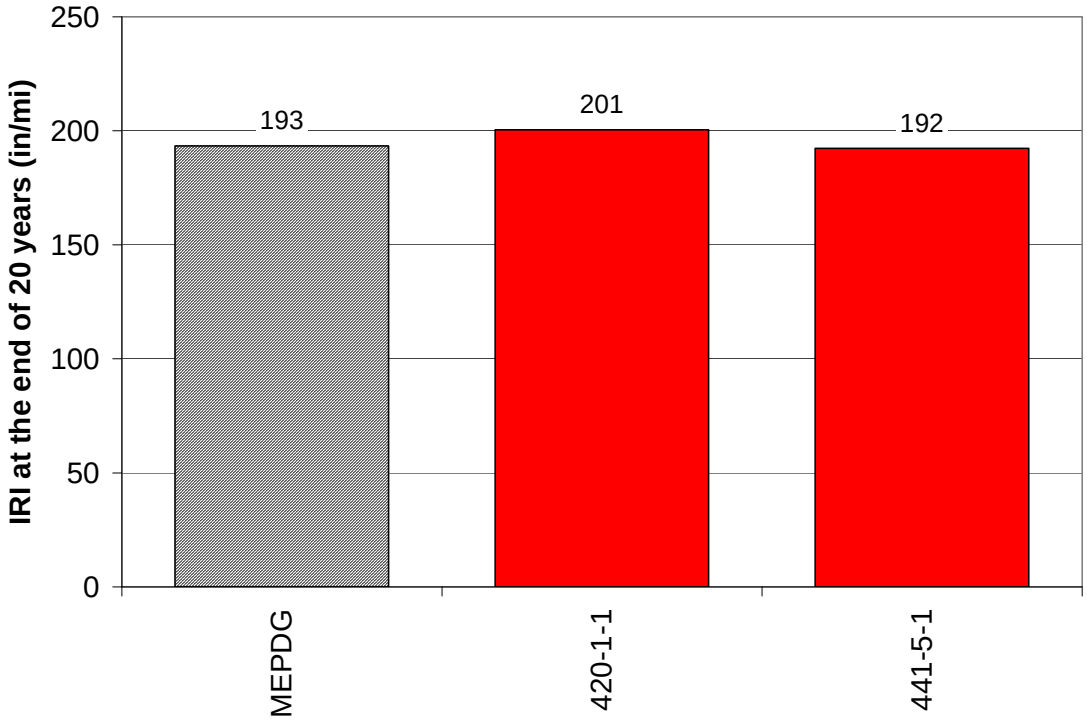


Figure I-71. Comparison of predicted smoothness using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-2 traffic sites.

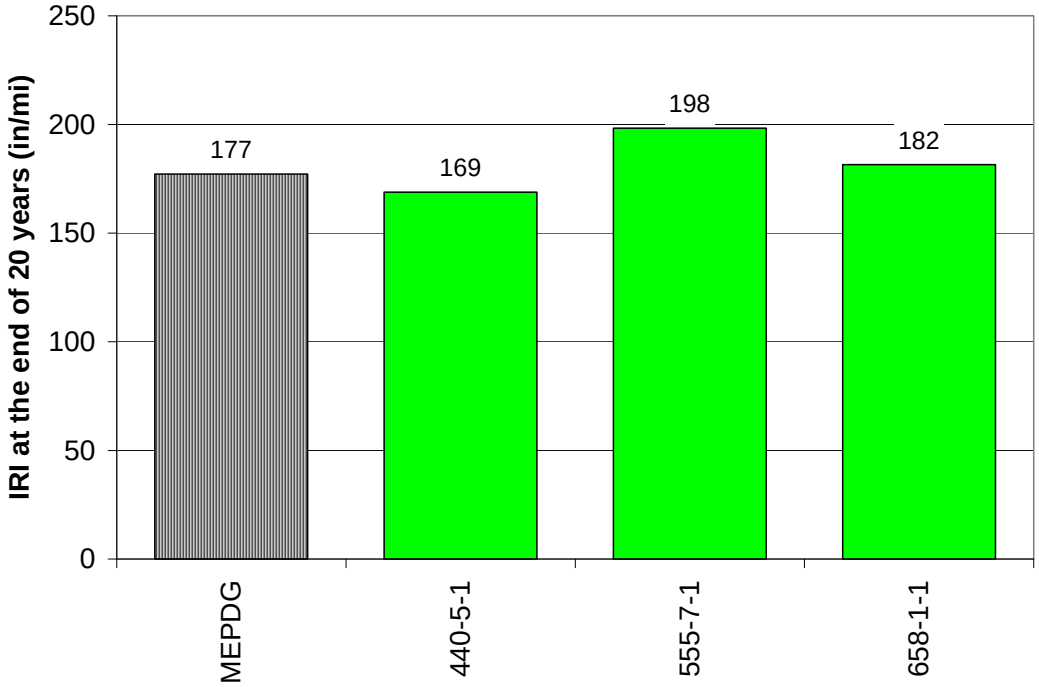


Figure I-72. Comparison of predicted smoothness using truck class and weight data from the MEPDG (level 3) and MoDOT's TTC-6 traffic sites.

A comparison of performance predictions indicate that the site-specific traffic inputs were not significantly different from the MEPDG default traffic inputs in predicting JPCP faulting and smoothness. The variability of faulting and smoothness predictions of MoDOT sites utilizing site-specific inputs were within a 5 percent range from the predicted values of level 3 inputs. The same trend was observed in all the three TTC groups.

However, in the TTC-1 group, the percentage of cracked slabs at sites 182-5-1 and 500-3-1 were significantly different from the level 3 predictions. The predicted cracking at the sites 182-5-1 and 500-3-1 were 35% and 48% different from the level 3 predictions respectively. The predicted cracking for all other sites in this group were not significantly different from the level 3 prediction. Similarly, in the TTC-6 group, the predicted cracking at the site 555-7-1 was 73% higher than the level 3 prediction. However, in the TTC-2 group, the level 1 cracking predictions were not significantly different from the level 3 prediction.

The cumulative amount of bottom-up fatigue damage followed a similar trend as the JPCP cracking. The top-down cracking, predictions followed a slightly different trend than those of slab cracking. For most sites in the TTC-1, the predicted values of top-down damage were significantly different from the level 3 predictions, whereas the differences in predicted damage were not significant in TTC groups 2 and 6.

The percentage of cracked slabs is a combined measure of both bottom-up and top-down cracking. Under typical conditions, although there is potential for either mode of cracking to occur, the cracking in a given slab can be initiated by only one mode, but not both. So it is meaningful to consider the percentage of slabs with cracks for further analysis.

Several traffic input parameters were considered to determine the reasons for variability in predicted slab cracking. In order to determine the reasons for variability in predicted cracking among TTC-1 sites, several traffic input parameters such as the percentage of dominant truck types, number of single, tandem and tridem axles, percent single or tandem axles exceeding legal limits, were examined. Out of several key traffic inputs, the percentage of tandem axles exceeding the MoDOT legal limits (34 kips for tandem axles) and the percentage of predominant truck class (Class 9 for TTC-1) appeared to have a profound effect on the predicted cracking.

Figure I-73 plots the predicted cracking at various MoDOT sites (TTC-1) with their corresponding percent tandem axles exceeding 34 kips and the percent class 9 trucks. These two traffic inputs together have a statistical correlation (coefficient of determination, R^2) of 0.86 with slab cracking. Site 182-5-1, where the predicted cracking was 35% higher than the level 3 prediction, have 21.9% of tandem axles exceeding the legal limit (over-loaded condition) and 76.8% of class 9 trucks. Site 500-3-1, where the predicted cracking was 48% lower than the level 3 prediction, have 0.4% of tandem axles in over-loaded condition and 76.3% of class 9 trucks. The MEPDG level 3 inputs assume the percentage of tandem axles in over-loaded condition to be 13.9% and the percent class 9 trucks in the TTC-1 is 74%. The performance predictions of JPCP will be reasonable and conservative as long as the percentage of over-weight axles, especially tandem axles, is not expected to exceed 15 percent on any roadway. This observation holds good only for the TTC-1 and MoDOT specific traffic conditions discussed in the report.

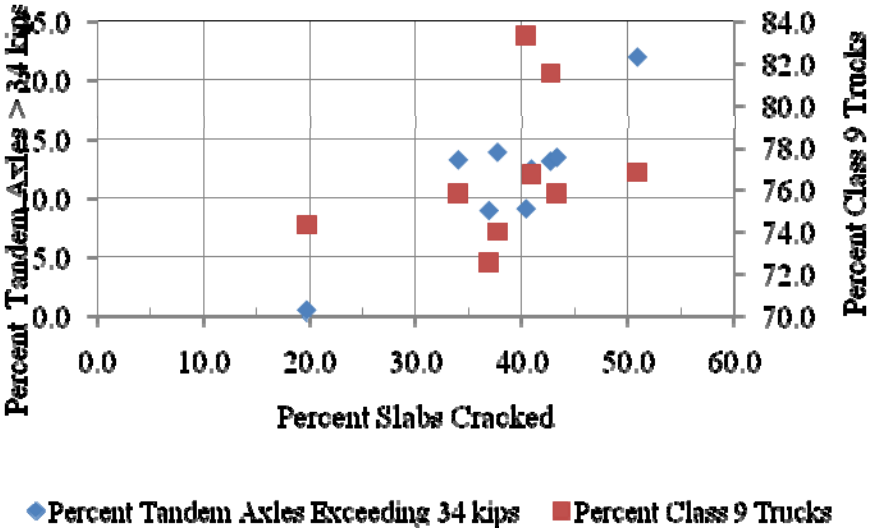


Figure I-73. Percentage of slabs cracked versus traffic inputs for TTC 1.

In the TTC-6 group, the cracking prediction for the site 555-7-1 was much higher (73%) than the level 3 predictions and other two sites. The traffic parameters of the site 555-7-1 were reasonably closer to those of other two sites and the level 3 inputs. However, the monthly adjustment factor for this site was significantly different from other sites, as shown in Figure I-74. This site 555-7-1 is a LTPP section on U.S. Route 54 (Rural Principal Arterial) in Miller County, MO. At this location, the average truck traffic in spring and summer (April through August) was 30% higher than the annual average. The roadway carried almost 46% higher and 38% lower than the annual average truck traffic in June and December, respectively.

Figure I-75 shows the typical variation in layer properties observed for the assumed JPCP features in this analysis. The figure illustrates how the properties of base course and subgrade could change for the assumed site conditions. It can be noted that the modulus of aggregate base bottomed out with a trough in non-freezing months (March to November). The truck traffic was about 35 % lower than the annual average in the months of December, January and February. In this case, assuming the annual average truck traffic constant for all the months would have resulted in under-prediction of distresses, as a significant proportion of truck traffic was re-distributed from warmer months to frozen base conditions in winter. Any significant deviation in the seasonal distribution of truck traffic could result in under-prediction or over-prediction of distresses. This seasonal variation in truck traffic at this site in conjunction with the seasonal variation in the layer properties of the JPCP could be cited as reasons for the observed variability in percent slab cracking.

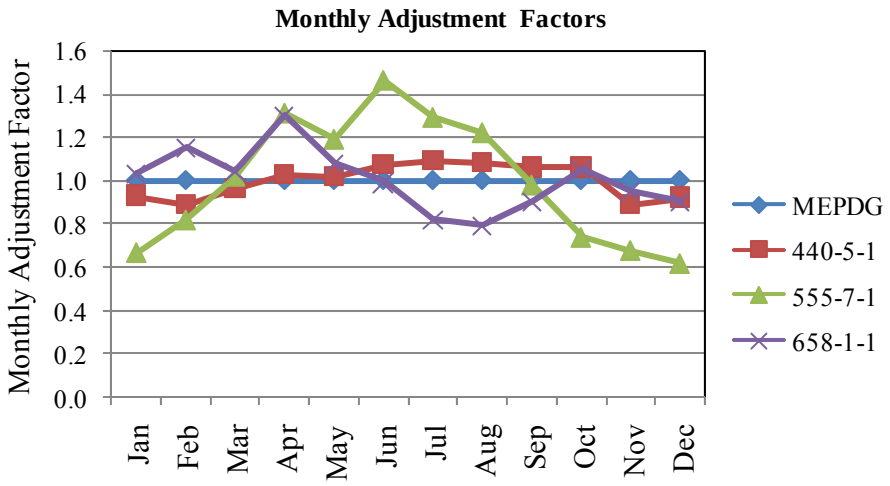


Figure I-74. Monthly adjustment factors for TTG 6 sites.

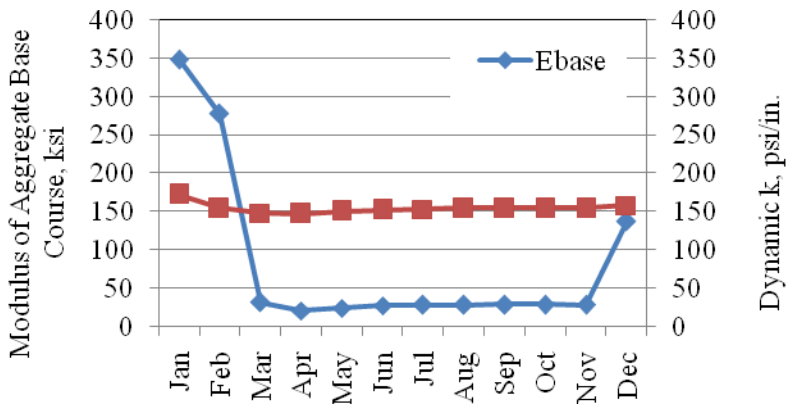


Figure I-75. Typical seasonal variation of base and subgrade properties.

3.2.5 Summary and Conclusions

The following conclusions can be drawn from the traffic data analysis performed as part of this study. However, it should be borne in mind that these findings are applicable to Missouri's rural Interstate and primary highway system only, since most of the data analyzed were from these classes of pavements. Urban Interstates and farm-to-market or haul roads need more WIM data representation. However, it is assumed in the interim that the load data derived for rural Interstate highways will hold true for urban Interstates as well. Volume and vehicle class distributions need to be better quantified for urban Interstates (which is already being done using MoDOT's existing traffic infrastructure).

- The quality of available data for continuous WIM sites is generally adequate.
- The quality of portable WIM data collection needs improvement (not presented here, but that was one of the reasons to select only the permanent sites for analysis in this study).

- The definition of the TTC groups in the MEPDG adequately describes the vehicle class distributions in MoDOT.
- MEPDG defaults for vehicle class distributions, axle load spectra, axles per truck, and hourly truck usage can be used in routine design. It is recommended that special data collection efforts be made when designing roadways where it is suspected that load spectra will be very different for the key truck and axle types than the default values proposed in the MEPDG.
- To a large extent, the MEPDG default monthly adjustments can be used for routine design purposes. However, significant monthly variations exist on some roadways. If the monthly truck distributions are suspected to be very different for a roadway under consideration, special data collection efforts to capture this information is recommended.

For each of the sites evaluated, traffic data files in a format that can be read into the MEPDG have been created and stored in the project database.

3.3 HMA MATERIALS CHARACTERIZATION

3.3.1 Overview

The intent of the laboratory test program undertaken by MoDOT in support of the MEPDG is to focus on four products, which are listed below:

- Develop a set of default material property values that can be used in the flexible pavement design process for the Level 1 inputs for HMA.
- Determine the effect of air voids on the dynamic modulus, permanent deformation coefficients, and indirect tensile strength and failure strain over a range of mixtures used in Missouri.
- Confirm the dynamic modulus regression equation included in the MEPDG software for conventional and specialty (polymer modified asphalt [PMA]or SMA) mixtures.
- Provide HMA material property data to improve on the local calibration process and reduce the error term for the predicted distresses.

Based on MoDOT's laboratory testing needs assessment (discussed in chapter 2), the following tests were conducted to determine the required HMA mixture inputs for typical MoDOT mixes representing current materials and construction practices:

- Compressive dynamic modulus, E*.
- Compressive repeated load permanent deformation testing.
- Indirect tensile strength, failure strain, and creep compliance.

The dynamic modulus and repeated load permanent deformation testing was conducted by personnel from MoDOT's central materials laboratory. The indirect tensile strength and creep compliance testing was performed by under a MoDOT contract by the University of Missouri at Rolla (Richardson & Lusher 2008).

In late 2004, MoDOT embarked on procuring a suitable device to perform the mixture testing. At the outset, a policy decision was made to use the Simple Performance Tester (SPT) developed under NCHRP Project 9-29 for the E* and repeated load testing rather than the AASHTO TP 62. An IPC Global SPT equipment (see Figure I-76) was acquired through a competitive bid process to perform the required testing and was housed MoDOT.

Figure I-76. SPT equipment used by MoDOT to perform HMA mixture testing.

3.3.2 Dynamic Modulus Testing

Level 1 Testing and Data Analysis

Nine different mixture types and three air void levels (4, 6.5, and 9) were included in the lab testing program for E*. Key details regarding these mixes are presented in Table I-16.

Table I-16. Details of MoDOT mixes tested for E*.

Mix Designation	Mix Type	Mix No.	AC Binder Type	AC Binder Gs	Gmm	Gmb	Gsb	Air Voids	VMA	VFA	Nini	Ndes	Nmax	Asphalt Content (by weight)
SP125	SMA[1]	05-94	PG 76-22	1.027	2.706	2.598	2.961	4.0	18.0	78		100		6.5
SP125	PMA[2]	05-119	PG 76-22	1.030	2.448	2.349	2.616	4.0	15.2	74	9	125	205	5.6
SP190	PMA	05-106	PG 76-22	1.033	2.450	2.352	2.564	4.0	13.1	70	9	125	205	5.1
SP190	Neat[3]	06-46	PG 64-22	1.036	2.598	2.493	2.726	4.0	13.1	69	8	100	160	5.0
SP190	PMA	05-115	PG 70-22	1.033	2.464	2.365	2.600	4.0	13.3	70	8	100	160	4.7
SP 190		4-95												
SP250	Neat	05-121	PG 64-22	1.030	2.519	2.417	2.629	4.0	12.8	69	8	100	160	5.1
SP250	PMA	06-18	PG 70-22	1.033	2.482	2.382	2.601	4.0	12.1	67	8	100	160	4.0
BP-1	Marshall	05-161	PG 64-22	1.033	2.459	2.355	2.567	4.2	13.5	69				5.7

1 Stone matrix asphalt (with fibers).
2 Polymer (SBS or SBR) modified asphalt binder.
3 Unmodified.
4 Note that information for 4-95 was missing.

A frequency sweep (25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz, and 0.1Hz) of the dynamic modulus testing was conducted at four temperatures (40, 70, 100, and 130 ^{o}F) for each mix type (except mix 5-115) and air void level. Master curves and the corresponding shift factors were developed by shifting laboratory frequency sweep data from different temperatures to a reference temperature of 70 ^{o}F. No testing was conducted at 70 ^{o}F for mix 5-115, and its master curve was developed at a reference temperature of 100 ^{o}F.

Table I-17 gives the model parameters and the shift factors for each mix at each air void level. With these model parameters and shift factors, any E* value at the temperature of interest and/or the loading rate of interest for the specific mix can be obtained for design purposes.

Table I-17. Model parameters and shift factors for MoDOT typical mixes (materials library).

		Model parameters				Shift Factors			
Mix type	Sample AV level	α	β	γ	δ	a(40)	a(70)	a(100)	a(130)
5-115	6	3.2540	-1.2919	0.4714	3.3031	0.0175	1	64.159	3999.956
5-115	9	4.0618	-1.5509	0.4250	2.4603	0.0052	1	90.253	4998.997
5-115	4	5.4324	-1.8352	0.3315	1.2767	0.0055	1	69.662	4999.772
5-106	4	5.6883	-1.9470	0.2492	0.9695	0.0037	1	270.742	9995.096
5-106	6	4.3771	-1.4366	0.2804	2.2345	0.0032	1	138.010	9998.123
5-106	9	4.5750	-1.4784	0.2429	2.0800	0.0024	1	144.850	7499.986
5-94	4	4.4156	-1.4411	0.2869	2.3049	0.0035	1	201.606	9989.977
5-94	9	3.2472	-1.3113	0.4727	3.2813	0.0252	1	128.747	9990.842
5-94	6	3.1509	-0.9828	0.4181	3.4256	0.0030	1	44.422	2499.741
4-95	4	3.0838	-0.8171	0.4483	3.4848	0.0008	1	6.575	349.673
4-95	9	3.1458	-1.1225	0.5407	3.2579	0.0075	1	25.863	765.494
4-95	6	3.6727	-1.4760	0.4725	2.8397	0.0107	1	37.112	1400.726
6-46	4	4.0532	-1.2468	0.4116	2.6469	0.0030	1	40.669	1999.219
6-46	9	3.8610	-1.0283	0.4023	2.7827	0.0017	1	22.938	1399.935
6-46	6	2.8322	-0.9393	0.6331	3.6934	0.0037	1	26.854	1299.942
6-18	4	2.7390	-1.2971	0.6374	3.7491	0.0076	1	34.067	1302.498
6-18	9	3.1619	-1.2333	0.5578	3.3002	0.0032	1	33.341	1302.499
6-18	6	2.7644	-1.2516	0.6915	3.6262	0.0072	1	32.539	1800.063
5-161	4	3.4517	-1.4099	0.4039	3.1131	0.0027	1	98.543	4499.493
5-161	9	4.0941	-1.7430	0.3748	2.4037	0.0018	1	49.726	7200.093
5-161	6	6.1236	-2.1451	0.2917	0.5028	0.0018	1	100.394	4999.988
5-121	4	3.6723	-0.6130	0.4808	3.0159	0.0001	-	1.000	19.999
5-121	9	3.3235	-0.3146	0.5809	3.1904	0.0002	-	1.000	26.612
5-121	6	2.8910	-0.1473	0.6678	3.6272	0.0003	-	1.000	33.707
5-119	4	5.4348	-1.9251	0.3217	1.2266	0.0045	1	71.940	9999.914
5-119	9	3.3763	-1.3668	0.3923	3.1462	0.0013	1	72.455	9999.913
5-119	6	6.1240	-1.7636	0.2668	0.6774	0.0093	1	57.423	9999.841

Verification of the Accuracy of MEPDG Dynamic Modulus Prediction Equation (Level 2)

To verify the accuracy of the MEPDG E* predictive model for MoDOT mixes at input Level 2, the master curves for seven mixtures (with the exception of Mix Nos. 5-94 and 5-161; no mix information was not available) at three air void levels were developed directly from the dynamic modulus prediction equation included in the MEPDG (ARA 2004), as shown in equation 1.

$$\log E^* = 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a$$
$$-0.802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1+e^{(-0.603313-0.313351\log(f)-0.393532\log(\eta))}} \quad (1)$$

where:

E* = dynamic modulus, psi
η = bitumen viscosity, 10^6 Poise.
f = loading frequency, Hz
V_a = air void content, %
V_{beff} = effective bitumen content, % by volume
ρ_{34} = cumulative % retained on the ¾ in sieve
ρ_{38} = cumulative % retained on the 3/8 in sieve
ρ_4 = cumulative % retained on the No.4 in sieve
ρ_{200} = cumulative % retained on the ¾ in sieve

Because the viscosity of the asphalt binder (η) at the temperature of interest is not available, the ASTM viscosity temperature relationship (equation 2) is used.

$$\log\log\eta = A + VTS\log T_R \quad (2)$$

where:

η = bitumen viscosity, cP.
TR = temperature, Rankine
A = regression intercept
VTS = regression slope of viscosity temperature susceptibility.

The A and VTS regression parameters (unaged) are determined based on the binder PG grade according to the default value provided in the MEPDG software.

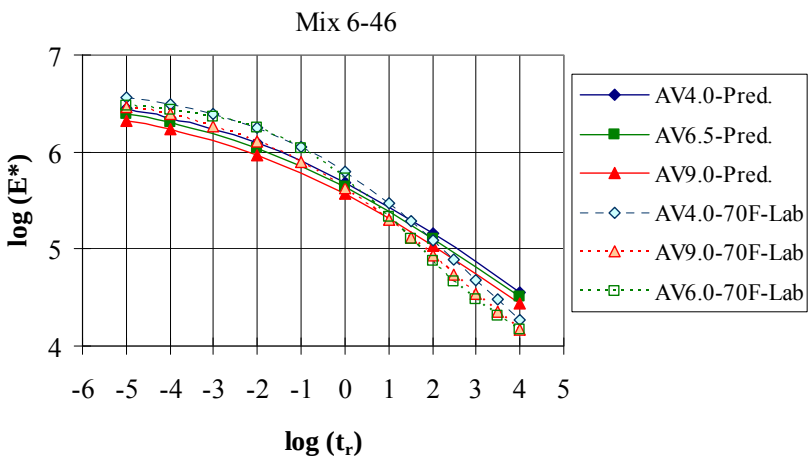
Table I-18 lists the gradation and binder properties of the seven mixtures. With materials volumetric properties, gradation information and binder viscosity values, the master curve for mixtures was generated at a reference temperature of 70 °F (at 100 °F reference temperature for mix 5-121).

Master Curves Comparison

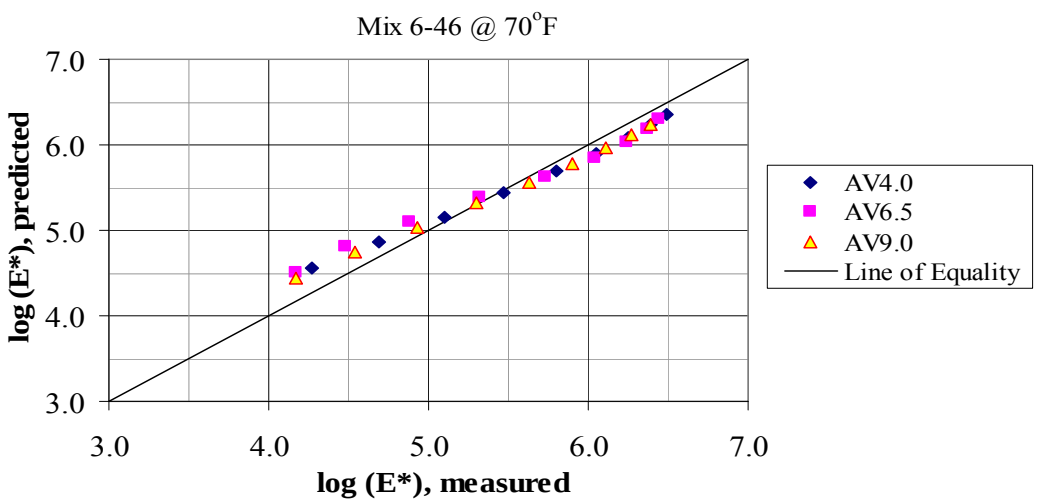
Figure I-77 through Figure I-83 compare the master curves developed based on laboratory testing with those developed from prediction equation for each mix. The first part of each figure (i.e., all figure [a]'s), the solid lines are from prediction equation and the dashed lines represent the master curves based on lab testing results. The curves are separated for each air void level. In the second part of each figure (i.e., all figure [b]'s), an equality line is drawn to show how the measured dynamic modulus (i.e., log E*) compares with the predicted values for each mix.

Table I-18. Mixture gradation and binder properties.

Mix ID	Mix Type	Mix Designation	$P_{3/4}$ %	$P_{3/8}$ %	P_{No4} %	P_{No200} %	Binder Type	A	VTS
05-94	SMA	SP125	100.0	59.1	27.9	8.9	PG 76-22	9.715	-3.208
05-119	PMA	SP125	100.0	80.3	51.9	5.2	PG 76-22	9.715	-3.208
05-106	PMA	SP190	100.0	74.2	40.1	4.5	PG 76-22	9.715	-3.208
05-115	PMA	SP190	100.0	74.3	37.5	3.9	PG 70-22	10.299	-3.426
06-18	PMA	SP250	89.7	62.0	41.8	3.4	PG 70-22	10.299	-3.426
06-46	Neat	SP190	98.5	76.7	43.7	3.6	PG 64-22	10.980	-3.680
05-121	Neat	SP250	88.7	71.8	37.9	4.7	PG 64-22	10.980	-3.680

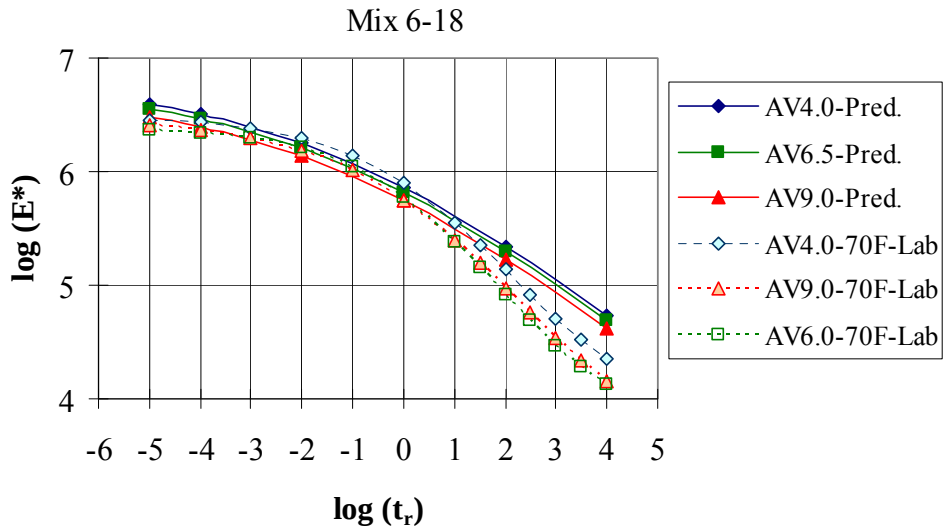


(a)

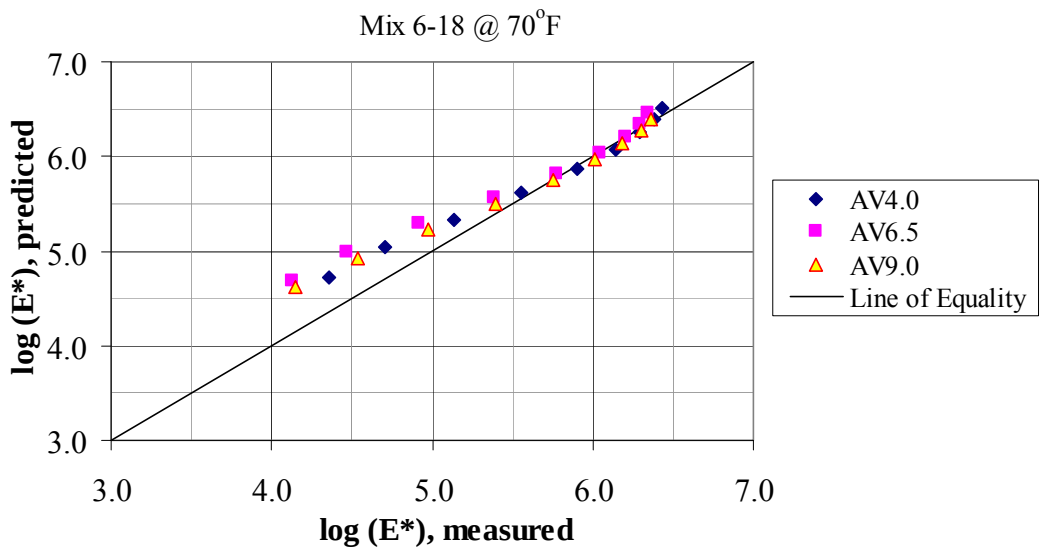


(b)

Figure I-77. Mix 6-46 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

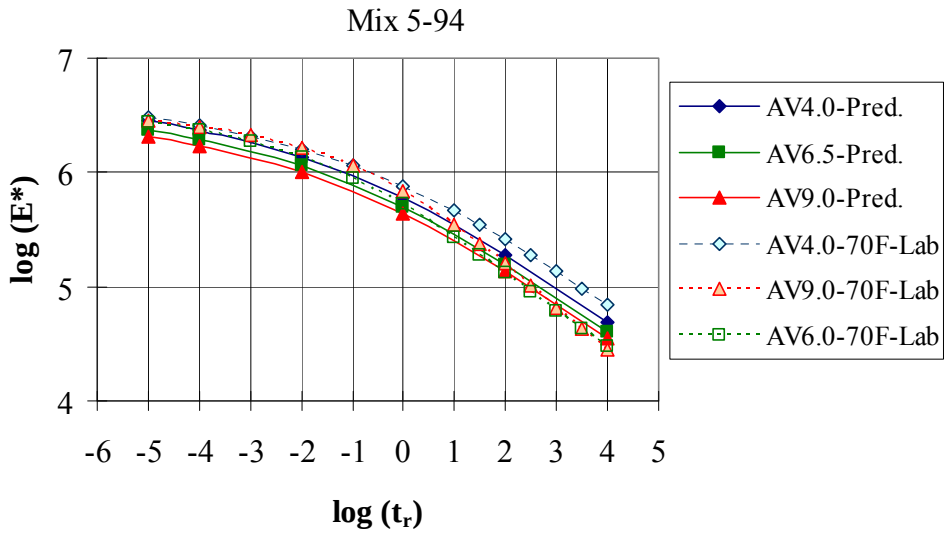


(a)

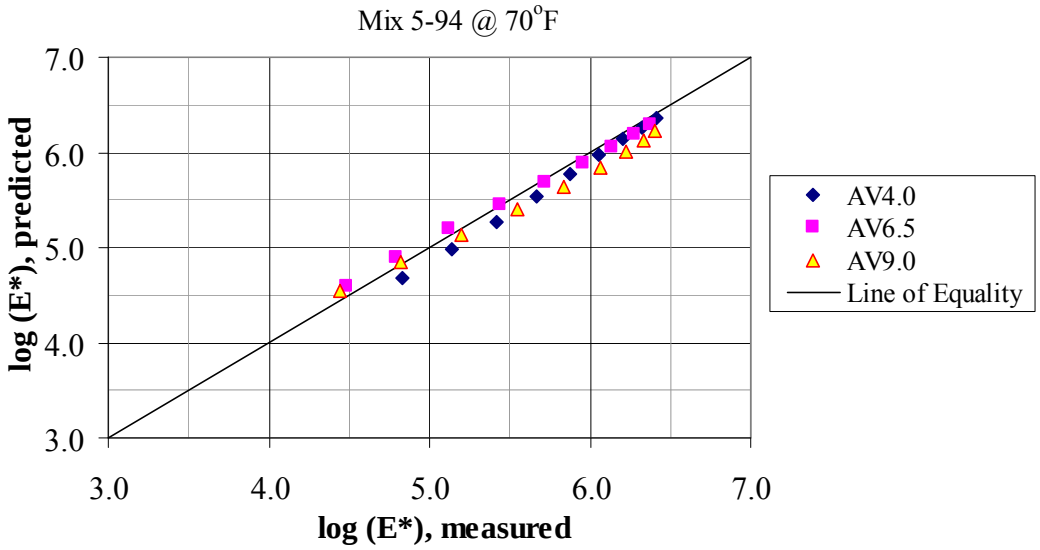


(b)

Figure I-78. Mix 6-18 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

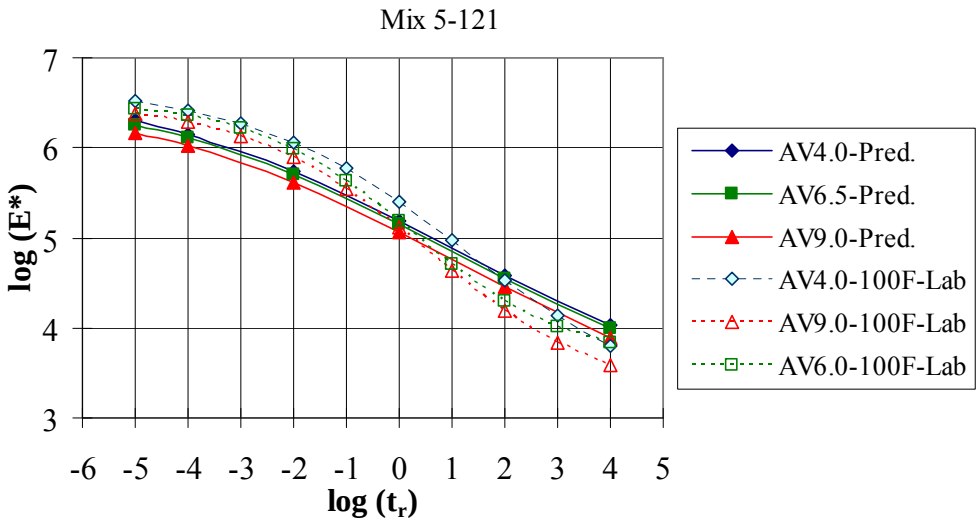


(a)

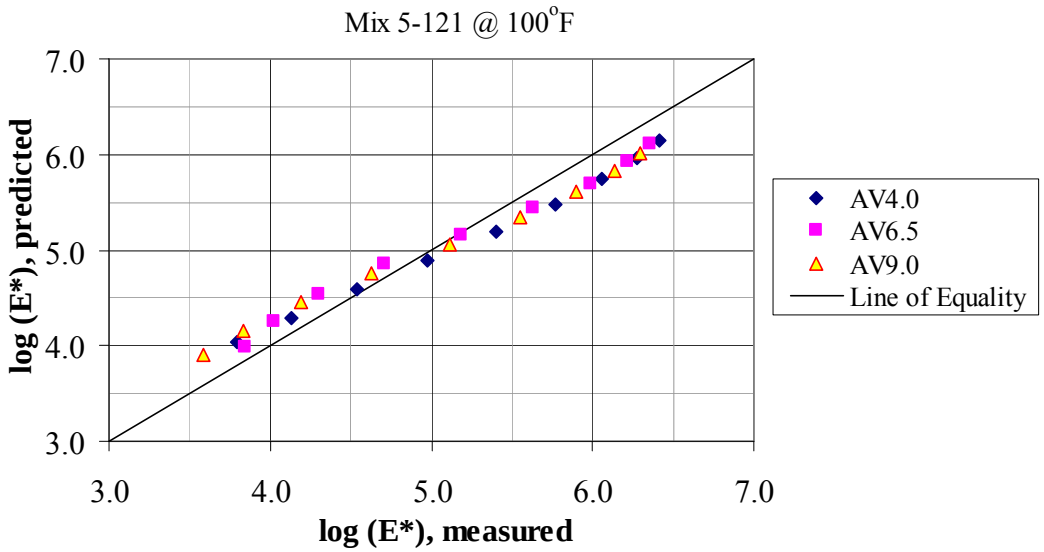


(b)

Figure I-79. Mix 5-94 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

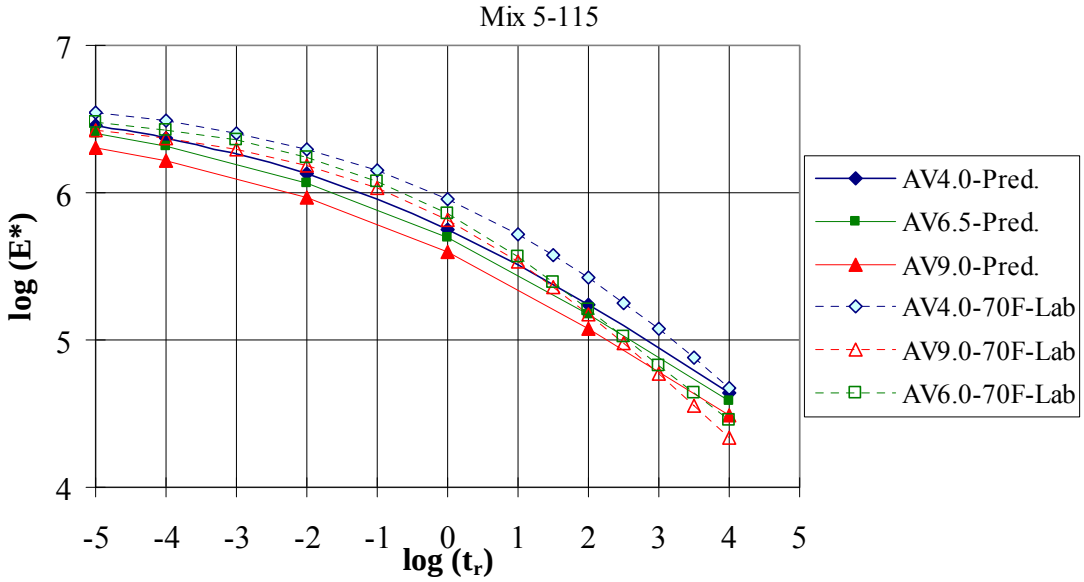


(a)

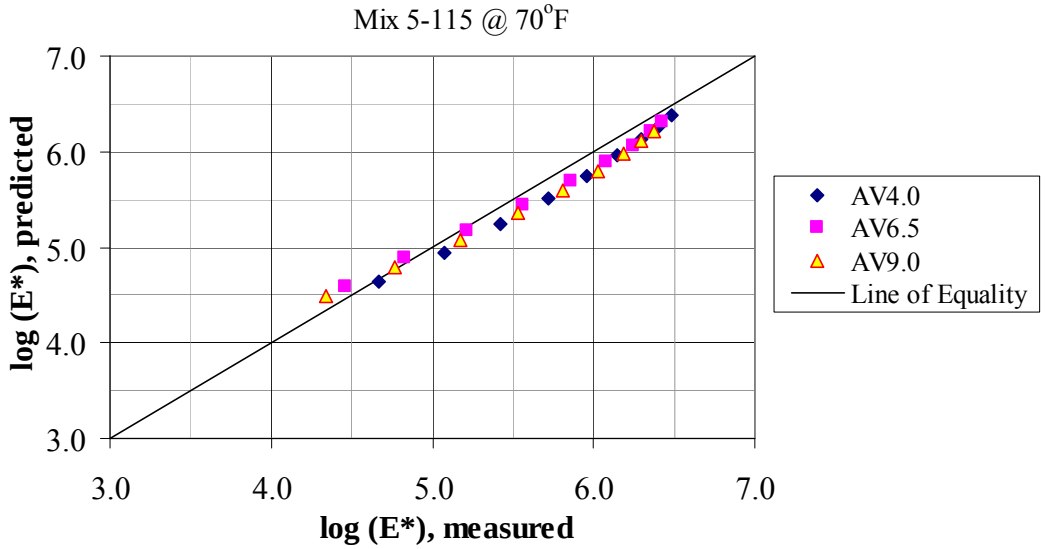


(b)

Figure I-80. Mix 5-121 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

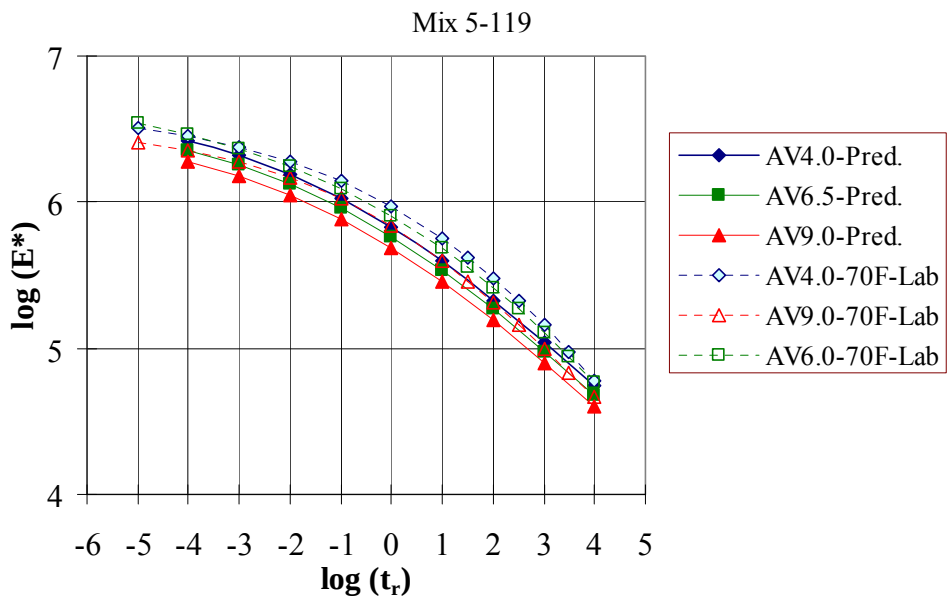


(a)

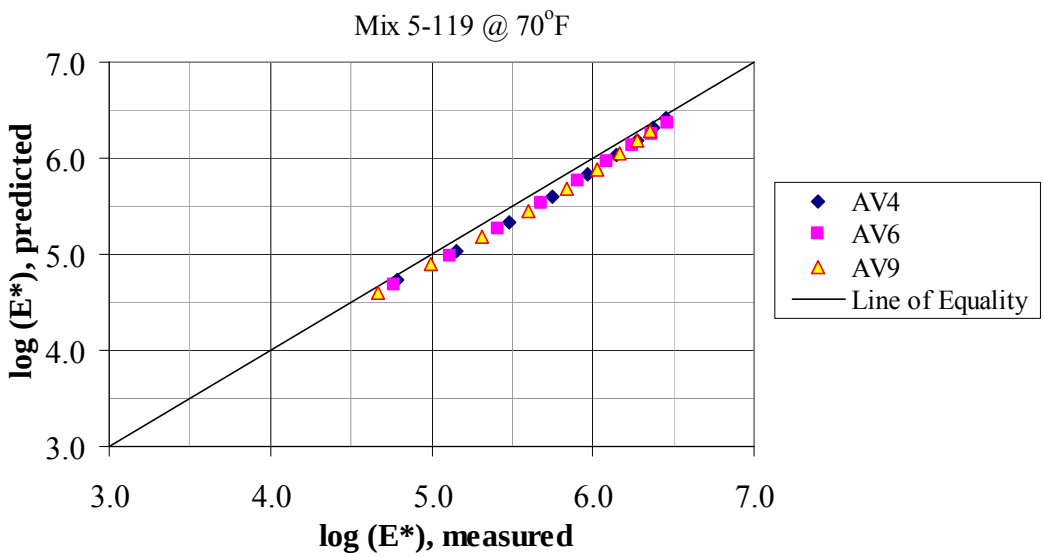


(b)

Figure I-81. Mix 5-115 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

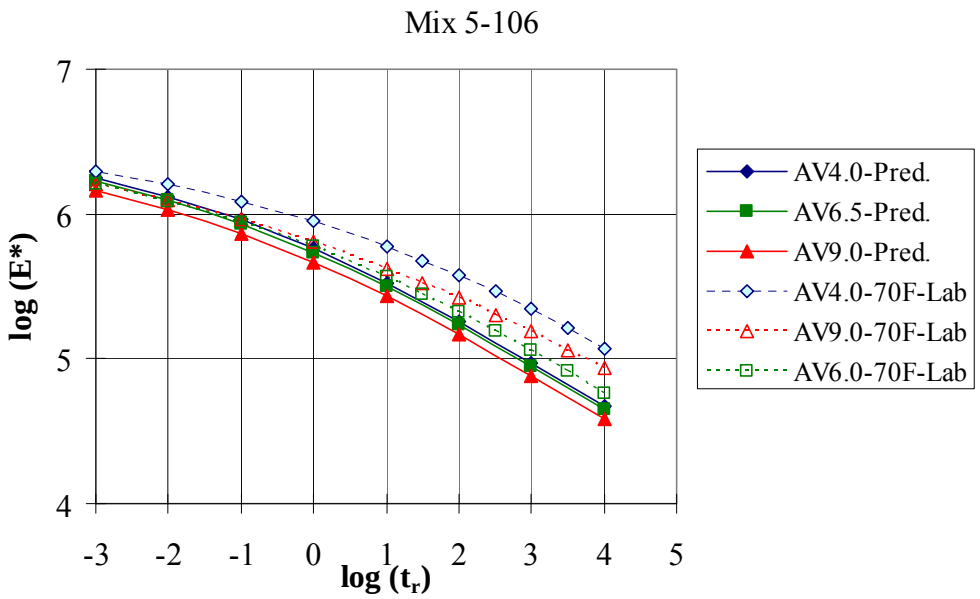


(a)

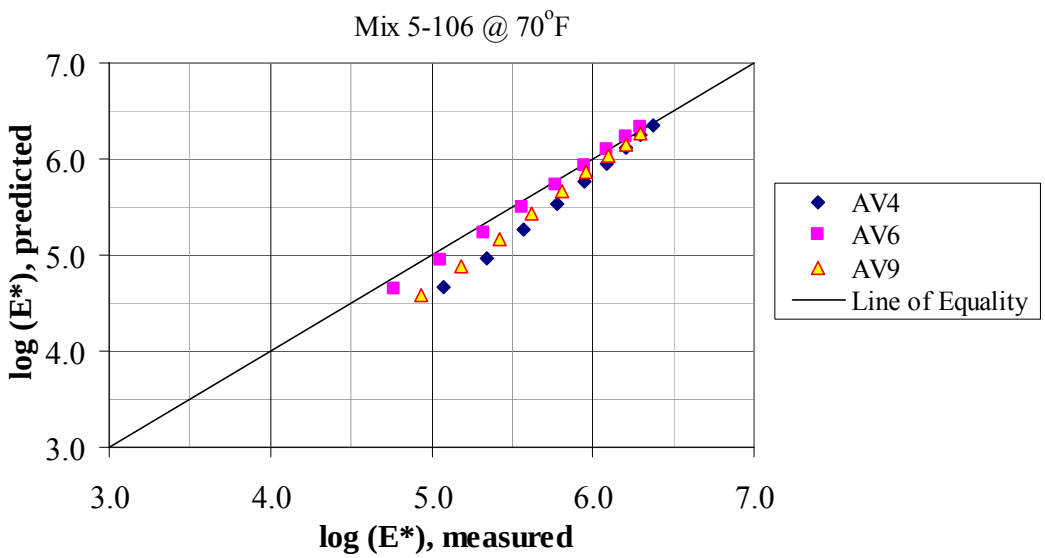


(b)

Figure I-82. Mix 5-119 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).



(a)



(b)

Figure I-83. Mix 5-106 @ 70°F reference temperature (a) Master curves from predicted and measured data at 3 air void levels (b) Comparison of the measured with the predicted log(E*).

The following conclusions can be drawn from Figure I-77 through Figure I-83:

- In general, the dynamic modulus prediction equation provides a reasonable prediction for the mixes evaluated. The equation has the tendency to underpredict the E* values, especially at high frequencies. In practice, this is not a major shortcoming.
- The effect of air voids on E* appears to be low when the air voids are within the range of 3.5% to 10%. For the same mix type, the master curves from three air void levels are close to each other.

Further statistical analysis, Analysis of Variance (ANOVA), was performed to investigate the effect of air voids on the E*. One-way ANOVA is conducted at each frequency level for all the seven mixtures measured E*, and the data is grouped using air void level as a factor. The null hypothesis for this study is $H_0 : \mu_1 = \mu_2 = \mu_3$ where μ_i represents the mean log(E*) at each air void level. A significance level α of 0.05 was used.

Table I-19 summarizes the results of the ANOVA. As shown, at high frequency levels (1000-10000 Hz), the null hypothesis has to be rejected, indicating that the variation of the log(E*) is due to the variation between the air void levels and not due to the random error. Hence, there is a difference in E*at the different air void levels at a significance level of 0.05. However, at all other frequencies (0.0001Hz to 100 Hz), the null hypothesis cannot be rejected, showing that there are no significant differences of the log(E*) at different air void levels. Therefore, it can be concluded that the air void effect on the dynamic modulus depends on the frequency (temperature) levels. At very high frequency levels (or low temperature levels), the air voids do have effect on E* whereas at most low to normal frequencies (high to medium temperatures), the air voids effect on E* values are not significant.

Table I-19. Summary of the ANOVA.

Frequency Level, Hz	P-value		Decision on Null Hypothesis
0.0001	0.588	>0.05	Accept
0.001	0.535	>0.05	Accept
0.01	0.5	>0.05	Accept
0.1	0.48	>0.05	Accept
1	0.445	>0.05	Accept
10	0.347	>0.05	Accept
100	0.177	>0.05	Accept
1000	0.047	<0.05	Reject
10000	0.014	<0.05	Reject

A complete general linear modeling (GLM) procedure was conducted using SAS statistical software. This procedure investigates whether the dynamic modulus from different air void levels for each mixture type at certain temperature and frequency can be considered as the same group. A same group number is assigned when the groups can be statistically considered as the same group. In total there are 168 different combinations in terms of 7 mixture types, 4 temperatures, and 6 frequencies. They are categorized into 5 grouping types, and the number of cases that fall into each grouping type is provided in Table I-20.

Table I-20. Number of cases for each grouping type.

Grouping type (AV4, AV6.5, AV9)	**Number of cases**	**Percentage (%)**
A, A, A	105	64.8
A, A (B), B	16	17.9
A, B, B	29	9.9
A, B, C	10	6.2
A, A, B	2	1.2
Total	162	100

For most cases (105 out of 162), the E* value from 3 different air void levels are considered as the same group with no significant statistical difference. Only 10 out of 162 cases treat the 3 air void levels completely different. Further, 27.8% cases consider the air void 6.5 group the same as the air void 9 group, and the air void 4 group and the air void 6.5 group are treated the same for 19.1% cases.

Combining the findings from figures and tables, it is suggested that the air void level is not a very significant factors in E* values. Its influence is low and will not always be a factor to differentiate master curves.

The master curves for all the mixtures at each air void level are plotted in figures Figure I-84 through Figure I-86 at a reference temperature of 70 ^{o}F. These curves are developed based on lab testing results. As can be seen, all the mixtures tend to have similar dynamic modulus at high frequencies (low temperatures). When the frequency goes down, different mixes show different modulus behavior. This may be related to the different binder effects. In general, at low frequencies (high temperatures), the polymer modified binder mix gives higher E* values. This is the case for all three air void levels.

The same procedure, complete GLM analysis using SAS statistical program is performed to evaluate the effect of binder type on the dynamic modulus. Table I-21 shows the results of the grouping based on binder types. For each temperature, the three binder types tend to be similar groups at higher frequency levels. When the frequency decreases, some differences can be noticed within the different binder types. At low frequencies, mixtures from binder PG64-22 should always be treated statistically different from the mixtures from binder PG76-22, which indicating different binder mixes (polymer or neat binder) behave differently at low frequency levels. This finding agrees with the results shown in figures Figure I-84 through Figure I-86.

Based on the data presented, Level 1 E* inputs for MoDOT typical mixes were developed and included into the project materials database for use in routine design.

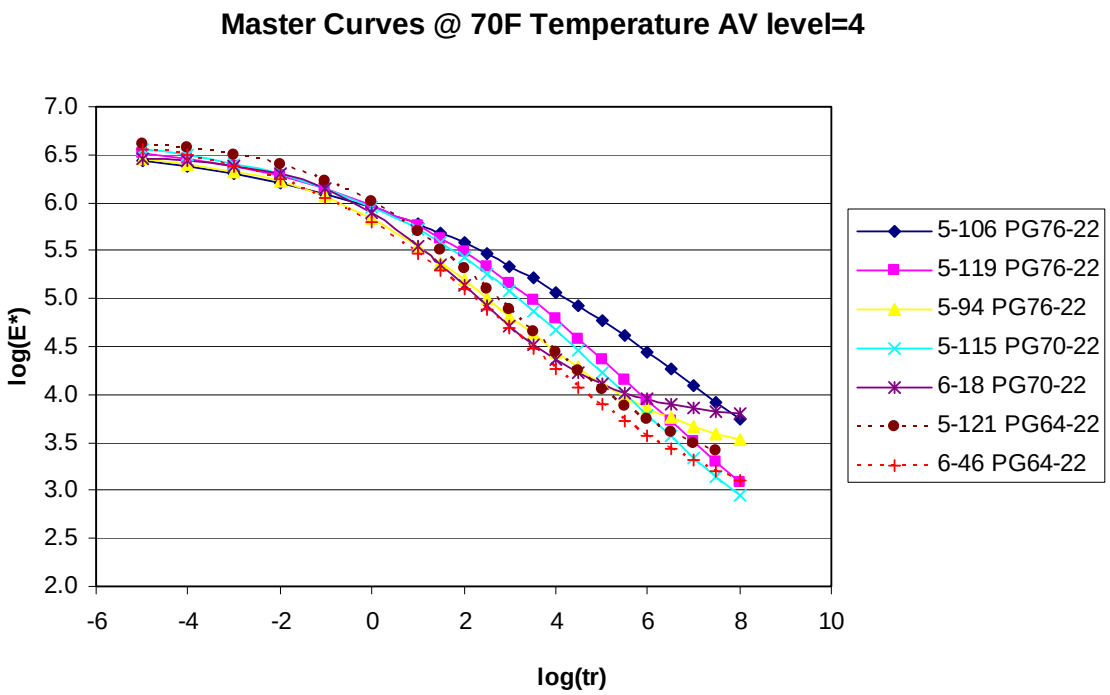


Figure I-84. Master curves for all mixes at air void level of 4% @ 70 °F reference temperature.

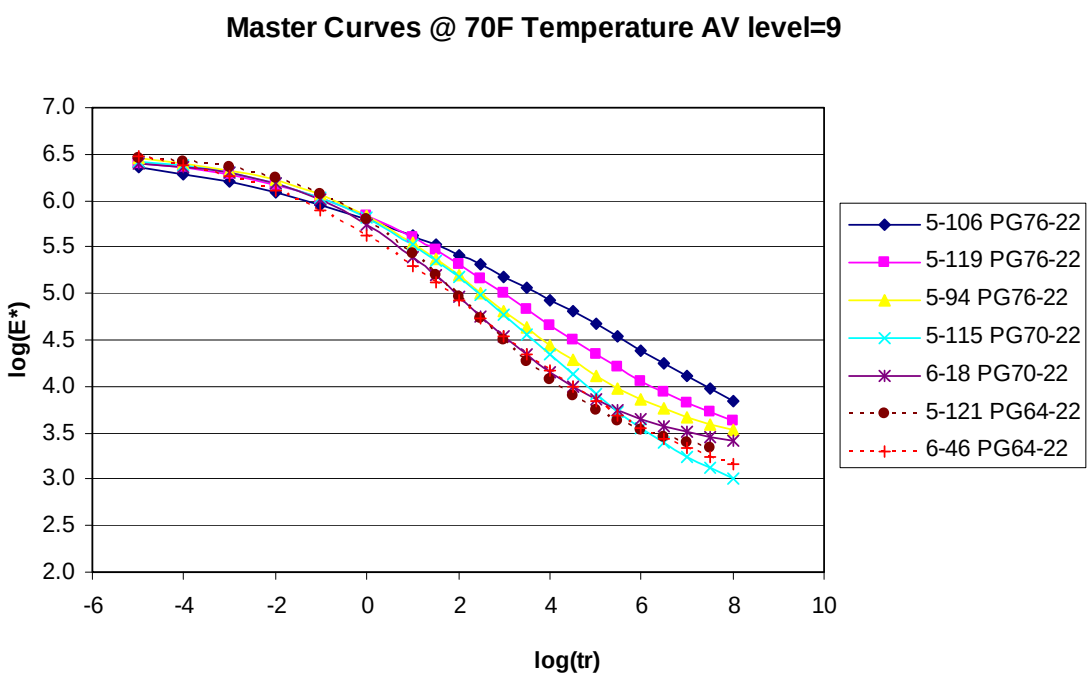


Figure I-85. Master curves for all mixes at air void level of 9% @ 70 °F reference temperature.

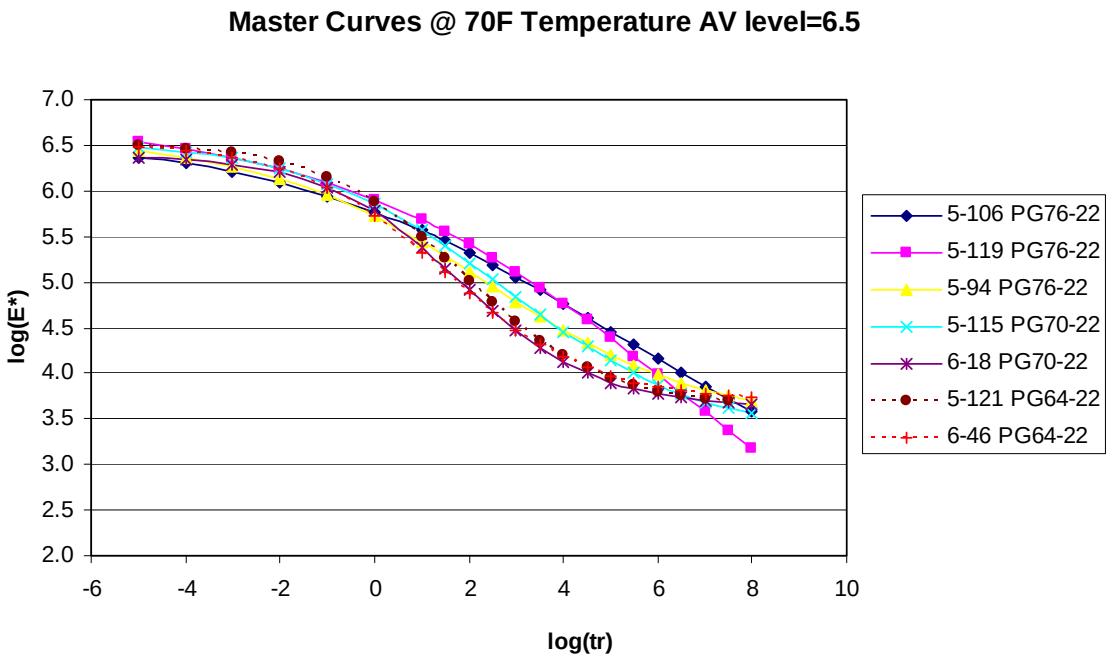


Figure I-86. Master curves for all mixes at air void level of 6.5% @ 70 °F reference temperature.

Table I-21. E* grouping based on binder type.

		Grouping		
Frequency (Hz)	Temperature (F)	PG64-22	PG70-22	PG76-22
25, 10, 5, 1	40	A	B	B
0.1	40	A	A, B	B
25, 10, 5, 1	70	A	A	A
0.5, 0.1	70	B	A, B	A
25, 10, 5, 1	100	A	A	A
0.5, 0.1	100	B	B	A
25	130	A	A	A
10, 5	130	B	A, B	A
1, 0.5, 0.1	130	B	B	A

3.3.3 Repeated Load Permanent Deformation Testing

The repeated load permanent deformation testing was performed to achieve two objectives: (1) use the data obtained to adjust the rutting model coefficients in the MEPDG to properly account for mix characteristics, and (2) develop correlations between the slope of the load versus permanent strain data for each specimen with the Asphalt Pavement Analyzer data to help MoDOT develop a better understanding of poor mixes. Originally, a comprehensive test matrix was developed for this testing (see Table I-22). However, since the SPT protocol for permanent deformation testing was not finalized in terms of confining pressures to be used, sample preparation for triaxial conditions, etc., the testing performed under this scope of work was limited.

Table I-22. HMA mixture parameters included in the experimental testing plan for measuring the permanent deformation coefficients or flow number (number of test specimens within each cell in accordance with SPT).

HMA Mix Type	Gradation & Aggregate Type	Asphalt Modifier	Air Voids, %		
			4	6.5	9
Marshall Designed Mixtures without Polymer Modified Asphalts	Limestone Coarse-Graded	None	NA	2	NA
	Limestone Fine-Graded	None	2	2	2
	Igneous Rock Coarse-Graded	None	NA	2	NA
	Igneous Rock Fine-Graded	None	2	2	2
Superpave Designed Mixtures without Polymer Modified Asphalts	Limestone Coarse-Graded	None	2	2	2
	Limestone Fine-Graded	None	NA	2	NA
	Igneous Rock Coarse-Graded	None	2	2	2
	Igneous Rock Fine-Graded	None	NA	2	NA
Polymer Modified & SMA Mixtures	Limestone Coarse-Graded	SBS	2	NA	2
	Limestone Coarse-Graded	SBR	NA	2	NA
	Limestone Fine-Graded	SBS	NA	2	NA
	Limestone Fine-Graded	SBR	2	NA	2

The laboratory testing that was performed under this effort is summarized in Table I-23. The nine mixes sampled and tested for E* were also tested for repeated load permanent deformation using the SPT and the protocols developed under NCHRP Project 9-29. Both confined and unconfined testing was performed. Repeated load permanent deformation testing was performed on the same specimens used for the E* testing. Therefore, data were available from three specimens for each mix at each air void level and temperature tested. Unconfined testing was done at a 10 psi nominal vertical compressive stress for all specimens tested at 130 ^{o}F, and triaxial testing was performed at 70 psi nominal deviator stress and 10 psi confining stress for all specimens tested at 0 ^{o}F. Due to the limited amount of testing performed, the data developed have limited use at this time.

Table I-23. Details of testing performed to characterize repeated load permanent deformation.

Mix Designation	Mix Type	Mix No.	AC Binder Type	Asphalt Content (by weight)	Air Voids Level Tested	Test Temperature, °F	Confining Pressure/ Deviator Stress, psi
SP125	SMA[1]	05-94	PG 76-22	6.5	4, 6.5, 9.0	130	0/10
SP125	PMA[2]	05-119	PG 76-22	5.6	4, 6.5, 9.0	70	10/70
SP190	PMA	05-106	PG 76-22	5.1	4, 6.5, 9.0	130	0/10
SP190	Neat[3]	06-46	PG 64-22	5.0	4, 6.5, 9.0	130	0/10
SP190	PMA	05-115	PG 70-22	4.7	4, 6.5, 9.0	70	
						130	0/10
SP 190		4-95			4, 6.5, 9.0, 13.0	130	0/10
SP250	Neat	05-121	PG 64-22	5.1	4, 6.5, 9.0	70	
						130	0/10
SP250	PMA	06-18	PG 70-22	4.0	4, 6.5, 9.0	130	0/10
BP-1	Marshall	05-161	PG 64-22	5.7	4, 6.5, 9.0	130	0/10

Nevertheless, a preliminary analysis of the data showed that PMA mixes for a given nominal maximum aggregate size (NMAS) and approximately equal binder contents have a flatter N-ε_p curve when compared to corresponding neat mixes. Figure I-87 shows a plot of the cumulative axial permanent strain at different number of loading cycles for HMA mixtures with 19.0mm NMAS tested at 130°F. This finding is significant and will need to be further validated with more comprehensive testing and analysis. If true, the testing indicates a different rut accumulation in PMA mixes compared to neat mixes. This behavior is along expected lines. Some of this research is being performed under NCHRP Project 9-30A. It is recommended that MoDOT follow this research project to further calibrate the HMA rutting model, particularly after more performance data on Missouri-specific sections being used in calibration becomes available.

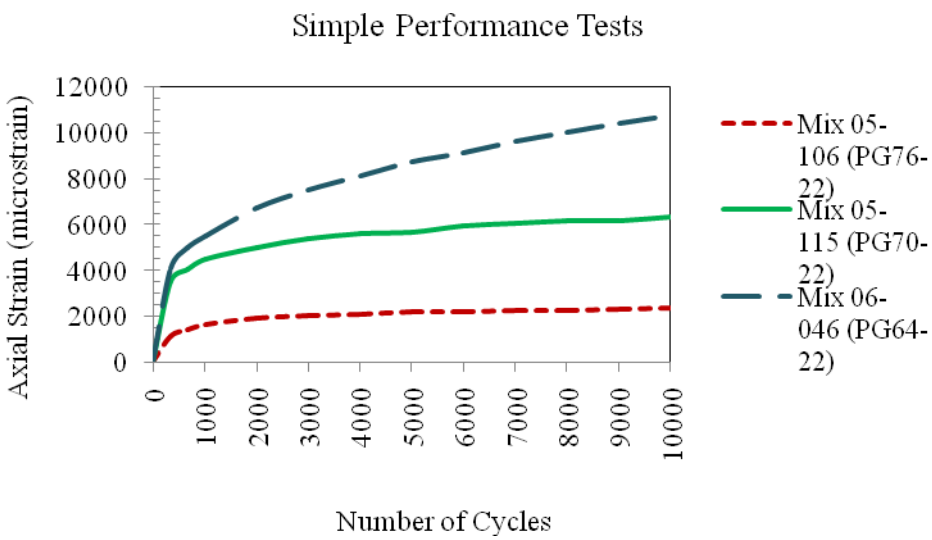


Figure I-87. Permanent deformation test results for MoDOT SP190 mixtures tested at 130°F.

3.3.4 Creep Compliance and Indirect Tensile Strength Testing

Creep compliance and indirect tensile (IDT) strength of HMA are the two primary inputs to predict the low-temperature or thermal cracking in the MEPDG. Missouri experiences thermal cracking problems over some part of the State. Therefore, MoDOT decided to develop a library of creep compliance and IDT strength values for typical Missouri mixes. The MEPDG thermal cracking model is very sensitive these inputs. The mixture testing for these inputs was performed at the Missouri University of Science and Technology (MUST) by Richardson and Lusher (2008). The following paragraphs provide an overview of the testing performed and key findings and uses of the data for Missouri conditions.

Trends such as increasing creep compliance and decreasing tensile strength with increasing percentage of air voids and/or temperature were confirmed. The presence of RAP in a mix tended to decrease the compliance (or in other words, increase the stiffness) and increase the tensile strength compared to similar mixes without RAP.

Mixes Sampled and Tested

Six plant-produced, wearing or surface course mixes were sampled and provided to the researchers by MoDOT. Four mixes were tested at three levels of percent air voids: 4, 6.5, and 9%, and two mixes were tested only at 6.5% air voids. Per the requirements of the MEPDG, creep testing was performed at 32°F (0°C), 14°F (-10°C), and -4°F (-20°C), and IDT strength testing was performed at 14°F (-10°C). Additional IDT strength testing was performed at 40°F (4.4°C) and 70°F (21°C) per MoDOT's requirements. In addition, Poisson's ratio was determined from the creep testing while tensile failure strain was determined from the IDT strength testing. Table I-24 presents the details of the testing matrix used. Table I-25 presents the mix volumetric properties for the mixes tested. As can be noted, various combinations of RAP and virgin mixes with modified and neat binders were tested.

Table I-24. HMA creep compliance and IDT strength test matrix (Richardson and Lusher, 2008).

HMA Mix Type	MoDOT ID [Description] % RAP** (Aggregate Type)	Virgin PG Binder Grade	No. Replicate Tests		
			4% Voids	6.5% Voids	9% Voids
Superpave	06-101 [SP125B] (Dolomite)	76-22 (modified)	3*	3*	3*
Superpave	06-150 [SP125C] 1 0% RAP (Limestone)	70-22 (modified)	3*	3*	3*
Superpave	06-125 [SP125C] (Limestone)	64-22	3*	3*	3*
Superpave	06-105 [SP125C] 10% RAP (Dolomite) 3	70-22 (modified)	3		
Superpave	(Stone Matrix) 06-84 [SP125BSM] (Porphry)	76-22 (modified)	3*	3*	3*
Marshall	07-123 [BP-1] 20% RAP (Dolomite)	64-22	3		

*Additional IDT strength testing at 4.4 and 21°C (40 and 70°F, respectively)
** Recycled Asphalt Pavement

Table I-25. HMA volumetric properties for the creep compliance and IDT mixes tested.

Mix ID	% Virgin Binder	%Binder in RAP	Total % Binder	% Fibers	G_{mm}
06-101	5.7	NA	5.7	0	2.515
06-150	5.0	4.8	5.5	0	2.467
06-125	6.5	NA	6.5	0	2.412
06-105	5.1	4.8	5.6	0	2.455
06-84	6.3	NA	6.3	0.3	2.436
07-123	4.2	5.7	5.3	0	2.501

In performing the testing, Richardson and Lusher (2008) used the MEPDG recommended AASHTO T322 as a reference. However, the researchers faced several issues when applying the protocol, which were resolved using expert advice from researchers around the country. Although these issues need to be addressed in a revision to AASHTO T322, they will perhaps have very little influence on the MEPDG implementation effort in Missouri. One significant item that needs to be mentioned is related to the load versus time profile for creep testing. The protocol requires this to be a step function with a rapid ramp up of the loading at the beginning followed by a hold period and subsequent unloading. The load application and removal is intended to be instantaneous (less than 0.1s). However, due to equipment limitations, the initial load "ramp" time (i.e., the elapsed time between initial load application and load stabilization to a +/- 2% creep) in the testing that was performed averaged 3 seconds in this study.

Creep Compliance Data

Table I-26 through Table I-31 provide the creep compliance test data for the various mixes. The values at air voids level of 6.5% may be used as MEPDG default inputs for creep compliance (this is close to the average air void level on MoDOT's HMA pavements and overlays).

Table I-32 presents a summary of the IDT strength values at 14 ^{o}F—a required input at all levels of the MEPDG. The IDT strength at an air void level of 6.5 percent can be used as a typical agency value for a given mix. However, if IDT strength is desired at other air void levels, figure Figure I-88, which presents a graphical representation of the IDT strength versus air voids for all the mixes tested, can be used instead.

Table I-26. Creep compliance test values for MoDOT ID: 06-125 SP125C mix with limestone aggregate and PG 64-22 neat binder (Richardson and Lusher, 2008).

Temp (deg C)	Time	06-125 (Voids = 4%)		06-125 (Voids = 6.5%)		06-125 (Voids = 9%)	
	(sec)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.5035E-07	0.03631	3.0510E-07	0.04425	3.3867E-07	0.04912
	2	2.5648E-07	0.03720	3.0997E-07	0.04496	3.4573E-07	0.05014
	5	2.6933E-07	0.03906	3.2352E-07	0.04692	3.5754E-07	0.05186
	10	2.8235E-07	0.04095	3.4009E-07	0.04933	3.7427E-07	0.05428
	20	2.9128E-07	0.04225	3.6010E-07	0.05223	3.9264E-07	0.05695
	50	3.1535E-07	0.04574	3.8300E-07	0.05555	4.1835E-07	0.06068
	100	3.2748E-07	0.04750	4.1431E-07	0.06009	4.4649E-07	0.06476
-10	1	3.3791E-07	0.04901	3.6567E-07	0.05304	4.1683E-07	0.06046
	2	3.4928E-07	0.05066	3.8180E-07	0.05538	4.2892E-07	0.06221
	5	3.7034E-07	0.05371	4.0938E-07	0.05938	4.5714E-07	0.06630
	10	3.9875E-07	0.05783	4.4683E-07	0.06481	4.9356E-07	0.07159
	20	4.2747E-07	0.06200	4.8141E-07	0.06982	5.3069E-07	0.07697
	50	4.7736E-07	0.06924	5.4865E-07	0.07957	5.9145E-07	0.08578
	100	5.2629E-07	0.07633	6.0627E-07	0.08793	6.4465E-07	0.09350
0	1	5.3193E-07	0.07715	5.6385E-07	0.08178	6.7142E-07	0.09738
	2	5.6947E-07	0.08260	6.0557E-07	0.08783	7.1841E-07	0.10420
	5	6.3890E-07	0.09266	6.9872E-07	0.10134	8.1813E-07	0.11866
	10	7.1948E-07	0.10435	8.0840E-07	0.11725	9.3953E-07	0.13627
	20	8.2759E-07	0.12003	9.5273E-07	0.13818	1.0931E-06	0.15854
	50	1.0377E-06	0.15051	1.2298E-06	0.17837	1.3791E-06	0.20002
	100	1.2568E-06	0.18228	1.5379E-06	0.22305	1.6955E-06	0.24591

Table I-27. Creep compliance test values for MoDOT ID: 06-101 SP125B mix with dolomite aggregate PG 76-22 modified binder (Richardson and Lusher, 2008).

Temp (deg C)	Time	06-101 (Voids = 4%)		06-101 (Voids = 6.5%)		06-101 (Voids = 9%)	
	(sec)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.1272E-07	0.03085	2.4003E-07	0.03481	2.8444E-07	0.04125
	2	2.1606E-07	0.03134	2.4822E-07	0.03600	2.8698E-07	0.04162
	5	2.2259E-07	0.03228	2.5550E-07	0.03706	2.9960E-07	0.04345
	10	2.3511E-07	0.03410	2.6741E-07	0.03878	3.1585E-07	0.04581
	20	2.4617E-07	0.03570	2.7939E-07	0.04052	3.3516E-07	0.04861
	50	2.6328E-07	0.03819	2.9706E-07	0.04308	3.5140E-07	0.05097
	100	2.7380E-07	0.03971	3.1193E-07	0.04524	3.7558E-07	0.05447
-10	1	2.6071E-07	0.03781	3.0755E-07	0.04461	3.7287E-07	0.05408
	2	2.6953E-07	0.03909	3.2101E-07	0.04656	3.8817E-07	0.05630
	5	2.8765E-07	0.04172	3.4047E-07	0.04938	4.1282E-07	0.05987
	10	3.0762E-07	0.04462	3.6382E-07	0.05277	4.3411E-07	0.06296
	20	3.2653E-07	0.04736	3.9391E-07	0.05713	4.6853E-07	0.06795
	50	3.6785E-07	0.05335	4.3838E-07	0.06358	5.1935E-07	0.07533
	100	4.0278E-07	0.05842	4.7890E-07	0.06946	5.6973E-07	0.08263
0	1	3.8947E-07	0.05649	4.3942E-07	0.06373	4.8861E-07	0.07087
	2	4.1800E-07	0.06063	4.7132E-07	0.06836	5.2329E-07	0.07590
	5	4.7754E-07	0.06926	5.3036E-07	0.07692	5.9067E-07	0.08567
	10	5.4781E-07	0.07945	5.9919E-07	0.08690	6.7225E-07	0.09750
	20	6.3849E-07	0.09261	6.9474E-07	0.10076	7.7699E-07	0.11269
	50	8.0632E-07	0.11695	8.6604E-07	0.12561	9.5867E-07	0.13904
	100	9.8017E-07	0.14216	1.0474E-06	0.15192	1.1556E-06	0.16761

Table I-28. Creep compliance test values for MoDOT ID: 06-84 SP125BSM SMA mix with Porphry aggregate and PG 76-22 modified binder (Richardson and Lusher, 2008).

Temp (deg C)	Time	06-84 (Voids = 4%)		06-84 (Voids = 6.5%)		06-84 (Voids = 9%)	
	(sec)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.5426E-07	0.03688	2.9047E-07	0.04213	3.6340E-07	0.05271
	2	2.6128E-07	0.03790	2.9604E-07	0.04294	3.6774E-07	0.05334
	5	2.7030E-07	0.03920	3.0591E-07	0.04437	3.8061E-07	0.05520
	10	2.8330E-07	0.04109	3.2202E-07	0.04670	3.9955E-07	0.05795
	20	2.9398E-07	0.04264	3.4097E-07	0.04945	4.2072E-07	0.06102
	50	3.1146E-07	0.04517	3.6314E-07	0.05267	4.4901E-07	0.06512
	100	3.2883E-07	0.04769	3.8628E-07	0.05603	4.7240E-07	0.06852
-10	1	3.5706E-07	0.05179	3.5774E-07	0.05189	5.0654E-07	0.07347
	2	3.6484E-07	0.05291	3.7019E-07	0.05369	5.1945E-07	0.07534
	5	3.8548E-07	0.05591	3.9085E-07	0.05669	5.4379E-07	0.07887
	10	4.0867E-07	0.05927	4.1908E-07	0.06078	5.8552E-07	0.08492
	20	4.4271E-07	0.06421	4.6059E-07	0.06680	6.3365E-07	0.09190
	50	4.8753E-07	0.07071	5.0960E-07	0.07391	7.1346E-07	0.10348
	100	5.4001E-07	0.07832	5.6664E-07	0.08218	7.9126E-07	0.11476
0	1	4.9589E-07	0.07192	4.9558E-07	0.07188	7.4524E-07	0.10809
	2	5.2990E-07	0.07686	5.2614E-07	0.07631	8.0206E-07	0.11633
	5	5.9431E-07	0.08620	5.9778E-07	0.08670	9.1754E-07	0.13308
	10	6.7615E-07	0.09807	6.8427E-07	0.09924	1.0566E-06	0.15324
	20	7.7898E-07	0.11298	8.0170E-07	0.11628	1.2460E-06	0.18072
	50	9.6964E-07	0.14063	1.0148E-06	0.14719	1.6149E-06	0.23423
	100	1.1634E-06	0.16874	1.2521E-06	0.18161	2.0361E-06	0.29531

Table I-29. Creep compliance test values for MoDOT ID: 06-150
SP125C SMA mix with dolomite aggregate and PG 70-22 modified binder
(Richardson and Lusher, 2008).

Temp	Time	06-150 (Voids = 4%)		06-150 (Voids = 6.5%)		06-150 (Voids = 9%)	
(deg C)	(sec)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)	D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.3270E-07	0.03375	2.7471E-07	0.03984	3.2558E-07	0.04722
	2	2.3364E-07	0.03389	2.7942E-07	0.04053	3.3127E-07	0.04805
	5	2.4020E-07	0.03484	2.8612E-07	0.04150	3.4147E-07	0.04953
	10	2.5333E-07	0.03674	2.9530E-07	0.04283	3.5699E-07	0.05178
	20	2.6562E-07	0.03853	3.0936E-07	0.04487	3.7511E-07	0.05441
	50	2.7686E-07	0.04016	3.2931E-07	0.04776	4.0184E-07	0.05828
	100	2.9248E-07	0.04242	3.4894E-07	0.05061	4.2234E-07	0.06126
-10	1	2.7076E-07	0.03927	3.4397E-07	0.04989	3.9128E-07	0.05675
	2	2.7845E-07	0.04039	3.5229E-07	0.05109	4.0149E-07	0.05823
	5	2.9297E-07	0.04249	3.7356E-07	0.05418	4.2930E-07	0.06227
	10	3.1444E-07	0.04560	4.0236E-07	0.05836	4.6357E-07	0.06724
	20	3.3663E-07	0.04882	4.2599E-07	0.06179	4.9991E-07	0.07251
	50	3.7557E-07	0.05447	4.7964E-07	0.06957	5.6571E-07	0.08205
	100	4.0644E-07	0.05895	5.2053E-07	0.07550	6.1993E-07	0.08991
0	1	3.6693E-07	0.05322	4.8603E-07	0.07049	6.5130E-07	0.09446
	2	3.8964E-07	0.05651	5.1387E-07	0.07453	6.9116E-07	0.10024
	5	4.2905E-07	0.06223	5.8161E-07	0.08436	7.8421E-07	0.11374
	10	4.7953E-07	0.06955	6.6901E-07	0.09703	8.9981E-07	0.13051
	20	5.4656E-07	0.07927	7.8147E-07	0.11334	1.0633E-06	0.15422
	50	6.6964E-07	0.09712	9.9636E-07	0.14451	1.3820E-06	0.20044
	100	8.0373E-07	0.11657	1.2394E-06	0.17976	1.7543E-06	0.25444

Table I-30. Creep compliance test values for MoDOT ID: 06-105 SP125C mix with limestone aggregate and PG 70-22 modified binder (Richardson and Lusher, 2008).

Temp (deg C)	Time (sec)	06-105 (Voids = 6.5%) D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.7026E-07	0.03920
	2	2.7292E-07	0.03958
	5	2.8299E-07	0.04104
	10	2.9788E-07	0.04320
	20	3.0996E-07	0.04496
	50	3.2931E-07	0.04776
	100	3.4218E-07	0.04963
-10	1	3.2643E-07	0.04734
	2	3.4122E-07	0.04949
	5	3.5722E-07	0.05181
	10	3.7983E-07	0.05509
	20	4.1038E-07	0.05952
	50	4.4907E-07	0.06513
	100	4.8786E-07	0.07076
0	1	4.3592E-07	0.06323
	2	4.5828E-07	0.06647
	5	5.0714E-07	0.07355
	10	5.6857E-07	0.08246
	20	6.4142E-07	0.09303
	50	7.7507E-07	0.11241
	100	9.1212E-07	0.13229

Table I-31. Creep compliance test values for MoDOT ID: 07-123 BP-1 Marshall mix with limestone aggregate and PG 64-22 neat binder (Richardson and Lusher, 2008).

Temp (deg C)	Time (sec)	07-123 (Voids = 6.5%) D(t) (1/psi)	D(t) (1/Gpa)
-20	1	2.4430E-07	0.03543
	2	2.4356E-07	0.03532
	5	2.5001E-07	0.03626
	10	2.5918E-07	0.03759
	20	2.7571E-07	0.03999
	50	2.9128E-07	0.04225
	100	3.0674E-07	0.04449
-10	1	2.9033E-07	0.04211
	2	3.0246E-07	0.04387
	5	3.2053E-07	0.04649
	10	3.3988E-07	0.04930
	20	3.6380E-07	0.05276
	50	4.0019E-07	0.05804
	100	4.2673E-07	0.06189
0	1	3.5911E-07	0.05208
	2	3.9380E-07	0.05712
	5	4.4563E-07	0.06463
	10	4.9442E-07	0.07171
	20	5.5972E-07	0.08118
	50	6.6436E-07	0.09636
	100	7.7751E-07	0.11277

Note: The creep compliance estimates for this mix were based on equivalent area method.

Table I-32. IDT strength values for typical MoDOT mixes at 14°F (Richardson and Lusher, 2008).

Mix ID	Average Air Voids, %	Tensile Strength, psi
07-123 (Marshall; BP-1, 64-22)	6.6	606
06-105 (SP125C; 70-22)	6.5	594
06-84 (SP125BSM; 76-22; SMA)	4.0	717
06-84 (SP125BSM; 76-22; SMA)	6.5	619
06-84 (SP125BSM; 76-22; SMA)	9.0	538
06-101 (SP125B; 76-22)	4.0	807
06-101 (SP125B; 76-22)	6.5	644
06-101 (SP125B; 76-22)	9.0	587
06-125 (SP125C; 64-22)	4.0	641
06-125 (SP125C; 64-22)	6.5	566
06-125 (SP125C; 64-22)	9.0	508
06-150 (SP125C; 70-22)	4.0	783
06-150 (SP125C; 70-22)	6.5	652
06-150 (SP125C; 70-22)	9.0	575

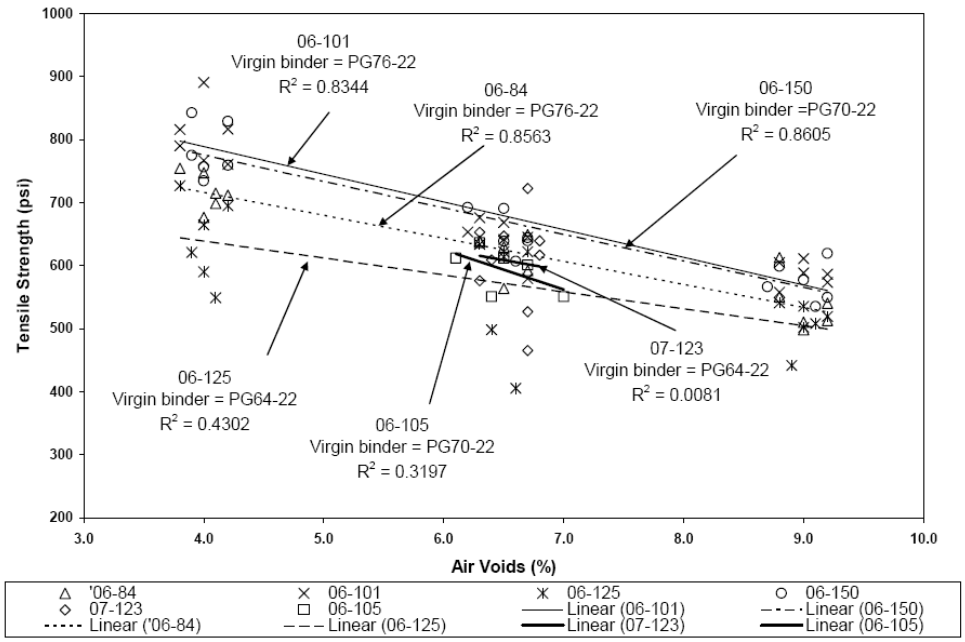


Figure I-88. Indirect tensile strength versus air voids for typical MoDOT mixes (Richardson and Lusher, 2008).

Data Analysis and Conclusions

Based on analysis of the creep compliance and tensile strength data, Richardson and Lusher (2008) concluded the following:

- Expected trends in the data were observed, such as increasing creep compliance and decreasing tensile strength with increasing air voids and temperature for the mixes tested.
- Presence of RAP in a mix tends to decrease the compliance and increase the tensile strength compared to a mix without RAP and same virgin binder grade, if everything else is similar.
- Due to the nature of the mixes (RAP versus virgin) and binders (modified versus neat) chosen for testing, it was not clear if one binder type was better than the other in term of the desirable HMA compliance and strength.
- The Marshall mixture was the least compliant of all mixes tested and produced highly variable tensile strength results.
- The SMA mixture did not lend itself properly to tensile strength testing because of the aggregate structure.

As part of this study, the MEPDG estimates (Level 3) of creep compliance for each mix tested were compared with the laboratory generated data (Level 1). The MEPDG defaults were obtained using the mix volumetric properties (effective binder content by volume, air voids, and binder grade) through the software interface. Figure I-89 through Figure I-94 compare the master curves of creep compliance measured at levels 1 and 3 for the MoDOT mixtures with their corresponding power functions.

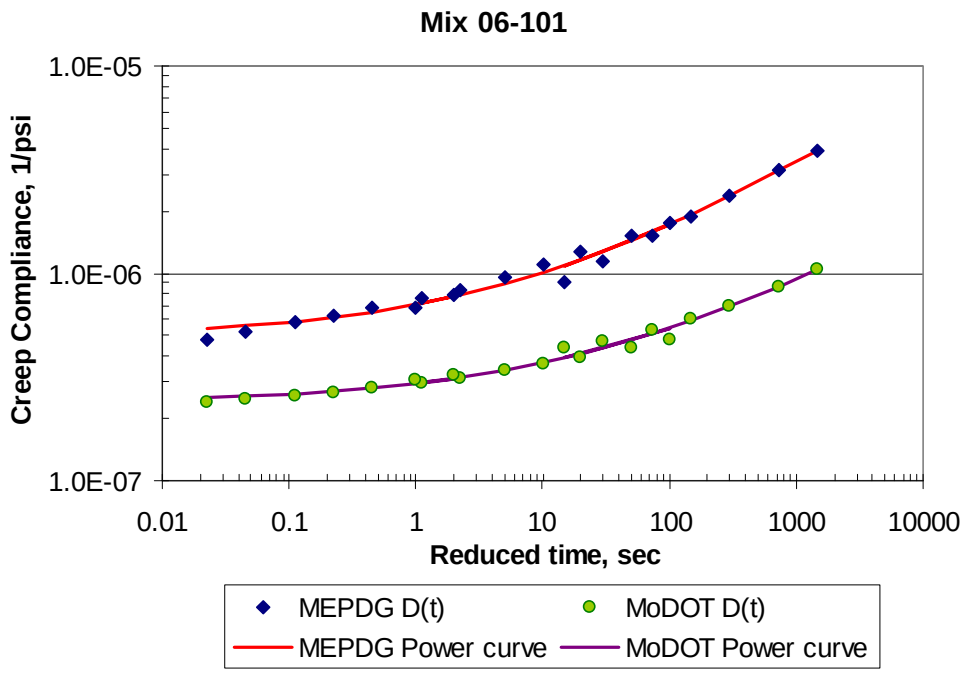


Figure I-89. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 06-101.

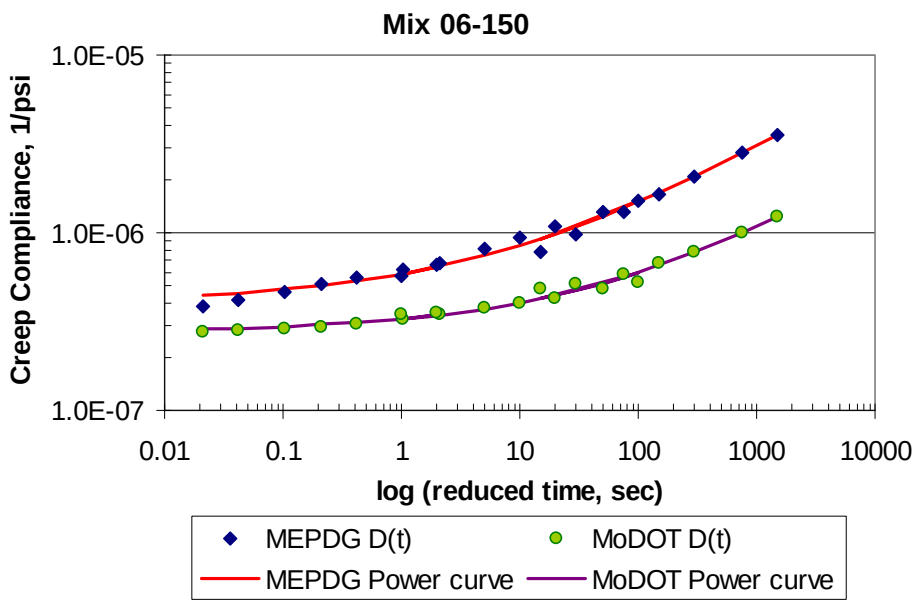


Figure I-90. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 06-150.

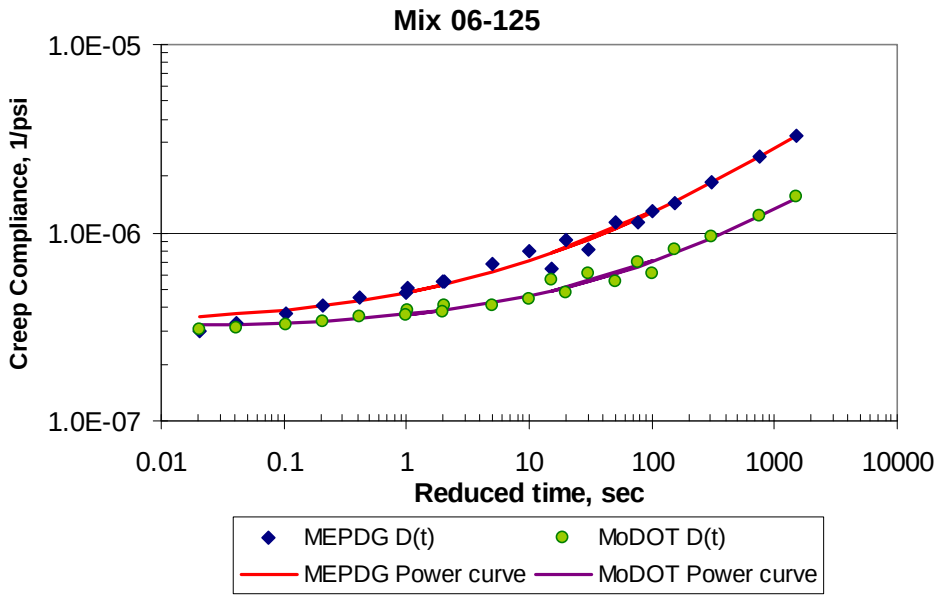


Figure I-91. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 06-125.

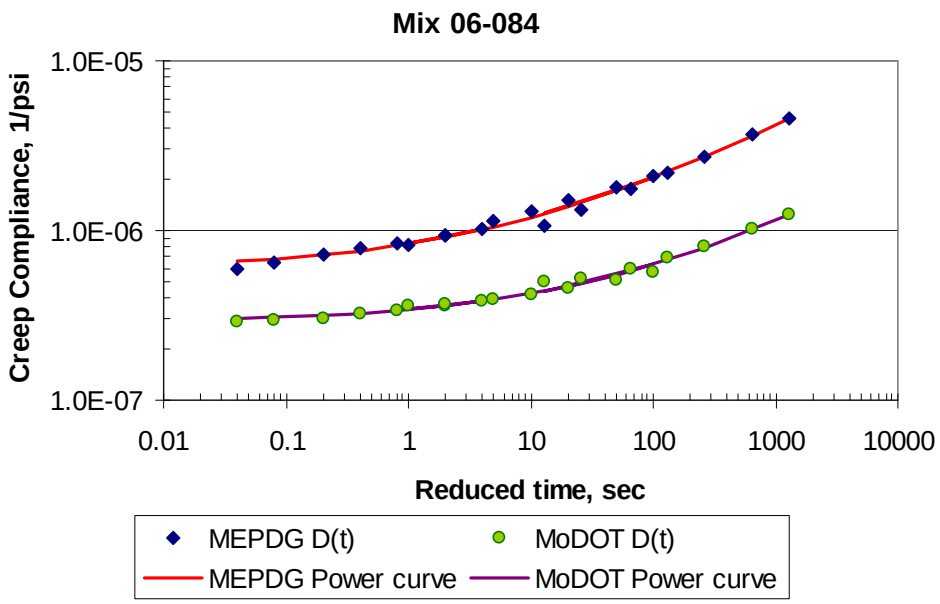


Figure I-92. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 06-84.

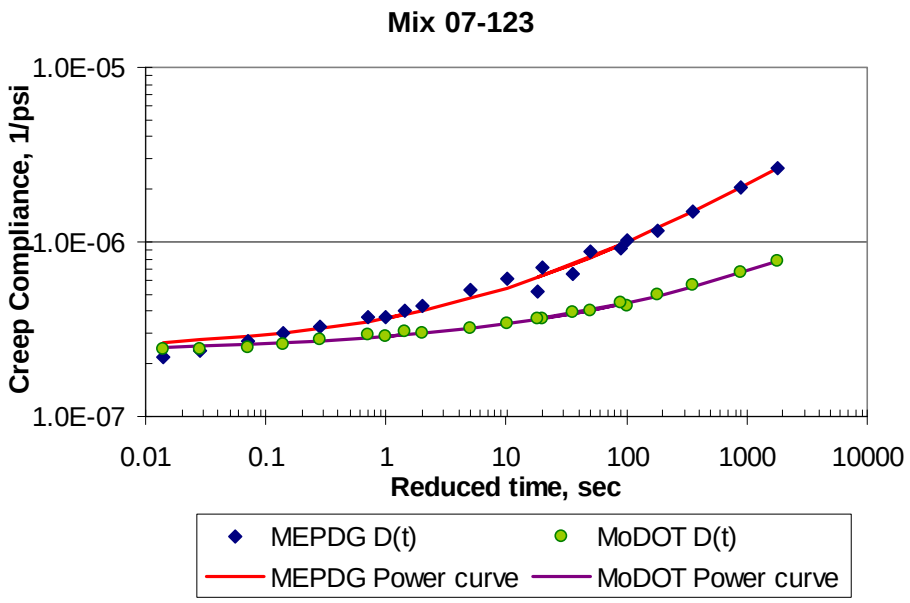


Figure I-93. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 07-123.

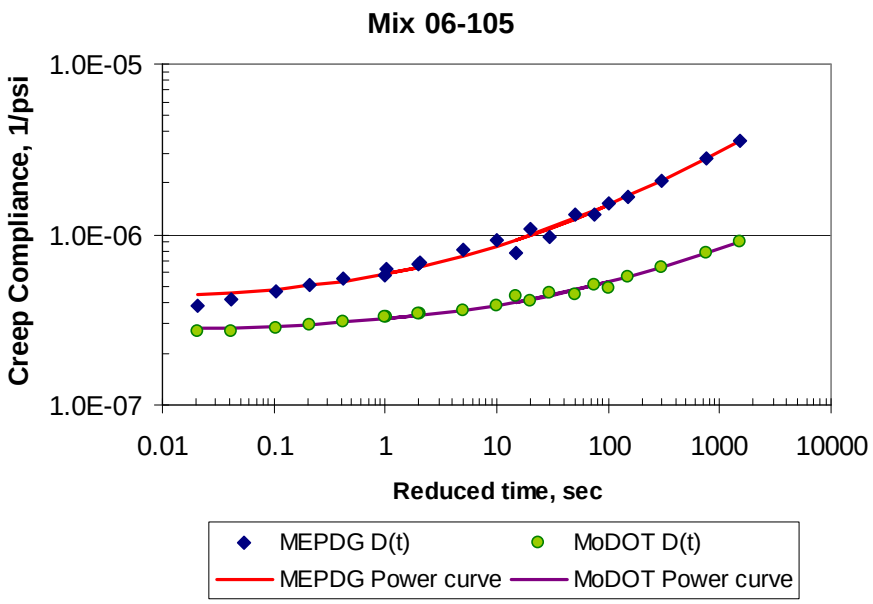


Figure I-94. Comparisons of laboratory tested creep compliance values with MEPDG estimated values for mix 06-105.

It can be seen from the figures that differences between the Level 1 and Level 3 estimates of creep compliance were significant for all mixes at all temperatures tested. This difference is more apparent for modified binders and at higher temperatures. As is evident in the figures Figure I-91 and Figure I-93, the difference in creep compliance between levels 1 and 3 is not significant at lower reduced loading times (i.e. lower temperatures) in mixtures containing PG

64-22 binder. At higher loading times (i.e. higher temperatures), the differences between Level 1 and Level 3 estimates increase at an increasing rate. The narrowing differences in creep compliance estimates at lower temperatures and widening differences at higher temperatures are observed in all mixtures. In general, the Level 3 estimate indicates more compliance in creep than level 1 estimate at higher binder grades and higher test temperatures. Thus, the MEPDG compliance estimates are higher and are more conservative and could lead to an under-prediction of thermal cracking. Of course, to evaluate this properly, the indirect tensile strengths also need to be compared. It was found, in general, that the level 3 estimated IDT strengths were lower than the corresponding level 1 strengths.

To more directly illustrate the true effects of decreased compliance and increased IDT strength, the thermal cracking predictions of the MEPDG, for a given site and structure, obtained using level 1 and level 3 inputs were compared. Level 1 inputs were obtained from the laboratory testing of HMA and are summarized in Table I-26 through Table I-32. Level 3 defaults were estimated using the MEPDG. Kirksville, which is located in the northern part of Missouri, was selected as the site of interest. The flexible pavement structure used in the baseline analysis was selected. The performance parameters of thermal cracking, crack length and crack depth, were predicted by the MEPDG, and are provided in Table I-33. The results indicate that the predicted parameters were significantly different between the two levels. The difference in the predictions of these two input levels apparently increased when the higher performance grade of the binder increased. The MEPDG underestimated thermal cracking when Level 3 inputs were used.

Table I-33. Comparison of Thermal Cracking Predictions between Input Levels 1 and 3.

Mix	Crack length, feet/mile		Crack depth, inches	
	Kirksville Level 1	Kirksville Level 3	Kirksville Level 1	Kirksville Level 3
06-105	1741.3	13.8	8.14	0.098
06-150	1578.2	13.8	6.82	0.098
06-125	1530.9	66.2	6.47	0.267
07-123	1731.0	316.8	8.05	0.943
06-101	1720.7	1.8	7.97	0.038
06-084	1474.7	0.1	6.07	0.008

The MEPDG recognizes this disparity between Level 1 and Level 3 estimates of compliance. The thermal cracking model is the only model in the MEPDG which allows for a change in the model reliability definition with a change in the input level used to characterize the materials. Therefore, based on this study, it is concluded that MoDOT should use laboratory test data, where possible, for HMA creep compliance and indirect tensile strength. Typical values for MoDOT mixes tested herein will be made available in the MoDOT MEPDG materials library.

3.4 PCC MATERIALS CHARACTERIZATION

3.4.1 Overview

The intent of the PCC laboratory test program was to develop:

- A set of default material property values that can be used in the rigid pavement design process for Level 1 inputs.
- Correlations between compressive strength and other strength parameters and modulus for PCC mix designs specific to Missouri for use as Level 2 inputs.
- Mix-specific or agency-specific strength gain relationships for use in the MEPDG procedure.

Based on MoDOT's laboratory testing needs assessment (discussed in chapter 2), the following tests were conducted to determine the required PCC mixture inputs for typical MoDOT mixes representing current materials and construction practices:

- Compressive strength.
- Flexural strength.
- Elastic modulus (chord modulus) and Poisson's ratio.
- Coefficient of thermal expansion.
- Shrinkage.

All the necessary testing equipment, with the exception of CTE equipment, was available at MoDOT's materials laboratory. Lacking a commercial supplier of the CTE test machine in accordance with the AASHTO TP60 protocol, MoDOT built a prototype machine following the blueprints and direction provided by the FHWA. A photograph of the MoDOT CTE test apparatus showing the waterbath, test frame, test specimen, and LVDT is shown in Figure I-95.

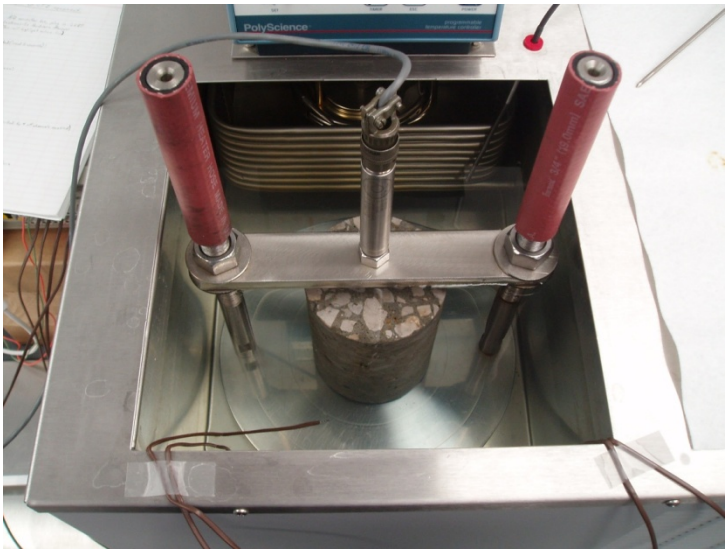


Figure I-95. CTE test apparatus built by MoDOT.

3.4.2 PCC Materials Test Plan

Mix designs to be included in the test program were selected based on the Missouri DOT mix design specifications. Based on a review of the typical mix designs used by MoDOT, the most critical mix design factors were identified as being:

- Addition of flyash content (15-25%—affects rate of strength gain and the strength gain model).
- Coarse aggregate gradation (common-B, typical-D, special cases-F—affects modulus properties).
- Coarse aggregate type (limestone, dolomite, igneous rock—affects modulus and strength properties, CTE, and shrinkage).

In light of the fact that MoDOT uses only Type I cement in its mixes and more or less uniform cement content and gradation for fine aggregate, the test setup did not include any variations to the cement type or cementitious material content or fine aggregate content. The suggested test matrix is shown in Table I-34.

Table I-34. Mix design parameters in the experimental plan.

		Coarse Aggregate Gradation		
		Gradation B or D (optimized)	Gradation D (typical)	Gradation F
Aggregate type	Limestone	FA=15%, w/c=0.42	FA=15%, w/c=0.42 FA=25%, w/c=0.42 FA=15%, w/c=0.35 FA=25%, w/c=0.35	FA=15%, w/c=0.42
	Dolomite	FA=15%, w/c=0.42	FA=15%, w/c=0.42	
	Igneous rock	FA=15%, w/c=0.42		

Using this template as a basis, MoDOT sampled six PCC mixes from the field. These mixes represented different gradation types from different geographic regions of the State. Characteristics of the selected mixes are shown in Table I-35. All the mixes selected for testing were made of limestone coarse aggregate. Table I-36 presents the planned testing program to obtain all the required materials data. This program was modified slightly based on testing expediencies.

Table I-35. Key constituents of the PCC mixes selected for testing.

Location	Gradation	Cement Content	Flyash Content	Total Cementitious Materials Content	Percent Flyash	Total Water	w/cm Ratio
I-44 in Laclede County	Gradation B	479	85	564	15%	215	0.38
US 412 in Dunklin County	Gradation B	445	111	556	20%	229	0.41
I-435 in Jackson County	Gradation B Optimized	510	90	600	15%	258	0.43
MO 367 in St. Louis County	Gradation D	441	110	551	20%	215	0.39
US 63 in Randolph County	Gradation D Optimized	432	108	540	20%	212	0.39
I-35 in Clinton County	Gradation F	517	91	608	15%	231	0.38

Table I-36. Testing plan and number of test specimens for each mix design.

Specimen	# of Specimens	Material properties	Test age	Sample numbers	Total volume
Cylinders 6”x12” (Total 18)	3	CTE (all ages)	All	C – 1, 2, 3	12000 cubic inch
	3	Modulus, Poisson’s ratio and compressive strength	7 days	C-4, 5, 6	
	3	Modulus, Poisson’s ratio and compressive strength	14 days	C-7, 8, 9	
	3	Modulus, Poisson’s ratio and compressive strength	28 days	C-10, 11, 12	
	3	Modulus, Poisson’s ratio and compressive strength	90 days	C-13, 14, 15	
	3	Modulus, Poisson’s ratio and compressive strength	Long-term estimate	No samples required, only estimates	
Beams 6”x6”x20” (Total 12)	3	Flexural strength	7 days	B-1, 2, 3	8640 cubic inch
	3	Flexural strength	14 days	B-4, 5, 6	
	3	Flexural strength	28 days	B-7, 8, 9	
	3	Flexural strength	90 days	B-10, 11, 12	
Total concrete volume to be batched (incl. 10% extra) = 17,000 cubic inch ~ 9.7 cft ~ 0.36 CY					

3.4.3 Summary of PCC Test Data

Compressive strength, flexural strength, modulus, and CTE data from the various mixes are presented in Table I-37 through Table I-40. Figure I-96, Figure I-97 and Figure I-98 present the average compressive strength, flexural strength, and elastic modulus gain for the MoDOT mixes along with the default (Level 3) strength gain model in the MEPDG.

Figure I-99 and Figure I-100 present the compressive strength versus flexural strength and compressive strength versus elastic modulus correlations for the mixes tested.

Table I-37. Summary of compressive strength test results.

MoDOT Mix Designation	Sample Age, days	Average Compressive Strength, psi
Gradation B	3	3343
Gradation B	7	4001
Gradation B	14	4390
Gradation B	28	4902
Gradation B	90	5421
Gradation B Opt	3	3472
Gradation B Opt	7	3936
Gradation B Opt	14	4474
Gradation B Opt	28	4857
Gradation B Opt	90	5606
Gradation D	3	3756
Gradation D	7	4472
Gradation D	14	4848
Gradation D	28	5082
Gradation D	90	5875
Gradation D Opt	3	3884
Gradation D Opt	7	4382
Gradation D Opt	14	4810
Gradation D Opt	28	5120
Gradation D Opt	90	5970
Gradation F	3	3243
Gradation F	7	3847
Gradation F	14	4502
Gradation F	28	4886
Gradation F	90	5643

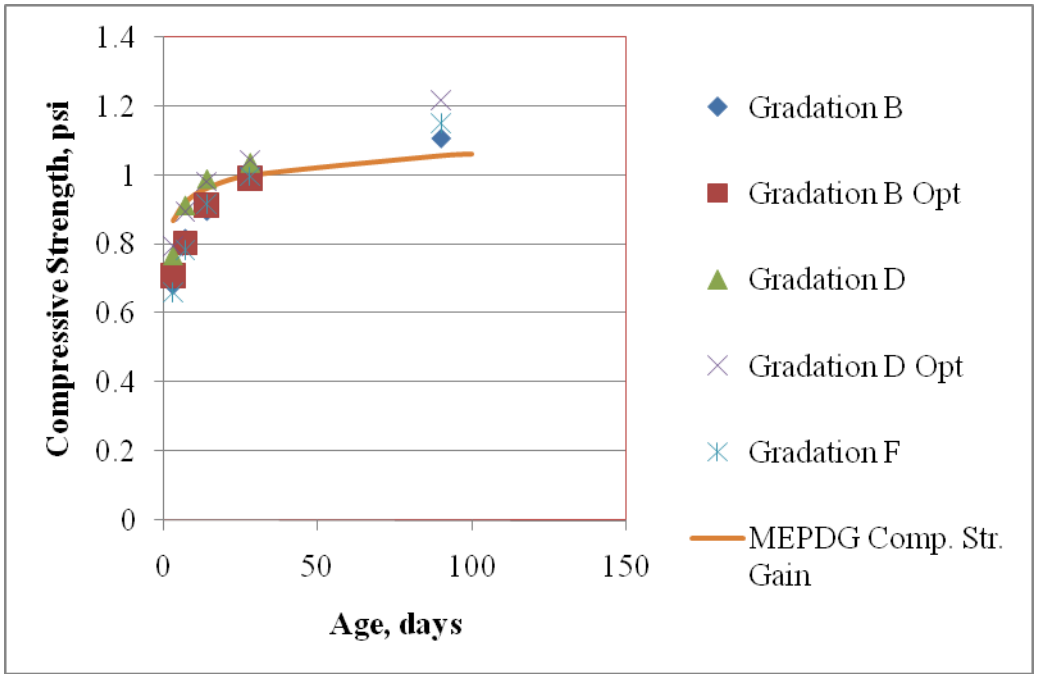


Figure I-96. Compressive strength gain compared to MEPDG Level 3 strength gain.

Table I-38. Summary of flexural strength test results.

MoDOT Mix Designation	Sample Age, days	Average Flexural Strength, psi
Gradation B	3	477
Gradation B	7	550
Gradation B	14	654
Gradation B	28	626
Gradation B	90	674
Gradation B Opt	3	564
Gradation B Opt	7	634
Gradation B Opt	14	652
Gradation B Opt	28	718
Gradation B Opt	90	788
Gradation D	3	587
Gradation D	7	595
Gradation D	14	640
Gradation D	28	655
Gradation D	90	725
Gradation D Opt	3	540
Gradation D Opt	7	583
Gradation D Opt	14	637
Gradation D Opt	28	744
Gradation D Opt	90	699
Gradation F	3	566
Gradation F	7	654
Gradation F	14	739
Gradation F	28	772
Gradation F	90	897

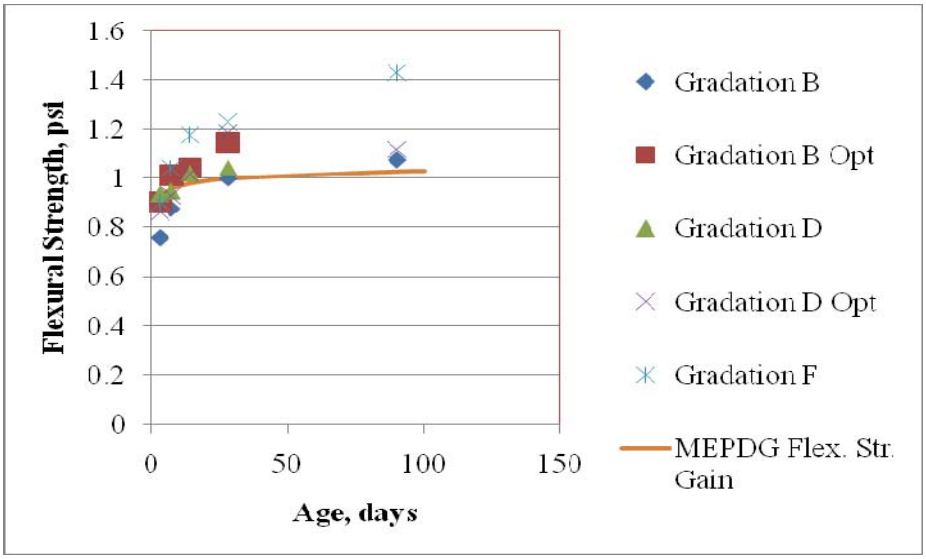


Figure I-97. Flexural strength gain compared to MEPDG Level 3 strength gain.

Table I-39. Summary of Poisson's ratio and elastic modulus test results.

MoDOT Mix Designation	Sample Age, days	Average Poisson's Ratio	Average Modulus of Elasticity, psi
Gradation B	3	0.16	3775772
Gradation B	7	0.19	4172195
Gradation B	14	0.17	4318238
Gradation B	28	0.17	4290195
Gradation B	90	0.21	4757531
Gradation B Opt	3	0.21	3729516
Gradation B Opt	7	0.20	3972549
Gradation B Opt	14	0.20	4161558
Gradation B Opt	28	0.21	4266237
Gradation B Opt	90	0.23	4632843
Gradation D	3	0.22	3835707
Gradation D	7	0.21	4291245
Gradation D	14	0.23	4271614
Gradation D	28	0.18	4452082
Gradation D	90	0.22	4974852
Gradation D Opt	3	0.09	4049615
Gradation D Opt	7	0.24	4239712
Gradation D Opt	14	0.21	4347735
Gradation D Opt	28	0.17	4958388
Gradation D Opt	90	0.22	4785520
Gradation F	3	0.22	3348184
Gradation F	7	0.20	--
Gradation F	14	0.26	4101783
Gradation F	28	0.26	4320960
Gradation F	90	0.24	4635612
		Avg = 0.20	

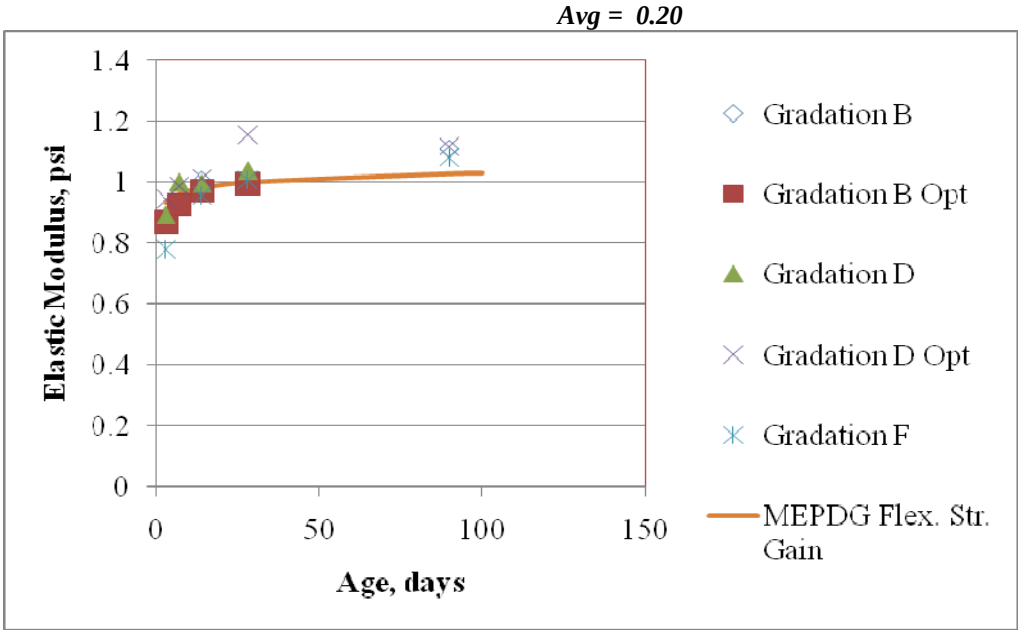


Figure I-98. Elastic modulus gain compared to MEPDG Level 3 modulus gain.

Table I-40. Summary of CTE test results.

MoDOT Mix Designation	Sample Age, days	Average of CTE, in/in/°F
Gradation B	3	--
Gradation B	7	4.85
Gradation B	14	--
Gradation B	28	5.00
Gradation B	90	5.06
Average CTE for Grad. B		**4.97**
Gradation B Opt	3	--
Gradation B Opt	7	4.38
Gradation B Opt	14	--
Gradation B Opt	28	4.94
Gradation B Opt	90	4.37
Average CTE for Grad. B Opt		**4.56**
Gradation D	3	--
Gradation D	7	4.16
Gradation D	14	--
Gradation D	28	4.44
Gradation D	90	4.68
Average CTE for Grad. D		**4.42**
Gradation D Opt	3	--
Gradation D Opt	7	3.82
Gradation D Opt	14	--
Gradation D Opt	28	3.87
Gradation D Opt	90	3.54
Average CTE for Grad. D Opt		**3.74**
Gradation F	3	--
Gradation F	7	3.93
Gradation F	14	--
Gradation F	28	4.07
Gradation F	90	4.71
Average CTE for Grad. F		**4.24**

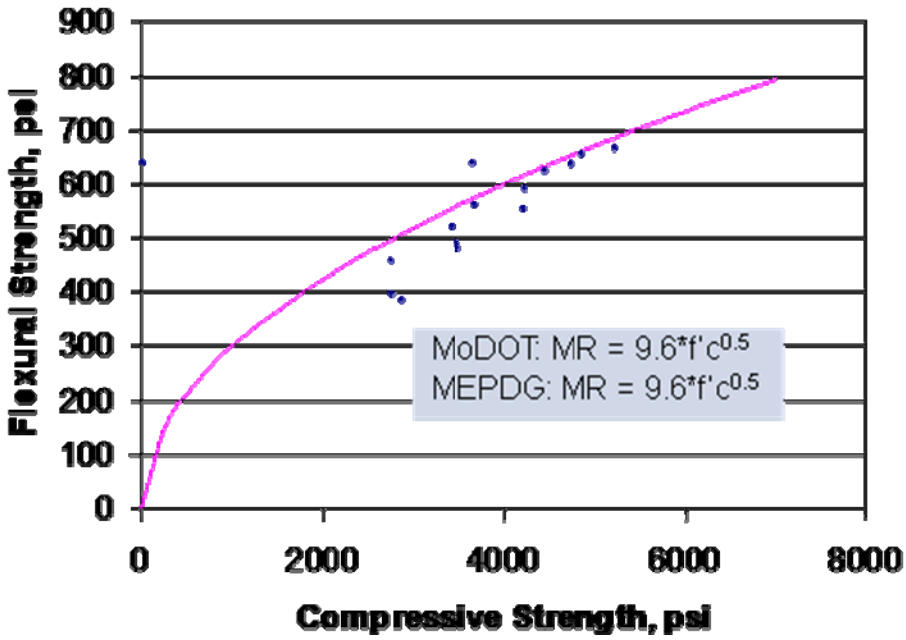


Figure I-99. Missouri-specific compressive versus flexural strength relationship.

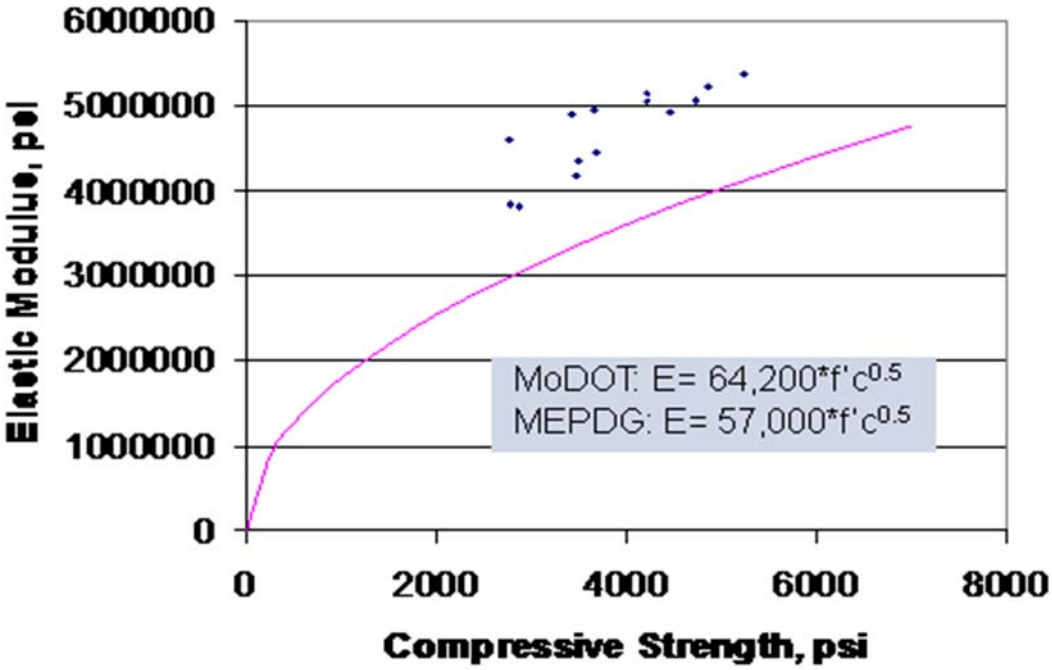


Figure I-100. Missouri-specific compressive strength versus elastic modulus relationship.

The following conclusions can be drawn regarding the data presented:

- The compressive strength, flexural strength, and elastic modulus are not significantly different statistically from the default values. The default compressive strengths in the MEPDG at Level 3 for these inputs are close to the respective average 28-day values. Based on the data presented, it is suggested the following Level 3 defaults be used:
    - Compressive strength: 4500 psi.
    - Flexural strength: 700 psi.
    - Elastic modulus: 4,500,000 psi.
- The strength and modulus gain for all the mixes tested for the range of ages over which they were tested is lower than the MEPDG model when the age of the pavement is less than 28 days. However, for ages greater than 28 days, the strength gain is higher than the MEPDG default. However, due to the lack of longer term data, it is recommended that the default strength and modulus gain model in the MEPDG be adopted along with the long-term strength/modulus ratio.
- The default compressive-to-flexural strength relationship in the MEPDG is very close to the MoDOT specific correlation. It is recommended that the MEPDG default correlation be retained.
- The MEDDG compressive strength-to-elastic modulus correlation underestimates the elastic modulus. It is recommended that this equation be replaced with a MoDOT-specific relationship shown in equation (2).

$$E = 64{,}200*(f'c)^{0.5} \qquad (2)$$

where,

E	=	PCC elastic modulus, psi.
f'_c	=	PCC compressive strength, psi.

- The lab tested CTE averages for each mix type appear to be lower than the national average. It is recommended that MoDOT perform additional confirmation testing and at the same time cross-verify the results with an independent laboratory such as FHWA's laboratory at the Turner0Fairbank Highway Research Facility. In the interim, it is recommended that the following test values obtained from MoDOT's testing of field cores as part of the local calibration effort be used for design purposes:

    - CTE of Gradation B = 5.4 in/in/^{o}F.
    - CTE of Gradation D = 5.0 in/in/^{o}F.
    - CTE of Gradation F =4.75 in/in/^{o}F.

  As can be noted, the field CTEs are higher than those noted for the corresponding mix types in Table I-40 and are closer to the MoDOT average field CTEs for concrete made with limestone.

- It is strongly recommended that, inasmuch as possible, all PCC properties for a given mix including cement factor, water cement ratio, strength, modulus, and CTE, be entered into the MEPDG software as a set of properties for a given mix type and not individual

entries. The materials library developed as part of this project contains a collection of PCC inputs for each MoDOT mix tested in this program.

3.5 UNBOUND MATERIALS CHARACTERIZATION

As noted earlier, two distinct sets of information are needed for unbound materials:

- M_r at optimum moisture for maximum dry density from laboratory testing or through correlations with other properties such as the penetration index from the DCP, or CBR values, etc.
- Soils gradations, Atterberg limits, unit weight, etc. from which more complex parameters such as the soil's drainage and suction characteristics are defined.

MoDOT has contracted with the Missouri University of Science and Technology (MUST) to develop M_r inputs for the dense graded aggregate base material (Type 5 dense graded aggregate base) and subgrade soils.

The other base type used in Missouri is the large stone base. This base material is used directly under the pavement as well as in embankments/fills. While it is difficult to characterize this material in the laboratory, in this study, as part of the local calibration effort, the modulus of the layer was estimated through backcalculation.

A summary of the results for these two base types and soils is presented in the following sections.

3.5.1 Type 5 Unbound Base

3.5.1.1 M_r Testing

For the M_r testing on unbound base materials, MoDOT sampled five stockpiles representing the expected variation of field conditions for this base material. The stockpile locations included JC Dolomite (JCD), Gasconade (Gasc), Bethany Falls (BF), Winterset (Win), and Plattin (Plat). The material was delivered to MUST's materials testing laboratories. For the purposes of this discussion, the material delivered from MoDOT is termed A-D to represent "As-Delivered" material. At MUST's testing laboratory, a blended material comprising of 95 percent A-D material and 5 percent fines (passing number 200 material) was created to simulate "in place" conditions. This was done to study the impact of the additional percentage of minus 200 material, assumed to be generated as the aggregate gets handled (loaded into trucks, hauled, dumped, shaped, and compacted) during the construction process, on its resilient modulus. This blended material with additional fines is designated W-F in the ensuing discussion.

Resilient modulus test specimens were prepared from these materials and tested in accordance with AASHTO T 307 as recommended by the MEPDG at optimum moisture content and maximum dry density. For each specimen tested, 15 M_r values at various combinations of confining and deviator stress levels are reported. The actual value of M_r to be used for design

will require an *a priori* knowledge of the stress state that is best suited for the in place material (including stresses due to overburden). However, to determine the stress state, knowledge of the M_r is essential. Therefore, by nature, determining the appropriate M_r for use in design is an iterative process which begins with assumptions regarding the following parameters:

- Design structure (layer types, thicknesses, and moduli)
- Loading.
- Location in the base where stresses are computed.
- The relationship between M_r and stress state.

The MEPDG articulates a methodology to estimate the M_r for unbound materials and soils. Essentially, without loss in accuracy, a typical design structure and a nominal wheel loading of 9000-lb (representing one-half of an 18-kip axle) can be assumed. Furthermore, the evaluation point where the stresses due to the applied loading and overburden are to be computed in the Type 5 base can be taken as being ¼th depth from the top of the layer. The MEPDG recommends that equation 3, termed the generalized constitutive model, be used to determine the relationship between the M_r and stress state. The nonlinear elastic coefficients and exponents of the constitutive model are determined by using linear or nonlinear regression analyses to fit the model to laboratory generated M_r test data.

$$M_r = k_1 p_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \qquad (3)$$

where

M_r = resilient modulus, psi

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$

σ_1 = major principal stress.

σ_2 = intermediate principal stress = σ_3 for M_r test on cylindrical specimen.

σ_3 = minor principal stress/confining pressure

τ_{oct} = octahedral shear stress

τ_{oct} = $\frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$

P_a = normalizing stress (atmospheric pressure)

k_1, k_2, k_3 = regression constants (obtained by fitting resilient modulus test data to equation 3)

The MEPDG (ARA 2004) recommends that the constitutive model coefficients determined for each test specimen be such that the multiple correlation coefficient, R^2, exceeds 0.90. Constitutive model coefficients from similar soils and test specimen conditions can be combined to obtain a "pooled" k1, k2, and k3. The MEPDG also notes that If the R^2 for a particular test specimen is less than 0.90, the test results and equipment should be checked for possible errors and/or test specimen disturbance. If no errors or disturbances are found, the use of a different constitutive relationship should be considered.

Using the laboratory tested M_r data, MUST researchers developed the k_1, k_2, k_3 terms for values for each Type 5 base material evaluated. Table I-41 through Table I-45 present a summary of the all the information developed by MUST researchers for each of the Type 5 bases evaluated under their study. As can be noted, the regression statistics are reasonable in almost all the cases establishing the validity of the testing and data analysis. In reviewing the data in the tables and the specific, M_r test results developed by MUST researchers (not presented herein), the following conclusions can be drawn:

- As is typical for coarse grained materials (like Type 5 bases), M_r increases with increasing bulk stress.
- As the applied bulk stress (the primary stress of interest to coarse grained material characterization) in the laboratory varied from 10 to 100 psi, the range of M_r from the testing varied from 20,000 to 100,000 psi.
- Additional fines did not seem to impact the M_r at optimum moisture much except for the material from Bethany Falls, i.e., the differences in the M_r values at any given stress state between the A-D and W-F materials do not appear to be significant. However, it is expected that, M_r of the W-F material will be affected more dramatically than the A-D materials at higher than optimum moisture contents typically experienced in the field (moisture changes in the unbound materials are modeled by the enhanced integrated climatic model [EICM] module of MEPDG). Therefore, if additional fines are indeed generated in the field, MoDOT should make a decision regarding the gradation parameters to be entered in the MEPDG for type 5 bases.

It should also be pointed out that the estimated M_r at optimum moisture content and maximum dry density is further modified in the field due to changes in base moisture as well as the modulus of changes of the underlying layer. The latter effects due to seasonal impacts of precipitation and ground water table depth as well as the drainability of the base layer are handled through the EICM module of the MEPDG (as pointed out above). However, the relationship between the subgrade and the base modulus is not explicitly handled in the MEPDG at the present time. The general theory in this regard was postulated by Barker and Brabston (1975) and can be summed up as stating that, the unbound base modulus is limited by its thickness and the modulus of the underlying unbound layers. This is because unbound materials cannot take tension and if the modular ratios between adjacent layers are large, the stiffer layers could decompact under loading. A rule of thumb estimate of the maximum allowable modular ratio between two adjacent unbound layers is 3. Considering this, it is recommended that the MEPDG default values be used for unbound base M_r after adjusting them appropriately for decompaction effects using Figure I- 101.

The M_r data developed from the MUST research have been entered into the materials library developed as part of this study.

Table I-41. Summary of resilient modulus test data and associated moisture contents and densities for A-D and W-F materials sampled from JC Dolomite (based on personal communication with MoDOT and MUST researchers).

Specimen	Target Dry Density (pcf)	Target Moisture Content (%)	As-Compacted Moisture Content [7th lift] (%)	Actual Dry Density (pcf)	Actual Moisture Content (%)	Degree Sat.* (%)	Deg.S** CoreLok (%)	Deg. S*** Wt. Avg (%)	k1	k2	k3	Rsq	Se/Sy
JCD A-D2	133.6	9.0	9.1	133.7	8.6	85.0	84.2	80.9	1818.332	0.635	-0.303	0.992	0.098
JCD A-D3	133.6	9.0	9.0	133.8	8.7	86.4	85.5	82.2	1509.215	0.652	-0.236	0.990	0.107
JCD A-D4	133.6	9.0	9.0	133.7	8.7	85.5	84.7	81.4	1556.384	0.667	-0.227	0.989	0.113
Average				133.8	8.7	85.7	84.8	81.5	1627.558	0.651	-0.257	0.963	0.197
SD				0.0452	0.0572	0.6889	0.6809	0.6498					
CV (%)				0.0338	0.6594	0.8042	0.8031	0.7974					
JCD W-F1	136.2	8.9	9.0	136.3	8.6	93.9	92.9	88.6	1130.693	0.683	-0.231	0.986	0.130
JCD W-F2	136.2	8.9	8.8	136.3	8.7	94.8	93.8	89.5	1434.249	0.642	-0.327	0.981	0.148
JCD W-F3	136.2	8.9	8.7	136.4	8.5	93.1	92.2	87.9	1378.216	0.670	-0.396	0.987	0.124
Average				136.3	8.6	93.9	93.0	88.7	1313.939	0.664	-0.322	0.954	0.219
SD				0.0336	0.0867	0.8444	0.8364	0.8022					
CV (%)				0.0246	1.0046	0.8988	0.8996	0.9044					

Table I-42. Summary of resilient modulus test data and associated moisture contents and densities for A-D and W-F materials sampled from Gasconade (based on personal communication with MoDOT and MUST researchers).

Specimen	Target Dry Density (pcf)	Target Moisture Content (%)	As-Compacted Moisture Content [7th lift] (%)	Actual Dry Density (pcf)	Actual Moisture Content (%)	Degree Sat.* (%)	Deg.S** CoreLok (%)	Deg. S*** Wt. Avg (%)	k1	k2	k3	Rsq	Se/Sy
Gasc A-D1	133.7	9.4	9.8	133.9	8.9	92.6	91.7	87.4	1456.506	0.783	-0.353	0.998	0.050
Gasc A-D2	133.7	9.4	9.9	133.8	8.8	91.6	90.7	86.4	1429.921	0.752	-0.323	0.997	0.058
Gasc A-D3	133.7	9.4	9.8	133.7	8.9	92.4	91.5	87.2	1298.257	0.796	-0.325	0.996	0.068
Average				133.8	8.9	92.2	91.3	87.0	1394.519	0.777	-0.334	0.989	0.110
SD				0.0647	0.0491	0.5420	0.5361	0.5074					
CV (%)				0.0484	0.5511	0.5877	0.5871	0.5833					
Gasc W-F1	134.4	9.1	9.2	134.4	8.8	92.8	91.8	87.0	1505.352	0.690	-0.406	0.989	0.112
Gasc W-F2	134.4	9.1	9.4	134.6	8.7	93.1	92.2	87.3	1461.368	0.689	-0.354	0.992	0.094
Gasc W-F3	134.4	9.1	9.5	134.6	8.8	93.6	92.6	87.7	1211.002	0.769	-0.344	0.993	0.089
Average				134.5	8.8	93.2	92.2	87.3	1390.806	0.714	-0.367	0.979	0.149
SD				0.1183	0.0188	0.3950	0.3879	0.3501					
CV (%)				0.0879	0.2149	0.4239	0.4207	0.4011					

Table I-43. Summary of resilient modulus test data and associated moisture contents and densities for A-D and W-F materials sampled from Plattin (based on personal communication with MoDOT and MUST researchers).

Specimen	Target Dry Density (pcf)	Target Moisture Content (%)	As-Compacted Moisture Content [7th lift] (%)	Actual Dry Density (pcf)	Actual Moisture Content (%)	Degree Sat.* (%)	Deg.S** CoreLok (%)	Deg. S*** Wt. Avg (%)	k1	k2	k3	Rsq	Se/Sy
Plat A-D1	133.9	8.0	8.1	133.9	7.6	85.3	84.3	83.7	2449.920	0.583	-0.279	0.988	0.121
Plat A-D2	133.9	8.0	7.8	134.1	7.3	82.5	81.6	81.0	3173.793	0.557	-0.289	0.988	0.120
Plat A-D3	133.9	8.0	7.9	134.2	7.4	82.9	82.0	81.4	2687.857	0.594	-0.273	0.989	0.114
Average				134.1	7.4	83.6	82.7	82.0	2769.219	0.577	-0.280	0.901	0.321
SD				0.1472	0.1703	1.4934	1.4807	1.4722					
CV (%)				0.1098	2.2869	1.7873	1.7914	1.7951					
Plat W-F1	134.4	7.9	6.8	135.6	6.9	82.3	81.4	79.6	3081.705	0.478	-0.135	0.985	0.133
Plat W-F2	134.4	7.9	8.2	134.2	7.9	88.5	87.5	85.7	2410.360	0.520	-0.214	0.986	0.129
Plat W-F3	134.4	7.9	8.0	134.6	7.6	86.9	85.9	84.1	2896.399	0.454	-0.281	0.974	0.173
Average				134.8	7.5	85.9	84.9	83.1	2794.575	0.483	-0.207	0.850	0.396
SD				0.7367	0.4908	3.1936	3.1799	3.1628					
CV (%)				0.5466	6.5681	3.7176	3.7451	3.8057					

Table I-44. Summary of resilient modulus test data and associated moisture contents and densities for A-D and W-F materials sampled from Winterset (based on personal communication with MoDOT and MUST researchers).

Specimen	Target Dry Density (pcf)	Target Moisture Content (%)	As-Compacted Moisture Content [7th lift] (%)	Actual Dry Density (pcf)	Actual Moisture Content (%)	Degree Sat.* (%)	Deg.S** CoreLok (%)	Deg. S*** Wt. Avg (%)	k1	k2	k3	Rsq	Se/Sy
Win A-D1	133.6	8.6	8.1	133.7	8.0	85.4	84.6	79.4	2103.090	0.581	-0.232	0.992	0.096
Win A-D2	133.6	8.6	8.5	133.9	8.0	85.9	85.1	79.8	2014.575	0.595	-0.266	0.984	0.137
Win A-D3	133.6	8.6	8.5	133.8	7.9	84.4	83.5	78.4	1722.742	0.650	-0.308	0.991	0.103
Average				133.8	8.0	85.2	84.4	79.2	1946.593	0.607	-0.266	0.952	0.224
SD				0.0809	0.0730	0.7909	0.7827	0.7319					
CV (%)				0.0605	0.9183	0.9278	0.9273	0.9239					
Win W-F1	134.4	8.0	7.9	134.5	7.9	86.9	86.0	80.6	2087.403	0.508	-0.268	0.965	0.201
Win W-F2	134.4	8.0	7.8	134.6	7.9	87.0	86.1	80.7	2025.851	0.547	-0.262	0.977	0.162
Win W-F3	134.4	8.0	8.2	134.5	7.9	86.8	85.9	80.5	2052.418	0.506	-0.215	0.969	0.189
Average				134.5	7.9	86.9	86.0	80.6	2054.856	0.520	-0.248	0.968	0.182
SD				0.0719	0.0164	0.1043	0.1019	0.0867					
CV (%)				0.0534	0.2069	0.1200	0.1185	0.1075					

Table I-45. Summary of resilient modulus test data and associated moisture contents and densities for A-D and W-F materials sampled from Bethany Falls (based on personal communication with MoDOT and MUST researchers).

Specimen	Target Dry Density (pcf)	Target Moisture Content (%)	As-Compacted Moisture Content [7th lift] (%)	Actual Dry Density (pcf)	Actual Moisture Content (%)	Degree Sat.* (%)	Deg.S** CoreLok (%)	Deg. S*** Wt. Avg (%)	k1	k2	k3	Rsq	Se/Sy
BF A-D1	138.0	7.4	7.2	138.2	6.8	86.1	85.1	81.3	2044.140	0.632	-0.320	0.994	0.086
BF A-D2	138.0	7.4	7.3	138.3	6.8	86.4	85.4	81.6	2281.441	0.541	-0.321	0.987	0.122
BF A-D3	138.0	7.4	7.3	138.3	6.7	85.9	84.9	81.1	2098.281	0.610	-0.328	0.988	0.120
Average				138.3	6.8	86.1	85.1	81.3	2139.855	0.594	-0.322	0.985	0.125
SD				0.0779	0.0223	0.2767	0.2723	0.2548					
CV (%)				0.0564	0.3291	0.3213	0.3199	0.3133					
BF W-F1	139.3	7.0	7.2	139.6	6.6	88.0	86.9	82.7	1887.359	0.685	-0.377	0.993	0.093
BF W-F2	139.3	7.0	7.1	139.5	6.6	88.6	87.5	83.3	1738.195	0.686	-0.368	0.993	0.093
BF W-F3	139.3	7.0	6.9	139.5	6.6	89.0	87.9	83.7	1876.223	0.640	-0.316	0.985	0.132
Average				139.5	6.6	88.6	87.4	83.2	1833.686	0.670	-0.354	0.981	0.143
SD				0.0329	0.0467	0.5028	0.4978	0.4794					
CV (%)				0.0236	0.7079	0.5678	0.5693	0.5759					

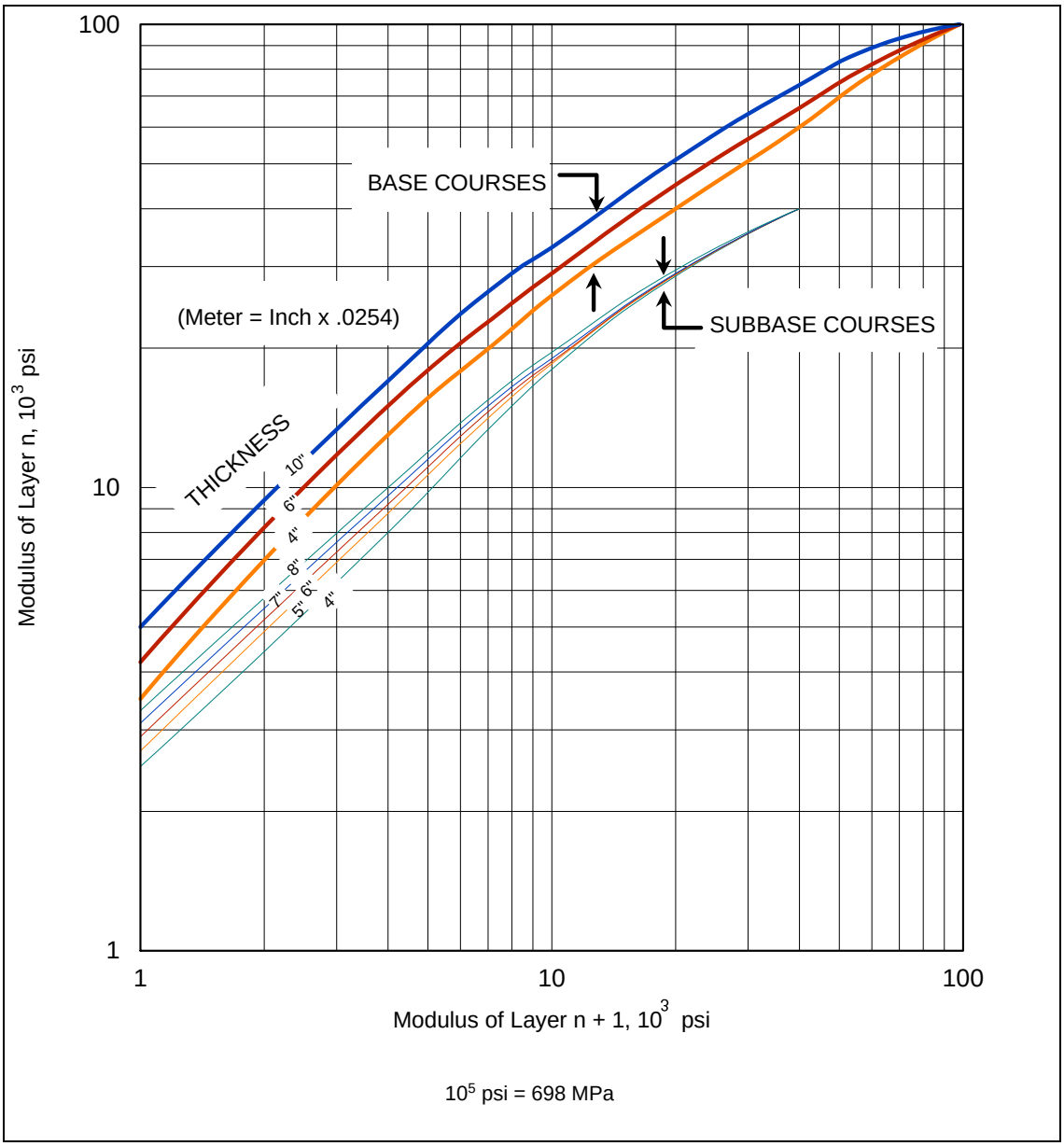


Figure I- 101. Limiting modulus criteria of unbound aggregate base and subbase layers (after Barker and Brabston, 1975).

3.5.1.2 Type 5 Materials Additional Data

Table I-46 presents the typical gradations and Atterberg's limits values for Type 5 materials by District. These data are essential for climatic effects modeling in the MEPDG. These data were tabulated based on a query of 1007 Type 1 & 5 base rock properties logged in at MoDOT's central materials laboratory since between January 2007 and April 2008. These inputs have been stored in the MoDOT MEPDG materials library and can be used in routine design.

Table I-46. Summary of default gradation and Atterberg's limits inputs for MoDOT Type 5 base by district.

District	Average LL	Average PL	Average PI	Average Gradation - % Passing				
				1"	1/2"	No. 4	No. 30	No. 200
1	22	17	5	100.0	75.5	45.0	21.4	15.3
2	0	0	0	100.0	66.6	35.5	16.2	8.1
3	6	5	1	100.0	67.4	33.9	16.3	9.6
4	16	13	3	100.0	75.2	44.0	20.2	12.9
5	4	4	1	100.0	77.9	45.0	20.6	11.0
6	10	8	2	100.0	70.4	37.9	18.5	11.2
7	2	2	0	100.0	57.3	26.5	10.8	7.5
8	8	7	2	100.0	77.5	45.7	19.1	10.6
9	4	3	0	100.0	76.4	43.5	20.5	9.1
10	0	0	0	100.0	74.1	46.7	17.1	7.2

3.5.2 Large Stone Base

As pointed out earlier, estimates of the modulus for the large stone base (LSB) or rock base were derived based on backcalculation of Falling Weight Deflectometer (FWD) data performed using the EVERCALC software from Washington DOT on MoDOT pavement management sections which included these bases as part of the local validation and calibration effort. Typical LSB backcalculated values by station from one 500-ft Missouri PMS sections are presented in Figure I-102. Figure I-103 presents the overall root mean square error (RMSE) from the backcalculation process by station. RMSE values less than 3 percent, although do not guarantee the engineering reasonableness of the backcalculated results, do indicate that a successful backcalculation has been completed. Such data can then be evaluated to see if the estimates of moduli pass the test of reasonableness. Based on this assessment the normalized backcalculated moduli (i.e., all FWD loads normalized to a 9,000 lb load level) for the LSB layer presented in figure I-101 range between 10,000 psi and 40,000 psi. Similar ranges of numbers were found on five other sections backcalculated.

On the basis of this information, and considering that seasonal variation in moisture contents will not change the modulus of this material due its excellent drainability and large aggregate structure, it is recommended that the LSB layer be modeled as a constant modulus material in the MEPDG with a representative modulus of 30,000 psi.

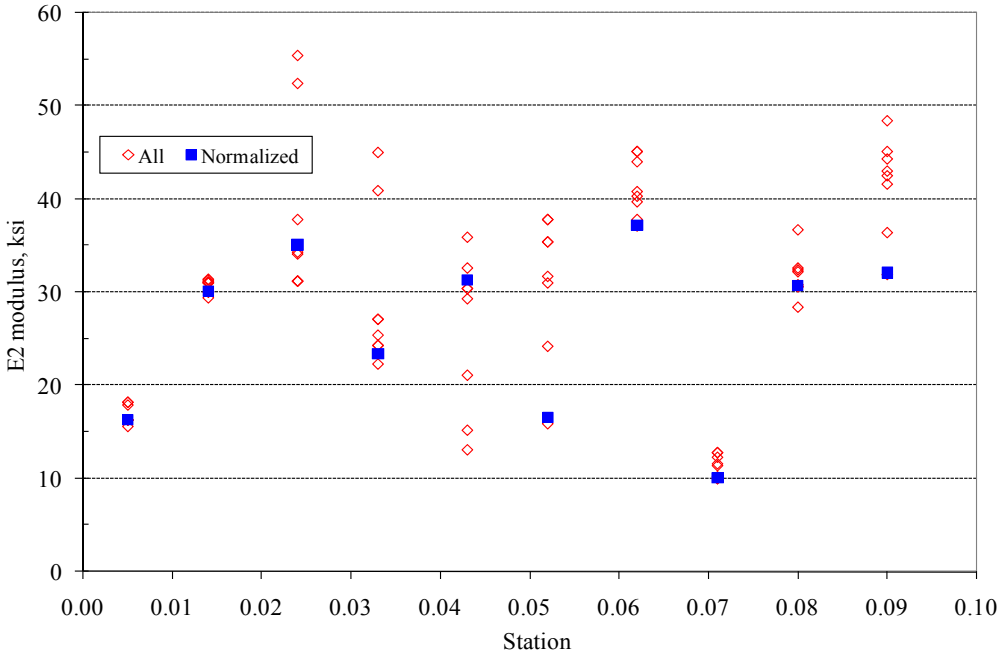


Figure I-102. Backcalculated large stone base moduli for MoDOT calibration section DS9.

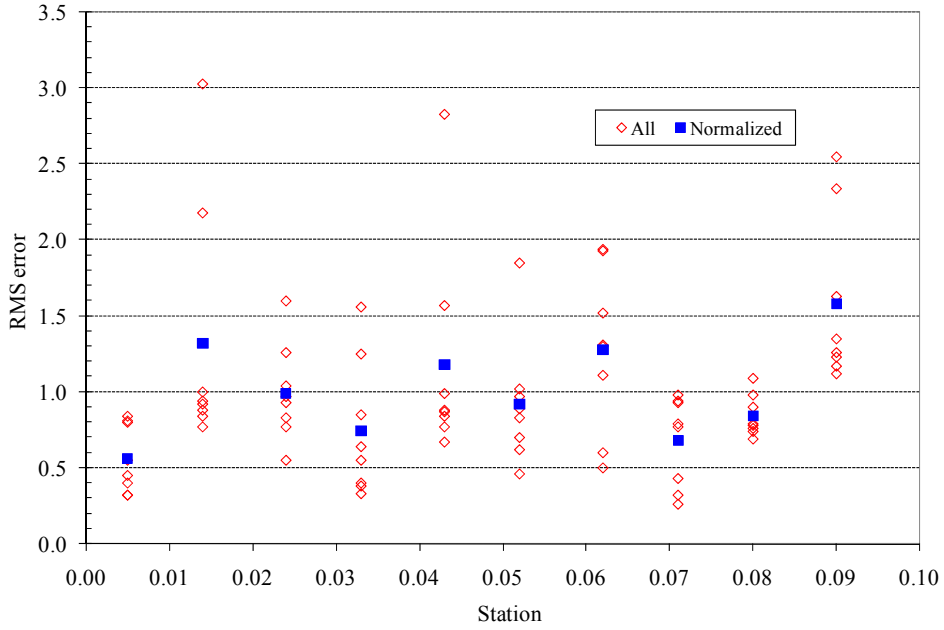


Figure I-103. Average RMSE from the backcalculation for MoDOT calibration section DS9.

3.5.3 Subgrade Soils Characterization

Under a MoDOT project titled "*Resilient Moduli of Typical Missouri Soils and Unbound Granular Base Materials,"* a testing program was recently concluded at MUST to determine the resilient moduli for common Missouri subgrade soils. The test plan included data from 27 common subgrade soils drawn from 99 Missouri soil series. The soils were selected from various geographic regions of Missouri as illustrated in Figure I-104. Soil class A-7-6 predominates in this group and is the typical soil type in Missouri. Other soil classes include A-2-4, A-2-6, A-4, A-6 and A-7-5. The subgrade soils were tested at two different conditions: one set at maximum dry density and optimum moisture content, at 95% MDD and the elevated moisture content. Resilient modulus testing for subgrade soils was conducted in accordance with the AASHTO T 307 test method.

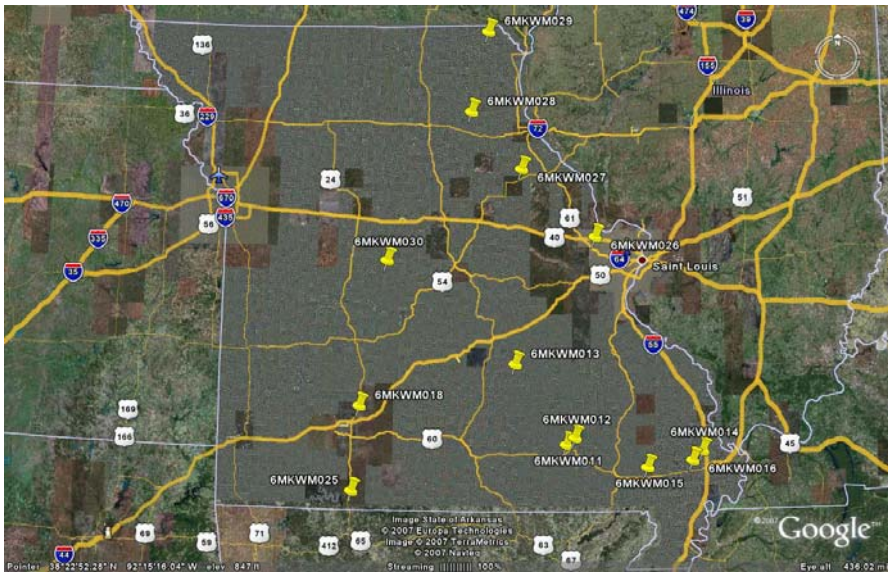


Figure I-104. Selected locations of soil sites for subgrade characterization (Richardson et al. 2008).

Table I-47 presents the information of each tested soil, including the soil series, the district number, AASHTO and ASTM soil classifications, specific gravities and Atterberg limits. Table I-48 presents the gradation of the soil samples tested. Table I-49 presents the maximum dry density (MDD), the optimum moisture content (OMC), percent degree of saturation (S), and the resilient modulus regression coefficients, k_1, k_2 and k_3 of all soils tested at the MDD and OMC. F For soils tested at wet of optimum, only the k_1, k_2, and k_3 coefficients are presented. These coefficients were obtained in accordance with equation 3. In order to obtain the coefficients presented in Table I-49, the resilient modulus test results at each applied stress state were regressed against the corresponding octahedral shear stress (τ_{oct}) and deviator stress (σ_d) applied to the soil specimen.

Table I-47. Properties of Missouri soils (from Richardson et al. 2008).

MoDOT ID	Soil Series	District	ASTM Soil Class	AASHTO Soil Class	Spec. Grav.	Liquid Limit	Plastic Limit	Plasticity Index
6MKWM011	Clarksville	9	CH	A-7-6	2.70	50	21	29
6MKWM012	Poynor/ Clarksville	9	SC	A-2-6	2.66	30	17	13
6MKWM013	Hobson-Lebanon	9	SC	A-4	2.66	23	13	10
6MKWM014	Lintonia	10		A-2-4	2.66	18*	17*	NP 1*
6MKWM015	Waverly	10	CL	A-6	2.68	32	18	14
6MKWM016	Sharkey	10	CH	A-7-6	2.70	63	21	42
6MKWM017	Sonsac	6	CH	A-7-6	2.70	65	23	42
6MKWM018	Wilderness-Viration	8	CH	A-7-6	2.70	62	28	34
6MKWM019	Darwin-Dockery-Chequest	2	CL	A-7-6	2.68	48	17	31
6MKWM020	Carlow-Dockery	2	CL	A-6	2.68	35	22	13
6MKWM021	Mexico-Leonard Putnam	2	CL	A-6	2.68	37	23	14
6MKWM022	Nixa-Clarksville Hailey	7	MH	A-7-5	2.68	61	33	28
6MKWM023	Cliquot-Bolivar	7	CL	A-7-6	2.68	46	24	22
6MKWM024	Eldorado-Newtonia	7	CL	A-7-6	2.68	49	24	25
6MKWM025	Rueter-Clarksville-	8	SC	A-7-6	2.66	44	19	25
6MKWM026	Glacial-residual	6	CH	A-7-6	2.70	57	18	39
6MKWM027	Belknap-Okaw-Twomile	3	SM	A-2-4	2.66	27	20	7
6MKWM028	Mexico-Armstrong	3	CH	A-7-6	2.70	52	22	30
6MKWM029	Vesser-Klum-Wakefiled	3	CL	A-6	2.68	34	20	14
6MKWM030	Hunington	5	SC	A-6	2.66	38	23	15
6MKWM031	Knox	1		A-4*	2.68	34*	21*	13*
6MKWM032	Sampsel-Snead-Polo	4	CH	A-7-6	2.70	60	20	40
6MKWM033	Mandeville-Norris-Bolivar	4	CL	A-7-6	2.68	43	17	26
6MKWM034	Knox-Sibley-Urban Land	4	CL	A-7-6	2.68	48	19	29
6MKWM035	Snead-Menfro-Oska	4	CL	A-6	2.68	36	21	15
6MKWM036	Marksburg-Sharpsburg-Sampsel	4	CH	A-7-6	2.70	55	24	31
6MKWM037	Useful	6	CL	A-7-6	2.70	48	19	29

Notes:

* Data reported in Richardson et al. 2008 are from the literature.

Table I-48. Gradations of Missouri Soils (from Richardson et al. 2008).

MoDOT ID	3”	2”	1½”	1”	¾”	3/8”	#4	#10	#40	#200
6MKWM011	100	100	100	96	95	92	88	86	74	59
6MKWM012	100	100	98	92	89	80	70	63	48	24
6MKWM013	100	100	100	97	96	93	91	89	75	37
6MKWM014	100	100	100	100	100	100	100	100	67	10
6MKWM015	100	100	100	100	100	100	100	100	100	82
6MKWM016	100	100	100	100	100	100	100	100	98	92
6MKWM017	100	100	100	100	100	96	91	89	83	67
6MKWM018	100	100	100	100	100	97	93	90	82	77
6MKWM019	100	100	100	100	100	100	99	99	93	74
6MKWM020	100	100	100	100	100	100	100	100	99	97
6MKWM021	100	100	100	100	100	100	100	100	96	82
6MKWM022	100	98	97	94	91	88	87	85	80	76
6MKWM023	100	100	100	100	100	100	100	99	97	90
6MKWM024	100	100	100	100	100	100	99	98	95	91
6MKWM025	100	100	100	100	100	96	86	72	56	46
6MKWM026	100	100	100	100	100	100	100	98	94	75
6MKWM027	100	100	98	91	83	65	54	46	33	25
6MKWM028	100	100	100	100	100	100	100	100	99	96
6MKWM029	100	100	100	100	100	98	96	94	88	75
6MKWM030	100	100	94	85	77	64	56	51	44	36
6MKWM031										82*
6MKWM032	100	100	100	100	100	100	100	100	99	97
6MKWM033	100	100	100	100	100	100	100	100	98	90
6MKWM034	100	100	100	100	100	100	100	100	99	98
6MKWM035	100	100	100	100	100	99	99	98	96	93
6MKWM036	100	100	100	100	100	100	100	100	98	96
6MKWM037	100	100	100	100	100	100	100	99	98	88

Notes:

* Data reported in Richardson et al. 2008 are from the literature.

Table I-49. MEPDG soils related inputs for Missouri soils (from Richardson et al. 2008).

District	Soil ID	AASHTO Soil Class	Condition	MDD (pcf)	OMC (%)	S (%)	Resilient Modulus Coefficients		
							k_1	k_2	k_3
1	6MKWM031	A-4	Optimum	108	17	83.1	644.6	0.72	-3.94
1	6MKWM031	A-4	Wet				556.5	0.13	-2.12
2	6MKWM019	A-7-6	Optimum	108	16	78.2	1218.8	0.20	-2.82
2	6MKWM020	A-6	Optimum	102	20	83.8	706.4	0.17	-5.27
2	6MKWM020	A-6	Wet				958.8	0.27	-6.34
2	6MKWM021	A-6	Optimum	101	18	73.6	1018.3	0.30	-2.62
2	6MKWM021	A-6	Wet				845.1	0.49	-4.94
3	6MKWM027	A-2-4	Optimum	126	9	75.5	974.0	0.45	-2.98
3	6MKWM028	A-7-6	Optimum	97	22	80.6	1635.3	0.07	-3.86
3	6MKWM028	A-7-6	Wet				1574.3	-0.46	-7.75
3	6MKWM029	A-6	Optimum	109	15	75.2	832.1	0.37	-4.61
4	6MKWM032	A-7-6	Optimum	94	24	81.8	1295.9	0.24	-2.11
4	6MKWM032	A-7-6	Wet				921.1	0.18	-5.74
4	6MKWM033	A-7-6	Optimum	103	18	77.4	1664.6	0.33	-1.89
4	6MKWM033	A-7-6	Wet				940.7	0.12	-5.55
4	6MKWM034	A-7-6	Optimum	101	19	77.7	1322.4	0.25	-1.71
4	6MKWM034	A-7-6	Wet				1033.3	0.10	-5.06
4	6MKWM035	A-6	Optimum	100	18	71.8	1881.9	0.22	-1.51
4	6MKWM035	A-6	Wet				1071.2	0.16	-4.85
4	6MKWM036	A-7-6	Optimum	94	23	78.4	1185.5	0.18	-2.58
4	6MKWM036	A-7-6	Wet				1980.4	0.24	-6.66
5	6MKWM030	A-6	Optimum	118	12	78.5	1529.0	0.15	-1.72
6	6MKWM017	A-7-6	Optimum	97	20	73.3	1261.7	0.40	-4.79
6	6MKWM026	A-7-6	Optimum	99	22	84.6	847.3	0.11	-5.20
6	6MKWM037	A-7-6	Optimum	105	19	85.9	1046.2	0.27	-4.57
6	6MKWM037	A-7-6	Wet				847.5	-0.08	-5.09
7	6MKWM022	A-7-5	Optimum	80	32	73.6	1667.4	0.28	-2.61
7	6MKWM023	A-7-6	Optimum	100	20	79.7	1567.7	0.26	-2.29
7	6MKWM023	A-7-6	Wet				1182.5	0.04	-5.51
7	6MKWM024	A-7-6	Optimum	96	21	75.8	1297.9	0.32	-2.24
7	6MKWM024	A-7-6	Wet				1452.8	0.20	-7.74
8	6MKWM018	A-7-6	Optimum	91	27	85.6	1106.4	-0.98	-4.42
8	6MKWM025	A-7-6	Optimum	112	13	71.7	1187.3	0.21	-2.21
8	6MKWM025	A-7-6	Wet				2274.8	0.35	-4.25
9	6MKWM011	A-7-6	Optimum	101	20	80.8	1627.1	0.28	-1.77
9	6MKWM012	A-2-6	Optimum	120	11	76.4	2185.4	0.32	-1.27
9	6MKWM013	A-4	Optimum	121	11	78.7	2241.3	0.29	-2.17
10	6MKWM014	A-2-4	Optimum	110	12	62.7	984.1	0.54	-2.66
10	6MKWM014	A-2-4	Wet				1007.2	0.42	-2.89
10	6MKWM015	A-6	Optimum	112	15	81.5	614.0	0.49	-2.58
10	6MKWM015	A-6	Wet				529.1	0.17	-1.29
10	6MKWM016	A-7-6	Optimum	96	21	75.1	1059.4	0.16	-2.96
10	6MKWM016	A-7-6	Wet				933.3	0.26	-6.45

Resilient Modulus Estimation for Typical Missouri Sections

The k_1, k_2, and k_3 regression coefficients presented in Table I-49 can be directly entered into the MEPDG at level 1. However, the MEPDG documentation (ARA 2004) cautions that the analysis tool that accepts level 1 inputs within the MEPDG is uncalibrated and results in long run times. For this reason, level 2 or 3 inputs are recommended for routine design in Missouri. At levels 2 or 3, the MEPDG requires the user to input a laboratory-derived M_r value (presumably determined at the correct stress state for a given design situation) at OMC and MDD conditions. The regression coefficients presented for the various soil types in Table I-49 can be used in conjunction with equation 3 to determine the appropriate M_r to use for a given subgrade soil provide the applied τ_{oct} and σ_d for the design situation at hand are known. However, in order to compute these stresses using elastic layer theory, the soil's M_r needs to be known *a priori*. To overcome this mutual dependence of stresses and modulus, iterative techniques are often employed to solve for the M_r. In these situations, the user starts from a initial seed M_r value and iterates through the stress computations and modulus estimation until two successive M_r estimates converge to a predetermined tolerance level (say within 5 percent of each other).

To simplify the application of this approach for production-level design in MoDOT, the regression coefficients presented in Table I-49 were employed along with equation 3 to determine the stress-adjusted M_r at OMC and MDD as well as at "wet of optimum" conditions for typical MoDOT pavement sections. These sections are presented in Figure I-105 and represent the typical cross-sections used for Interstate or Primary, Other Principal Arterial, and Minor routes in Missouri. An 18,000 lb axle load was used in the analysis. The stresses were computed 18 inches below the surface of subgrade following the recommendations presented in the MEPDG (ARA 2004).

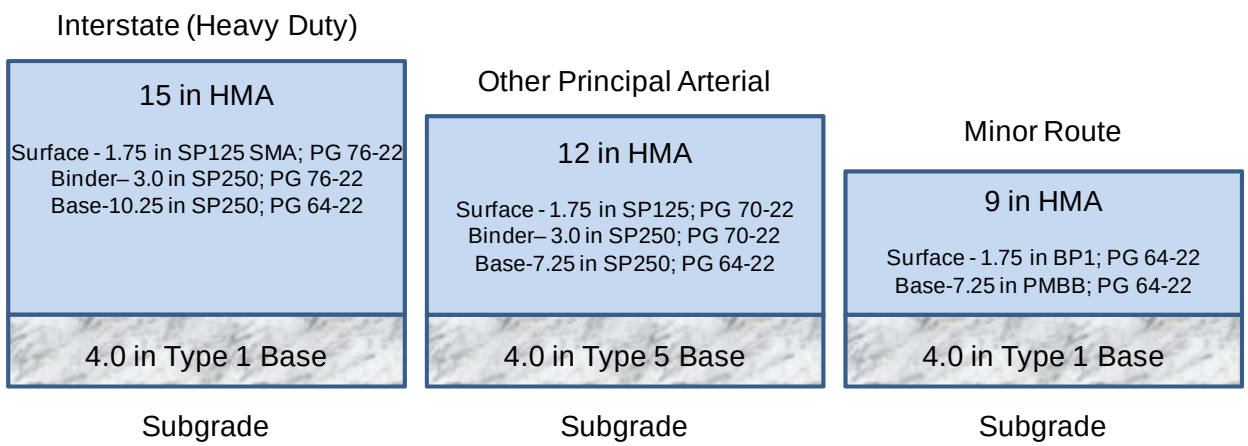


Figure I-105. Typical cross-sections for which stress-adjusted M_r was estimated.

Table I-50 presents the "stress-adjusted" resilient moduli at optimum and wet of optimum conditions calculated for each soil type for the three typical sections selected. A few major observations are noted below:

Table I-50. Resilient modulus inputs for Missouri soils.

District	Soil ID	Type	Condition	MDD (pcf)	OMC (%)	Interstate	Principal Arterial	Minor Route
9	6MKWM011	A-7-6	Optimum	101	20	18,848	18,371	17,848
9	6MKWM012	A-2-6	Optimum	120	11	25,368	24,675	23,927
9	6MKWM013	A-4	Optimum	121	11	26,101	25,483	24,811
10	6MKWM014	A-2-4	Optimum	110	12	9,417	8,970	8,497
10	6MKWM014	A-2-4	Wet			10,482	10,108	9,704
10	6MKWM015	A-6	Optimum	112	15	6,122	5,862	5,586
10	6MKWM015	A-6	Wet			6,736	6,637	6,528
10	6MKWM016	A-7-6	Optimum	96	21	13,163	13,003	12,821
10	6MKWM016	A-7-6	Wet			10,182	10,005	9,801
6	6MKWM017	A-7-6	Optimum	97	20	12,647	12,218	11,748
8	6MKWM018	A-7-6	Optimum	91	27	33,282	37,148	41,972
2	6MKWM019	A-7-6	Optimum	108	16	14,824	14,596	14,340
2	6MKWM020	A-6	Optimum	102	20	8,414	8,333	8,236
2	6MKWM020	A-6	Wet			10,440	10,253	10,039
2	6MKWM021	A-6	Optimum	101	18	11,454	11,152	10,821
2	6MKWM021	A-6	Wet			7,952	7,621	7,264
7	6MKWM022	A-7-5	Optimum	80	32	18,657	18,154	17,598
7	6MKWM023	A-7-6	Optimum	100	20	18,233	17,813	17,350
7	6MKWM023	A-7-6	Wet			15,547	15,613	15,673
7	6MKWM024	A-7-6	Optimum	96	21	14,436	14,017	13,560
7	6MKWM024	A-7-6	Wet			16,275	16,119	15,927
8	6MKWM025	A-7-6	Optimum	112	13	14,523	14,279	14,008
8	6MKWM025	A-7-6	Wet			24,404	23,744	23,019
6	6MKWM026	A-7-6	Optimum	99	22	10,588	10,549	10,498
3	6MKWM027	A-2-4	Optimum	126	9	10,130	9,765	9,375
3	6MKWM028	A-7-6	Optimum	97	22	21,577	21,554	21,514
3	6MKWM028	A-7-6	Wet			29,340	31,101	33,147
3	6MKWM029	A-6	Optimum	109	15	8,689	8,435	8,155
5	6MKWM030	A-6	Optimum	118	12	19,709	19,481	19,227
1	6MKWM031	A-4	Optimum	108	17	5,276	4,945	4,598
1	6MKWM031	A-4	Wet			7,225	7,159	7,084
4	6MKWM032	A-7-6	Optimum	94	24	15,326	14,998	14,634
4	6MKWM032	A-7-6	Wet			10,813	10,703	10,570
4	6MKWM033	A-7-6	Optimum	103	18	18,526	17,966	17,358
4	6MKWM033	A-7-6	Wet			11,647	11,605	11,549
4	6MKWM034	A-7-6	Optimum	101	19	15,637	15,280	14,887
4	6MKWM034	A-7-6	Wet			13,082	13,051	13,006
4	6MKWM035	A-6	Optimum	100	18	22,830	22,369	21,861
4	6MKWM035	A-6	Wet			12,960	12,841	12,701
4	6MKWM036	A-7-6	Optimum	94	23	14,615	14,404	14,166
4	6MKWM036	A-7-6	Wet			21,933	21,609	21,229
6	6MKWM037	A-7-6	Optimum	105	19	11,784	11,554	11,296
6	6MKWM037	A-7-6	Wet			12,215	12,403	12,605

- The predominant soil type in Missouri is the A-7-6 soils. At optimum moisture content, the average M_r for the A-7-6 soils was approximately 14,500 psi with a coefficient of variability of 16.7%. At elevated moisture, the average M_r was approximately 12,750 psi. The second predominant type of soil is the A-6. The average resilient modulus of the A-6 soils was 8,500 psi.
- In the case of A-2-4 and A-4 soils instances, where data are available for the same soil type, the wet of optimum M_r values were greater than the corresponding M_r values at optimum. This was unexpected. However, the project team did not have additional information to determine the reasons for these unexpected results. It is plausible that the degree of saturation of the tested specimens may hold the key to the answer. For A-6 and A-7-6 soils, the wet of optimum M_r values were lower than the corresponding M_r values at optimum as expected, however, the differences were not very large.
- There is not a great difference in the stresses computed underneath the various pavement cross-sections (including overburden) and hence the M_r values computed under the various cross-sections are more or less the same.
- Within the same AASHTO soil class, the gradation differences between soils can results in significant differences in M_r.

On the basis of these observations, it is recommended that MoDOT adopt the M_r values determined at OMC and MDD for the Other Principal Arterial cross-section for routine design purposes. Table I-51 presents typical statistical averages that can be used as level 3 inputs. Note that the perceived outliers in the measured M_r values presented in Table I-50 such as single data points, high variability within the sample set were removed in preparing Table I-51. Also note that, in Table I-51, where gradation differences were noted for soils belonging to the same AASHTO class, the M_r values were tabulated separately (i.e., for A-6 soils).

Table I-51. Typical resilient moduli for Missouri soils.

AASHTO Soil Classification	Condition	Average M_r, psi	Range of M_r, psi		Remarks
			Maximum	Minimum	
A-7-6	Optimum	14,500	18,500	10,500	
A-7-6	Wet	12,750	16,000	10,000	
A-2-4	Optimum	9,500	9,765	8,970	Only 2 soil series
A-2-4	Wet	10,100	N.A.	N.A.	Single soil series
A-4 (1)	Optimum	5,000	N.A.	N.A.	Single soil series in District 1
A-4 (1)	Wet	7,200	N.A.	N.A.	
A-4 (9)	Optimum	25,500	N.A.	N.A.	
A-6 (CL)	Optimum	8,500	11,152	5,862	The soil series were classified as CL
A-6 (CL)	Wet	8,000	10,253	6,637	
A-6 (SC)	Optimum	19,500	N.A.	N.A.	This soil series in District 5 was classified as SC
A-2-6	Optimum	25,000	N.A.	N.A.	Single soil series

3.6 PROJECT DATABASE

A project database was created in Microsoft Access to house all the materials testing information gathered from Missouri literature, laboratory testing, or even field testing performed as part of this study by MoDOT and its contractors. The data analyzed as part of this study were also stored in the database. The database is segmented into two parts:

1. Library information – which contains the information described above.
2. Calibration information – which contains all the inventory, testing, traffic, and site information related to each Missouri PMS section used in the analysis.

The database was modeled after the LTPP database and contains many of its features including separation of data into inventory, testing, and monitoring information. This database will be a part of the final deliverable on this project. An overview diagram of the calibration segment of the database is presented in Figure I-106.

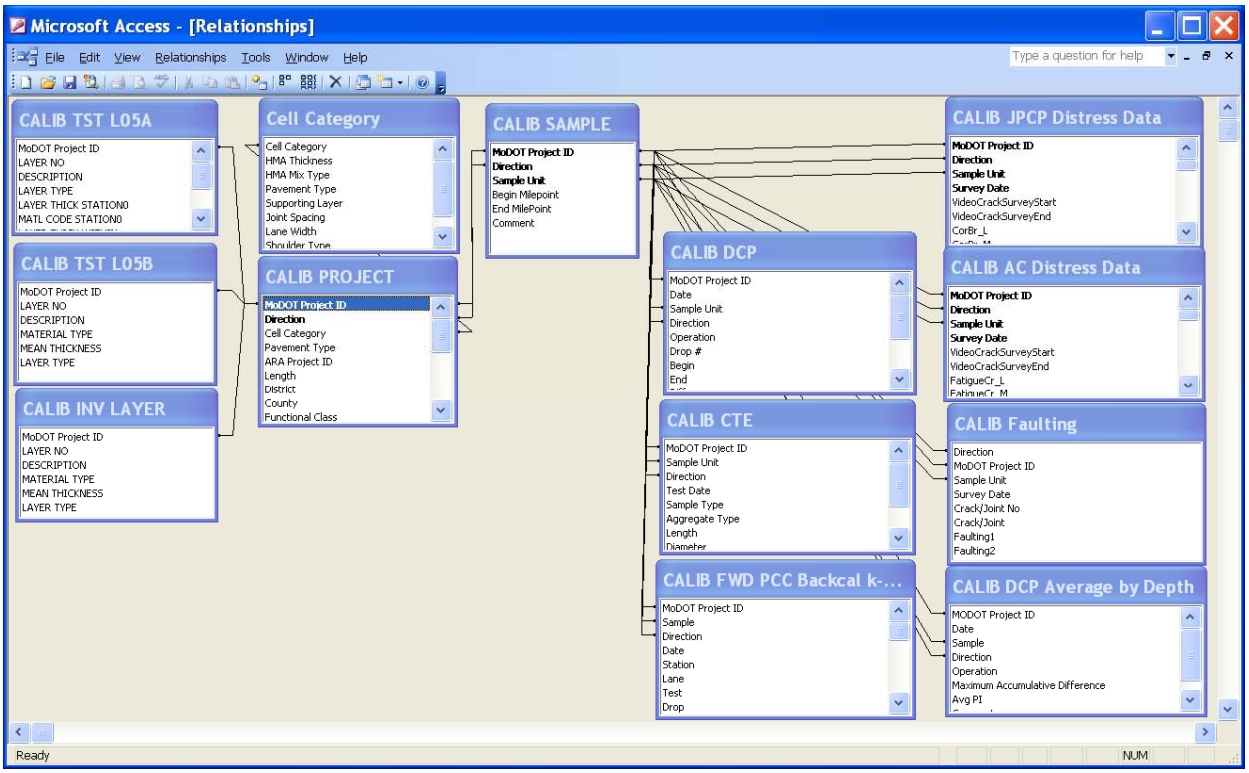


Figure I-106. Overview of the data tables and their linkages in MoDOT's relational database used to store the MEPDG related information.

CHAPTER 4. SENSITIVITY ANALYSIS

The MEPDG includes several models to predict pavement distress and smoothness, which form the basis for a designer to adequately optimize the trial design to meet performance requirements. The model outputs are therefore affected by the design and site-related inputs. The degree to which the output of a given model is affected by a particular input is largely a function of the distress mechanism modeled in the MEPDG and the empirical observations of the phenomenon that relate the two in the field calibration data. For most part, these linkages have been well established and explained for each distress or smoothness model in the documentation of the MEPDG (ARA 2004). However, model dependencies on specific inputs are best demonstrated through sensitivity analyses.

A sensitivity analysis is the process of varying model input parameters over a reasonable range and observing the relative change in model response. It provides a systematic review of how the model predictions change by varying inputs. Input parameters can cover the range of uncertainty in values of variables such as PCC strength, HMA dynamic modulus, and layer thicknesses within an agency's typical practice. The sensitivity of these parameters to model outputs can be observed. Typically, the observed changes in model response are critical to developing recommendations regarding model improvement, establishing procedures for obtaining a given input and desired levels of accuracy, selection of default values, and so on. In other words, inputs for which the model is relatively sensitive would require more detailed characterization, as opposed to inputs for which the model is relatively insensitive.

The effect of interaction between several input parameters is a critical factor to consider in the review and application of results from a sensitivity analysis. The sensitivity of MEPDG model inputs should be assessed in a relative manner; thus, an input parameter that is sensitive for a given design at a given site (climate, subgrade, traffic) may not be sensitive for another design and site situation. The baseline design around which a sensitivity analysis is performed should always be considered. The baseline designs should, therefore, inasmuch as possible represent "typical" designs for the results to hold true for generalization. For meaningful analysis and data interpretation, the baseline designs should produce distresses that are neither too minimal nor too excessive. Actually the level of reliability (or conservatism) of the baseline design has a large effect on the sensitivity of inputs.

This chapter presents the results of a sensitivity analysis conducted for the various pavement types of interest to MoDOT. The main objectives of this task performed under this study were to:

- Assess the robustness and quality of the performance models definitions.
- Determine factors that mostly contribute to the output variability.
- Assess the impact of MoDOT specific site and design inputs on key design types and distress types of interest.
- Establish the region in the space of input factors for which the model variation is maximum.
- Determine key input variables to evaluate during the validation process and consider for empirical adjustments of global calibration models.
- Identify optimal or instability regions within the factor space to be used in calibration.

4.1 SELECTION OF BASELINE DESIGNS FOR NEW HMA PAVEMENT AND NEW JPCP

The main criterion involved in the selection of a baseline design for HMA and JPCP was to incorporate typical and current MoDOT design and construction practices, paving material properties, traffic characteristics and climate. The baseline designs selected also include structure and material properties corresponding to new construction of typical State highways in Missouri. Furthermore, equivalent site conditions (i.e., the same climate, subgrade type, traffic level) and design period were selected for the HMA and JPCP baseline designs.

The inputs to the MEPDG for each baseline case and their respective ranges to conduct the sensitivity analysis were determined from Missouri-specific information—i.e., a general range of values observed in Missouri pavement sections and as reported in relevant documents such as design reports, specifications, and databases. The outputs of interest were key distress types for each pavement type analyzed predicted using nationally calibrated performance (distress) models contained in the MEPDG.

The following MoDOT projects were selected as baseline designs:

HMA baseline design:

- Project ID: DS9-E-S1
- Highway: US-54 Eastbound S1, Camden County (Pavement structure)
- Climate conditions: Camden county climate
- Construction and traffic opening months: October and November respectively

PCC baseline design:

- Project ID - J0P0572
- Highway: US-60 Westbound S1, Butler County (Pavement structure)
- Climate conditions: Camden county climate
- Construction and traffic opening months: October and November respectively

4.2 SENSITIVITY ANALYSES PROCESS

Using the baseline designs, alternate designs were developed by varying the key inputs one at a time. The key inputs that were varied in the sensitivity analyses were selected based on the structural mechanisms modeled in the MEPDG and determined as critical for performance prediction in the development of the MEPDG (ARA 2004). Every effort was made to recognize and factor in the interdependence of some of the inputs as much as possible. For example, simultaneous changes were made to HMA mix types and mix properties, PCC strength and modulus and shrinkage, etc. There were, however, certain limitations in handling all potential interactions between the inputs because the relationships of some of the parameters are very material- or site-specific and are often not well established through sources in literature.

Finally, the results were analyzed to rank order the inputs based on their impact on the models of interest. To achieve this, relative comparisons were made to determine the inputs that most affect performance.

4.3 BASELINE DESIGN AND INPUTS FOR SENSITIVITY ANALYSES

To predict pavement performance, the MEPDG requires the following information/inputs:

- General information.
    - Design life/analysis period.
    - Base/subgrade construction date.
    - Pavement construction date.
    - Traffic opening date.
- Site/project identification (general project location information)
- Analysis parameters.
    - Initial smoothness (International Roughness Index, IRI) value.
    - Terminal distress and smoothness (IRI) values.
- Traffic.
    - Growth factors.
    - Volume adjustment factors.
    - Axle load factors
    - General traffic inputs (axles per truck, axle configuration, etc.).
- Climate.
- Structure.
    - Layer thicknesses.
    - Layer material types and properties.
    - Design features for JPCP.
    - HMA Design Properties for HMA pavements.

Detailed descriptions of all inputs selected for the baseline new HMA pavement and new JPCP are presented in the following sections.

4.3.1 MEPDG Inputs for the Baseline HMA Pavement

The baseline new HMA pavement structure consists of four layers placed over the natural subgrade and is presented in Figure I-107. The pavement layers include:

- HMA surface course (layer 1)
- HMA binder course (layer 2)
- HMA base course (layer 3)
- Dense graded aggregate base (DGAB) material (layer 4)
- Natural subgrade

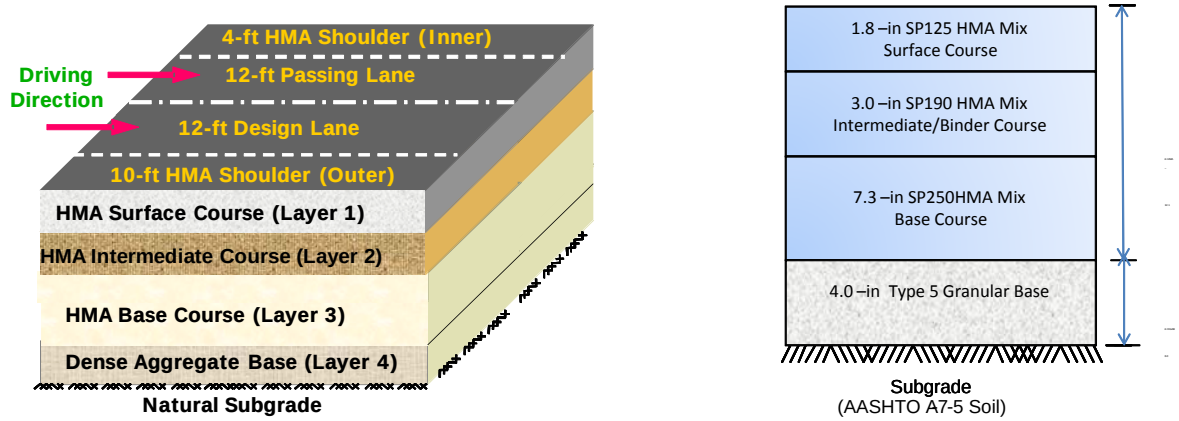


Figure I-107. Pavement structure for the selected HMA baseline design.

Table I-52 presents the various MEPDG input values associated with the baseline HMA design. Table I-53 lists key inputs whose impact on the MEPDG outputs was studied along with the ranges over which these inputs were varied.

The following comments are offered with regard to selecting the range of values for the baseline design and sensitivity analyses. The discussion is presented for various inputs by input category:

General Information

The MEPDG requires the following basic information for new HMA pavement design analysis and performance prediction:

- Design life/analysis period.
- Base/subgrade construction date.
- Pavement construction date.
- Traffic opening date.

A design life of 20 years was selected consistent with PCC design sensitivity analyses to be discussed in the next section of this chapter. MoDOT's paving season typically begins around May and ends around October. The baseline HMA pavement was assumed to be constructed in September/October and opened to traffic in November of the same year.

Site/Project Identification

Site/project location information is used to identify the project under design and for record keeping purposes. Information required is not relevant to sensitivity analysis.

Analysis Parameters

Analysis parameters required by the MEPDG are as follows:

- Initial smoothness characterized using the IRI.
- Terminal distress and IRI values.

Table I-52. MEPDG inputs assumed for the baseline HMA pavement section.

Input Category	Input Variable and Selected/Assumed Input Value
General Information	• Design life/analysis period – 20 yr • Base/subgrade construction date – September 1997 • Pavement construction date – October 1997 • Traffic opening date – November 1997
Site/Project Identification	• Identification information for project and site, which have no impact on predicted performance
Analysis Parameter	• Initial IRI: 63 in/mile • Terminal performance criteria not of significance to sensitivity analyses
Climate	• Climate: Camden, MO (central location)
Traffic	• Traffic: Data from DS9-E-S1 applied. See Volume II for more details. Some key inputs include: o Cumulative trucks on design lane - 19.6 million o 2-way AADTT - 4,000 o Directional distribution - 50 percent. o Lane distribution factor - 95 percent o Growth rate 4.32 percent (linear). o Vehicle distribution: Principal Arterial (74% Class 9). o MEPDG defaults for all other inputs.
Structure Material Properties	• Layer 1 – HMA surface course mix o Thickness – 1.75 -in o Gradation* – 0, 14.8, 46, 5.1 o Superpave binder grade – SP 70-22 o Volumetrics – AC = 11%, Air voids = 7%, Unit weight = 142.7 pcf • Layer 2 – HMA binder course mix o Thickness – 1.75 -in o Gradation* – 0, 19.3, 46.3, 4 o Superpave binder grade – SP 70-22 o Volumetrics – AC = 9%, Air voids = 7%, Unit weight = 144.7 pcf • Layer 3 – HMA base course mix o Thickness – 7.31 -in o Gradation* – 6.3, 36.5, 61, 4.3 o Superpave binder grade – SP 64-22 o Volumetrics – AC = 8%, Air voids = 7%, Unit weight = 144.7 pcf • Layer 4 – DGAB o Thickness – 4.0 -in o Resilient modulus (M_r)=29,500 psi, Plasticity Index = 1, Maximum Dry Density = 127.7 pcf • Layer 5 – Subgrade o Resilient modulus (M_r): 17,500 psi, Plasticity Index = 24, Maximum Dry Density = 102.0 pcf o Soil classification – A-7-5

* Gradation represented as Cumulative percent retained on ¾” sieve, 3/8” sieve, # 4 sieve, percent passing #200 sieve

Table I-53. Ranges of inputs used in the new HMA models sensitivity analysis.

MEPDG Input Parameter	Range of Input values changed one at a time (*indicates the baseline representative design)
Climate (weather stations in MO) (See Figure I-108)	• Camden County* • Cape Girardeau • Columbia • Jefferson City • Joplin • Kansas City 1, Kansas City 2 • Kirksville • Lee's Summit • Poplar Bluff • Rolla • Sedalia • Springfield • St. Charles • St. Joseph • St. Louis1, St. Louis 2 • West Plains
Construction season	• January • April • July • October*
HMA thickness	10 -, 12-*, 14-, 16-in (varying the bituminous base thickness only)
HMA type (surface course)	Superpave HMA Mix Surface Course Binder grading • PG 64-22 • PG 70-22* • PG 76-22
HMA volumetric binder content	Baseline binder content (surface course = 11%, binder course = 9%, and base course = 8%) +1%, +2%, -1%, -2% of the baseline binder content across all three HMA layers.
HMA air voids content	5, 6, 7*, 8, 9%for all three HMA layers (7% air voids used for all three layers in the baseline design)
Base type[#,^]	• LSB with M_r of 60,000 psi • LSB with M_r of 40,000 psi • DGAB* with M_r of 29,500 psi • Gravel with 7% passing #200 sieve (M_r of 29,500 psi) • Gravel with 15% passing #200 sieve (M_r of 29,500 psi)
Subgrade type[^]	• Fine grained (A-6, A-7-5*, A-4) • Coarse grained (A-3, A-1-a)
Depth of water table	• 3 ft* • 10 ft • 20 ft

*Baseline design input values
[#]Base modulus values are based on results typically obtained from the back calculation of FWD test data
[^]Default MEPDG gradations and resilient modulus values will be used, where applicable.

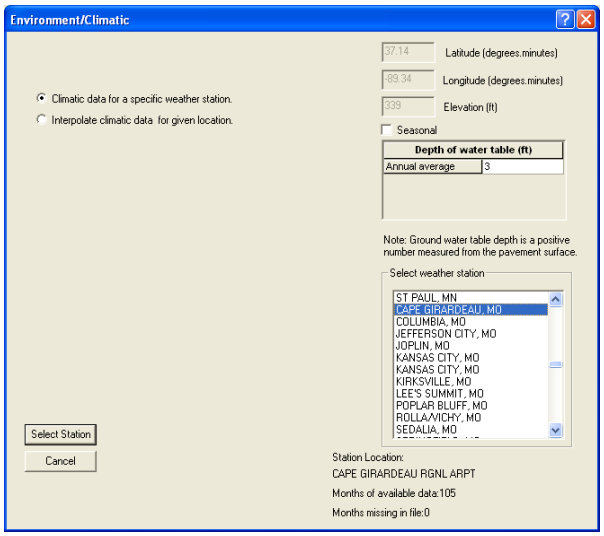


Figure I-108. Geographic location of weather stations selected for the sensitivity study.

For initial IRI, a value of 63 in/mile was used for all the sensitivity runs. On average, the project team has determined that this value is reasonable to assume based on MoDOT specifications.

For the purposes of sensitivity analysis, only the initial IRI at construction is required. Terminal distresses and IRI values are largely inconsequential in a sensitivity analysis and therefore the MEPDG defaults for the various distresses were retained for the analyses. However, terminal or threshold distress and IRI values and associated reliability levels are very important from a design standpoint and for developing policy/guidance on terminal distress/IRI values for used in design. Terminal distress/IRI is typically set according to service category of the roadway, traffic levels, and so on and should always be set keeping in mind the design reliability.

Traffic

Traffic inputs used in the baseline design and sensitivity analyses represent site specific traffic volume data and where necessary MEPDG default values which were established using national average conditions based on LTPP data. It was found during this study that the national traffic defaults represent Missouri conditions and therefore it was reasonable to use MEPDG defaults for the analyses.

However, base year traffic data were adjusted so that a reasonable level of distress can be observed, to adequately demonstrate the sensitivity of the various parameters considered. For example, the original MoDOT section selected as a baseline HMA design had a two-way AADTT of approximately 1,500 trucks. This section, as predicted by the MEPDG, has seen little to no distress during its service life. The AADTT was increased to 4,000 trucks during the sensitivity analyses so that a reasonable distress can be predicted for evaluating the sensitivity of the various input parameters listed in Table I-54. Site WIM data backcast over the duration of data collection showed an average linear traffic growth of 4.3%. This growth rate was maintained in the sensitivity analyses. A plot of the truck traffic growth (from an initial AADTT of 4,000 to an AADTT of 7,440 after 20 years) is presented in Figure I-109.

Figure I-109. Plot showing growth in AADTT for the 20- year design period.

Climate

The MEPDG requires several climate-related inputs for analysis. Specific inputs required include rainfall, temperature, humidity, sunshine/heat radiation, and wind speed. The data required are mostly routinely collected by weather stations located throughout the U.S. For the MEPDG, required climate data are obtained primarily from weather stations located closest to the pavement project under design. To facilitate implementation of the MEPDG, the software contains a default database of climatic data from over approximately 800 weather stations located throughout the U.S. including 16 weather stations in Missouri. Each weather station contains approximately 9 years of climate data.

The baseline design, located in Camden County, utilized a virtual weather station created by interpolating weather data from the six closest stations in the database. The sensitivity study utilized weather data from the other Missouri stations in the MEPDG climatic database. The Missouri weather stations the MEPDG database, shown in Figure I-107, represent all climatic zones and weather patterns of the State. However, data for the St. Charles and St. Joseph locations were not of good quality, so these weather stations were not included in the sensitivity study.

Structure (Definition of Layer Material Types, Properties, and Thicknesses)

As presented in Figure I-107 and Table I-52, the baseline new HMA pavement structure consists of the following layers:

- 1.3-inch HMA surface course SP 125 HMA mix.
- 3.0-inch HMA binder course SP 190 HMA mix.
- 7.3-inch HMA base course SP 250 HMA mix.
- 4.0-inch DGAB Type 5 base material.
- Natural subgrade (AASHTO A-7-5).

Key material properties of the baseline design are summarized in Table I-52, and the range of inputs used are summarized in Table I-53.

4.3.2 MEPDG Inputs for the Baseline New JPCP Design

The baseline new JPCP pavement structure was developed using the same selection criteria and information from the same sources as for the HMA baseline design. The baseline new JPCP structure is presented in Figure I-110. It consists of two layers placed over the natural subgrade:

- 10-in PCC surface layer.
- 6.0-in DGAB.
- Natural subgrade (AASHTO A-2-4).

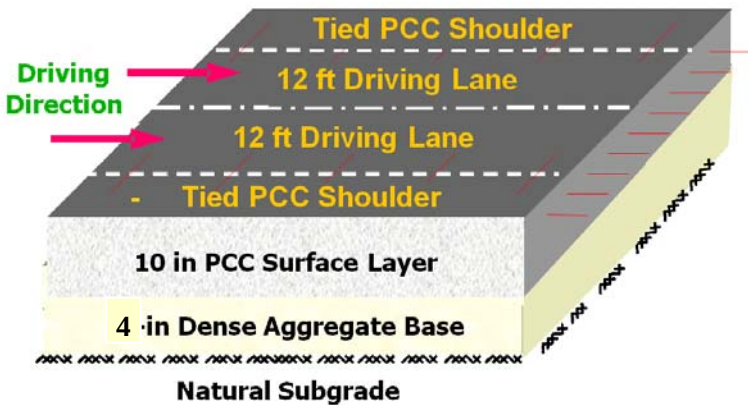


Figure I-110. Structure for the selected JPCP baseline design.

Table I-54 presents the various MEPDG input values associated with the baseline HMA design. Table I-55 lists key inputs whose impact on the MEPDG outputs was studied along with the ranges over which these inputs were varied.

The following comments are offered with regard to selecting the range of values for the baseline design and sensitivity analyses.

General Information

The inputs in this category are similar to those selected for the HMA baseline design and were based on the same selection criteria. A design life of 20 years was selected based on current recommendations for all new pavement designs.

Site/Project Identification

Site/project location information is used to identify the project under design and for record keeping purposes. Information required is not relevant to sensitivity analysis.

Table I-54. MEPDG inputs assumed for the baseline JPCP section.

<table>
<tr><th>Input Category</th><th>Input Variable and Selected/Assumed Input Value</th></tr>
<tr><td>General Information</td><td>• Design life/analysis period – 20 yr
• Pavement construction date – October 1997
• Traffic opening date – November 1997</td></tr>
<tr><td>Site/Project Identification</td><td>• Identification information for project and site, which have no impact on predicted performance</td></tr>
<tr><td>Analysis Parameter</td><td>• Initial IRI: 63 in/mile
• Terminal performance criteria not of significance to sensitivity analyses</td></tr>
<tr><td>Climate</td><td>• Climate: Camden, MO (central location)</td></tr>
<tr><td>Traffic</td><td>• Traffic: Data from DS9-E-S1 applied. See Volume II for more details. Some key inputs include:
o Cumulative trucks on design lane – 65.4 million
o 2-way AADTT - 16,300
o Directional distribution - 50 percent.
o Lane distribution factor - 95 percent
o Growth rate 1.64 percent (linear).
o Vehicle distribution: Principal Arterial (74% Class 9).
o MEPDG defaults for all other inputs.</td></tr>
<tr><td>Structure Material Properties</td><td>• Layer 1 – PCC
o Thickness – 10-in
o Unit weight - 145 pcf
o PCC coefficient of thermal expansion = 5.46 in/in/^{O}F
o Mix design: Type 1 cement, cement content =560 lb/yd^3, Water to cementitious materials ratio (w/c) = 0.397, Coarse aggregate type = limestone,
o Curing type – curing compound
o Strength: Level 1 data
<table>
<tr><th>PCC age</th><th>Epcc, psi</th><th>MR, psi</th></tr>
<tr><td>7 Day</td><td>4172195</td><td>550</td></tr>
<tr><td>14 Day</td><td>4318238</td><td>654</td></tr>
<tr><td>28 Day</td><td>4290195</td><td>626</td></tr>
<tr><td>90 Day</td><td>4757531</td><td>674</td></tr>
<tr><td>20 Year/28 Day</td><td>1.2</td><td>1.2</td></tr>
</table>
• Layer 2 – DGAB
o Thickness – 4.0 -in
o Resilient modulus (M_r)=18,000 psi, Plasticity Index = 1
• Layer 3 – Subgrade
o Resilient modulus (M_r): 10,000 psi, Plasticity Index = 24
o Soil classification – A-7-5</td></tr>
<tr><td>Structure Design Features</td><td>• Joint spacing – 15 feet
• Sealant type – Liquid
• Dowel – 1.5-in diameter dowels @ 12-in spacing
• Shoulder – Tied PCC shoulder with long term LTE of 40%
• Erodibility index of base – Fairly erodible category 4
• Base-PCC Interface – Full friction contact with loss of friction at 240 months</td></tr>
</table>

Table I-55. Ranges of inputs used in the new JPCP models sensitivity analysis.

MEPDG Input Parameter	Range of Input values changed one at a time (*indicates the baseline representative design)
Climate (weather stations in MO) (See Figure I-108)	• Camden County* • Cape Girardeau • Columbia • Jefferson City • Joplin • Kansas City 1, Kansas City 2 • Kirksville • Lee's Summit • Poplar Bluff • Rolla • Sedalia • Springfield • St. Charles • St. Joseph • St. Louis1, St. Louis 2 • West Plains
Construction season	• January • April • July • October*
PCC thickness	8-, 9-, 10-*, 11-, 12-, 13- in
Slab width	12-*, 13-, 14- feet
Joint spacing	12-, 15-*, 18-feet
PCC Mix (see discussion on PCC materials in next section)	• Gradation B* • Gradation B optimized • Gradation D • Gradation D optimized • Gradation F
Shoulder type	• HMA • Tied PCC
Base type#	• LSB with M_r of 60,000 psi • LSB with M_r of 40,000 psi • DGAB* with M_r of 29,500 psi • Gravel with 7% passing #200 sieve (M_r of 29,500 psi) • Gravel with 15% passing #200 sieve (M_r of 29,500 psi)
Subgrade type^	• Fine grained (A-6, A-7-5*, A-4) • Coarse grained (A-3, A-1-a)
Depth of water table	• 3 ft* • 10 ft • 20 ft

*Baseline design input values

#Base modulus values are based on results typically obtained from the back calculation of FWD test data

^Default MEPDG gradations and resilient modulus values will be used, where applicable.

Analysis Parameters

The discussion presented for HMA baseline design holds true for JPCP baseline design as well.

Traffic

The discussion presented for HMA baseline design holds true for JPCP baseline design as well. The two-way AADTT and growth factor inputs were 16,300 trucks and 1.6%, respectively. The growth factor was backcalculated from WIM data for that section, and the AADTT input selected was that required to produce reasonable levels of distress in the pavement so as to evaluate the sensitivity of the models to various input parameters. The total number of cumulative truck traffic over the 20 years with the projected growth is 65.4 million trucks. This growth rate was maintained in all cases for the sensitivity analyses. A plot of the truck traffic growth (from an initial AADTT of 16,300 to an AADTT of 21,516 after 20 years) is presented in Figure I-111.

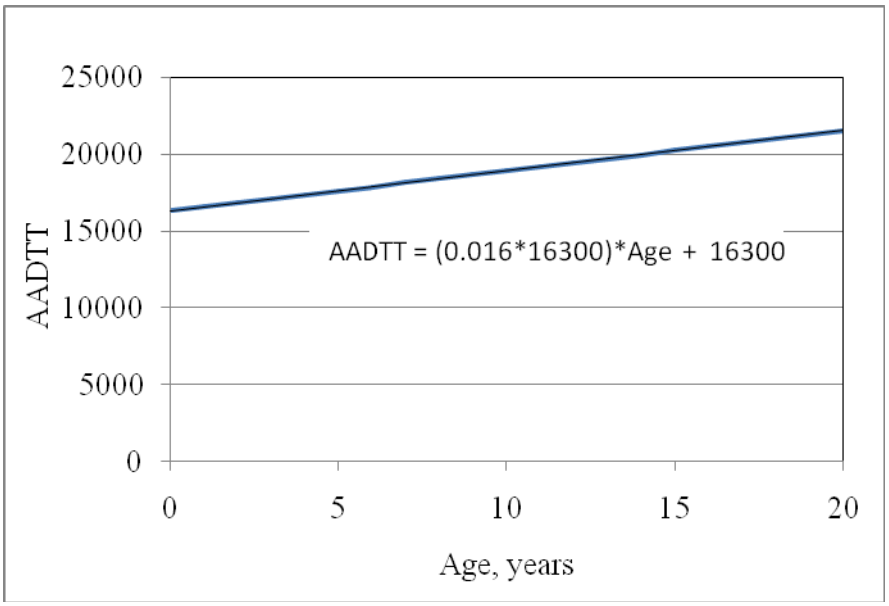


Figure I-111. Plot showing growth in AADTT for the 20- year design period of JPCP baseline design.

Climate

Climate inputs were the same as for baseline HMA pavement and HMA sensitivity analyses.

Structure (Definition of Layer Material Types, Properties, and Thicknesses)

Inputs for the structure include those corresponding to the different layers as well as those corresponding to the design features of the JPCP. As presented in Figure I-110 and Table I-54, the baseline JPCP structure consists of the following layers:

- 10-inch PCC surface layer.
- 4.0-inch DGAB Type 5 base material.
- Natural subgrade (AASHTO A-7-5).

Key material properties and design features of the baseline design are summarized in Table I-54, and the range of inputs used is summarized in Table I-55. Details of the PCC layer properties are discussed in this section, as they were not provided in Table I-55.

The MEPDG requires the following PCC inputs for analysis. Depending on data availability, they can be configured at Level 1, 2, or 3:

- General properties:
    - Unit weight.
    - Poisson's ratio.
- Thermal properties:
    - Coefficient of thermal expansion.
    - Thermal conductivity.
    - Heat capacity.
- Mix properties:
    - Cement type.
    - Cementitious material content.
    - Water to cementitious material ratio.
    - Reversible shrinkage.
    - Time to develop 50 percent of ultimate shrinkage.
- Strength:
    - Flexural strength (time function).
    - Elastic modulus (time function).

MEPDG Level 1 inputs were obtained for typical MoDOT mixes from laboratory testing. Mix design and test results were provided for the following standard MoDOT gradations and associated projects.

- Gradation B used on
    - I-44 in Laclede County
    - US 412 in Dunklin County
- Gradation B Optimized used on I-435 in Jackson County
- Gradation D used on MO 367 in St. Louis County
- Gradation D Optimized used on US 63 in Randolph County and Macon County
- Gradation F used on I-35 in Clinton County

For the sensitivity analyses, PCC mixes were characterized by the gradation. The associated mix and strength properties for each gradation were used as inputs to the MEPDG for analysis purposes. For gradation B, the average values from the two projects listed above were used. Table I-56 presents the mix design inputs to the MEPDG analyses and also includes the PCC CTE values. Table I-57 tabulates the strength results from laboratory tests at different ages as required for MEPDG analyses.

Table I-56. PCC mix design inputs and PCC CTE used in MEPDG sensitivity analyses.

Mix Gradation ID	Project and Location	Cementitious Material (lb/yd^3)	Water (lb/yd^3)	w/cm ratio	CTE in/in/^{O}F
Gradation B	*I-44 in Laclede County*	*564*	*215*	*0.381*	-
Gradation B	*US 412 in Dunklin County*	*556*	*229*	*0.412*	-
Average Gradation B (used in MEPDG analyses for Gradation B)		560	222	0.397	5.40
Gradation B Opt	I-435 in Jackson County	600	258	0.430	5.06
Gradation D	MO 367 in St. Louis County	551	215	0.390	4.90
Gradation D Opt	US 63 in Randolph / Macon County	540	212	0.393	4.25
Gradation F	I-35 in Clinton County	608	231	0.380	4.75

Several other inputs used in PCC Materials and Design Features categories were those recommended as defaults in the MEPDG. These input values were also same as those used during the global calibration process. The following is a list of default values that were also used in the sensitivity analyses:

- PCC Poisson's ratio: 0.20
- PCC thermal conductivity: 1.25 BTU/ hr-ft/°F
- PCC heat capacity 0.28 bTU/lb/°F
- Reversible shrinkage (percent of ultimate shrinkage): 50 percent
- Time to develop 50 percent of ultimate shrinkage: 35 days
- Permanent curl/warp effective temperature gradient: -10°F

Table I-57. Strength properties for PCC mixtures used in sensitivity analyses (laboratory test results).

Mix ID (County)	Age at testing, days	Compressive Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio	Flexural strength, psi
Gradation B (Dunklin county)	*3*	*2934*	*4090658*	*0.16*	*414*
	7	*3537*	*4479212*	*0.15*	*498*
	14	*3768*	*4707686*	*0.15*	*587*
	28	*4348*	*5042242*	*0.14*	*619*
	90	*4904*	*5224673*	*0.20*	*656*
Gradation B (Laclede county)	*3*	*3855*	*3460886*	*0.15*	*540*
	7	*4464*	*3865177*	*0.24*	*602*
	14	*5012*	*4058606*	*0.19*	*721*
	28	*5456*	*3538148*	*0.21*	*633*
	90	*5938*	*4290389*	*0.22*	*692*
Gradation B average (used in MEPDG analyses)	3	3395	3775772	0.16	477
	7	4001	4172195	0.19	550
	14	4390	4383146	0.17	654
	28	4902	4290195	0.17	626
	90	5421	4757531	0.21	674
Gradation B optimized (Jackson county)	3	3472	3729516	0.21	564
	7	3936	3972548	0.20	634
	14	4474	4161557	0.20	652
	28	4857	4266236	0.21	718
	90	5667	4632843	0.23	664
Gradation D (St. Louis county)	3	3756	3835707	0.22	587
	7	4472	4291245	0.21	595
	14	4848	4271614	0.23	640
	28	5082	4452082	0.18	655
	90	5852	4974852	0.22	664
Gradation D optimized (Randolph county)	3	3884	4049615	0.09	540
	7	4382	4239712	0.24	583
	14	4810	4347735	0.21	927
	28	5120	4958388	0.17	744
	90	5970	4785520	0.22	699
Gradation F (Clinton county)	3	3243	3348184	0.22	566
	7	3847	2516396	0.20	654
	14	4502	4101783	0.26	739
	28	4886	4320960	0.26	772
	90	5643	4635612	0.24	620

4.4 SENSITIVITY STUDY

4.4.1 HMA Sensitivity Study

As stated earlier, the sensitivities of the following HMA models were examined:

- Alligator bottom up fatigue cracking.
- Rutting (total at surface).
- Thermal cracking.
- IRI.

Figure I-112 through Figure I-115 present a summary of the sensitivity of the various HMA performance models to the selected inputs discussed in Table I-53. Table I-58 summarizes the impact of each of the inputs on the MEPDG predicted HMA pavement distress and smoothness outputs in terms of:

- Magnitude of impact
    - None—negligible variance in measured output when compared to baseline over the entire range of input considered.
    - Low— 1 to 5 percent variance in measured output when compared to baseline over the entire range of input considered.
    - Moderate—5 to 20 percent variance in measured output when compared to baseline over the entire range of input considered.
    - High—greater than 20 percent variance in measured output when compared to baseline over the entire range of input considered.
- Directionality of impact – Direct proportionality or inverse proportionality represented by symbols (↑) and (↓), respectively.

Based on the information presented in the table and four figures above, the following conclusions can be drawn regarding the sensitivity of each distress prediction model:

- Alligator Cracking
    - The model predictions show that alligator cracking is significantly affected by HMA thickness, air voids, as well as asphalt binder type and content. Higher thicknesses, higher asphalt contents, and lower air voids lead to lower alligator cracking and vice versa. Higher PG binder grade shows lesser cracking.
      Note 1: HMA binder contents and air voids are interlinked in the real world to some degree. Therefore the overall impact may be lower than considering each of these parameters independently. Likewise, owing to these interaction effects, the significance of one variable might be lower if the other is changed. That is, for mixes with a higher binder content, the impact of changing air voids could be less significant.
      Note 2: The effect of binder PG grade could vary for a different climate location. For example, the effect of changing the binder grade from PG 70-22 to PG 76-22 might show relatively a larger benefit in a warmer climate than the baseline case shown. Also, changing the grade to PG 64-22 might show a smaller increase in alligator cracking in a colder climate.
    - Base type (DGAB versus large stone base) also shows significant impact with the stiffer bases producing lower alligator cracking. This is as expected.

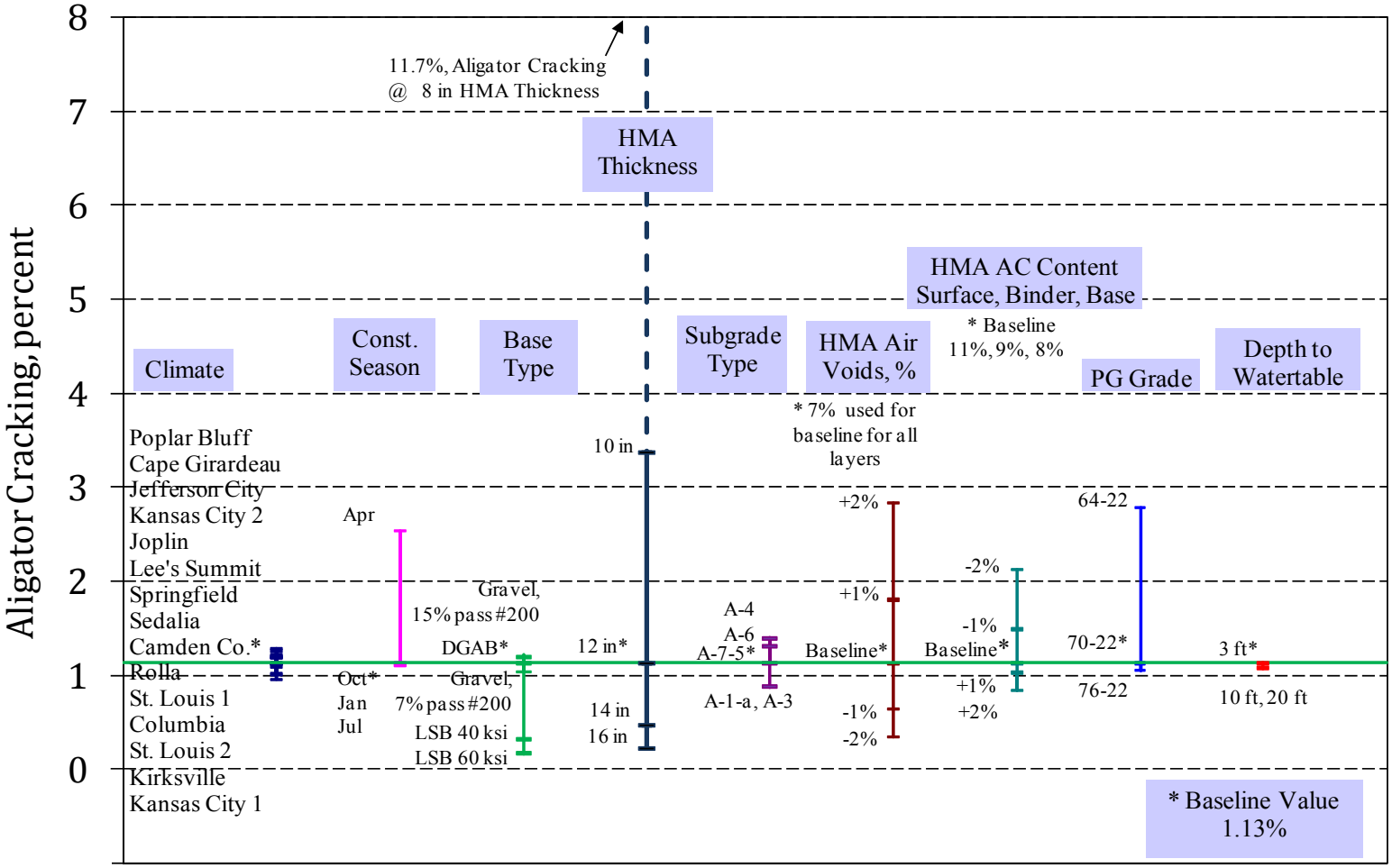


Figure I-112. Summary of HMA alligator cracking sensitivity to the various site, materials, and structure-related inputs.

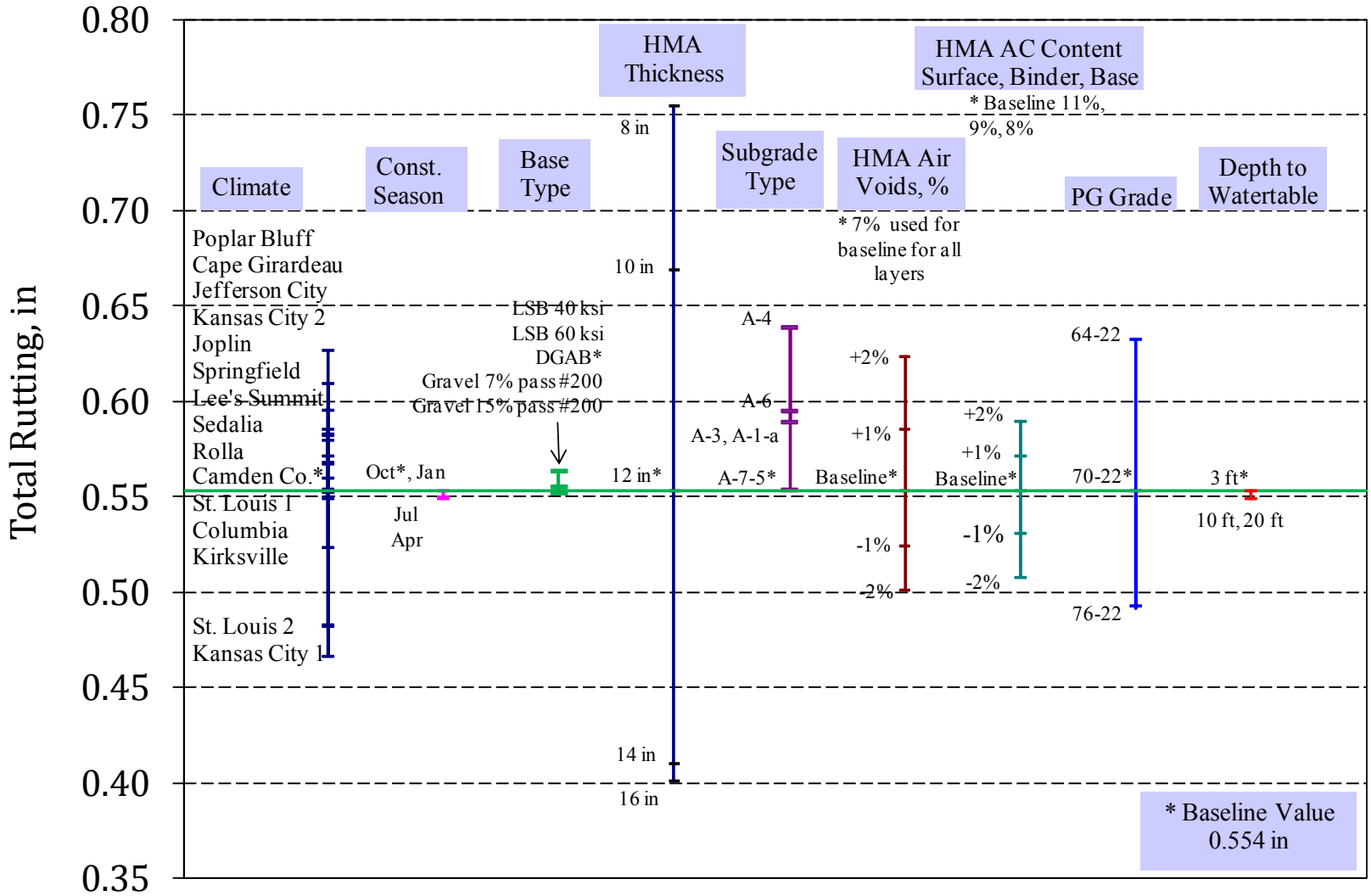


Figure I-113. Summary of total rutting sensitivity to the various site, materials, and structure-related inputs.

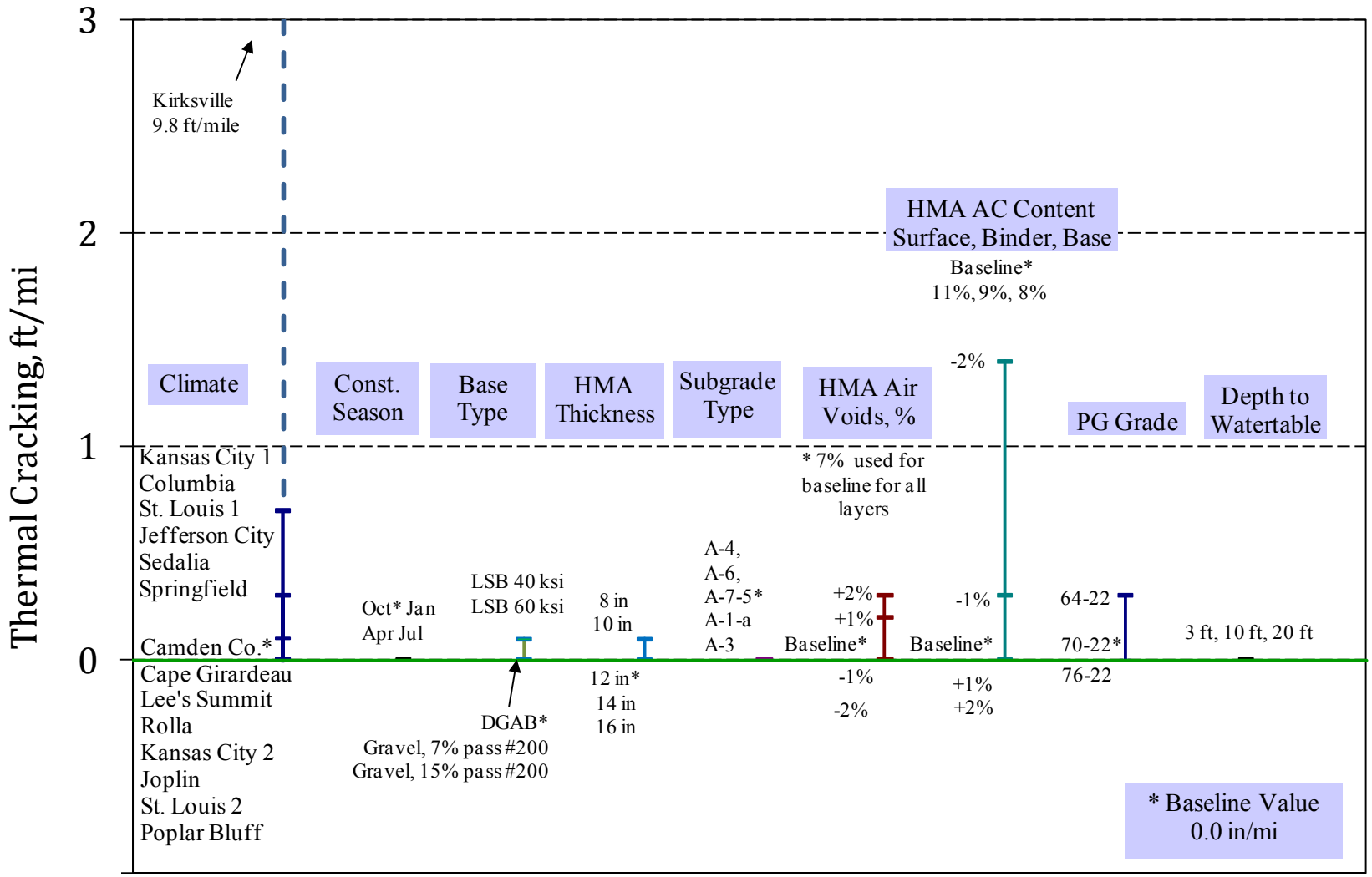


Figure I-114. Summary of HMA thermal cracking sensitivity to the various site, materials, and structure-related inputs.

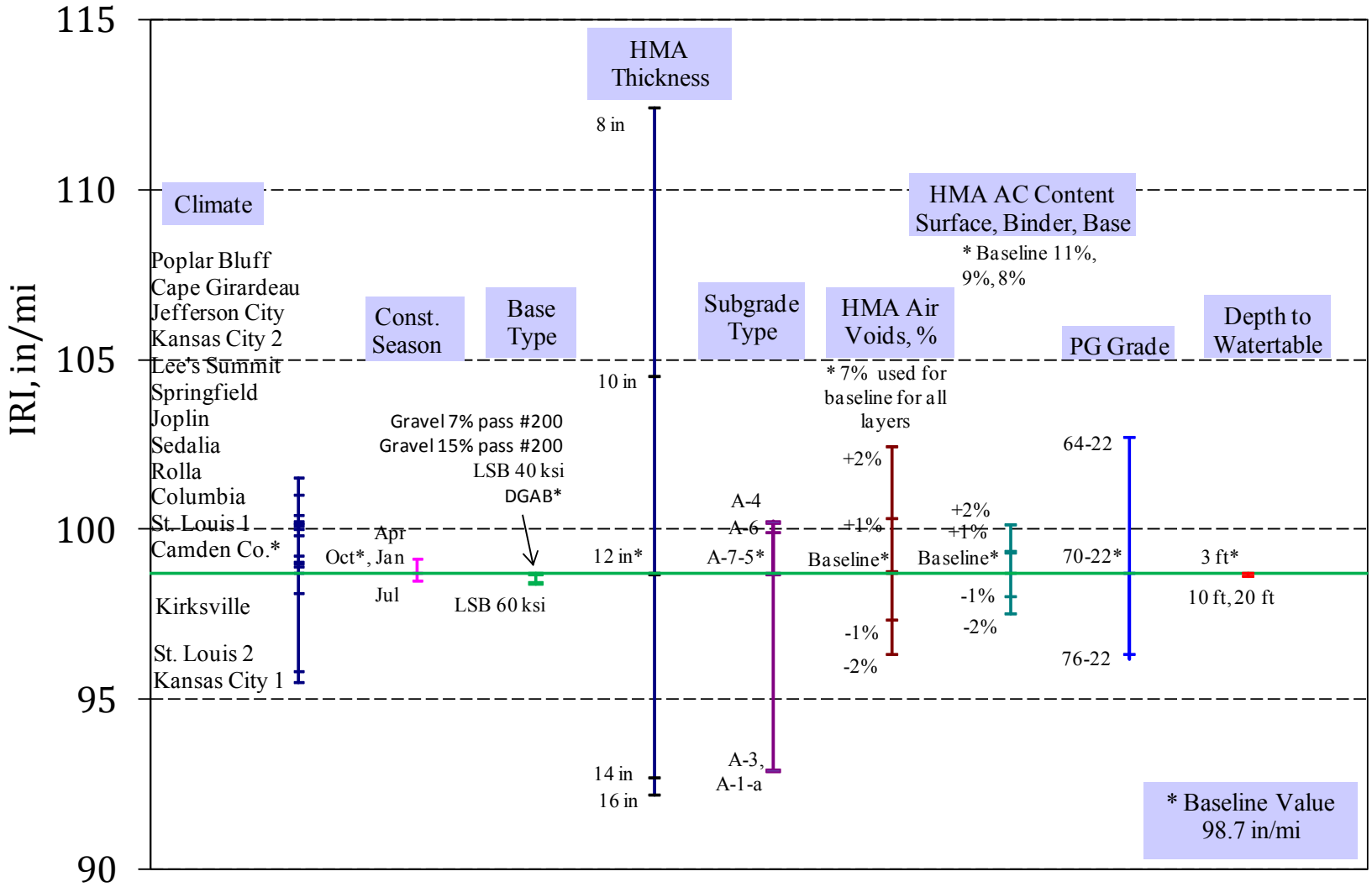


Figure I-115. Summary of HMA IRI sensitivity to the various site, materials, and structure-related inputs.

Table I-58. Summary of the relative sensitivity of various MoDOT-specific design inputs on MEPDG HMA pavement distress and smoothness predictions.

Input Variable	HMA Distress/Smoothness Model			
	Alligator Fatigue Cracking	**Rutting**	**Thermal (Transverse) Cracking**	**IRI**
Construction season	High[1] ↑	None	None	Low[1]↑
HMA thickness	High[2] ↓	High[2] ↓	None	High[2] ↓
HMA air voids content	High[2] ↑	High[2] ↑	None-Low	Low[2] ↑
HMA volumetric binder content	High[2] ↓	High[2] ↑	Low[2]↓	Low↑
HMA binder type (higher PG grade)	High[2] ↓	High[2] ↓	Low[2]↑	High ↓
Base type (Base Modulus)	High ↓	Low ↓	None	Low ↓
Natural Subgrade type/modulus	Moderate ↓	High ↓	None	High ↓
Climate (warm temperatures)	Low↑	High ↑	Low↓	Moderate ↑
Depth of water table	Low ↓	Low ↓	None	None-Low↓

1. Sensitive to subgrade moisture state and distress increases for construction in spring.
2. The impact of increasing the input value is not same as the impact of decreasing the value of the given input, i.e., the change in performance when the input is increased is not proportional to the change in performance when the value is decreased. For example, the increase in rutting by decreasing HMA thickness by 2 inches is more than the decrease in rutting by increasing thickness by 2 inches. Likewise, increasing air void content by 2% is more detrimental the pavement from the standpoint of alligator cracking than the benefit derived from reducing air voids by 2%.

  - The construction season also appears to have a significant effect on performance, particularly the construction in spring time when the subgrade and unbound layer moisture contents are higher.
  - Base type (DGAB versus asphalt treated) also shows noticeable impact, with the stiffer bases such as the 50-50 recycled mixes producing lower alligator cracking. This is as expected.
  - There could be considerable variation in the predicted alligator cracking from one traffic site to the other highlighting the importance of accurately characterizing traffic data (site to site variations primarily include volume variations).
  - The remainder of the variables such as subgrade type, climate, pavement construction month, and depth of water table have low to no impact on the predicted alligator cracking. This appears to be reasonable considering the structural thickness of the baseline design.
- Total Rutting (includes HMA layer, base, and subgrade rutting)
  - Contrary to expectations, HMA thickness has the highest impact on rutting. However, as will be discussed later in the calibration section of the report, the rutting model requires a recalibration to remove thickness bias. Post local calibration, the impact of thickness is not expected to be higher.
  - Air voids, asphalt binder grade, and binder content have a significant impact on the predicted rutting. Higher binder contents and higher air voids result in higher

rutting, as expected. However, since binder contents and air voids tend to go in different directions in the "real world" their true impact may be over exaggerated for a good mix. Additionally, a binder grade resulting in a softer mix results in higher rutting.
    - Climatic variations in Missouri also show a significant impact on the predicted rutting. The trends, supported by climatic data indicating high temperatures over consecutive months show that higher the average summer temperatures, greater the predicted rutting. The interaction of climate and binder properties is important to consider in rutting predictions.
    - Stiffer subgrades, and to some extent stiffer base layers, resulted in a lower predicted rutting. Therefore, material stiffness as a whole can be considered to have a moderate-high impact on the total rutting predicted.
    - The depth to water table has no impact on total rutting for the conditions evaluated.
- Transverse Cracking
    - The predictions show that transverse cracking is highly affected by climate, asphalt binder content, and binder type. Relatively higher transverse cracking was predicted for the Kirksville location. In general, colder temperatures as determined from climate records for the Kirksville location result in higher transverse thermal cracking if the PG grade is held constant.
    - The predictions indicate that the air voids have minimal impact on transverse cracking. However, this observation is not true. The MEPDG Level 3 estimates indicate more compliance of HMA mixtures than actual laboratory measurements, which results in under-estimation of transverse cracking. So it is essential to use Level 1 data for HMA mixtures.
    - All other factors studied have negligible to low impact.
- Ride Quality or IRI
    - The impact of the various site and design factors studied on IRI prediction can, for the most part, be estimated by examining how they impact alligator cracking and rutting. For example, if an increasing value of given factor increases alligator cracking and rutting, it can be expected to increase IRI by the same magnitude. On the other hand, if it has equal but opposite effect on alligator cracking and rutting, it can be expected to have a minimal impact on IRI.
    - It is observed that pavement thickness has the most significant effect on IRI with thicker pavements exhibiting lower IRI.
    - Climate, subgrade type, air voids, and PG Grade of the binder have reasonable impact on IRI with stiffer and more stable layers being more beneficial to minimize IRI.
    - The remaining factors studied have a low impact on IRI.
    - Initial IRI has a linear effect on the terminal IRI. The value to the terminal IRI depends on the value of the initial IRI. It is vital to build smoother pavements (lower initial IRI) to achieve lower terminal IRI.

Based on the observations noted, the most significant inputs that influence the predicted distresses in HMA pavements include:

- HMA thickness.
- HMA binder and air void content (and stiffness or E*).
- HMA binder grade.
- Climate.
- Stiffness of base and subgrade materials.
- Initial IRI

4.4.2 JPCP Models Sensitivity Analysis

The sensitivity study was performed to determine how various critical inputs affect the distress models:

- Transverse "fatigue" cracking.
- Transverse joint faulting.
- IRI or ride quality.

Figure I-116 through Figure I-118 present a summary of the sensitivity of the various JPCP performance models to the selected inputs discussed in Table I-55. Table I-59 summarizes the impact of each of the inputs on the MEPDG predicted distresses based on the same criteria and format shown in Table I-58 for HMA pavements.

Based on the information presented, the following conclusions can be drawn regarding the sensitivity of each distress prediction model:

- Mid-slab or Fatigue Cracking
    - As expected, slab design features such as thickness, transverse joint spacing, and slab width all have a major impact on the predicted amounts of transverse cracking. There appears to be a counter-intuitive trend with slab width values at 13 ft and 14 ft—the 13-ft-wide slab shows higher cracking than the 14-ft slab. This occurs due to the wheel load location with respect to the longitudinal edge of the slab, which typically increases with larger slab width values. With a 14-ft slab, the critical edge distance shifts from the outer edge to the inner edge of the slab which results in a slightly smaller edge distance with a 14-ft slab compared to a 13-ft slab. Moreover, the load transfer efficiency (LTE) across the inner longitudinal edge of the slab is set at a higher level (70%) than across the lane-shoulder joint.
    - Tied shoulders have a large impact, as can be noted by comparing the cracking for a 12-ft slab with that of a slab with HMA shoulder.

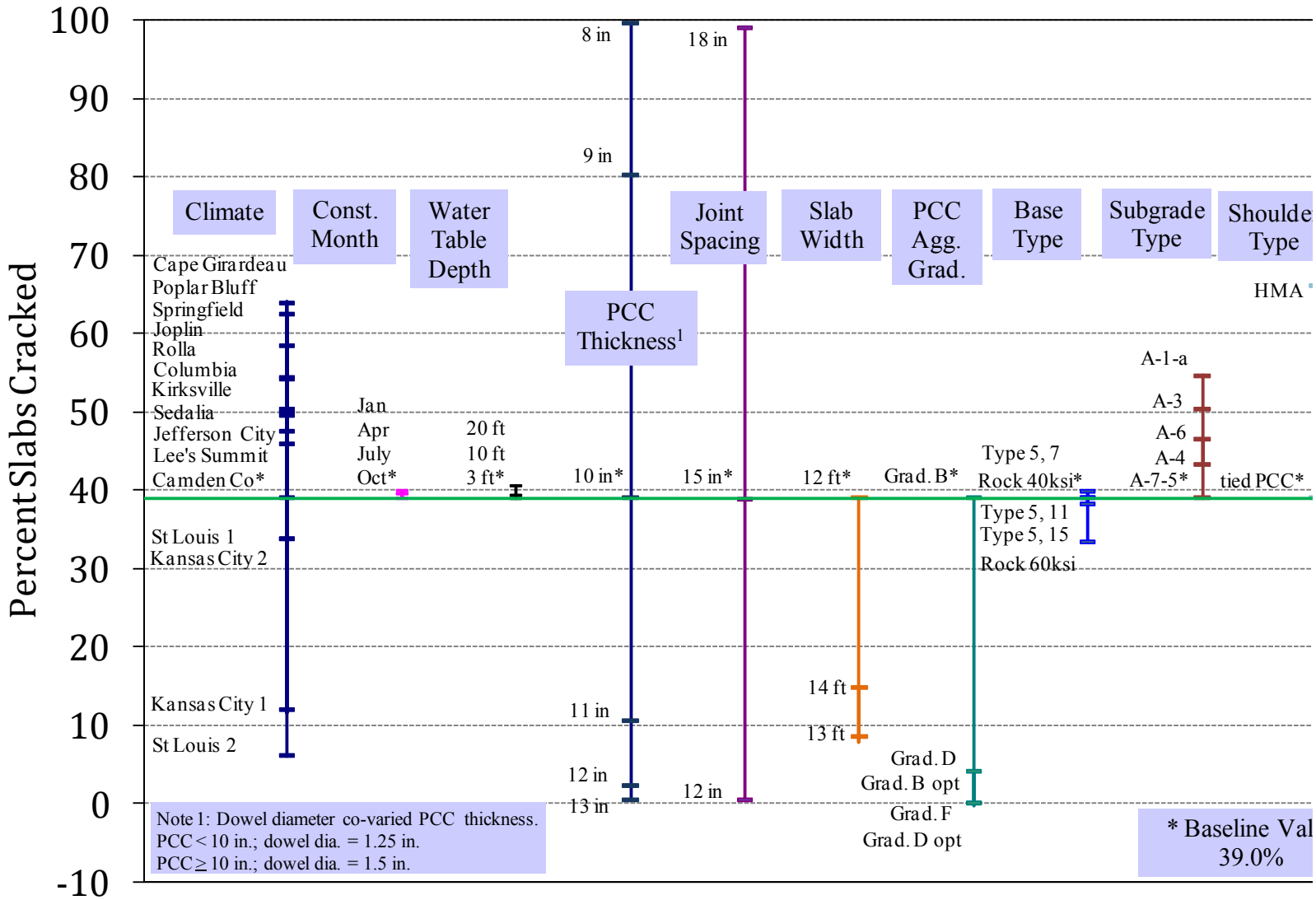


Figure I-116. Summary of JPCP fatigue (transverse) cracking sensitivity to the various site, materials, and structure-related inputs.

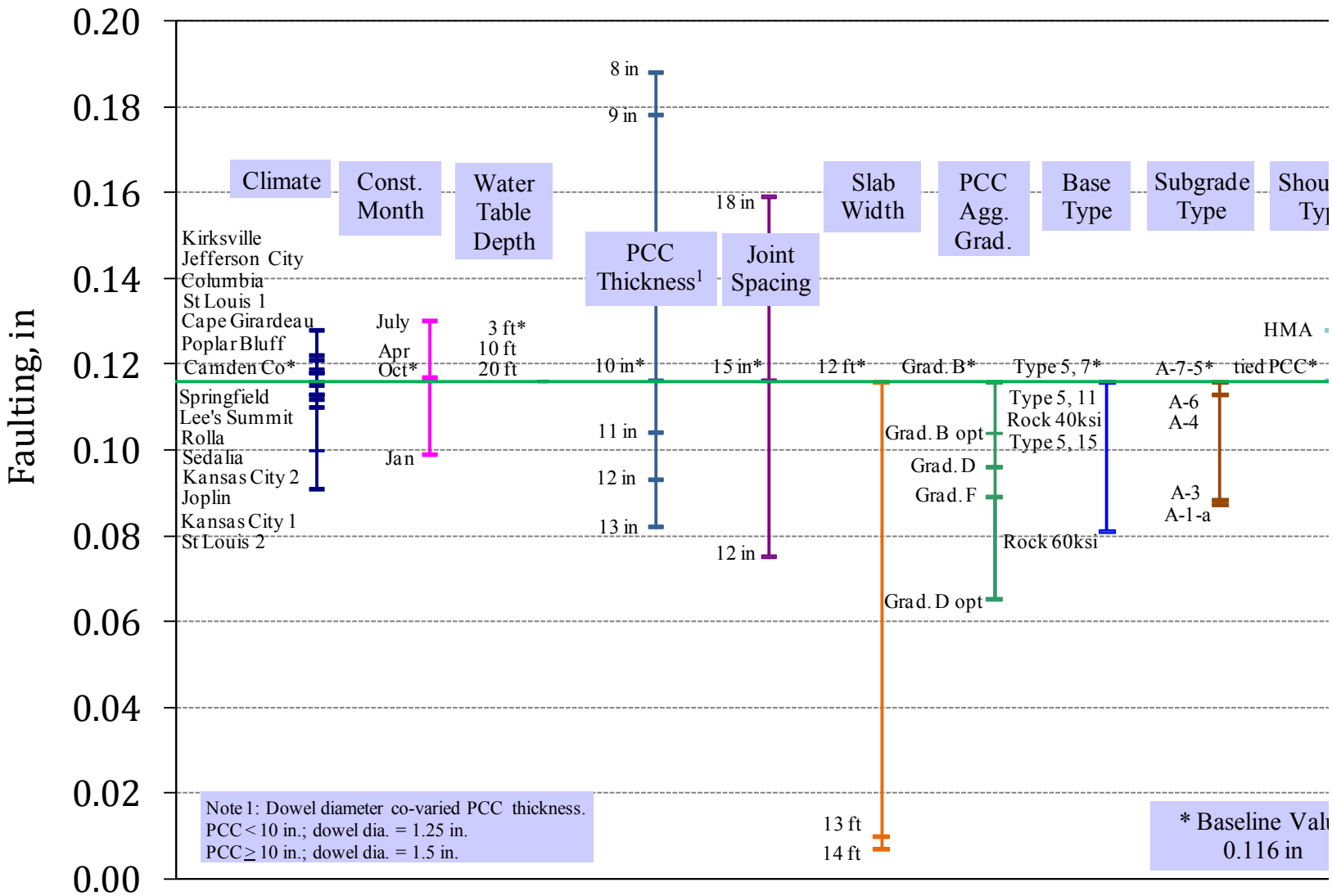


Figure I-117. Summary of JPCP mean joint faulting to the various site, materials, and structure-related inputs.

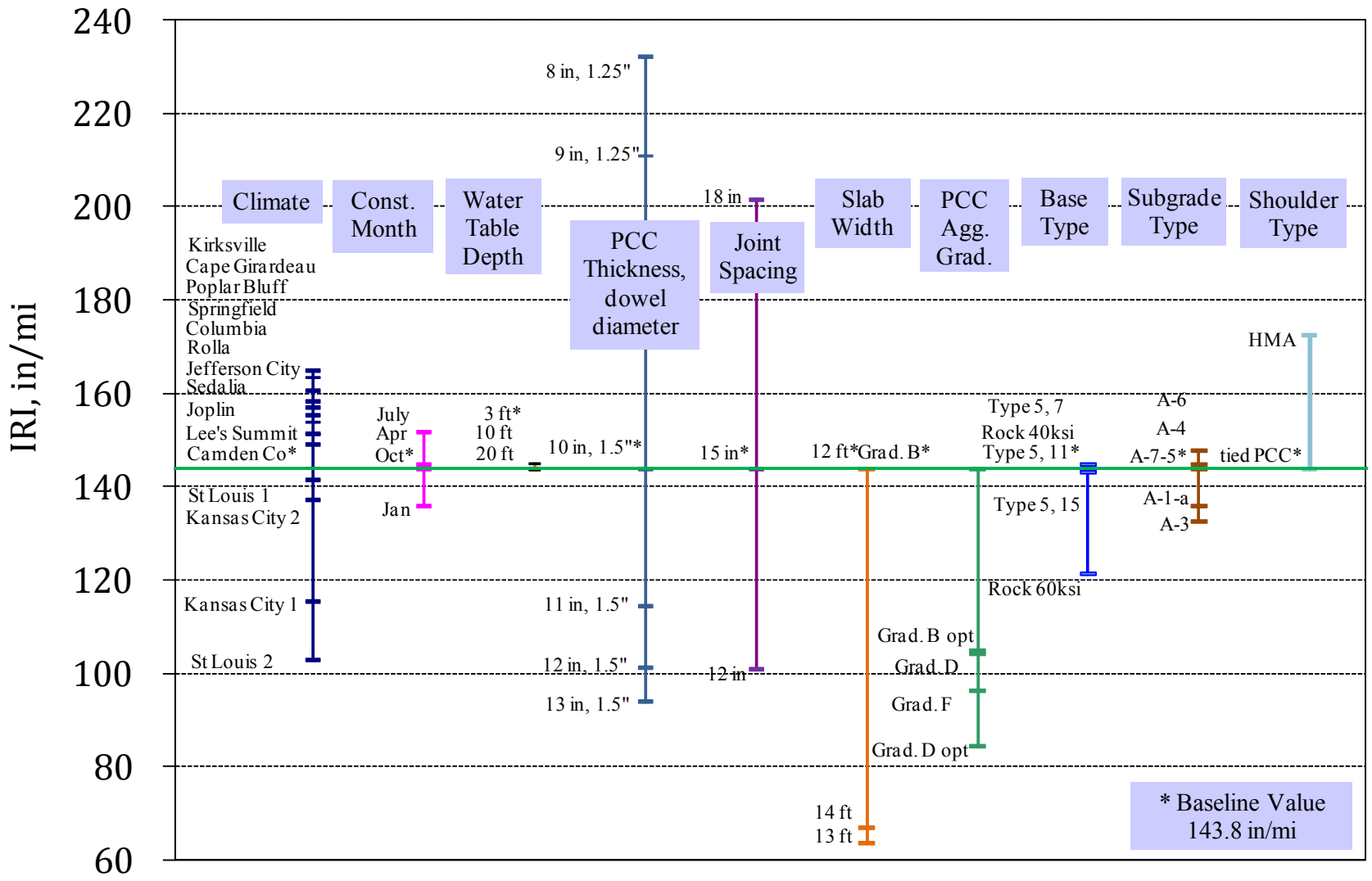


Figure I-118. Summary of JPCP IRI to the various site, materials, and structure-related inputs.

Table I-59. Summary of the relative sensitivity of various MoDOT-specific design inputs on MEPDG JPCP distress and smoothness predictions.

Input Variable	HMA Distress/Smoothness Model		
	Transverse fatigue cracking	**Joint faulting**	**IRI**
Construction season	None	High[1]	Low[1]
Climate (warm temperatures)	High[2]	High ↓	Moderate[3]
PCC thickness	High[4] ↓	High[4] ↓	High[4] ↓
Joint spacing	High[4] ↑	High[4] ↑	High[4] ↑
Slab width	High[4] ↓	High[4] ↓	High[4] ↓
PCC gradation and mix	High[5]	High[5]	High[5]
Shoulder type	High[6]	High[6]	High[6]
Base type (Base Modulus)	Moderate ↓	High ↓	High ↓
Natural Subgrade type /modulus	High ↑	High ↓	Moderate ↓
Depth of water table	Low	Low ↓	None-Low↓

1. Sensitive to temperature at time of construction. Construction in warm temperatures causes higher level of distress.
2. Trend is dependent on data in specific climate files and the quality of data.
3. The trend depends on relative increase or decrease of faulting and transverse cracking.
4. The impact of increasing the input value is not same as the impact of decreasing the value of the given input, i.e., the change in performance when the input is increased is not proportional to the change in performance when the value is decreased.
5. Distress increases for HMA shoulder

Among material properties, CTE or aggregate type and PCC strength when treated individually have shown significant impact on the predicted cracking in other studies, as presented in

- Table I-60. However, when the interaction of strength, CTE, cementitious materials content, etc. are considered together (as embodied in the PCC mixes considered in the sensitivity study), the impact of the individual material parameters appear to balance each other out to some degree. Mix B, with a high CTE and relatively lower flexural strength, shows the highest level of cracking. The opposite holds true for Mix F. Interestingly, the combined effects of CTE and flexural strength are apparent in Mix B optimized and Mix D, both of which have very similar levels of cracking. Although Mix D has lower flexural strength, the low CTE value is beneficial to result in distress levels equivalent to Mix B optimized.

Table I-60. Summary of PCC mix properties on JPCP transverse cracking predictions.

•

Gradation	CTE in/in/^{O}F	Elastic Modulus, psi (28 days)	Flexural Strength, psi (28 days)	Percent Slab Cracking
B average (used in MEPDG analyses)	5.4	4,290,195	626	39.0
B optimized (Jackson county)	5.06	4,266,236	718	0.1
D (St. Louis county)	4.9	4,452,082	655	4
D optimized (Randolph county)	4.25	4,958,388	744	0
F (Clinton county)	4.75	4,320,960	772	0

- Along expected lines, the strength of the base course shows moderate impact and the subgrade type shows a relatively larger impact on slab cracking. Stiffer subgrades result in larger cracking due to increased subgrade k-values that result in higher curling stresses in the slab. However, higher modulus base materials reduce cracking as the base contributes to the effective slab thickness in the analyses process.
- Missouri climate locations used in the sensitivity study appear to produce a reasonable effect on slab cracking. Typically, these trends are attributed to data in specific climatic locations. For example, extreme weather conditions recorded within the available data can influence results more dramatically than average conditions would. Therefore, the sensitivity results could be significantly affected by the quality of the climatic data stored. The accuracy of these predictions can be improved by the addition of more climatic data in the weather station files and checking the accuracy of the existing climate data.
- Depth of water table and construction month did not appear to have a large impact on cracking. However, in practical cases, it might have a larger effect on field due to changes in built-in effective curl/warp conditions during different months and periods of paving.

- Joint Faulting
  - When dowel diameter is considered on its own, it has a large impact on joint faulting for a given thickness. Likewise, slab thickness when considered alone also shows a large impact on faulting. When dowel diameter and PCC thickness are co-varied, as considered in the current sensitivity analyses, the combined impact is not as significant but is still considerable. However, the large effect of this combined parameter can mostly be attributed to the dowel diameter alone.
  - Short jointed or widened slab pavements seem to have a significantly lower faulting when compared to long jointed slabs or standard width pavements, respectively. This is along expected lines as the joint opening or closing and slab deflections reduce in these cases.
  - When CTE is varied over the chosen range for a given mix, it has a significant impact on the predicted faulting with higher CTE concrete producing higher faulting. However, when all mix properties are taken together, the impact of CTE, still shows to control faulting predictions. Faulting trends appear to match trends in CTE values and there is little effect of PCC strength.
  - PCC strength (through its correlative impact on modulus), by itself has a small impact on joint faulting.
  - Construction month has a significant impact suggesting that paving in warmer temperatures results in higher faulting and vice-versa. This is mainly due to higher PCC set temperature causing a larger contraction in the PCC slab as the temperature drops causing larger joint openings and reduced load transfer across the joint. This condition accelerates faulting development.
  - Base type and subgrade type have a reasonable impact on the predicted faulting. Higher amount of fines, which increases erosion under the slab, is the most critical factor that controls this distress.

    - Climate also shows a reasonable impact on faulting. However, the effect of climate is best demonstrated when it is considered in combination with the subgrade type. Subgrades with higher fines in sections located in wet climates result in higher faulting than coarse grained subgrades located in dry climate zones.
    - Shoulder type appears to have a moderate impact on faulting. Slabs with tied shoulder undergo lower deflections under traffic loads resulting in lesser faulting over its design life.
    - Depth to water table shows no effect on joint faulting.
- Ride Quality or IRI
    - Not surprisingly, the impact of the various site and design factors studied on IRI prediction can, for the most part, be estimated by examining how they impact joint faulting, and, to some extent, fatigue cracking.
    - PCC thickness and dowel diameter, PCC properties, CTE, slab width, transverse joint spacing, base type, and edge support are all key parameters to reduce IRI.
    - Climatic location has a fairly significant effect, mostly because of its influence on slab cracking.
    - Subgrade type and construction month have moderate impact on IRI.
    - Water table depth has no impact on IRI.
    - Initial IRI has large effect on predicted IRI.

Based on the information presented, the most significant inputs for JPCP design which should be estimated with care are the following:

- PCC slab thickness in combination with joint load transfer (dowels and dowel diameter).
- Joint spacing.
- PCC mix properties including CTE, strength, and modulus (there is a need to test for these properties from a single batch of mix).
- Lane edge support (widened slab).
- Base type and subgrade type.
- Month of construction.
- Climate.
- Initial IRI.

CHAPTER 5. SUMMARY OF LOCAL VALIDATON AND CALIBRATION STUDIES

5.1 BACKGROUND

The validity and practicality of the design methodology proposed in the MEPDG is centered on its distress and smoothness prediction models. The prediction models in the MEPDG were validated using Missouri data from non-LTPP and LTPP projects. For non-LTPP and LTPP projects, a project database containing pavement design, materials, climate, traffic, and performance data from several flexible, composite, and rigid pavement projects across Missouri was assembled. Although several pavement types were of interest to Missouri, the model validation effort was limited to the following:

- New or reconstructed HMA pavements.
- New or reconstructed JPCP.
- HMA overlaid existing HMA pavements.
- HMA overlaid intact PCC pavement.
- HMA overlaid fractured (rubblized) PCC pavements.
- Unbonded JPCP overlays of existing JPCP.

The following performance indicators and thus, MEPDG prediction models, were investigated in this project:

- HMA surfaced pavements.
    - Total rutting.
    - Load-related alligator cracking, bottom initiated cracks.
    - Transverse "thermal" cracking.
    - Smoothness (measured as International Roughness Index [IRI]).
- JPCP
    - Mean transverse joint faulting.
    - Load-related transverse slab cracking (includes both bottom and surface initiated fatigue cracks).
    - Smoothness (IRI).

These performance indicators/models are the predominant structural and functional distresses that occur on the selected pavement types in the State and are the basis for triggering maintenance and rehabilitation. Other models such as longitudinal (top down) cracking of HMA pavements and reflection cracking of HMA overlays, although of interest to MoDOT, were not investigated since they are subjects of active national research at this point in time.

Note that a detailed description of these models has been presented in Volume II of this report.

5.2 DATA ASSEMBLY

The project team, working with MoDOT engineers, assembled a list of potential pavement projects in Missouri for inclusion in the project database. The criteria for selecting projects for inclusion into the project database included:

- Data availability (at least 2 to 3 years of some performance data for at least one distress type of interest must be available).
- Project age (project must be old enough to have adequate amounts of distress/IRI data and young enough to reflect current MoDOT pavement design and construction philosophy).

The identified projects are presented in Table I-61 and Table I-62. Data collection and assembly included but was not limited to the following:

- Non-LTPP new HMA and LTPP HMA overlaid pavements.
    - Each 500-ft unit was sampled according to a sampling plan developed by the project team.
    - Each sample unit was cored and the cores tested in the lab for the following:
        - HMA thickness.
        - HMA bulk specific gravity, air voids, gradation, maximum specific gravity, and asphalt content.
        - Other key inputs such as binder type, HMA creep compliance, etc., were obtained by reviewing MoDOT pavement design and construction records.
    - State-specific defaults of unbound material properties (Atterberg limits, gradation, and so on) were obtained from MoDOT records were used for material characterization.
    - Initial average annual daily truck traffic (AADTT) and traffic growth rate were computed using historical truck traffic counts obtained from MoDOT.
    - Initial IRI was backcasted using historical IRI data provided by MoDOT.
    - Dynamic cone penetrometer (DCP) testing was performed on HMA projects with crushed stone bases. However, DCP test data were not used to characterize the resilient modulus of crushed stone materials and subgrade soils.
    - The resilient modulus of crushed stone materials and subgrade soils were obtained from the FWD data. FWD deflection test data were provided by MoDOT. Using these data, pavement layer moduli (e.g., HMA, base, underlying intact and rubblized PCC layers) were backcalculated. For flexible pavements, backcalculation was done using the layer modulus backcalculation EVERCALC.
    - Automated and manual distress surveys were conducted and made available by MoDOT. The data were used to characterize pavement condition (measured alligator cracking, transverse cracking, rutting, and IRI).

Table I-61. Local Missouri pavements projects identified for inclusion in validation/calibration database.

MoDOT Project ID	ARA Project ID	Pavement Type	Direction	County	Functional Class	Route	Construction Year	Number Samples
J5P0418	DS7	HMA	NB	Boone	Principal Arterial	MO-63	2000	4
J4S0110D	DS8	HMA	EB	Ray/Clay		MO-210	2002	2
J6P0112	DS6	HMA	NB	Iron	Minor Arterial	MO-21	1999	3
J6P0112	DS6	HMA	SB	Iron	Minor Arterial	MO-21	1999	3
J5P0368	DS9	HMA	EB	Camden	Principal Arterial	MO-54	1997	3
J5U0441D	DS11	HMA	SB	Cole	Principal Arterial	MO-179	2000	1
J8P0454	DS10	HMA	SB	Taney	Principal Arterial	MO-65	2000	2
5-P-54-34	A3	JPCP	WB	Callaway	Principal Arterial	US-54	1975	3
BRS-44(11)	A2	JPCP	EB	Buchanan	Major Collector	MO-6	1987	1
BRS-44(11)	A2	JPCP	WB	Buchanan	Major Collector	MO-6	1987	1
1-S-116-25	A1	JPCP	WB	Caldwell	Major Collector	MO-116	1989	2
J1U0402	B1	JPCP	WB	Buchanan	Major Collector or Minor Arterial	MO-6	1995	1
ACSTP-1136(7)	C1	JPCP	WB	Scott	Major Collector	RT-AB	1997	1
J7P0601B	DS5	HMA	NB	McDonald	Principal Arterial	US-71	2002	3
J7P0492	DS5	HMA	SB	McDonald	Principal Arterial	US-71	2002	3
J4U0029F	D2	JPCP		Clay	Freeway	MO-152	1998	
J4P0889	D1	JPCP	WB	Platte	Minor Arterial	MO-92	1995	1
J5P0381	D3	JPCP	EB	Boone	Minor Arterial	RT-TT	1995	1
J1I0541	E1	JPCP	SB	Harrison	Interstate	IS-35	1996	3
J7P0490	E4	JPCP	SB	Newton	Principal Arterial	US-71	1997	2
J7P0427G	E2	JPCP	SB	Newton	Principal Arterial	US-71	1995	2
J0P0571	F8	JPCP	EB	Butler	Principal Arterial	US-60	1997	2
J0P0572	F9	JPCP	WB	Butler	Principal Arterial	US-60	1997	2
J4P0861D	F7	JPCP	SB	Henry	Principal Arterial	MO-7	1997	2
J0U0412C	F10	JPCP	EB	Cape Girardeau	Principal Arterial	MO-74	1995	1
J5P0409	F3	JPCP	WB	Callaway	Freeway	US-54	1994	3
J1P0489B	F1	JPCP	WB	Caldwell	Principal Arterial	US-36	1996	4
J3P0284	F2	JPCP	EB	Marion	Principal Arterial	US-24	1997	1
J5P0412C	F6	JPCP	EB	Callaway	Principal Arterial	US-54	1994	3
J5P0411C	F5	JPCP	EB	Callaway	Principal Arterial	US-54	1993	3
J5P0410	F4	JPCP	WB	Callaway	Freeway/Principal Arterial	US-54	1994	1
J5P0621	G1	JPCP	SB	Callaway	Freeway/Principal Arterial	US-63	1994	2
J6S064I	G3	JPCP	EB	Jefferson	Principal Arterial	RT-M	1998	3
J6S064E	G2	JPCP	EB	Jefferson	Principal Arterial	RT-M	1998	2
J6I1486	UB4	UBOL	SB	St. Louis	Interstate	IS-255	2004	2
J0I0833	UB3	UBOL	SB	Pemiscot	Interstate	IS-55	2002	4
J8I0633	UB2	UBOL	WB	Greene	Interstate	IS-44	2000	2
J1I0634	UB1	UBOL	SB	Atchison	Interstate	IS-29	1998	2
J0P0600D	UB5	UBOL	EB	Pemiscot	Interstate	US-412	2003	4

Table I-62. Local Missouri LTPP pavements projects identified for inclusion in validation/calibration database.

LTPP Project ID	Pavement Type	Direction	County	Functional Class	Route	Construction (Overlay) Year	Number Samples
29_0501_1	HMA	NB	Taney	RPA - Other	US-65	1981	1
29_0502_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0503_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0504_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0505_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0506_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0507_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0508_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0509_2	HMA/HMA	NB	Taney	RPA - Other	US-65	1998 (1998)	1
29_0661	HMA/Rubblized	NB	Harrison	RPA - Interstate	IH-35	1975 (1992)	1
29_0662	HMA/Rubblized	NB	Harrison	RPA - Interstate	IH-35	1975 (1992)	1
29_0663	HMA/Rubblized	NB	Harrison	RPA - Interstate	IH-35	1975 (1992)	1
29_0664	HMA/Rubblized	NB	Harrison	RPA - Interstate	IH-35	1975 (1992)	1
29_0701_1	JPCP	NB	Jefferson	UPA	US-67	1955	1
29_0801_1	HMA	SB	Christian	RLC	RT-65	1998	1
29_0802_1	HMA	SB	Christian	RLC	RT-65	1998	1
29_0807_1	JPCP	SB	Christian	RLC	RT-65	1998	1
29_0808_1	JPCP	SB	Christian	RLC	RT-65	1998	1
29_1002_1	HMA	WB	Cole	RMC	RT-3	1986	1
29_1005_1	HMA	WB	Miller	RPA – Other	US-54	1974	1
29_1008_1	HMA	SB	Jasper	RPA – Other	RT-171	1986	1
29_1010_1	HMA	EB	Pulaski	RPA - Interstate	IH-44	1980	1
29_1010_2	HMA/HMA	EB	Pulaski	RPA - Interstate	IH-44	1980 (1998)	1
29_5393_2	HMA/PCC	NB	St. Charles	Rural Minor Arterial	SH-79	1957 (1987)	1
29_6067_1	HMA	EB	Carter	RPA – Other	US-60	1965	1
29_7073_1	HMA/PCC	NB	Livingston	RPA – Other	US-65	1974 (1987)	1
29_A330_2	HMA	WB	Miller	RPA - Other	US-54	1974	1
29_A340_1	HMA	WB	Miller	RPA – Other	US-54	1974	1
29_A601b	JPCP	EB	Washington	RPA - Other	SH-8	1969	1
29_A603_2	HMA/PCC	EB	Washington	RPA – Other	SH-8	1969 (1998)	1
29_A604_2	HMA/PCC	EB	Washington	RPA – Other	SH-8	1968 (1998)	1
29_A606_2	HMA/PCC	EB	Washington	RPA – Other	SH-8	1969(1998)	1
29_A607_2	HMA/PCC	EB	Washington	RPA – Other	SH-8	1969(1998)	1
29_A608_2	HMA/PCC	EB	Washington	RPA – Other	SH-8	1969(1998)	1
29_A801_1	HMA	NB	Ralls	RLC	US-61	1998	1
29_A802_1	HMA	NB	Ralls	RLC	US-61	1998	1
29_A807_1	JPCP	NB	Ralls	RLC	US-61	1998	1
29_A808_1	JPCP	NB	Ralls	RLC	US-61	1998	1
29_B330_1	HMA	WB	Cole	RMC	RT-C	1986	1
29_B340_1	HMA	WB	Cole	RMC	RT-C	1986	1

RMC = rural major collector, RLC = rural local collector, RPA = rural principal arterial.

- Non-LTPP MoDOT new JPCP and unbonded JPCP over existing PCC pavements.
    - Each 500-ft unit was sampled according to a sampling plan developed by the project team (see figure 5).
    - Each sample unit was cored and the cores were tested in the lab for the following:
        - PCC layer thicknesses.
        - PCC compressive strength and coefficient of thermal expansion among others.
        - PCC elastic modulus and flexural strength was obtained through the use of State specific correlations based on PCC compressive strength. The correlations are presented below as follows:

$$E_{PCC} = 64200 * \sqrt{f_c^{'}} \qquad (1)$$

$$MR = 9.6 * \sqrt{f_c^{'}} \qquad (2)$$

    - State-specific defaults of unbound material properties (Atterberg limits, gradation, and so on) obtained from MoDOT records were used for material characterization.
    - Initial AADTT and traffic growth rate was computed using historical truck traffic counts obtained from MoDOT.
    - Initial IRI was backcasted using historical IRI data provided by MoDOT.
    - FWD deflection test data were provided by MoDOT. Using these data, pavement layer moduli and modulus of subgrade reaction were backcalculated. Rigid pavement backcalculation was done based on the AASHTO AREA method.
    - Automated and manual distress surveys were provided by MoDOT. The data were used to characterize pavement condition (transverse cracking, faulting, and IRI).
- LTPP MoDOT HMA and PCC pavements.
    - Review of LTPP traffic, inventory, materials, and distress databases.
    - Key inputs were obtained as described below:
        - <u>Asphalt mix volumetric properties</u>: These were obtained through lab testing of asphalt concrete (AC) cores extracted by LTPP.
        - <u>PCC strength and modulus</u>: Depending on the project type (SPS-8 versus GPS sections), the 14-, 28-, 365-day or long-term modulus of rupture (flexural strength), elastic modulus, compressive strength, and so on were tested and are available in the LTPP database. The available data were used to estimate Level 1 MEPDG inputs (i.e., 14-, 28-, and 90-day MR and E_{PCC}) for SPS projects and Level 3 MEPDG inputs (i.e., 28-day MR and E_{PCC}) for GPS projects.
        - <u>PCC coefficient of thermal expansion (CTE)</u>: For both the SPS and GPS projects PCC materials, CTE values were measured by LTPP.
        - <u>Unbound aggregate materials and soils inputs for climate modeling</u>: These were determined using the LTPP lab tested gradation and Atterberg limit values.

- Resilient modulus of unbound aggregate materials used as base or subbases: Default MEPDG values were adopted based on the material AASHTO soil classification determined using LTPP lab tested gradation and Atterberg limit values.
- Subgrade resilient modulus (tested at optimum moisture): Obtained through backcalculation using FWD deflection test data provided by LTPP. Rigid pavement backcalculation was done based on the AASHTO AREA method. For flexible pavements, backcalculation was done using EVERCALC.

For both non-LTPP and LTPP flexible and rigid pavements, subgrade lab resilient modulus (M_r) at optimum moisture content is the required MEPDG input when the Integrated Climatic Model (ICM) is used to determine the effect of seasonal changes in moisture within the subgrade layer on M_r over time. The reasonably adjusted M_r is then used in the program to backcalculate a k-value for each month for rigid and composite pavements and used for analysis. For flexible pavements, the seasonally adjusted values M_r are used directly as inputs for analysis. The MEPDG allows for the direct input for in-situ subgrade k-values for rehabilitated JPCP (e.g. CPR, unbounded JPCP overlay, bonded PCC over existing JPCP and so on). The procedure for obtaining lab tested M_r values for the subgrade is presented below:

- For both HMA pavements and JPCP, FWD data from the LTPP database or MoDOT were used to backcalculate a long-term in-situ subgrade resilient modulus or modulus of subgrade reaction (k-value). The point in time for backcalculation was selected in such a way that the backcalculated subgrade moduli or k-value represent in-situ conditions presumably when equilibrium moisture contents are reached in the field and when the subgrade is not saturated or frozen (summer months).
- The backcalculated subgrade resilient modulus value for HMA pavements was multiplied by a factor of 0.35 to convert it to a lab tested in situ resilient modulus value.
- An appropriate subgrade lab resilient modulus (M_r) at optimum moisture content value was then determined through trial and error (by running the MEPDG and using it to estimate long-term in situ subgrade resilient modulus for HMA pavements or k-value for PCC pavements and matching the MEPDG long term HMA subgrade resilient modulus or PCC pavement k-value estimate to that obtained through backcalculation).

5.3 MEPDG MODEL VALIDATION

A summary of the comparisons between the MEPDG predicted and field measured distress/IRI for all the pavement types evaluated is presented in this section. Details are presented in Volume II of this report.

Several methods (statistical or otherwise) were used singly or in combination to evaluate model adequacy. The analyses performed are described in greater detail in Volume II (Chapter 4) and are briefly summarized below:

- Statistical approach for model validation (used when the measured distress/IRI values were well above zero)
    - Determine model prediction capability—assessed by determining the correlation between measured and MEPDG predicted distress/IRI. The diagnostic statistic used for making this comparison was the coefficient of determination, R^2. The estimated R^2 was compared with R^2 obtained from NCHRP 1-40D (see Table I-63). Engineering judgment was then used to determine the reasonableness of the SEE.
    - Estimate model accuracy—the diagnostic statistic standard error of the estimate (SEE) was used to determine model accuracy. The estimated SEE was compared with SEE obtained from NCHRP 1-40D (see Table I-63). Engineering judgment was then used to determine the reasonableness of the SEE.
    - Determine bias—bias was determined by performing linear regression using the measured and MEPDG predicted distress/IRI and performing the following three hypothesis tests in the sequence listed.
        - Hypothesis 1: Determining whether the linear regression model developed using measured and MEPDG predicted distress/IRI has an intercept of zero.
        - Hypothesis 2: Determine whether the linear regression model developed using measured and MEPDG predicted distress/IRI has a slope of 1.0.
        - Hypothesis 3: Determine whether the measured and MEPDG predicted distress/IRI represented the same population of distress/IRI using a paired t-test.

      A significance level, α, of 0.05 or 5 percent was assumed for all hypothesis testing. A rejection of any of the null hypothesis implied that the model was biased and therefore there was no need for further testing. Models that successfully passed all three tests were deemed to be unbiased.
- Non-statistical approach for model validation (used when measured distress/IRI was zero or close to zero for all sections evaluated)
    - A simple comparison was made of measured and predicted distress/IRI categorized into as many groupings as needed. The range of each group was determined based on the engineering judgment. The goal was to determine how often measured and predicted distress/IRI remained in the same group. Measured and predicted distress remaining in the same group implied reasonable and accurate predictions without bias, while measured and predicted distress residing in different groups suggested otherwise.

The results of both the non-statistical and statistical analysis, as appropriately applied, were used to determine overall MEPDG distress/IRI models adequacy. Where the MEPDG models were deemed inadequate for Missouri conditions the models were recalibrated. The recalibrated models were again evaluated for prediction capacity, accuracy, sensitivity and bias. The results are presented in the following sections.

Table I-63. Summary of NCHRP 1-40D new HMA pavement and new JPCP model statistics.

Pavement Type	Performance Model	Model Statistics		
		Coefficient of Determination, R^2	Standard Error of Estimate, SEE	Number of Data Points, N
New HMA	Alligator cracking	0.275	5.01 percent	405
	Transverse "thermal" cracking	Level 1*: 0.344 Level 2*: 0.218 Level 3*: 0.057	—	—
	Rutting	0.58	0.107 in.	334
	IRI	0.56	18.9 in./mi.	1926
New JPCP	Transverse "slab" cracking	0.85	4.52 percent	1505
	Transverse joint faulting	0.58	0.033 in.	1239
	IRI	0.60	17.1 in./mi.	163

*Level of inputs used for calibration.

5.3.1 Comparison of Measured and Predicted Distress/Smoothness for New HMA and HMA Surfaced Pavements (including HMA Overlays of existing HMA and PCC Pavements)

5.3.1.1 Alligator "Bottom-Up Fatigue" Cracking

Most of the projects selected for validation were newer sections in relatively good condition and reported no alligator cracking. Hence, it was not possible to assess model accuracy and bias using conventional diagnostic statistical techniques. Therefore, a simple non-statistical comparison of measured and predicted alligator cracking was done based on the approach described earlier. The results are presented in

Table I-64. For this comparison, alligator cracking was categorized into eight groups. The range of each group increased with increasing alligator cracking. The following observations were made:

- A vast majority of the measured and predicted alligator cracking (approximately 96 percent) fell within the 0-2 percent grouping.
- The nationally calibrated MEPDG model appears to both under-predict and over-predict alligator cracking slightly.

There is a need for data from older deteriorated pavements in order to determine model adequacy. However, until such data are available for further consideration, it is recommended that MoDOT use the nationally calibrated MEPDG alligator cracking model for routine design.

Table I-64. Comparison of measured and predicted alligator cracking (percent of all measurements).

Measured Alligator Cracking, percent area	MEPDG Predicted Alligator Cracking, percent area							
	0-2	2-5	5-10	10-20	20-40	40-60	60-80	80-100
0-2	148	0	1	0	0	0	0	0
2-5	2	0	1	0	0	0	0	0
5-10	1	0	0	0	0	0	0	0
10-20	1	0	0	0	0	0	0	0
20-40	0	0	0	0	0	0	0	0
40-60	0	0	0	0	0	0	0	0
60-80	0	0	0	0	0	0	0	0
80-100	0	0	0	0	0	0	0	0

5.3.1.2 HMA Transverse "Thermal" Cracking

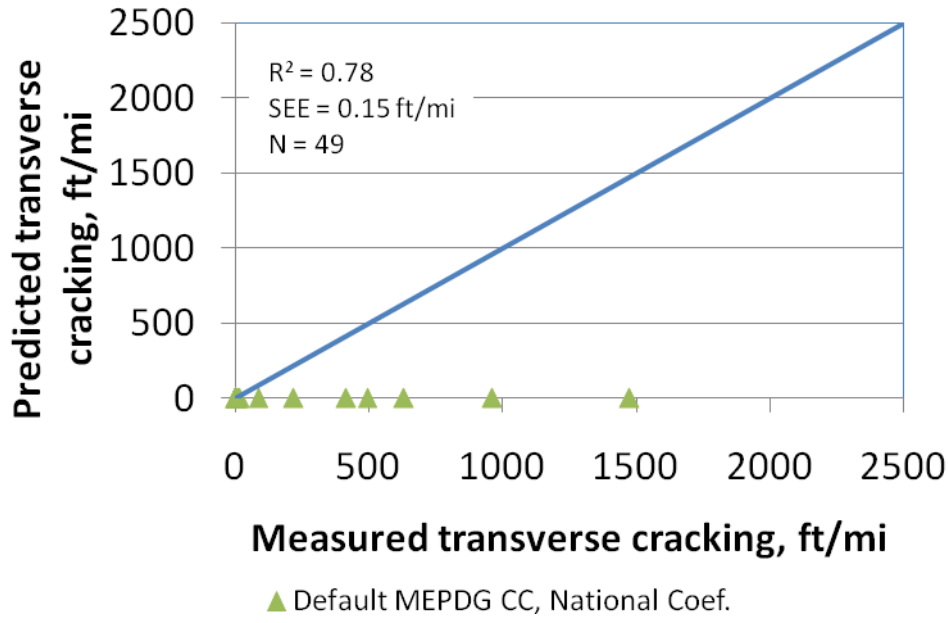
An adequate amount of thermal cracking data was available for a statistical evaluation of the nationally calibrated model. Model validation was performed at two levels using only new HMA pavements (since overlays may have datasets confounded with reflection cracking) with HMA mixtures for which MoDOT specific HMA creep compliance and tensile strengths were available. The validation effort included:

- Checking the goodness of fit and bias of the nationally calibrated MEPDG model for transverse cracking predictions using MEPDG default (Level 3) HMA creep compliance and tensile strength inputs. The goal was to determine whether applying national defaults for key HMA properties along with national model coefficients would result in reasonable predictions of HMA transverse cracking.
- Checking the goodness of fit and bias of nationally calibrated MEPDG model for transverse cracking predictions using MoDOT specific (Level 1) HMA creep compliance and tensile strength inputs. The goal was to determine if using MoDOT specific HMA creep compliance and tensile strengths significantly improve the prediction accuracy and bias.

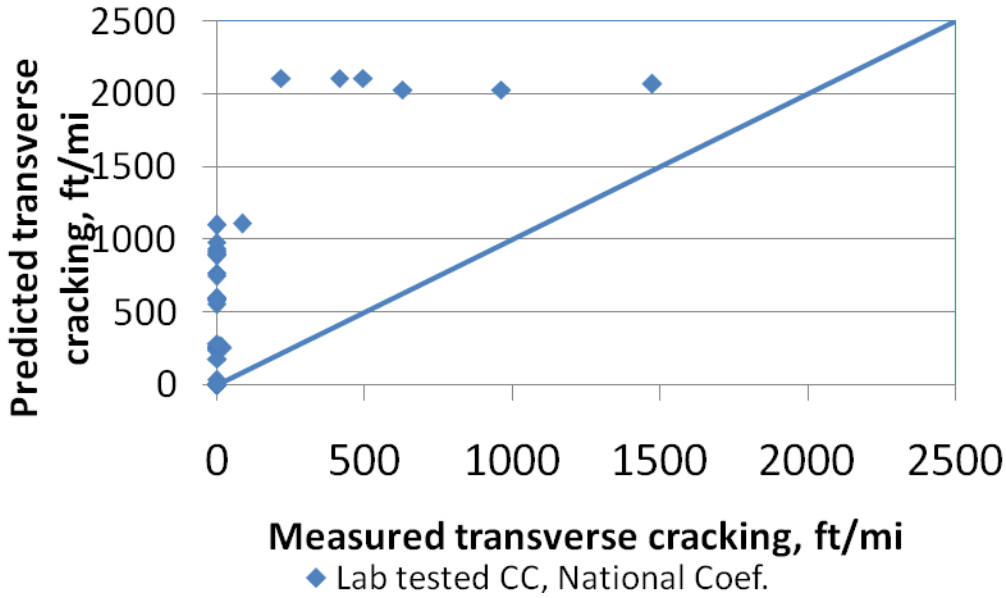
Figure I-119 and Table I-65 indicate that:

- The MEPDG national model and MEPDG default values of HMA creep compliance and tensile strength appear to underpredict the measured cracking. Note that it was discussed in Chapter 3 that the MEPDG default compliance values were significantly higher than lab measured values. The MEPDG default tensile strengths on the other hand were slightly lower than the lab measured values.
- Use of MoDOT specific HMA creep compliance and tensile strengths improved the predictions somewhat. However, the predictions still show a significant bias (overprediction) of the thermal cracking in some cases.

Therefore, there was a need to recalibrate the transverse cracking model. Recalibration involved replacing national MEPDG HMA creep compliance and HMA tensile strength values with MoDOT specific values and modifying the local calibration coefficient, β_t from 1.5 to 0.625 to reduce bias. The results of the recalibration effort are presented in Figure I-121 and Table I-67.



(a) MEPDG default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model

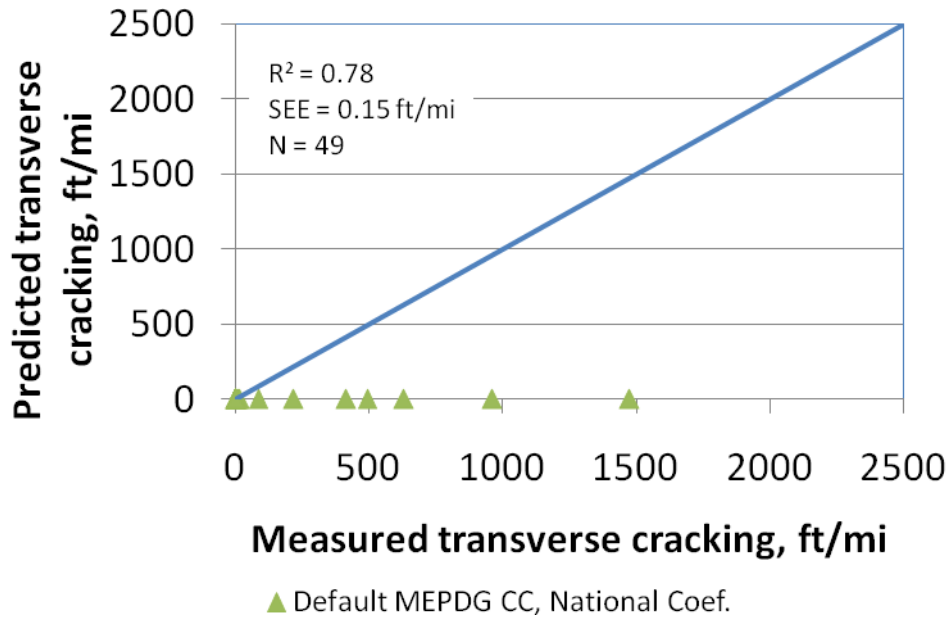


(b) MoDOT default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model

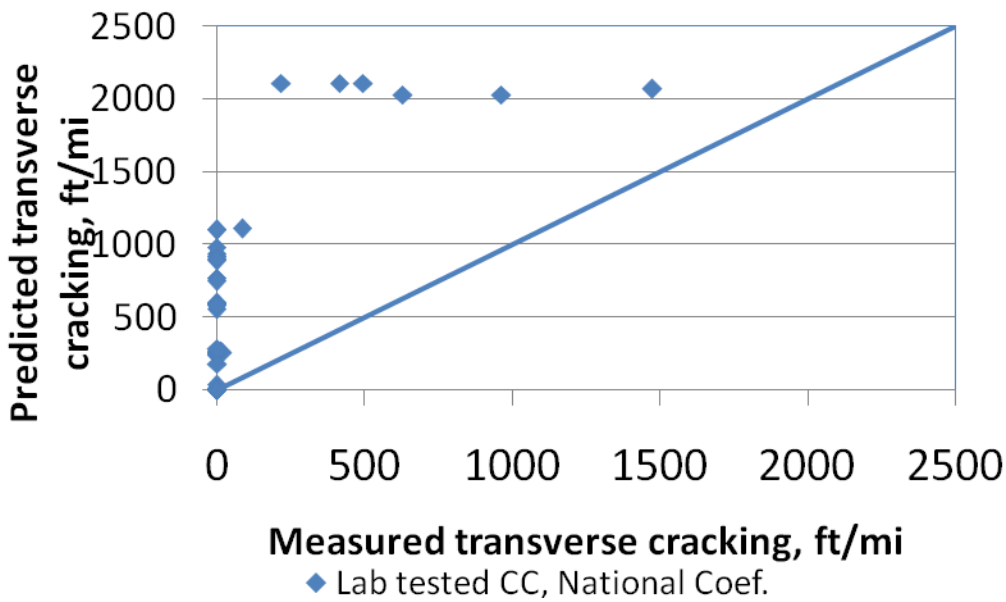
Figure I-119. Plot showing measured versus MEPDG predicted transverse cracking.

Table I-65. Statistical comparison of measured and predicted transverse cracking data.

MEPDG default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model					
Goodness of Fit $R^2 = 0.78$ SEE = 0.15 ft/mi N = 49					
Hypothesis Testing (MEPDG Defaults)					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	0.035	0.1250	-0.0103	0.0820
(2) Ho: Slope = 1.0	1	0.0011	<0.0001	0.0009	0.0012
(3) Ho: Measured transverse cracking – MEPDG transverse cracking = 0	117	—	<0.0001	—	—
MoDOT default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model					
Goodness of Fit (MEPDG Defaults) $R^2 = 0.52$ SEE = 459 ft/mi N = 49					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	425	<0.0001	288	562
(2) Ho: Slope = 1.0	1	2.207	<0.0001	1.59	2.82
(3) Ho: Measured transverse cracking – MEPDG transverse cracking = 0	117	—	<0.0001	—	—



(c) MEPDG default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model



(d) MoDOT default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model

Figure I-120. Plot showing measured versus MEPDG predicted transverse cracking.

Table I-66. Statistical comparison of measured and predicted transverse cracking data.

MEPDG default HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model					
Goodness of Fit $R^2 = 0.78$ SEE = 0.15 ft/mi N = 49					
Hypothesis Testing (MEPDG Defaults)					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	0.035	0.1250	-0.0103	0.0820
(2) Ho: Slope = 1.0	1	0.0011	<0.0001	0.0009	0.0012
(3) Ho: Measured transverse cracking – MEPDG transverse cracking = 0	117	—	<0.0001	—	—
MoDOT specific HMA creep compliance and HMA tensile strength along with nationally calibrated MEPDG transverse cracking model					
Goodness of Fit (MEPDG Defaults) $R^2 = 0.52$ SEE = 459 ft/mi N = 49					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	425	<0.0001	288	562
(2) Ho: Slope = 1.0	1	2.207	<0.0001	1.59	2.82
(3) Ho: Measured transverse cracking – MEPDG transverse cracking = 0	117	—	<0.0001	—	—

The adjusted transverse cracking model has an excellent goodness of fit but is slightly biased. However, considering that the bias of the recalibrated model is more favorable than the nationally calibrated model, it is recommended for use by MoDOT.

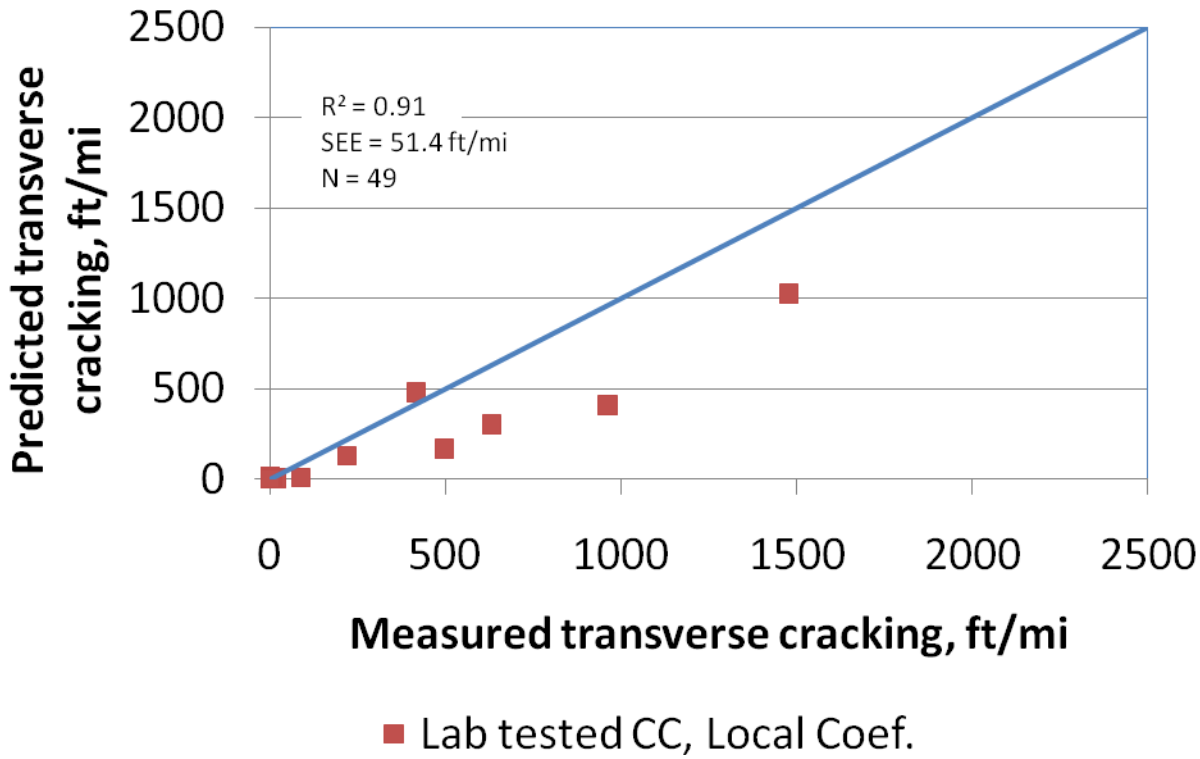


Figure I-121. Plot showing measured versus predicted transverse cracking (for various stages of model and HMA material properties inputs).

Table I-67. Statistical comparison of measured and predicted transverse cracking data.

Goodness of Fit $R^2 = 0.91$ SEE = 51.4 ft/mi N = 49					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	0.035	0.907	-16.2	14.5
(2) Ho: Slope = 1.0	1	0.60	<0.0001	0.54	0.65
(3) Ho: Measured transverse cracking – MEPDG transverse cracking = 0	117	—	00041	—	—

A limited sensitivity analysis was performed to determine the reasonableness of the locally calibrated thermal cracking model. A plot showing the changes in predicted thermal cracking for different levels of model inputs is presented in Figure I-122. The following conclusions can be drawn by comparing the information presented in this plot with the sensitivity of the nationally calibrated model (Figure I-114):

- Judging by the scale of the predicted cracking (y-axis), the locally calibrated model predicts significantly more cracking as expected. This is largely because, in Figure I-114, the MEPDG default values were used for creep compliance and tensile strength to predict thermal cracking. Recall that it was discussed earlier in Chapter 2 that the MEPDG default compliance values were higher than the MoDOT specific values and hence they resulted in lower predictions of thermal cracking.
- The effect of climate is more pronounced on the predicted cracking but the relative order of the climate stations is still the same, i.e., the coldest location had the highest predicted thermal cracking and vice versa generally speaking.
- The sensitivity of the recalibrated model brings to fore another important aspect of the thermal cracking prediction. It can be note that the thermal cracking predictions for Kansas City 1 and Kansas City 2, which are essentially two different weather stations from the same city located a few miles apart, are significantly different. Similarly, although to a lesser degree, the same is true for climate stations St. Louis 1 and St. Louis 2. This points to the high degree of importance attached to good quality climate data for thermal cracking predictions.
- The ranking of the factors influencing thermal cracking is slightly different, and perhaps more accurate, for the locally calibrated model. The factors arranged by their order of importance are: Climate, asphalt grade, air voids, and HMA thickness. A few unexpected findings were that the binder content has no impact. This was more an artifact of the way the sensitivity analysis was conducted rather than a true effect in that in performing this analysis creep compliance and tensile strength were not varied as a function of binder content since these data were not available. In reality, if such data were available, the impact of binder content would likely be larger and proportional to the impact of air voids on the predicted thermal cracking. Another variance between the sensitivities of the national and local models was the impact of construction season on the predicted thermal cracking. The nationally calibrated model is insensitive to the construction season while the locally calibrated model shows slight sensitivity with pavement constructed in the Fall months showing a higher amount of predicted thermal cracking.

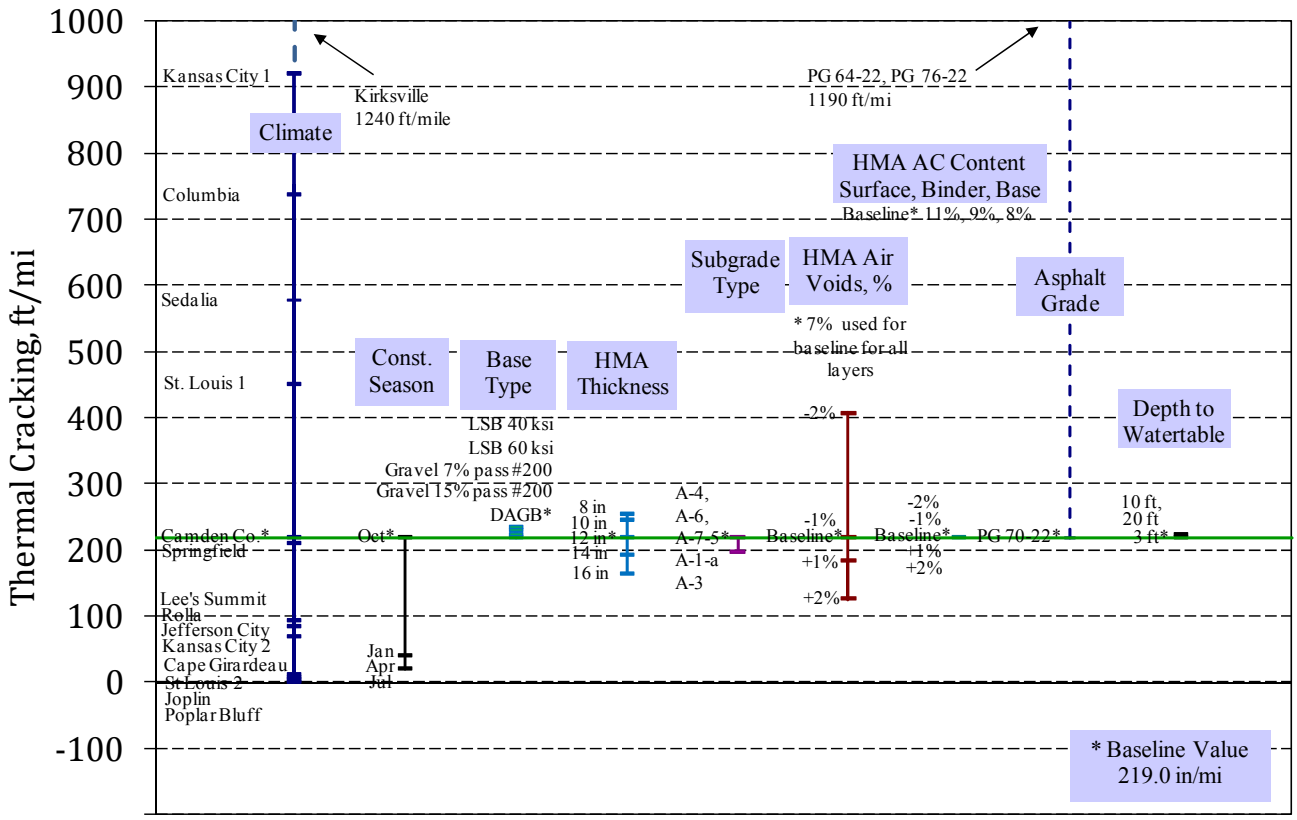


Figure I-122. Plot showing sensitivity analysis results of locally calibrated HMA transverse cracking model.

5.3.1.3 Total Rutting

A statistical comparison of measured and predicted total rutting was performed to determine the model's predictive capacity. The results are presented in Figure I-123 and Table I-68 and they indicate that the nationally calibrated model overpredicts total rutting indicating a bias. A poor correlation can also be noted from Table I-68 (i.e., R^2 was 0.32). The prediction accuracy (i.e., SEE) was, however, comparable to that from the national calibration.

Based on the bias and poor correlation noted between the predicted and measured total rutting a need to recalibrate the nationally calibrated rutting model was necessary to make it more suitable for Missouri conditions. Recalibration involved modifying the local calibration coefficients of the HMA, base, and subgrade rutting submodels (see equation 40 in Volume II of this report) of the total rutting model. Specifically, state/regional calibration coefficients β_{1r} for the HMA submodel, β_{S1} for the unbound base submodel, and β_{S2} for the subgrade submodel were modified to improve the model's predictive capacity.

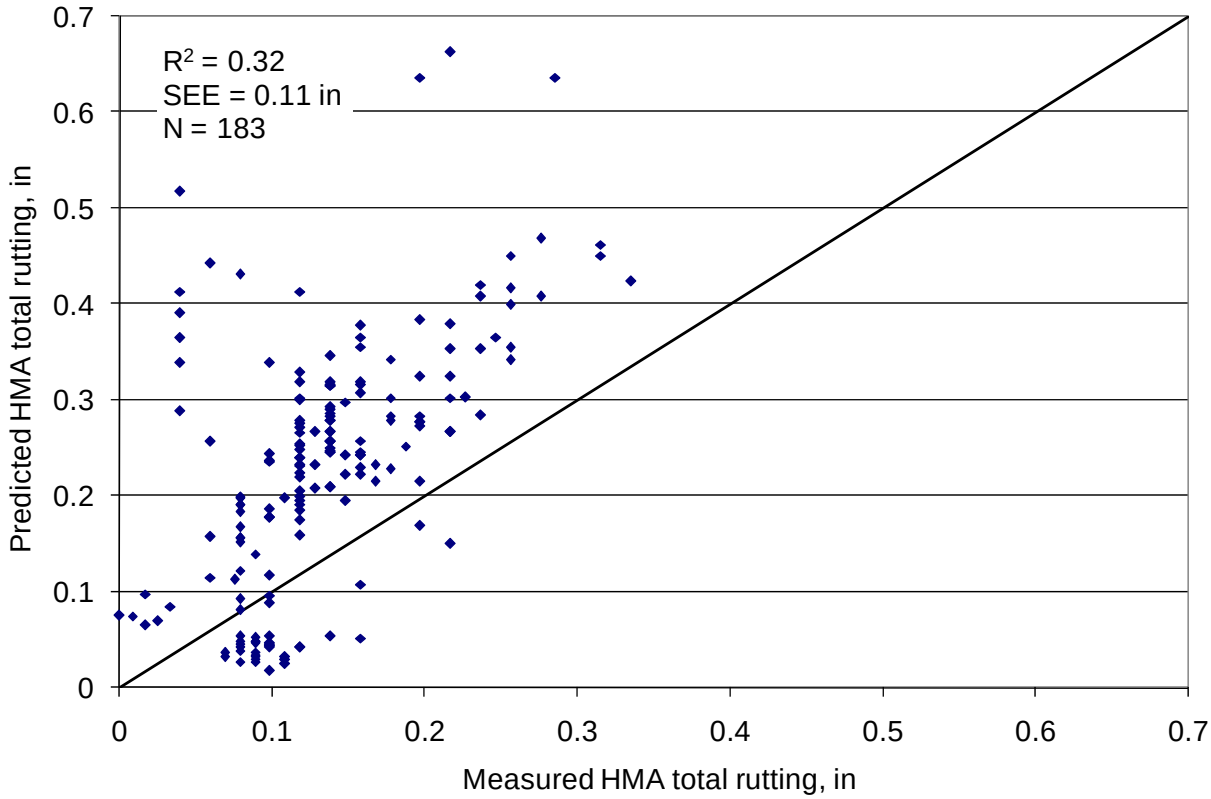


Figure I-123. Plot of measured versus MEPDG predicted new HMA pavement total rutting.

Table I-68. Statistical comparison of measured and predicted new HMA pavement MEPDG model rutting data.

Goodness of Fit

N = 183
R^2 = 0.32
SEE = 0.11 in

Hypothesis Testing

Hypothesis	DF	Parameter Estimate	p-value (Pr > |t|)	95 Percent Confidence Limits	
(1) Ho: Intercept = 0	1	0.071	0.0003	0.0329	0.1095
(2) Ho: Slope = 1.0	1	1.667	<0.0001	1.5535	1.7813
(3) Ho: Measured Rutting – MEPDG Predicted Rutting = 0	183	—	<0.0001	—	—

The recalibrated model, including new model coefficients, is as presented below for new HMA pavements:

$$TRUT = 1.07*ACRUT + 0.01*BASERUT + 0.4375*SUBGRUT \quad (40)$$

where

TRUT	=	Total rutting
ACRUT	=	Rutting in the asphalt layers predicted using the 1-40D models (see Chapter 2)
BASERUT	=	Rutting in the base layer predicted using the 1-40D models (see Chapter 2)
SUBGRUT	=	Rutting in the subgrade layer predicted using the 1-40D models (see Chapter 2)
β_{1r}	=	HMA rutting prediction local calibration factor = 1.07
β_{2r}	=	1.0
β_{3r}	=	1.0
β_{s1}	=	Unbound base rutting prediction local calibration factor = 0.01
β_{s2}	=	Subgrade rutting prediction local calibration factor = 0.4375

For HMA overlays, the model coefficients are as follows:

Pavement Type	**Rutting Submodels Local Calibration Coefficients**		
	HMA (β_{r1})	**Base (β_{s1})**	**Subgrade (β_{s1})**
HMA overlaid HMA	1.07	0	0
HMA overlaid Intact PCC	1.07	0	0
HMA overlaid Rubblized PCC	1.07	0	0

A statistical comparison of measured and predicted rutting using recalibrated rutting model was performed to determine its reasonableness. The results are presented in Figure I-124 and Table I-69. The results indicate the following:

- A fair correlation between measured and MEPDG predicted rutting.
- SEE much less than that reported for the national MEPDG rutting model.
- No significant bias in predicted and measured rutting as indicated by the results of testing hypotheses (1), (2), and (3).

A limited sensitivity analysis was performed to determine the reasonableness of the locally calibrated total rutting model. A plot showing the changes in predicted total rutting for different levels of model inputs is presented in Figure I-125. The following conclusions can be drawn by comparing the information presented in this plot with the sensitivity of the nationally calibrated model (Figure I-113):

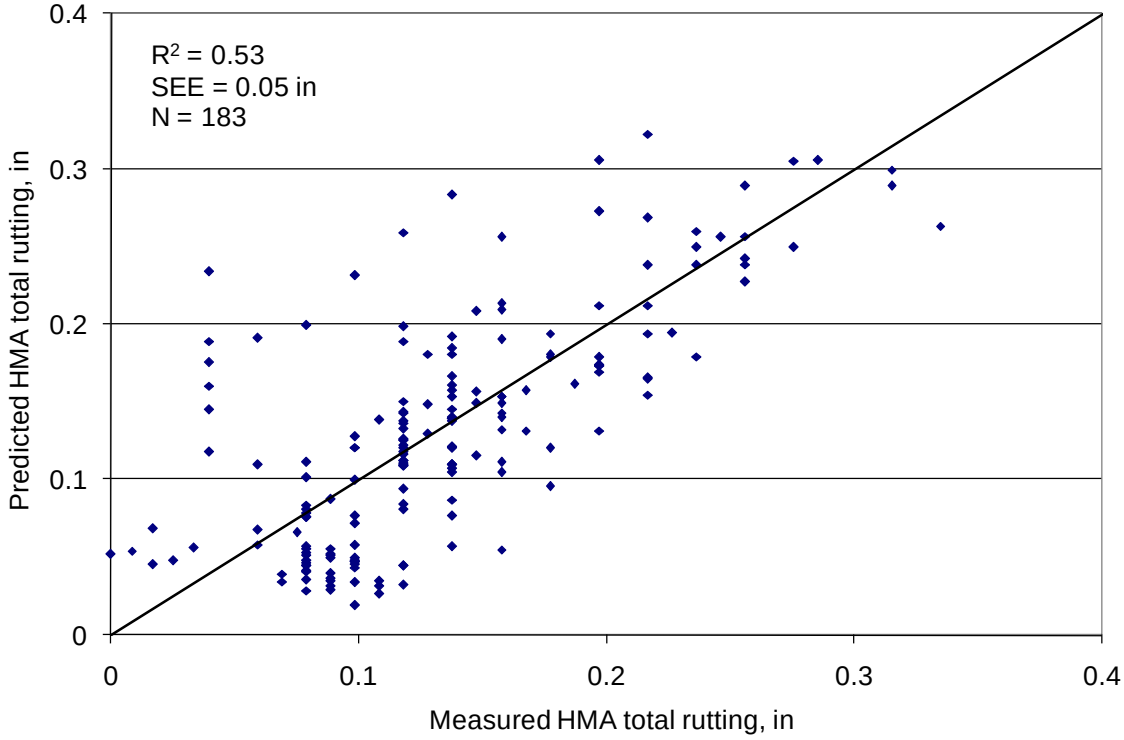


Figure I-124. Plot of measured versus locally calibrated model predicted new HMA pavement total rutting.

Table I-69. Statistical comparison of the measured and predicted rutting from the recalibrated model.

Goodness of Fit N = 183 R^2 = 0.52 SEE = 0.051 in					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	0.017	0.05	0.0000	0.035
(2) Ho: Slope = 1.0	1	0.974	0.322	0.9234	1.025
(3) Ho: Measured Rutting – MEPDG Predicted Rutting = 0	183	—	0.943	—	—

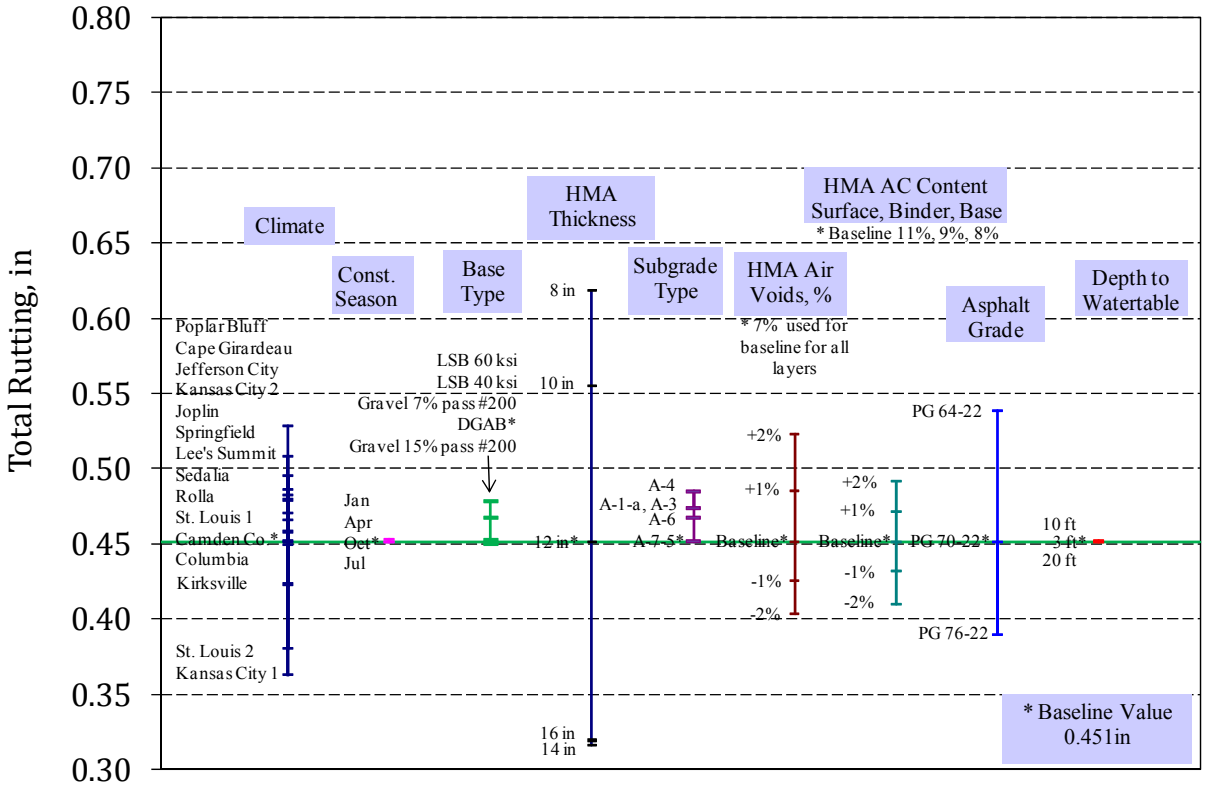


Figure I-125. Plot showing sensitivity analysis results of locally calibrated rutting model.

- The overall magnitude of the predicted baseline rutting is lower for the locally calibrated model.
- The ranking of the factors influencing total rutting is different, and perhaps more accurate, for the locally calibrated model. The factors arranged by their order of importance are: HMA thickness, climate, asphalt grade, air voids, and asphalt content. Key differences between the nationally calibrated and locally calibrated total rutting models are noted below:
    - The effect of climate is more pronounced on the predicted cracking but the relative order of the climate stations is still the same, i.e., the coldest location had the highest predicted thermal cracking and vice versa generally speaking. There is a general agreement in the ordering of the weather stations and the magnitude of rutting predicted, i.e., the hotter locations had higher predicted rutting than the cooler locations.
    - The impact of binder type is more pronounced.
    - Subgrade type and modulus have a smaller impact on the predicted rutting.

The sensitivity results showed that the recalibrated model is reasonable.

5.3.1.4 New HMA and HMA/HMA Pavement Smoothness (IRI)

A statistical comparison of the measured and MEPDG predicted IRI was performed to determine the nationally calibrated IRI model's predictive ability and accuracy. Note that MEPDG predicted IRI reflects the adjustments made to both the rutting and transverse cracking prediction models as part of local calibration effort. The results are presented in Table I-70 and Figure I-126.

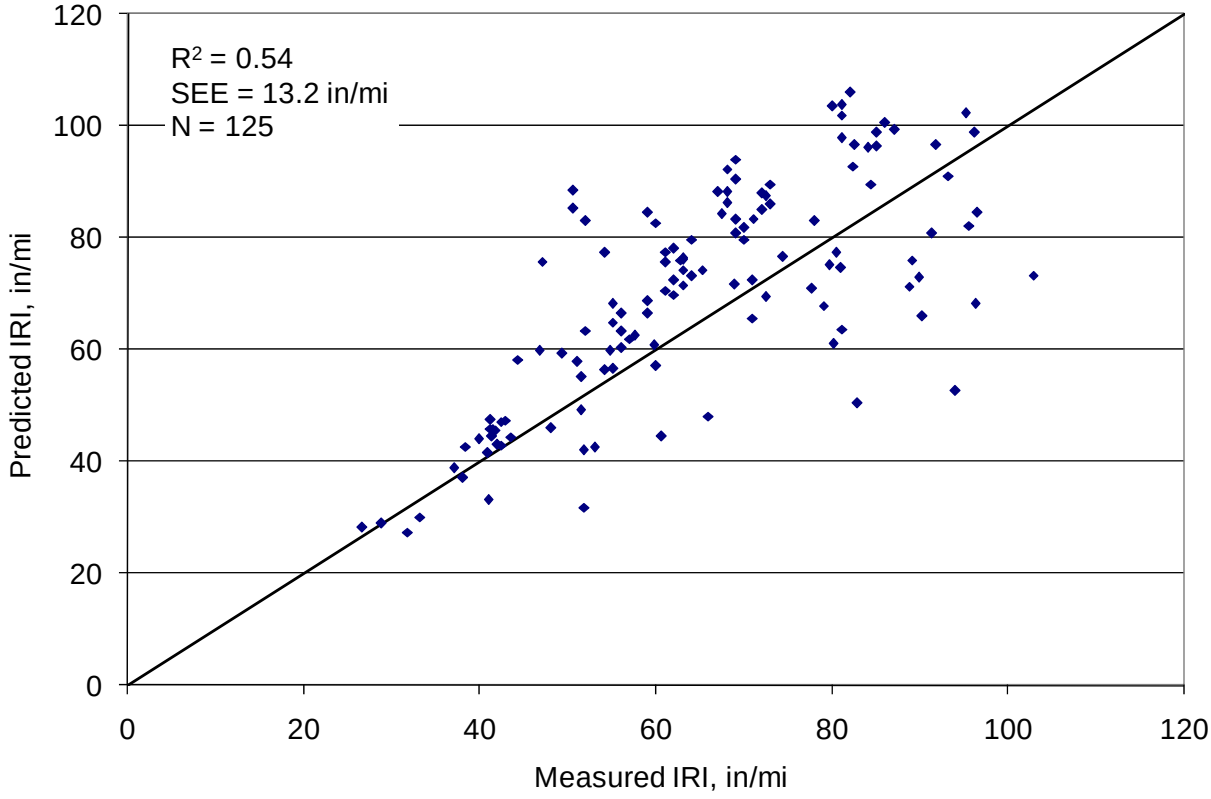


Figure I-126. Plot of measured versus MEPDG predicted IRI (nationally calibrated) for new HMA pavements and HMA overlays of existing HMA pavements.

The results indicate the following:

- There is bias in predicted and measured IRI as indicated by the results from testing hypothesis (1) and (3) in Table I-70. It can be gathered from Figure I-126 that the MEPDG slightly underestimates the IRI for the higher magnitudes of measured IRI. However, this was not very significant.
- There is a reasonable correlation between the measured and MEPDG predicted IRI (R^2 = 0.54).
- The SEE is less than that reported for the national MEPDG IRI model.

Recalibration was performed to remove the identified bias and improve the prediction accuracy using pavement sections with good quality measured IRI data. The recalibrated model, including new model coefficients, is as presented below:

Table I-70. Statistical comparison of measured and MEPDG predicted IRI (nationally calibrated) data for new HMA pavements and HMA overlays of existing HMA pavements.

Goodness of Fit

N = 125
R^2 = 0.54
SEE = 13.2 in/mi

Hypothesis Testing

Hypothesis	DF	Parameter Estimate	p-value (Pr > |t|)	95 Percent Confidence Limits	
(1) Ho: Intercept = 0	1	13.5	0.0037	4.48	22.52
(2) Ho: Slope = 1.0	1	1.03	0.0953	0.994	1.066
(3) Ho: Measured IRI – MEPDG Predicted IRI = 0	183	—	0.0182	—	—

$$IRI = IRI_o + 0.01(SF) + 0.975(FC_{Total}) + 0.008(TC) + 17.7(RD)$$

where:

IRI_o = Initial IRI after construction, in/mi
SF = Site factor (see below).
FC_{Total} = Area of fatigue cracking (combined alligator and longitudinal, cracking in the wheel path), percent of total lane area. All load related cracks are combined on an area basis – length of cracks is multiplied by 1 foot to convert length into an area basis. For HMA overlaid HMA pavements, the reflected cracking in the wheel path is included.
NTC = Length of transverse cracking (including the reflection of transverse cracks for HMA overlaid HMA pavements), ft/mi. Transverse cracking was computed using locally calibrated model.
NRD = Average rut depth (using MoDOT locally calibrated model), in
$\alpha_1, \alpha_2, \alpha_3, \alpha_4$ = model local calibration coefficients.

The site factor (SF) is calculated in accordance with the following equation:

$$SF = Age(0.02003(PI+1) + 0.007947(\mathrm{Pr}\,ecip+1) + 0.000636(FI+1))$$

where:

Age = Pavement age, years
PI = Percent plasticity index of the subgrade soil
FI = Average annual freezing index, degree F days
$Precip$ = Average annual precipitation or rainfall, in

A statistical comparison of measured and predicted HMA IRI using the locally calibrated model was done to determine the recalibrated model's prediction capacity, accuracy, and bias. The results are presented in Figure I-127 and Table I-71. The results indicate the following:

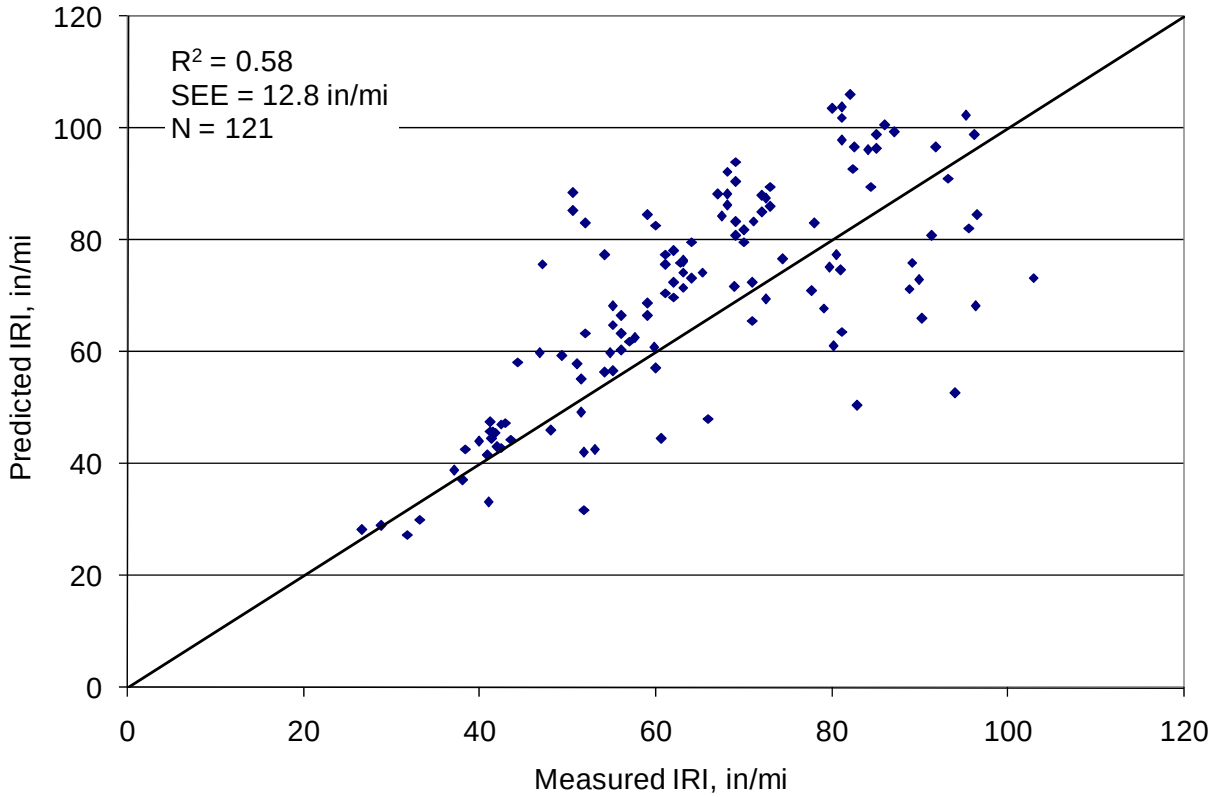


Figure I-127. Plot of measured versus MEPDG predicted IRI (locally calibrated) for new HMA pavements and HMA overlays of existing HMA pavements.

Table I-71. Statistical comparison of measured and predicted IRI (locally calibrated) for new HMA pavements and HMA overlays of existing HMA pavements.

Goodness of Fit

N = 125
R^2 = 0.53
SEE = 13.2 in/mi

Hypothesis Testing

Hypothesis	DF	Parameter Estimate	p-value (Pr > |t|)	95 Percent Confidence Limits	
(1) Ho: Intercept = 0	1	12.41	0.0092*	3.42	21.4
(2) Ho: Slope = 1.0	1	0.978	0.225	0.942	1.013
(3) Ho: Measured IRI – MEPDG Predicted IRI = 0	183	—	0.6265	—	—

*Borderline when compared to a significance level of 0.05.

- A reasonable correlation exists between measured and predicted IRI from the recalibrated HMA IRI model.
- The SEE was lower than that of the original MEPDG HMA IRI model
- Although hypothesis (1) was rejected and hypotheses (2) and (3) were accepted, the levels of bias reported were deemed reasonable.

Overall, the model can be used in routine design. A limited sensitivity analysis was performed to determine the reasonableness of the locally calibrated IRI model. A plot showing the changes in predicted IRI for different levels of model inputs is presented in Figure I-128. The information presented confirmed that the sensitivity of the locally calibrated IRI model for this pavement type was similar to the nationally calibrated model. However, the magnitude of the predicted IRI was lower overall for the locally calibrated model.

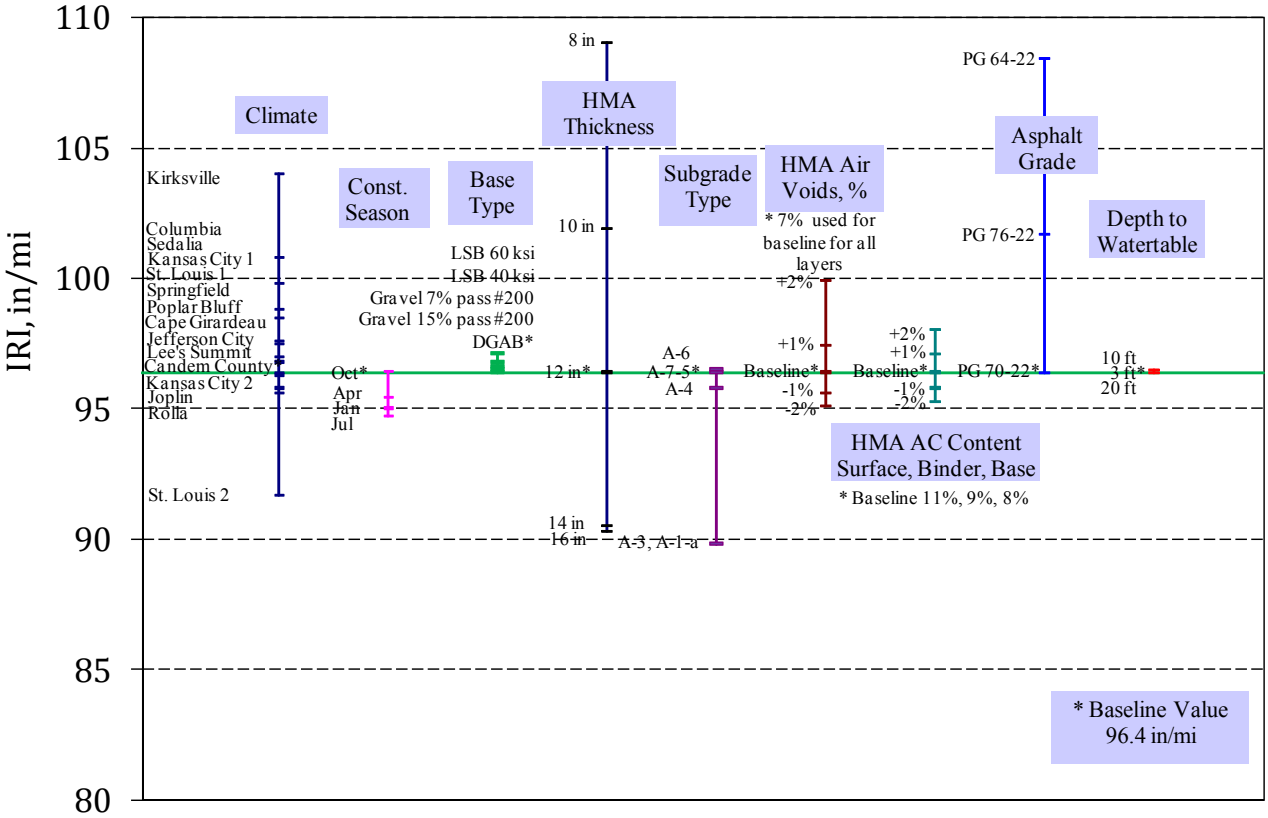


Figure I-128. Plot showing sensitivity analysis results of locally calibrated HMA IRI model.

5.3.1.5 HMA/PCC Pavement Smoothness (IRI)

Although limited amount of information was available for this pavement type, the data were evaluated statistically to assess the model's predictive capacity and reasonableness. The results are presented in Table I-72 and Figure I-129.

Table I-72. Diagnostic statistics used for evaluating the HMA/PCC IRI model (nationally calibrated).

Goodness of Fit N = 11 R^2 = 0.65 SEE = 5.3 in/mi					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	21.8	0.0974	-4.75	48.28
(2) Ho: Slope = 1.0	1	0.945	0.0259*	0.898	0.992
(3) Ho: Measured IRI – MEPDG Predicted IRI = 0	183	—	0.05	—	—

*Borderline when compared to a significance level of 0.05.

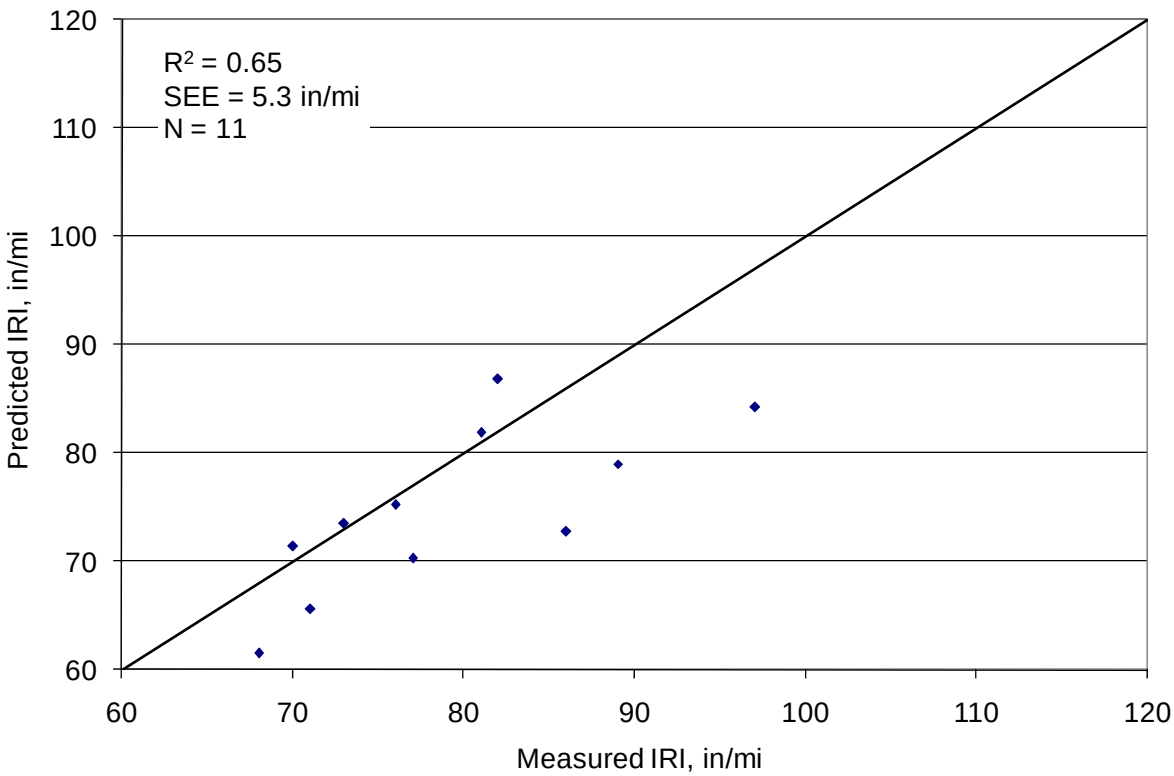


Figure I-129. Plot of measured versus predicted IRI (nationally calibrated) for HMA overlays of existing PCC pavements.

From the limited data analyzed, the following can be noted:

- A reasonable correlation exists between measured and predicted IRI from the nationally calibrated IRI model.
- SEE was lower than that of the original MEPDG composite HMA IRI model
- Although hypothesis (2) was rejected and hypotheses (1) and (3) were accepted, the levels of bias reported were deemed reasonable.

Recalibration of the nationally calibrated model was not deemed necessary for this pavement type at the present time.

5.3.2 Comparison of Measured and Predicted Distress/Smoothness for New JPCP

5.3.2.1 Transverse Slab Cracking

Although a majority of cracking magnitudes (including time series information) reported for the projects evaluated for this pavement type was zero, there were sufficient numbers of distressed projects to make it possible to evaluate this model statistically. The results of a statistical evaluation of the nationally calibrated MEPDG JPCP transverse cracking prediction model are presented in Figure I-130 and

Table I-73.

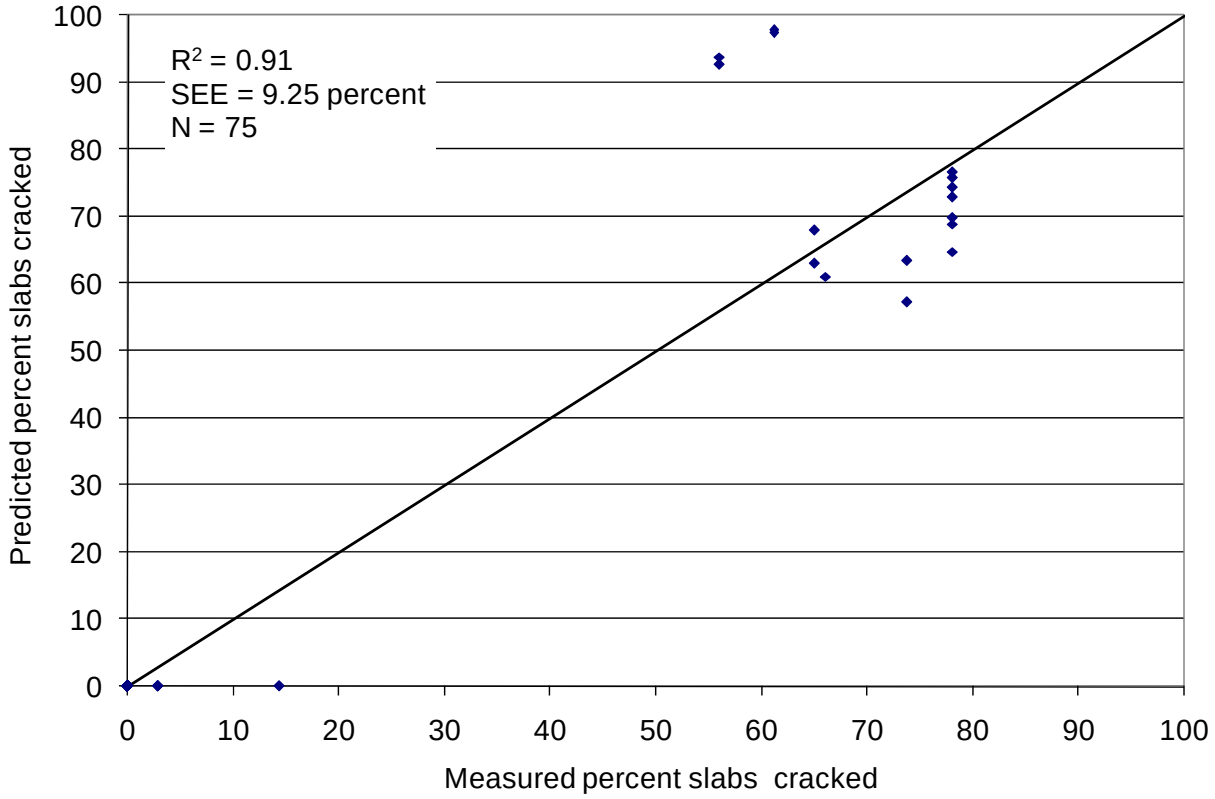


Figure I-130. Plot of measured versus MEPDG predicted JPCP transverse cracking.

Table I-73. Statistical comparison of measured and MEPDG predicted transverse cracking.

Goodness of Fit					
N = 75 R^2 = 0.91 SEE = 9.25 percent					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	0.1784	0.8821	-2.21	2.56
(2) Ho: Slope = 1.0	1	1.0352	0.2811	0.9705	1.099
(3) Ho: Measured Transverse Cracking – MEPDG Predicted Transverse Cracking = 0	75	—	0.5283	—	—

The results indicate the following:

- An excellent correlation exists between the measured and MEPDG predicted transverse cracking.
- The SEE is higher than that reported for the national MEPDG transverse cracking model.
- No bias in predicted and measured transverse cracking as indicated by the results of hypotheses (1), (2), and (3).

Therefore, despite the high SEE (owing perhaps to the limited number of data points from distressed projects), it can be concluded that the MEPDG nationally calibrated transverse cracking model was adequate. Recalibration of this model was is not imperative for its use in routine design. However, as additional data becomes available, the model validation exercise can be repeated to verify the findings and to improve the model accuracy.

5.3.2.2 Mean Transverse Joint Faulting

A review of the faulting performance data contained in the calibration database showed that the projects shortlisted for evaluation[a] had very low magnitudes of faulting (mean joint faulting ranged from 0 in to 0.04 in). This is typical for doweled JPCP with joint spacings less than 20 ft. Thus, a non-statistical approach was adopted for evaluating the MEPDG JPCP faulting model. The results of this evaluation are summarized in Table I-74.

Table I-74. Comparison of measured and predicted mean transverse joint faulting (numbers in table represent percentage of all data points).

Measured Transverse Joint Faulting, in	**MEPDG Predicted Transverse Joint Faulting, in**	
	0 - 0.03 in	0.03 – 0.05
0 - 0.03	94%	2%
0.03 – 0.05	4%	0%

*Total data points = 68.

[a] Note that candidate JPCP sections whose designs are not representative of current MoDOT design philosophy were excluded from this analysis. These include undoweled pavements and pavements with greater than 20 ft joint spacings or lane widths less than 12 ft.

As shown in Table I-74, a majority of the measured and predicted faulting data (approximately 94 percent) fell within the same grouping. For the levels of faulting evaluated in this analysis, the JPCP transverse joint faulting model predicted faulting with reasonable accuracy and without significant bias. Higher levels of faulting present on moderate to highly distressed pavements were not evaluated, as none of the projects included in the analysis experienced this level of faulting. Recalibration of this model is not imperative for its use in routine design. However, as additional data becomes available, the model validation exercise can be repeated to verify the findings and to improve the model accuracy.

5.3.2.3 JPCP Smoothness (IRI)

A statistical comparison of the measured and MEPDG predicted IRI using the nationally calibrated JPCP IRI model was performed. The results of this analysis are presented in Figure I-131 and

Table I-75. The following inferences can be drawn for this data:

- An adequate correlation exists between measured and MEPDG predicted IRI from the nationally calibrated smoothness model.
- An SEE less than that reported for the national MEPDG JPCP IRI model was obtained.
- A bias in the JPCP IRI predictions was observed as indicated by the results of testing hypotheses (1), (2), and (3) (see

Table I-75).

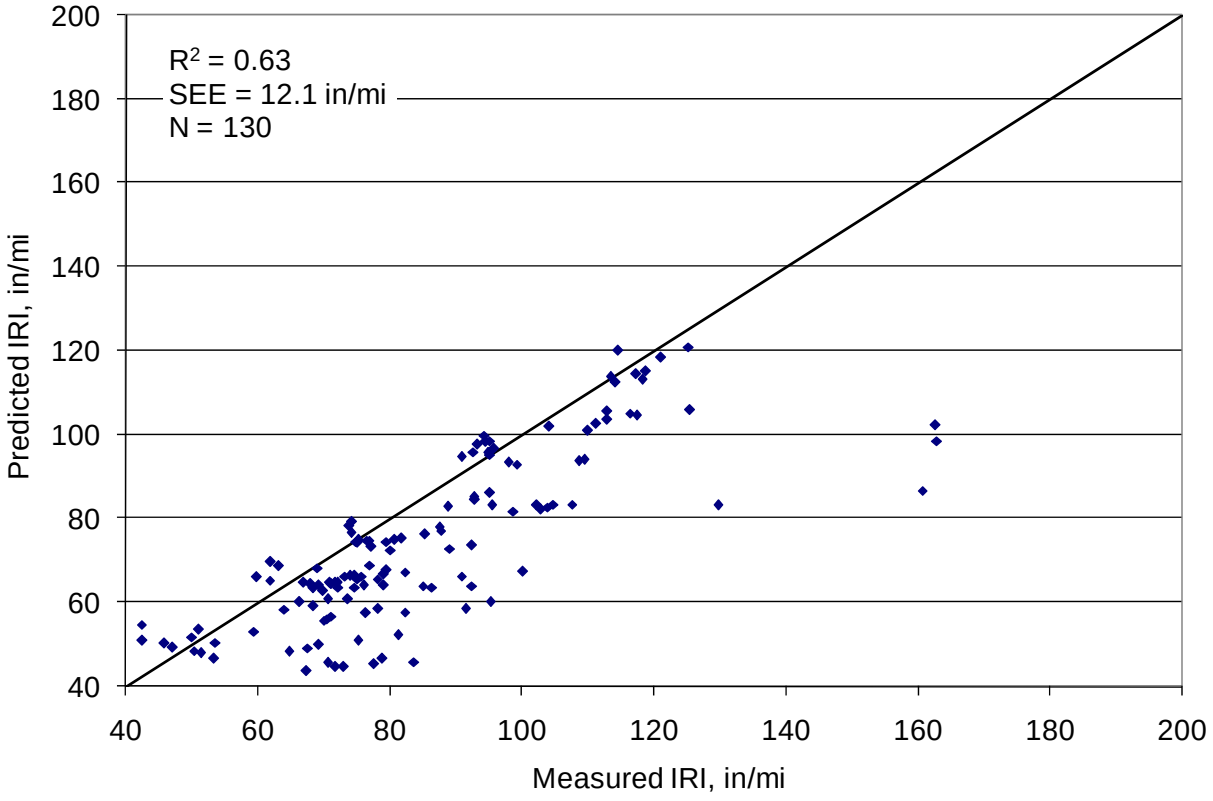


Figure I-131. Plot of measured versus MEPDG predicted JPCP IRI (nationally calibrated model).

Table I-75. Statistical comparison of measured and MEPDG predicted JPCP IRI (nationally calibrated model).

Goodness of Fit					
N = 130 R^2 = 0.63 SEE = 12.1in/mi					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	14.76	0.0005	6.55	22.97
(2) Ho: Slope = 1.0	1	1.0352	< 0.0001	0.8362	0.8858
(3) Ho: Measured IRI – MEPDG Predicted IRI = 0	75	—	< 0.0001	—	—

Although the model's predictive capacity was adequate, there was a need to recalibrate the national model to remove the significant bias identified. Recalibration involved modifying the original MEPDG JPCP IRI prediction model as follows:

$$IRI = IRI_I + C1*CRK + C2*SPALL + C3*TFAULT + C4*SF$$

Where:

IRI	=	Predicted IRI, in/mi
IRI_I	=	Initial smoothness measured as IRI, in/mi
CRK	=	Percent slabs with transverse cracks (all severities)
SPALL	=	Percentage of joints with spalling (medium and high severities)
TFAULT	=	Total joint faulting cumulated per mi, in
C1, C2, C3, C4	=	Recalibration coefficients
SF	=	Site factor (see below)

$$SF = AGE\ (1+0.5556*FI)\ (1+P_{200})*10^{-6}$$

where:

AGE	=	Pavement age, yr.
FI	=	Freezing index, °F-days.
P_{200}	=	Percent subgrade material passing No. 200 sieve.

As shown in equations above, all four model coefficients can be modified as needed to improve predicted JPCP IRI by removing bias.

Recalibration also involved reviewing the measured and predicted JPCP IRI to determine the possible sources of bias. The review indicated that there was no obvious source of bias.

Recalibration was performed using all the selected projects described earlier. The recalibrated model, including new model coefficients, is:

$$IRI = IRI_I + 0.82*CRK + 1.17*SPALL + 1.43*TFAULT + 66.8*SF$$

Where all variables are as previously defined.

A statistical comparison of measured and predicted JPCP IRI for the recalibrated model was done to determine recalibrated model prediction capacity and accuracy. The results are presented in Figure I-132 and

Table I-76 and indicate the following:

- A very good correlation exists between the measured and predicted IRI from the recalibrated JPCP IRI model.
- An SEE approximately equal to the nationally calibrated MEPDG JPCP IRI model was obtained.
- No significant levels of bias as indicated by the results of testing for hypotheses (1), (2), and (3) in

Table I-76.

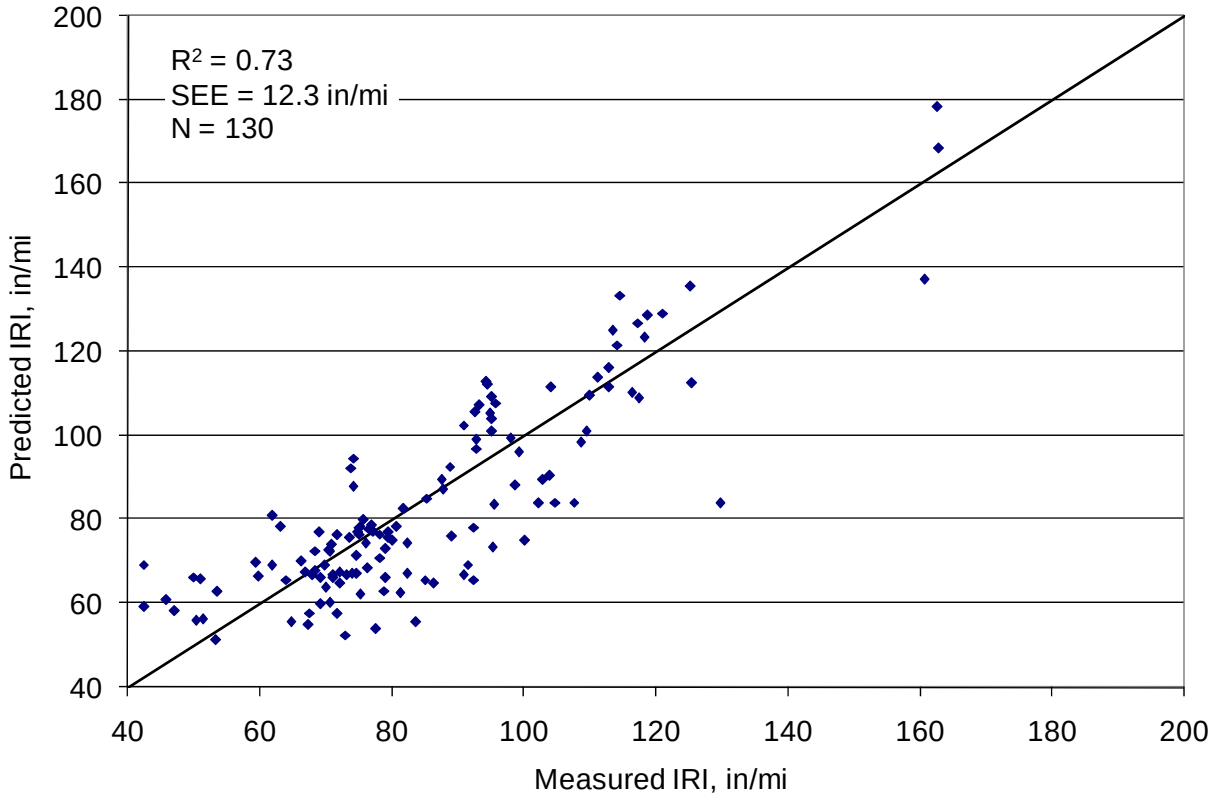


Figure I-132. Plot of measured versus predicted IRI using the locally calibrated JPCP IRI model.

Table I-76. Statistical comparison of measured and predicted IRI from the locally calibrated JPCP IRI model.

Goodness of Fit					
N = 130 R^2 = 0.73 SEE = 12.3in/mi					
Hypothesis Testing					
Hypothesis	**DF**	**Parameter Estimate**	**p-value (Pr > |t|)**	**95 Percent Confidence Limits**	
(1) Ho: Intercept = 0	1	7.614	0.073	-0.72	15.94
(2) Ho: Slope = 1.0	1	0.977	0.0645	0.953	1.001
(3) Ho: Measured Rutting – MEPDG Predicted Rutting = 0	130	—	0.1831	—	—

A limited sensitivity analysis was performed to determine the reasonableness of the locally calibrated JPCP IRI model. A plot showing the changes in predicted IRI for different levels of model inputs is presented in Figure I-133. The information presented confirmed that the sensitivity of the locally calibrated IRI model for this pavement type was similar to the nationally calibrated model. However, the magnitude of the predicted IRI was higher overall for the locally calibrated model.

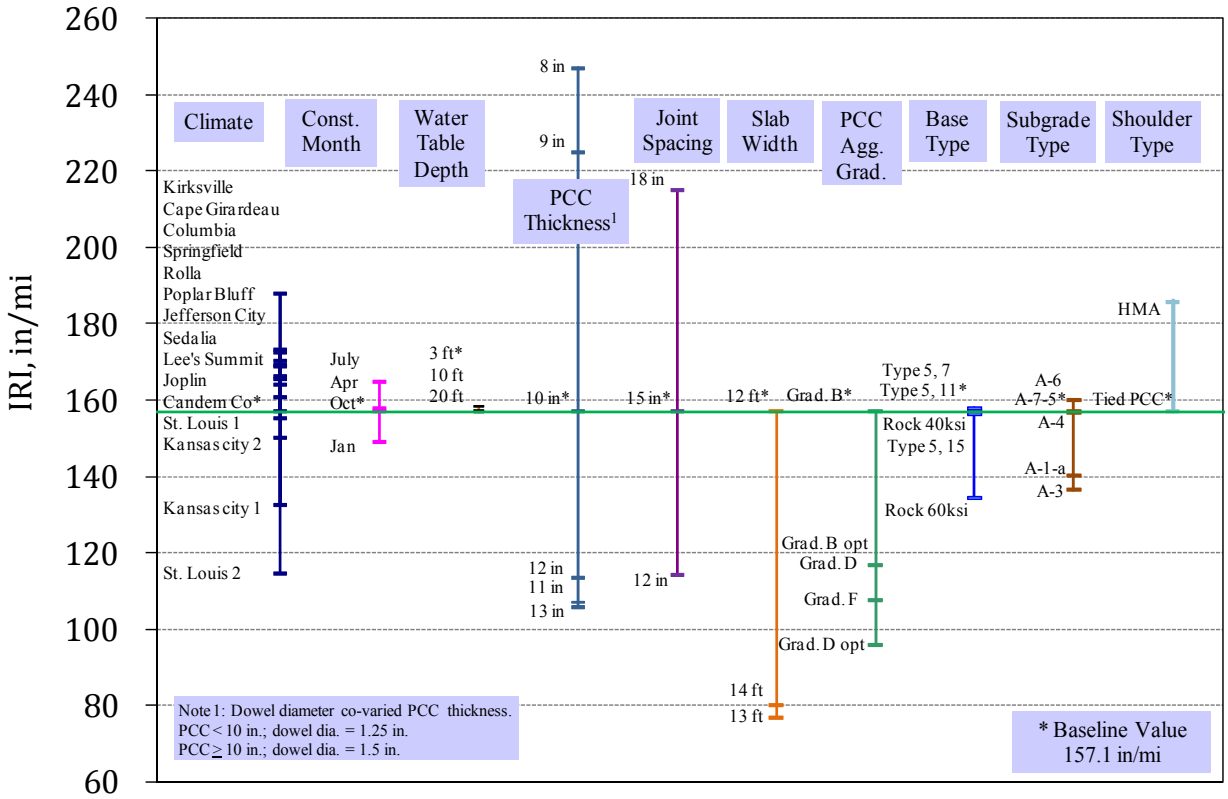


Figure I-133. Plot showing sensitivity analysis results of locally calibrated JPCP IRI model.

CHAPTER 6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY OF FINDINGS AND CONCLUSIONS

The Missouri State highway system consists of over 71,000 lane-miles of pavement. Functional classes of pavements represented on the system range from low-volume rural collectors to multi-lane, high-volume urban Interstates. A breakdown of the functional classes according to pavement type shows that the NHS and arterial highways make up roughly 35 percent of system while carrying a majority of the traffic. Like many other States, Missouri is also dealing with an aging highway infrastructure, ever-increasing traffic demands, user expectations, and funding limitations. MoDOT is tackling these issues using innovative strategies that cover various aspects of project delivery, including innovative contracting, performance specifications, and the use of modern design and construction practices. It is in this context that MoDOT decided to adopt the MEPDG.

MoDOT's MEPDG implementation efforts began in 2004. The effort received continuous support from MoDOT's top management, internal and external stakeholders including several of MoDOT's business units and offices, industry, and academia. The overall flow of work on this effort followed that depicted in Figure I-134. The following sections summarize the implementation scope and the major findings of the study.

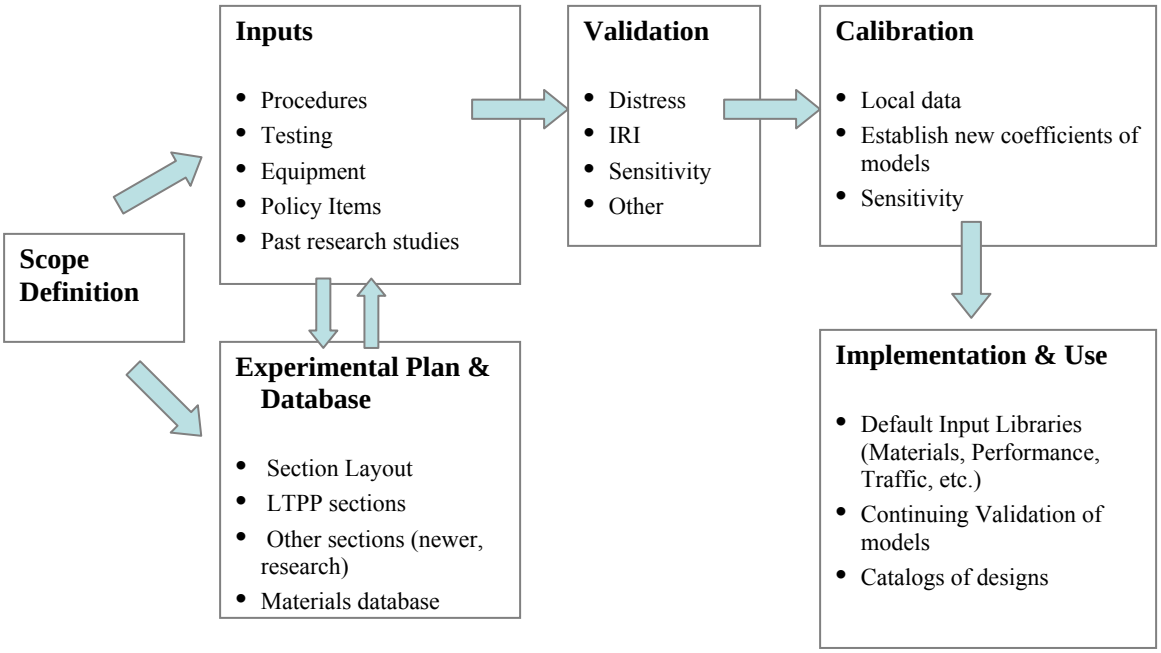


Figure I-134. Overview of the implementation process the MEPDG in Missouri.

6.1.1 Scope of the MEPDG Implementation Effort

The scope of the implementation effort focused primarily on the pavement types of interest to the MoDOT alternate bidding process and included:

- New pavements
  - New HMA deep strength (with a crushed stone or a large stone base).
  - New JPCP with dowel bars (with a crushed stone or large stone base).
- Rehabilitated pavements
  - HMA overlays of rubblized PCC pavements.
  - UBOL.

Other pavement types of interest that were investigated to the degree to which data were available included:

- HMA overlays of existing HMA pavements.
- HMA overlays of existing PCC pavements.

The MEPDG is also expected to be implemented on what MoDOT terms as their heavy and medium duty pavements (the classification of which is based on cumulative ESAL estimates over the design life).

6.1.2 Preparation of a Road Map and Assessment of Needs

Early in the project, a comprehensive road map was developed covering all the activities to be accomplished under this study and beyond. Simultaneously, MoDOT's current laboratory and field materials characterizing capabilities, traffic data collection program, and current and future design features of interest for each selected pavement type were discussed. Based on this discussion, gaps in current capabilities were identified and policy decisions were taken to allocate resources to fill the various gaps. The products of this needs assessment were:

- Definition of experimental factorials arranged by design and site features of interest for each pavement type of interest.
- Identification of Missouri-specific LTPP and pavement management system sections representing the designs of interest and a population of the experimental factorials.
- Development of detailed laboratory and field testing plans, data collection plans, and data review processes to support the validation and calibration effort, as well as future MEPDG implementation.
- Identification of a database structure and methodology to house all the relevant test and field data.
- Assessment of training needs and MEPDG roll-out plans.

6.1.3 Input Characterization Studies

Under this critical and time-consuming task, all the inputs needed to implement the MEPDG in Missouri and to perform local validation and calibration studies were assembled, processed, and analyzed. This involved the following activities:

1. Traffic data – review and analysis of Missouri's continuous WIM data.
2. Materials data – sampling and testing of prevailing MoDOT PCC, HMA, and unbound materials. Analysis of the data to confirm or reject MEPDG's nationally calibrated data (Level 3 defaults) and to establish input data libraries to support the MEPDG implementation into the future.
3. Field testing data – assembling, reviewing, and analyzing all field data needed to characterize MoDOT's pavement management sections. The subactivities included:

    3.1 Field coring and sampling of PCC and HMA materials.
    3.2 Testing of PCC cores to determine layer thicknesses and long-term compressive strength and CTE.
    3.3 Testing of HMA cores to determine lift thicknesses, densities, gradations, and binder contents by weight.
    3.4 FWD testing and backcalculation to determine subgrade M_r for HMA sections and k-value for PCC sections.
    3.5 DCP testing of unbound materials and computation of Penetration Index values by layer type.

<u>Laboratory Testing</u>

MoDOT purchased an SPT in 2004 to characterize their typical HMA mixes. Based on the laboratory testing plan developed as part of the needs assessment, MoDOT staff performed the necessary laboratory testing to determine E* and repeated load permanent deformation characteristics of field sampled mixes. Similarly, MoDOT built a CTE testing machine and performed a full suite of testing internally to characterize PCC mixes including strength, modulus, Poisson's ratio, and CTE.

Additionally, MoDOT contracted with MUST to characterize typical field sampled HMA mixes for creep compliance and IDT strength and unbound materials and typical soils for M_r and other index properties.

<u>Field Testing</u>

All field testing—including locating (using global positioning system [GPS] coordinates and linear referencing methods) and permanently marking 500-ft sample units used in local validation and calibration, physical sampling and coring, and FWD and DCP testing—also was performed by MoDOT staff. Manual surveys of distress data were performed by MoDOT staff in 2005 and 2007 using the LTPP Distress Identification Manual as a basis to evaluate the performance of the selected pavement management sections. The resulting crack maps and distress quantification tables are a part of the project database. Finally, MoDOT staff also

performed testing of HMA and PCC cores in the laboratory to determine the properties listed above.

Data Analysis

As indicated, all the input data assembled were processed and analyzed under this study to achieve the following goals:

- Develop a set of default material property values that can be used in pavement design process for the Level 1 inputs.
- Accept or reject the MEPDG default inputs and correlations.
- Provide data to perform a meaningful sensitivity analysis.
- Provide data to improve the quality of the local calibration process and reduce the error term for the predicted distresses.

The review process established the following main points:

- The MEDPG traffic default inputs for vehicle class distributions, load spectra, 24-hour truck counts, and axles per truck agree reasonably well with MoDOT processed WIM data from their RPA-I, UPA-I, and RPA-O functional class sites.
- The climate data seems to provide sufficient coverage. However, a critical review of the quality of the data and adding additional data (weather stations) from neighboring states is recommended in the medium- to long-term. Some distresses such as JPCP cracking and thermal cracking are extremely sensitive to the quality of climate data.
- MEPDG Level 2 dynamic modulus estimation equation predicts the E* reasonably well for MoDOT's HMA made with both modified and unmodified binder.
- Limited data available from the repeated load permanent deformation testing indicate that the MEPDG permanent deformation model coefficients are dependent on mix type. However, a more comprehensive testing program and a more detailed analysis of the data are needed before final conclusions can be drawn.
- The MEPDG Level 3 creep compliance estimates are significantly higher than those determined from Level 1 testing.
- The MEPDG PCC Level 3 inputs for strength and modulus are in reasonable agreement with MoDOT's data. Moreover, the strength gain for typical MoDOT mixes is estimated relatively conservatively, but adequately, by the MEPDG.
- The MEPDG compressive strength to flexural strength relationship was considered adequate for MoDOT mixes. However, a new relationship between the compressive strength and elastic modulus was proposed.
- Laboratory CTE values for typical MoDOT mixes were lower than expected for the aggregate types tested, based on a comparison with national averages. However, long-term CTEs determined from field cores were higher for the same mix type. Since it could not be determined if this is an age related effect or testing variability, it is recommended that MoDOT verify their CTE test results with an independent laboratory. In the interim, the CTE values from field cores are recommended for use.
- MoDOT-specific coefficients for the generalized constitutive model to predict M_r at optimum moisture condition and maximum dry density as a function of stress state in the

granular base and subgrade soils were developed. These coefficients were used in deriving further guidance on the appropriate resilient moduli to be used in design.
- The material modulus estimates for the large stone bases were determined through backcalculation. It is recommended that this layer be treated as constant modulus material in design.
- Typical soil gradation and index properties to support climatic effects modeling at Level 2 were identified and catalogued for future use.

6.1.3 MEPDG Database

An MS Access database, modeled after the LTPP information management system database, was developed to house all the raw and processed data. The database is segmented at two levels—library information and calibration information. The library information tables contain details of all the laboratory studies performed by MoDOT and its contractors that relate to the MEPDG. The calibration database contains section-specific information on all MoDOT PMS sections including inventory, identification, sampling, laboratory testing, and field testing data (including data from nondestructive testing and manual surveys). It is expected that this database will serve as a satellite database to continue calibration efforts in Missouri into the future.

6.1.4 Sensitivity Studies

Model sensitivity studies were performed to study the effects of changing a design input on the distress and IRI model predictions for new HMA and JPCP pavement sections. As a first step in this analysis, typical MoDOT pavement sections were established as baseline designs. The MEPDG inputs for the baseline design and ranges of each input to be varied during the analysis were then configured using the data from the MoDOT specific input libraries. The sensitivity of key inputs on distress and IRI was studied. A limited amount of interaction effects analyses were also conducted. e.g., PCC materials interaction and PCC thickness and dowel size interaction.

Based on the information derived, the most significant inputs for JPCP design were found to be:

- PCC slab thickness in combination with joint load transfer (dowels and dowel diameter).
- Joint spacing.
- PCC mix properties including CTE, strength, and modulus (there is a need to test for these properties from a single batch of mix).
- Lane edge support (widened slab and tied shoulder)
- Base type and subgrade type.
- Month of construction.
- Climate.
- Traffic (truck weights and monthly adjustment factors).

The most significant inputs for HMA pavement design were found to be:

- HMA thickness.
- HMA binder grade.
- HMA binder and air void content (and stiffness or E*).

- Climate.
- Stiffness of base and subgrade materials.
- Traffic (truck weights and monthly adjustment factors).

The information was helpful in developing hypotheses and performing model validation and calibration.

6.1.5 Local Validation and Calibration

The dataset that populated the experimental factorial matrices for calibration and validation of the MEPDG models was comprised of the following sections and section characteristics for each pavement type.

- New JPCP
    - 25 MoDOT PMS sections; 55 500-ft sample units
        - Sections between 10 to 30 years of age.
        - Median age was 12 years.
    - 7 LTPP 500-ft sections (not used in global calibration)
        - Sections between 10 and 50 years of age.
        - Crushed stone base sections predominate.
        - Widened and tied PCC sections predominate (modern designs)
- New HMA sections
    - 6 MODOT sections; 22 500-ft sample units.
        - Sections between 5 to 10 years of age in 2007.
    - 14 LTPP 500-ft sections (not used in global calibration)
        - Sections between 10 and 35 years of age.
        - Crushed stone base sections with Marshall and Superpave mixes predominate.
        - Most sections are > 8 inches thick.
- HMA/HMA sections
    - 11 LTPP 500-ft sections.
- HMA/PCC sections
    - 5 LTPP 500-ft sections.
- Rubblized HMA pavements
    - 4 LTPP 500-ft sections.
- UBOL sections
    - 5 500-ft MoDOT PMS sections.

Based on several data availability and adequacy reviews, some of the identified sections were removed from consideration. A preliminary assessment of the data indicates the following salient points:

- Many distresses of interest to this study are either non-existing or low on the selected sections (particularly the newer MoDOT PMS sections) with the exception of a few older LTPP JPCP and HMA sections. This finding limits the statistical comparisons of the

MoDOT validation effort with the national effort even if the predictions match field performance.
- Very few rubblized, UBOL, and HMA/JPCP sections were available for detailed investigations.

The local validation and calibration was performed on the following models:

- New HMA, Rubblized HMA, and HMA/HMA
    - Alligator (bottom-up fatigue) cracking.
    - Rutting.
    - Thermal (temperature) cracking.
    - IRI

- New HMA/JPCP
    - Rutting.
    - Thermal (temperature) cracking.
    - IRI

- New JPCP and UBOL
    - Faulting.
    - Mid-panel transverse (bottom-up and top-down) cracking.
    - IRI

The following steps were executed to perform the validation/calibration effort:

1. Assemble best possible input data for each sample unit
    a) Backcast initial IRI from historical IRI data for each section collected from the ARAN vehicle (post-2002 data only).
    b) Backcast initial AADTT and compute growth rate from historical traffic data.
    c) Backcast PCC strength/modulus from long-term data for JPCP and UBOL (overlays only).
    d) Assume MODOT specific defaults where project specific data is not available.
2. Execute MEPDG runs.
3. Examine predicted versus measured distress plots.
4. Perform validation to assess bias in predictions and the model's predictive ability and accuracy.
5. Make suitable engineering and statistical analyses to calibrate models and to reduce bias and improve the model's predictive capability and accuracy.

The broad conclusions from the effort were that:

- The lack of moderately to highly deteriorated pavements close to the current design practices, limits the application of the MEPDG.
- For new HMA pavements and HMA overlaid HMA, intact PCC, and fractured PCC pavements, rutting was overpredicted by the MEPDG model and was locally calibrated. Thermal cracking was consistently underpredicted. It is recommended that, to

circumvent this problem, creep compliance and IDT strength estimates developed for typical MoDOT mixed as part of this project be used in conjunction with the locally calibrated thermal cracking model. For some of the older LTPP sections, fatigue cracking predictions were a concern. However, as a whole, the measured and predicted fatigue cracking were close to 0 percent. Therefore, this model was not recalibrated and the nationally calibrated model is recommended for use. The IRI model was recalibrated separately for new HMA pavements and HMA overlaid pavements.

- For new JPCP pavements, very low distresses were measured and predicted for both cracking and faulting. The exceptions were pavement sections with thin slabs and long joint spacings for which both the cracking and faulting were overpredicted. However, it was not recalibrated on the data from these sections. The nationally calibrated cracking and joint faulting models are recommended for use. The IRI prediction model was recalibrated.

6.2 RECOMMENDATIONS FOR MEDPG IMPLEMENTATION IN MODOT

Improved model validation using a more mature local (MoDOT PMS) data set in terms of pavement age and distresses remains a top issue. It is suggested that MoDOT continue monitoring the PMS sections selected as part of this project and other nominated sections for at least another 10 years. The data from these sections can then be used to periodically recalibrate the models in the future to make them more accurate for Missouri conditions.

In the interim, the following recommendations are made relative to use of the MEPDG in pavement design:

- The MEPDG is ready for use by MoDOT for new HMA and JPCP pavements, subject to the recommended inputs and local calibration provided from this study. There are, however, a number of additional activities needed on the short and long terms as recommended in this study.
- Some of the critical input related needs (e.g., verification of lab tested CTE values, estimation of design M_r from laboratory test data, etc.) need to be addressed and included in the design manual.
- The MEPDG models that were found to be unbiased can be used for Missouri conditions.
- The MEPDG models that were found to be biased (over or under prediction) were recalibrated using Missouri data. These models provide the best available prediction for modern Missouri asphalt and concrete pavements. The calibration factors in the MEPDG should be modified to reflect these new calibrations.
- Use of the MEPDG using the input library developed through this project and within the limitations noted in this report should be adequate for the next few years. This recommendation is based on the fact that the MEPDG models predict the current performance of most of MoDOT's modern pavement design well (lower amounts of distress measured and predicted). Since the design criteria for these pavements are relatively low values anyways, the MEPDG models appear to be reasonable within this range.

- A design manual needs to be developed based on the findings of this study that guides the user through input selection and proper execution of the MEPDG that includes the new distress/IRI model calibration coefficients. The design manual will include a sensitivity analysis for the key inputs to show to the user how it affects performance. This will give the designer more confidence in selection of an input.

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APPENDIX A

SITE-BY-SITE TRAFFIC DATA ANALYSIS FOR MISSOURI WEIGH IN MOTION SITES

SITE 29_182-1-1 (SHRP_ID NA)

STEP 1 CREATE NORMALIZED VEHICLE CLASS DISTRIBUTION

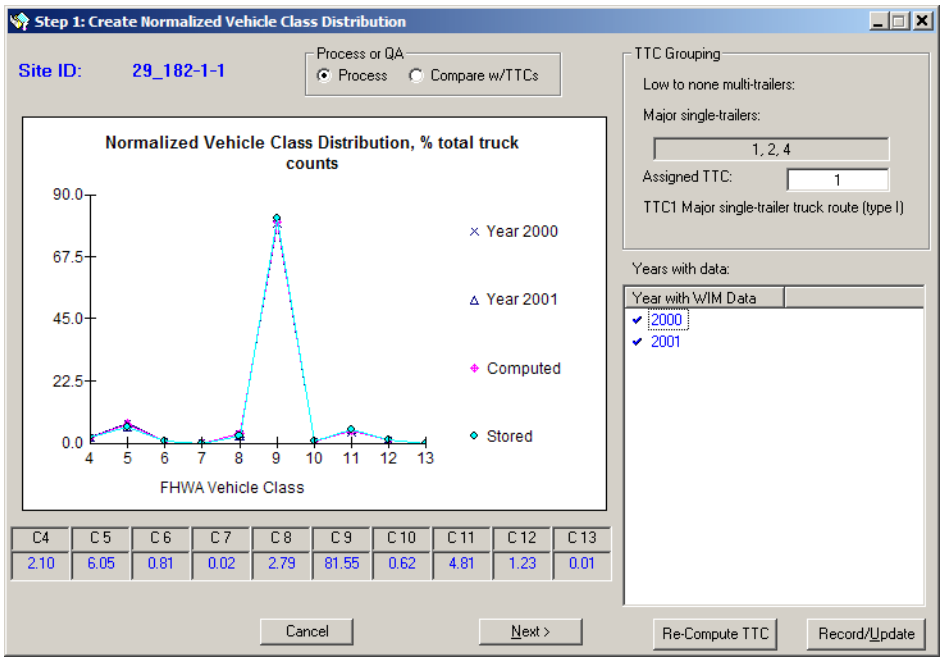


Figure A.1 Normalized vehicle class distribution for site 29_182-1-1 (SHRP_ID NA)

STEP 2 CREATE NORMALIZE LOAD SPECTRA BY VEHICLE CLASS AND FOR EACH MONTH

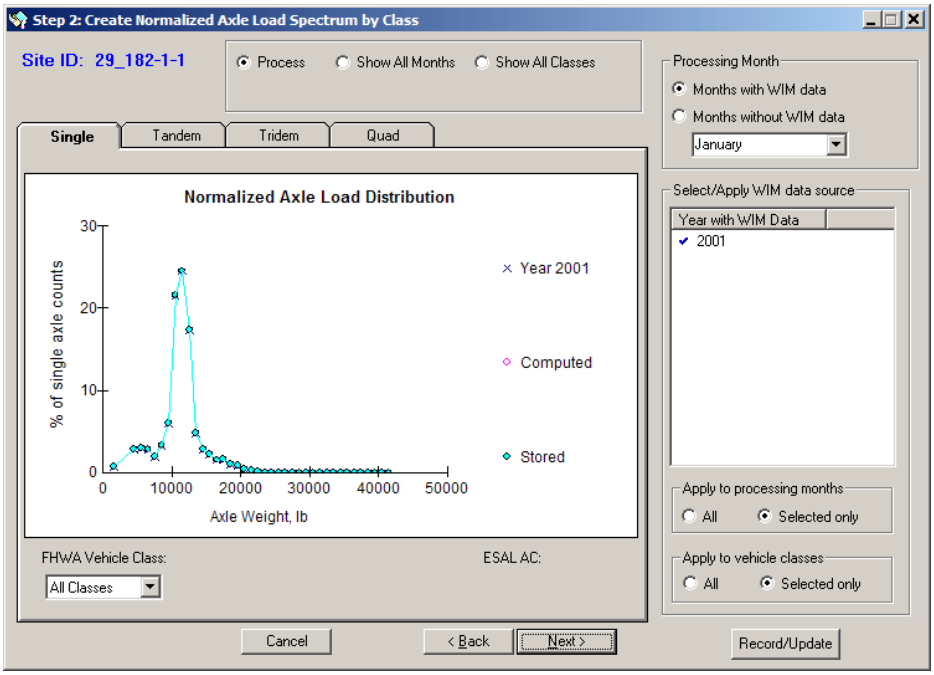


Figure A.2 Normalized single axle spectra for all classes for site 29_182-1-1 (SHRP_ID NA)

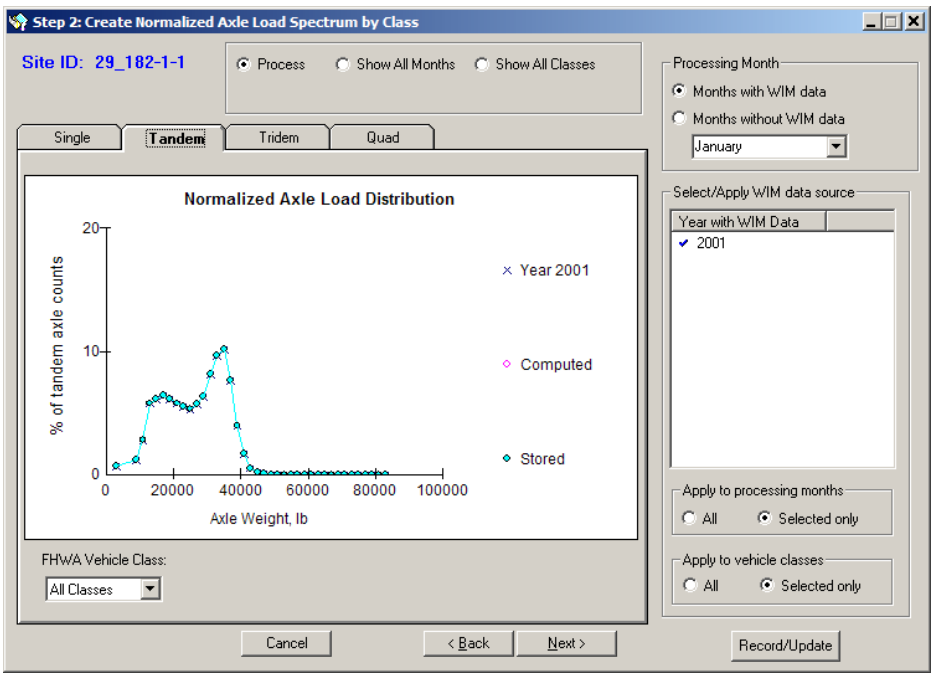


Figure A.3 Normalized tandem axle spectra for all classes for site 29_182-1-1 (SHRP_ID NA)

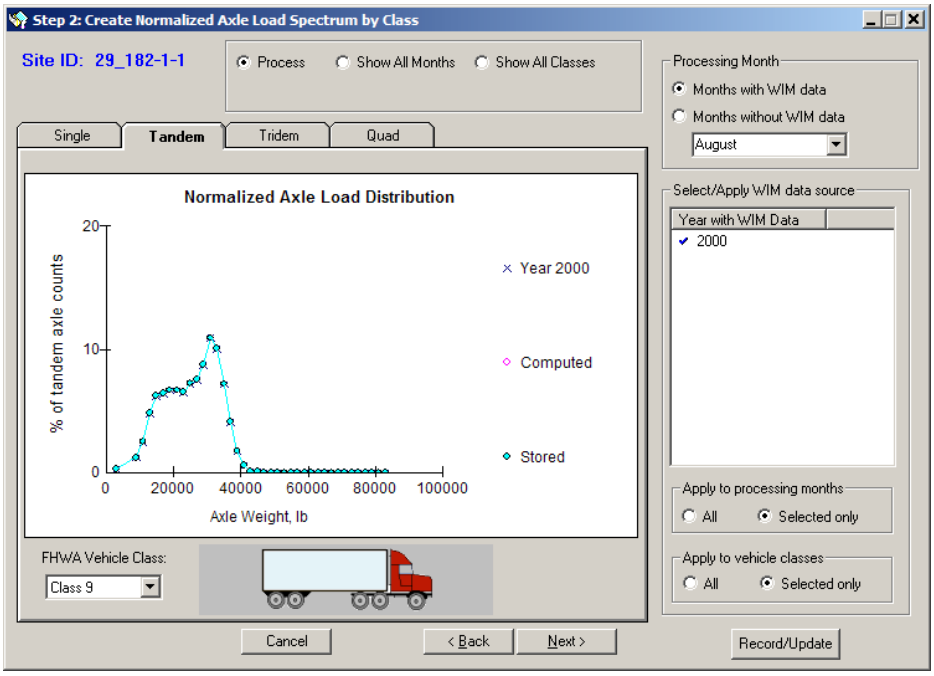
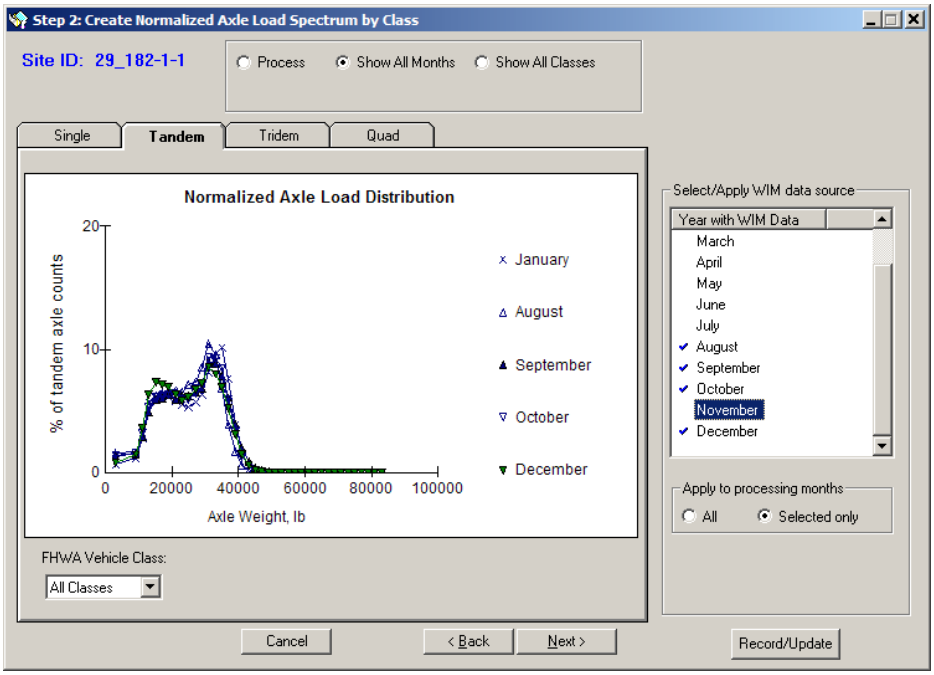


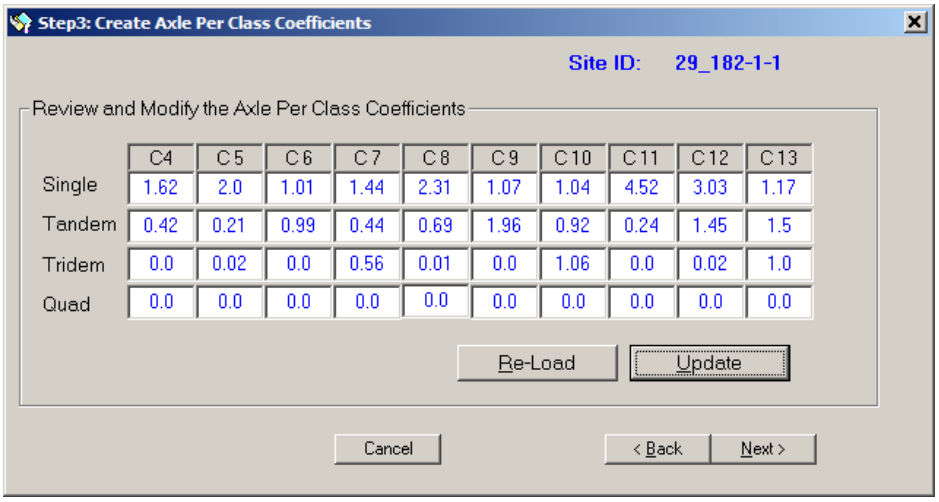
Figure A.4 Normalized tandem axle spectra by class for site 29_182-1-1 (SHRP_ID NA)



Comment: Consistent from month to month and year to year (2000 vs. 2001) – good data!

Figure A.5 Normalized tandem axle spectra for all classes for site 29_182-1-1 (SHRP_ID NA)

STEP 3 CREATE AXLE PER CLASS COEFFICIENTS



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.62	2.0	1.01	1.44	2.31	1.07	1.04	4.52	3.03	1.17
Tandem	0.42	0.21	0.99	0.44	0.69	1.96	0.92	0.24	1.45	1.5
Tridem	0.0	0.02	0.0	0.56	0.01	0.0	1.06	0.0	0.02	1.0
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.6 Axle class coefficients for site 29_182-1-1 (SHRP_ID NA)

STEP 4 DETERMINE BASE TRUCK VOLUME AND ANNUAL GROWTH

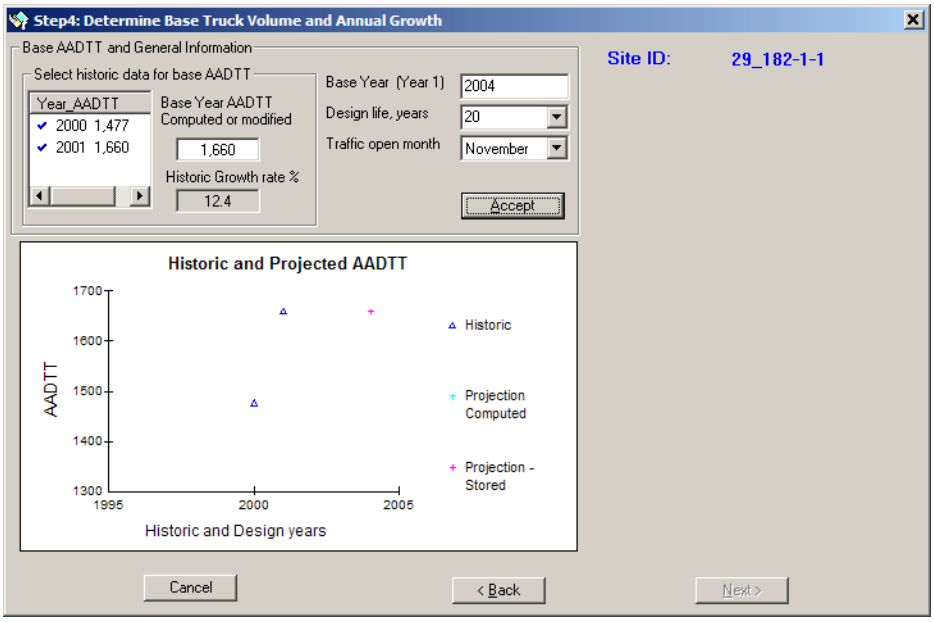


Figure A.7 Base truck volume and annual growth for site 29_182-1-1 (SHRP_ID NA)

STEP 5 DETERMINE MONTHLY TRUCK VOLUME ADJUSTMENT COEFFICIENTS

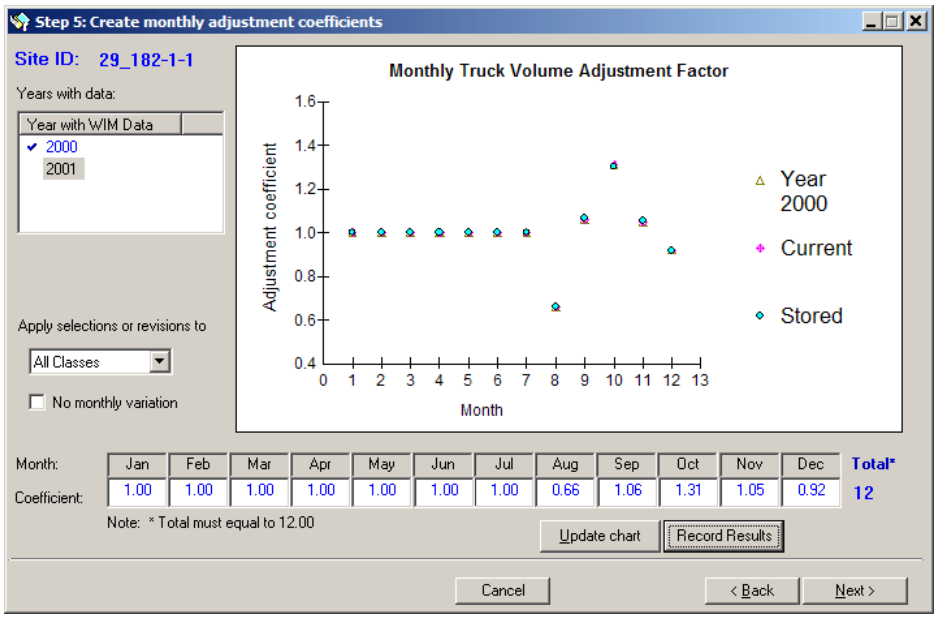
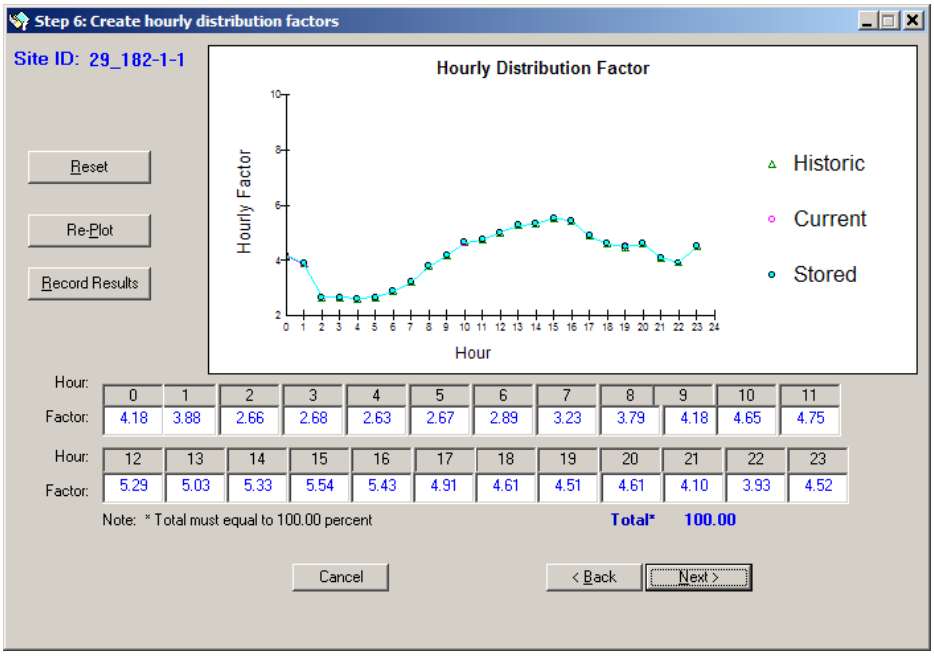


Figure A.8 Monthly truck volume adjustment coefficients for site 29_182-1-1 (SHRP_ID NA)

STEP 6 CREATE HOURLY TRUCK VOLUME DISTRIBUTION FACTORS



Comment: greatest through the day from about 9am to 11pm

Figure A.9 Hourly truck distribution factors for site 29_182-1-1 (SHRP_ID NA)

SITE 29_182-5-1 (SHRP_ID NA)

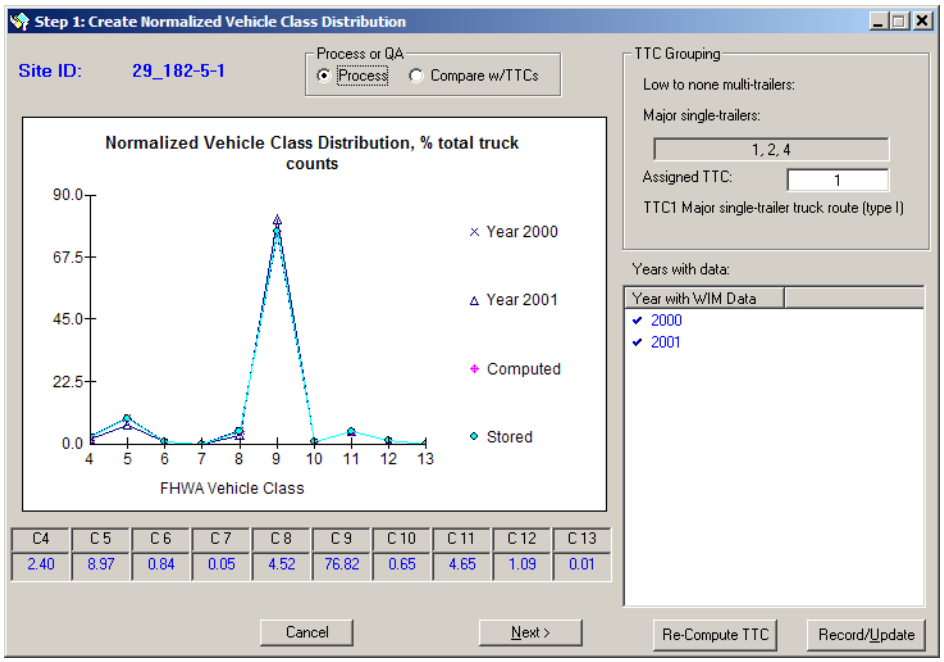


Figure A.10 Normalized vehicle class distribution for site 29_182-5-1 (SHRP_ID NA)

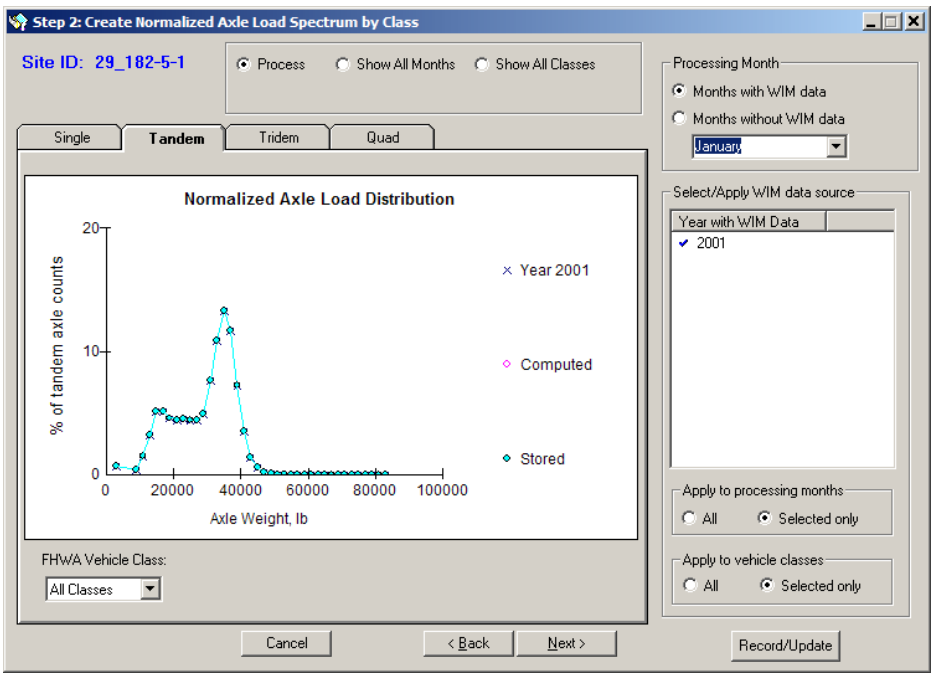
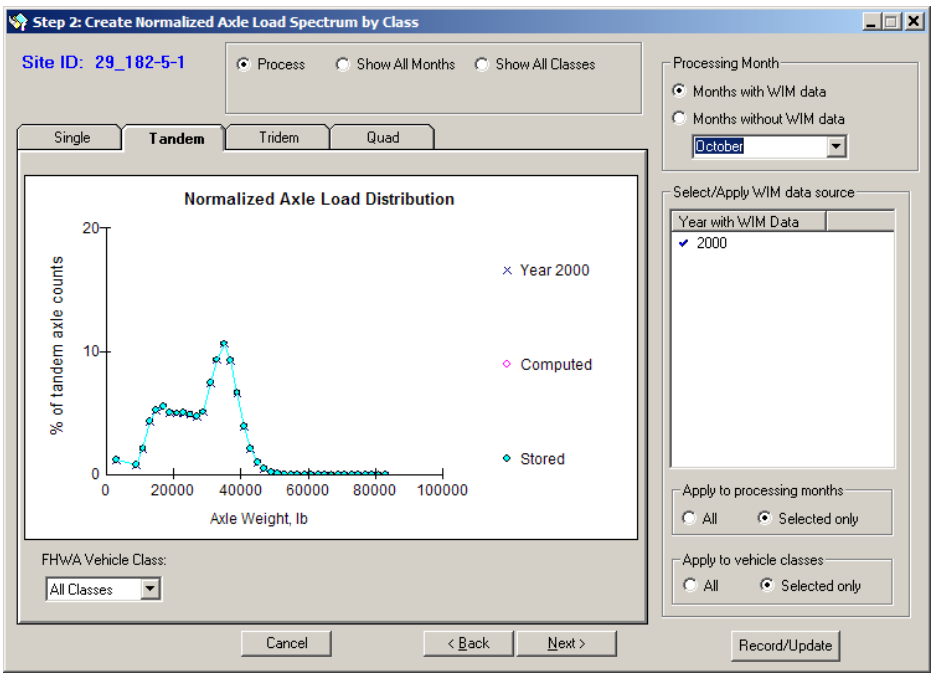


Figure A.11 Normalized tandem axle spectra by class for site 29_182-5-1 (SHRP_ID NA)



Comment; compare by month, January and October

Figure A.12 Normalized tandem axle spectra by class for site 29_182-5-1 (SHRP_ID NA)

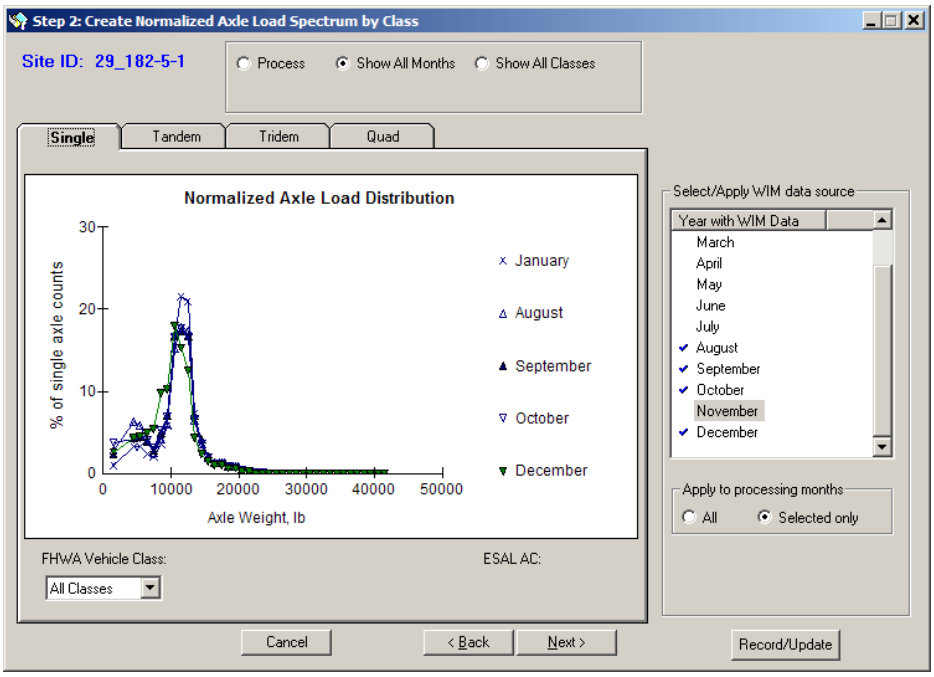


Figure A.13 Normalized single axle spectra for all classes for site 29_182-5-1 (SHRP_ID NA)

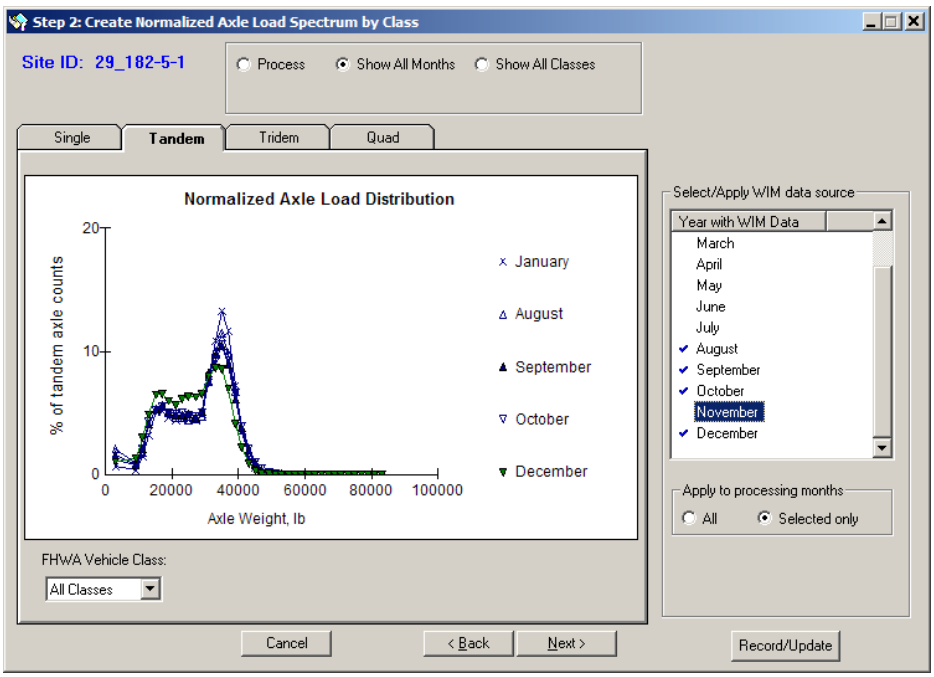


Figure A.14 Normalized tandem axle spectra for all classes for site 29_182-5-1 (SHRP_ID NA)

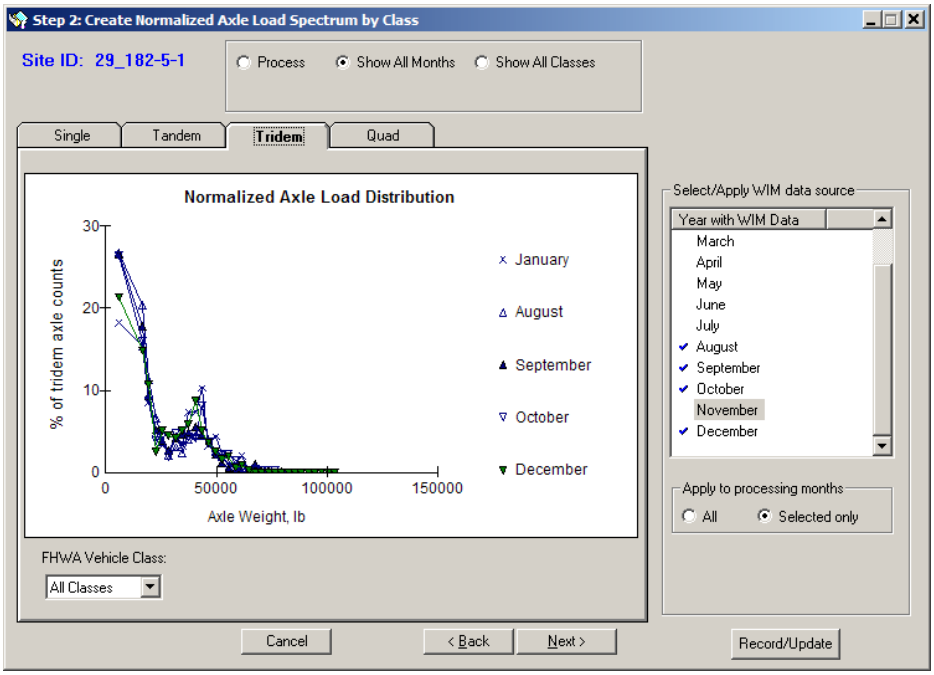
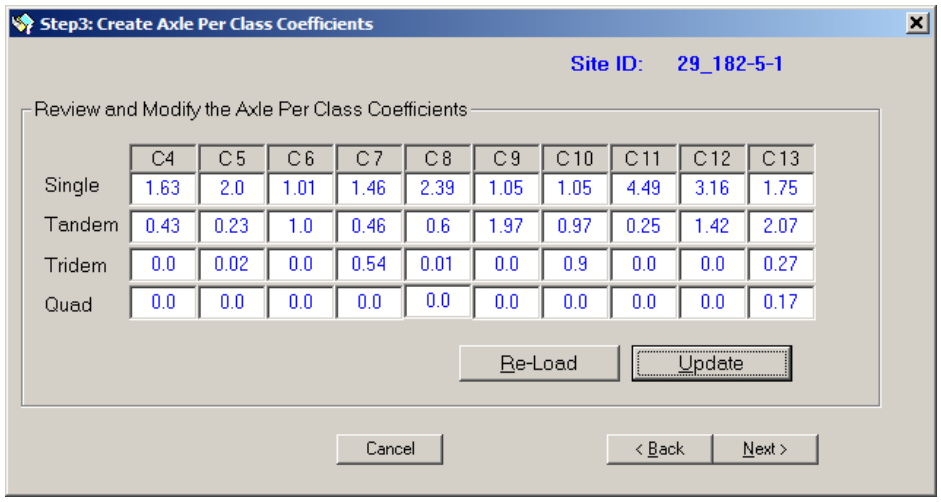


Figure A.15 Normalized tridem axle spectra for all classes for site 29_182-5-1 (SHRP_ID NA)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.63	2.0	1.01	1.46	2.39	1.05	1.05	4.49	3.16	1.75
Tandem	0.43	0.23	1.0	0.46	0.6	1.97	0.97	0.25	1.42	2.07
Tridem	0.0	0.02	0.0	0.54	0.01	0.0	0.9	0.0	0.0	0.27
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.17

Figure A.16 Axle class coefficients for site 29_182-5-1 (SHRP_ID NA)

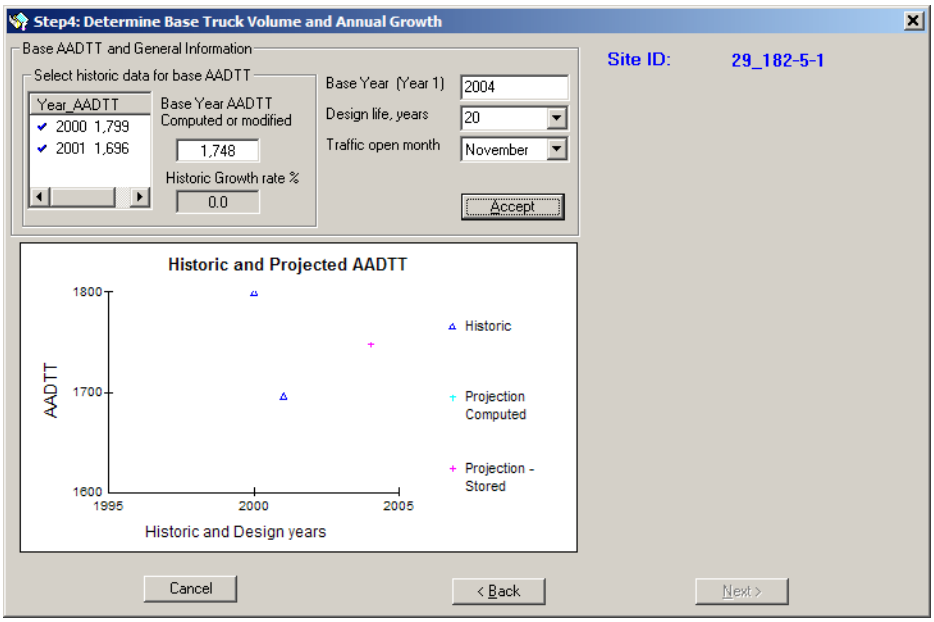


Figure A.17 Base truck volume and annual growth for site 29_182-5-1 (SHRP_ID NA)

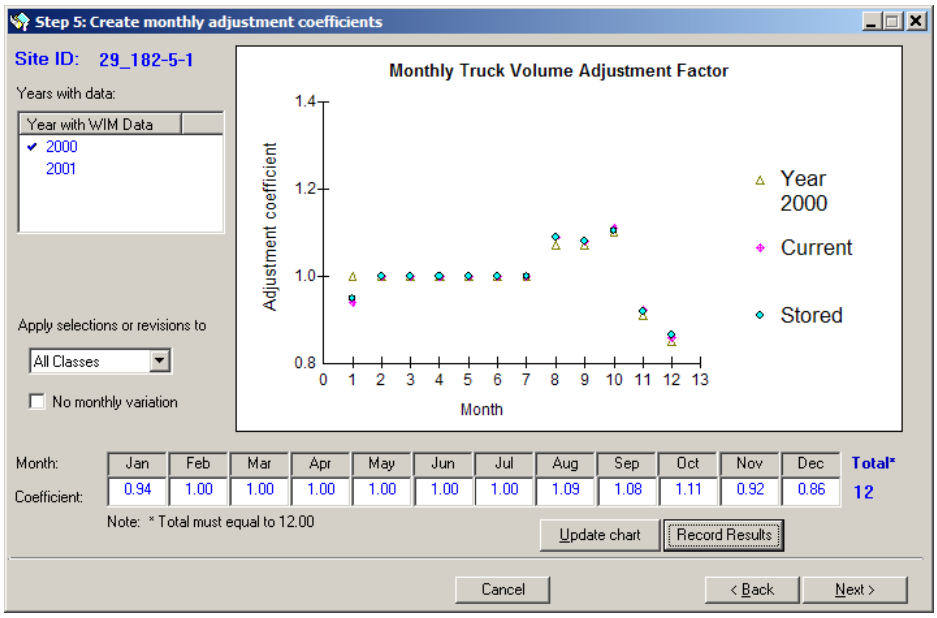
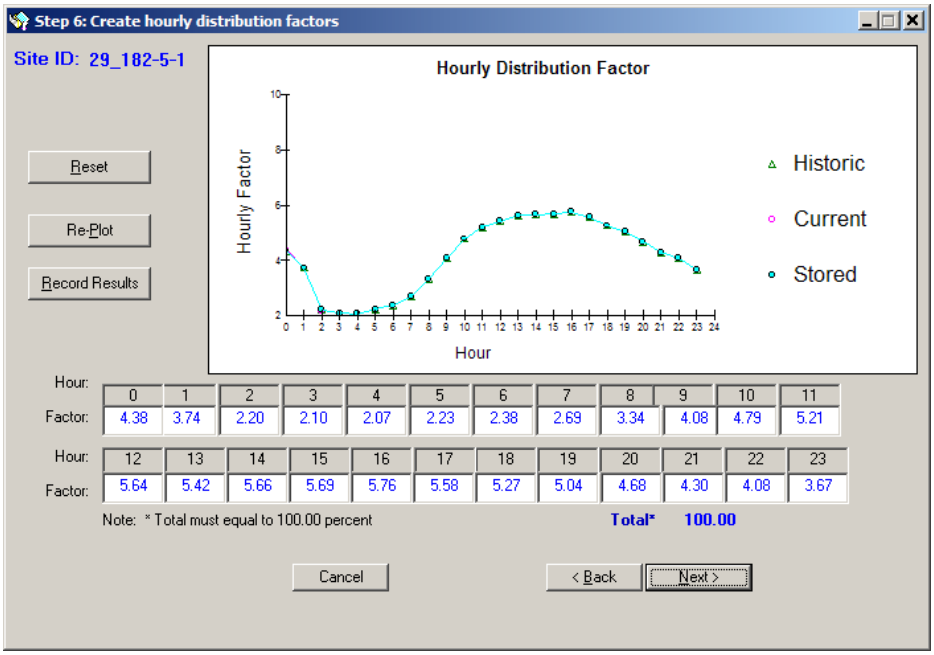


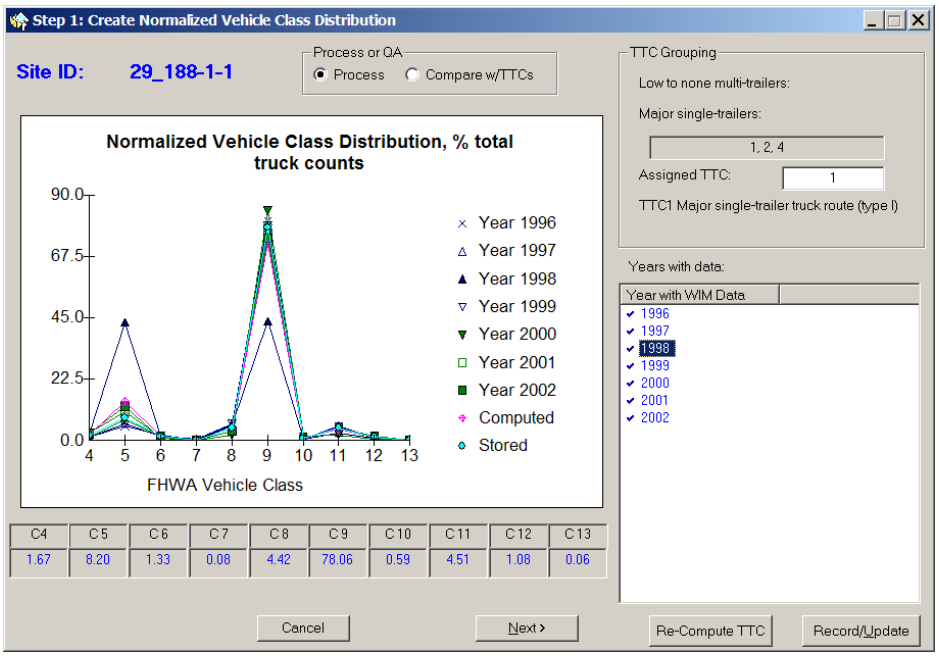
Figure A.18 Monthly truck volume adjustment coefficients for site 29_182-5-1 (SHRP_ID NA)



Comment: Mid-day: 10am to 5pm

Figure A.19 Hourly truck distribution factors for site 29_182-5-1 (SHRP_ID NA)

SITE 29_188-1-1 (SHRP_ID 6015)



Comment: Need to get rid of 1998.

Figure A.20 Normalized vehicle class distribution for site 29_188-1-1 (SHRP_ID 6015)

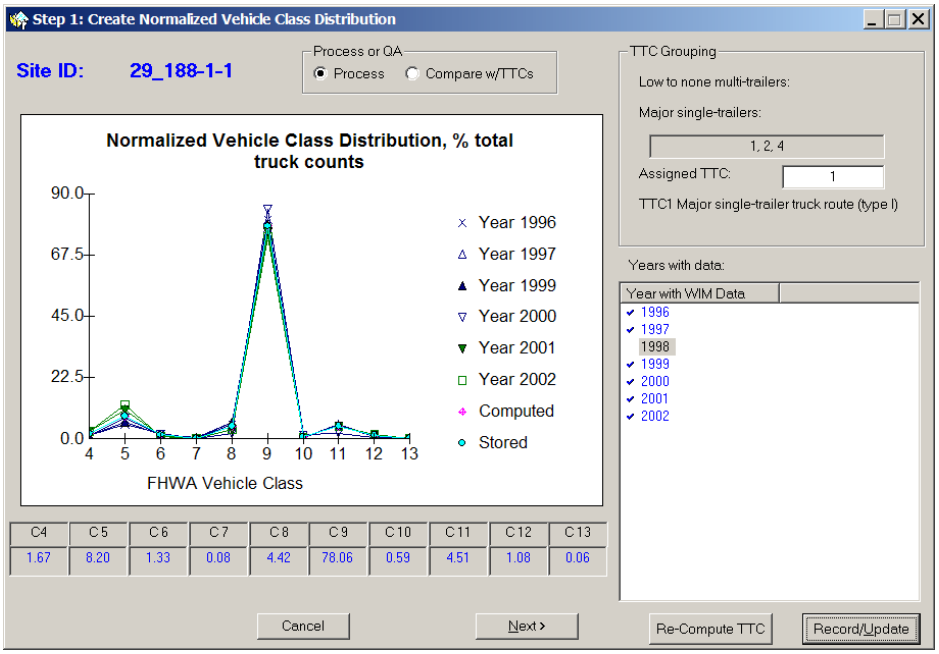


Figure A.21 Normalized vehicle class distribution (excluding 1998) for site 29_188-1-1 (SHRP_ID 6015)

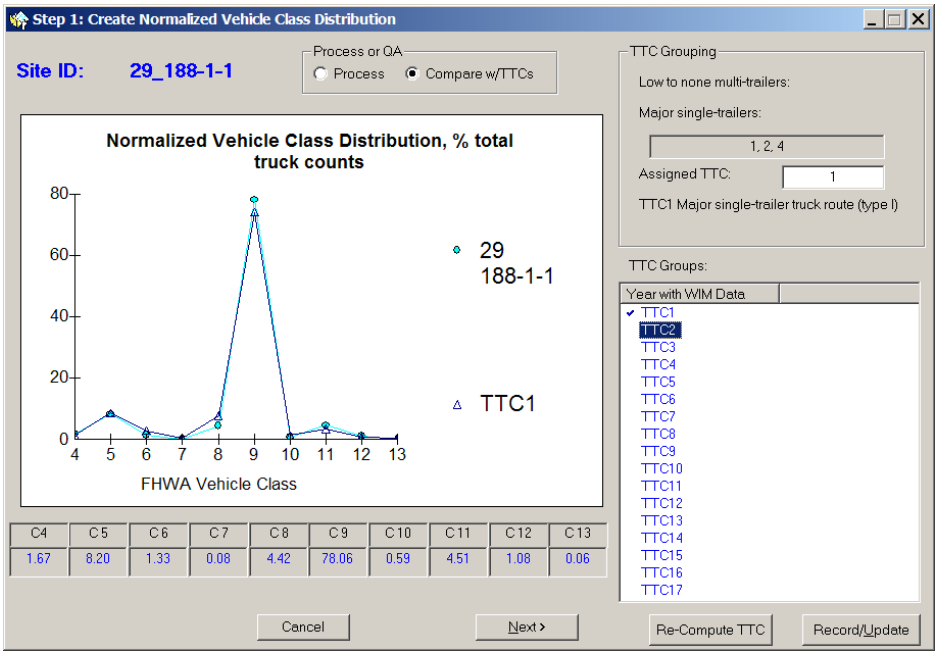


Figure A.22 Compare normalized vehicle class distribution with TTCs for site 29_188-1-1 (SHRP_ID 6015)

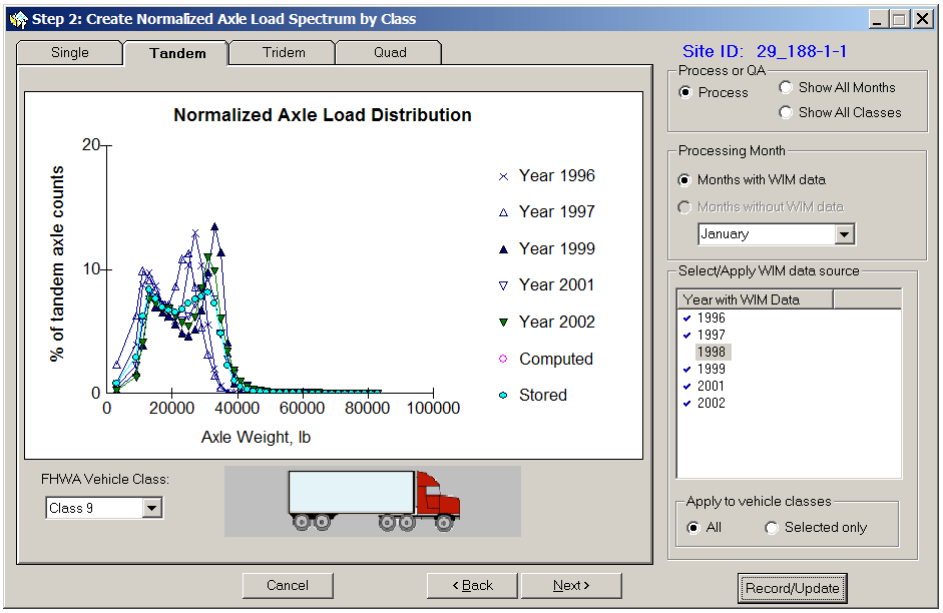


Figure A.23 Normalized tandem axle spectra by class for site 29_188-1-1 (SHRP_ID 6015)

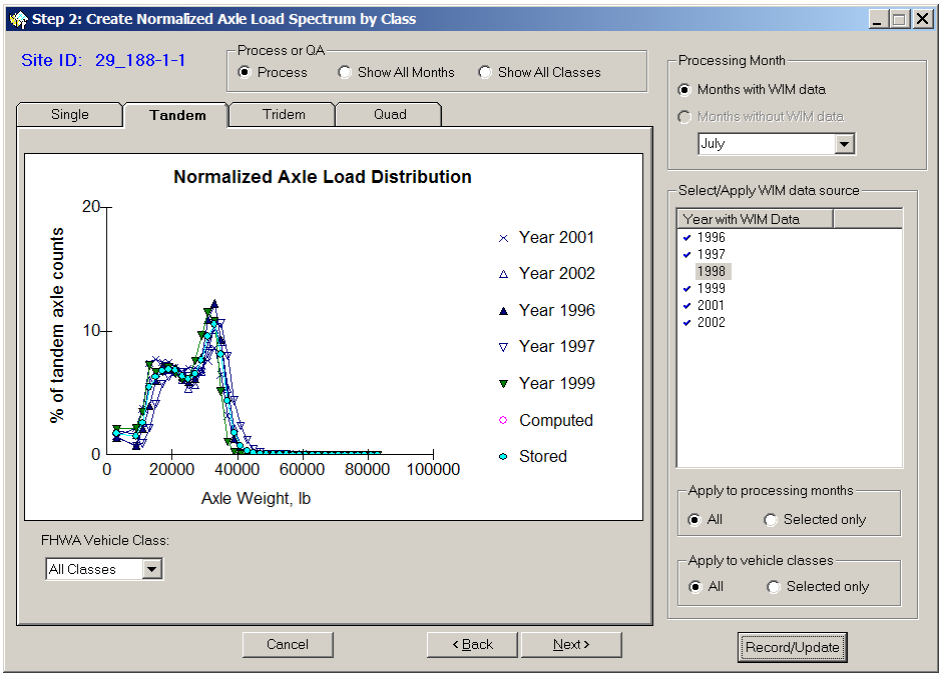


Figure A.24 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

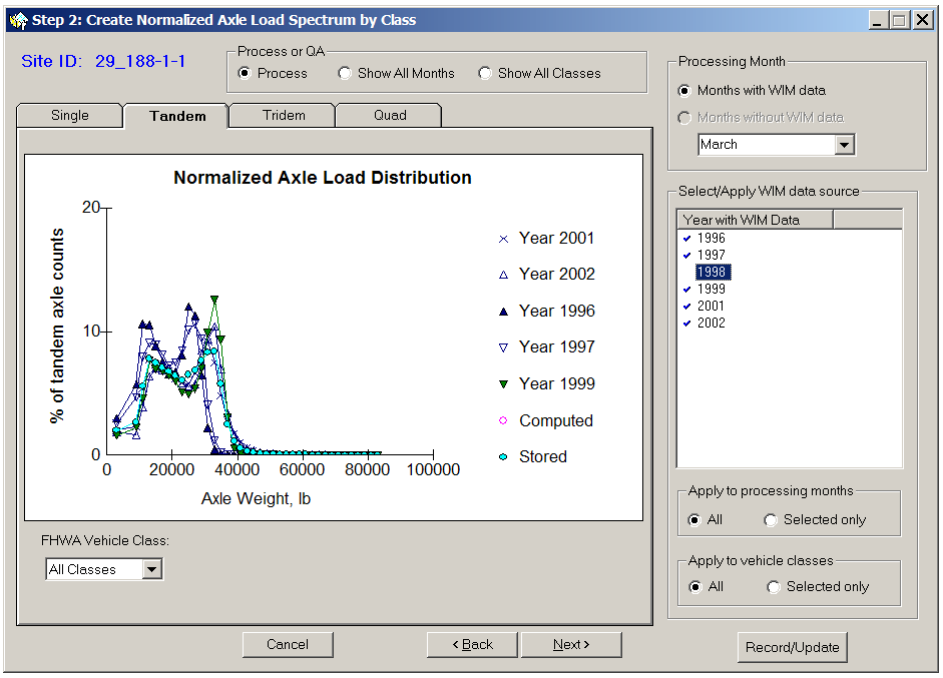


Figure A.25 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

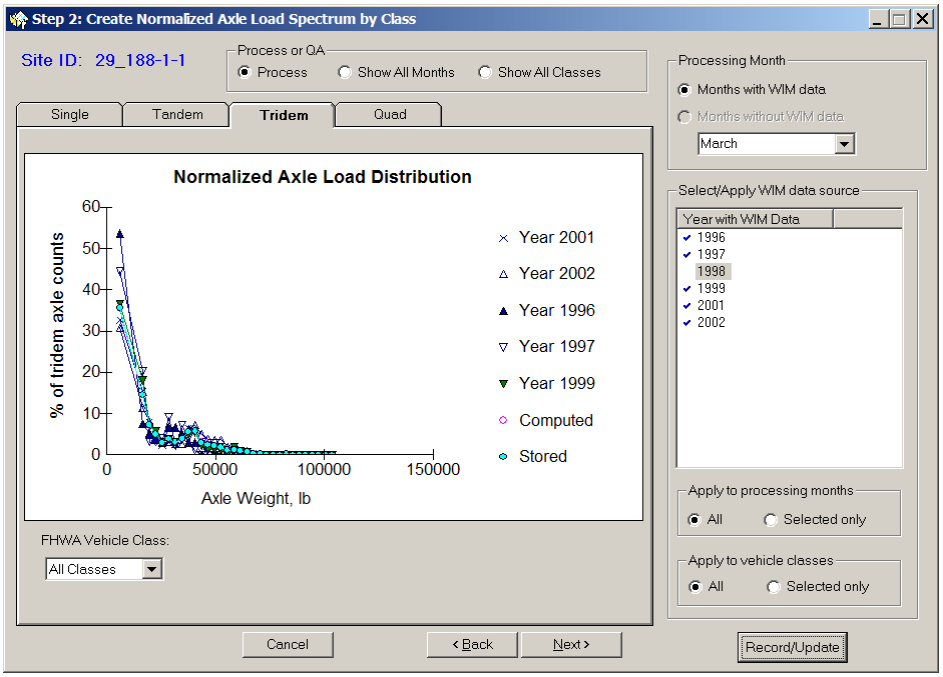


Figure A.26 Normalized tridem axle spectra for all classes in March for site 29_188-1-1 (SHRP_ID 6015)

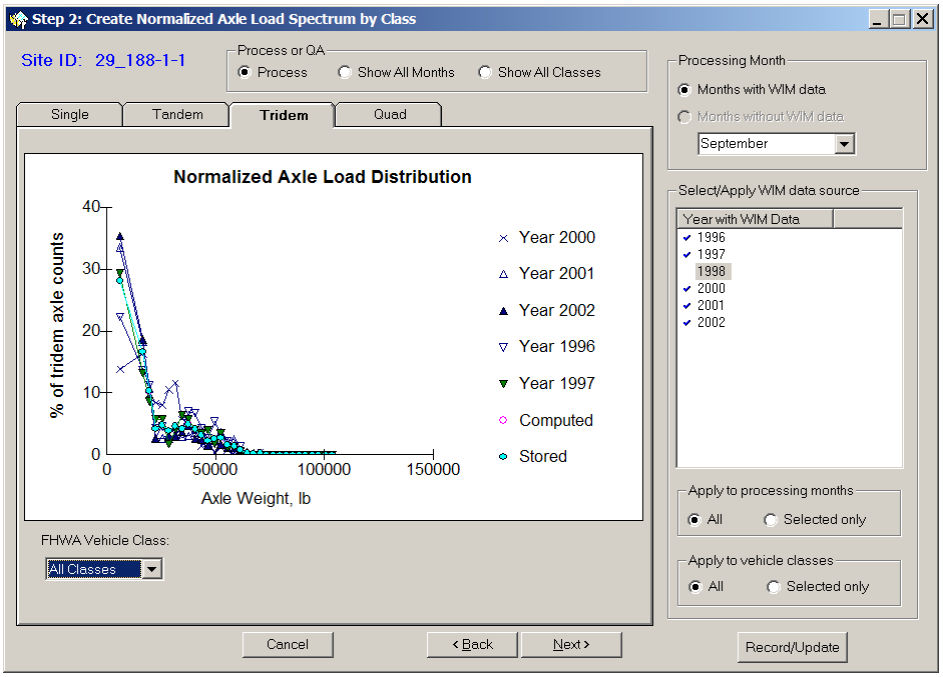


Figure A.27 Normalized tridem axle spectra for all classes in September for site 29_188-1-1 (SHRP_ID 6015)

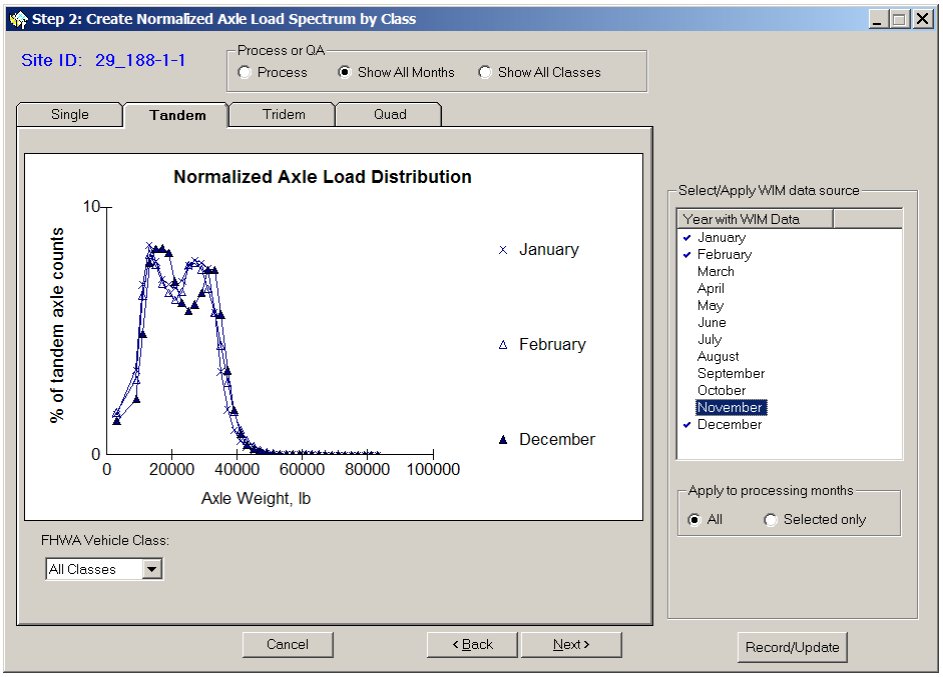


Figure A.28 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

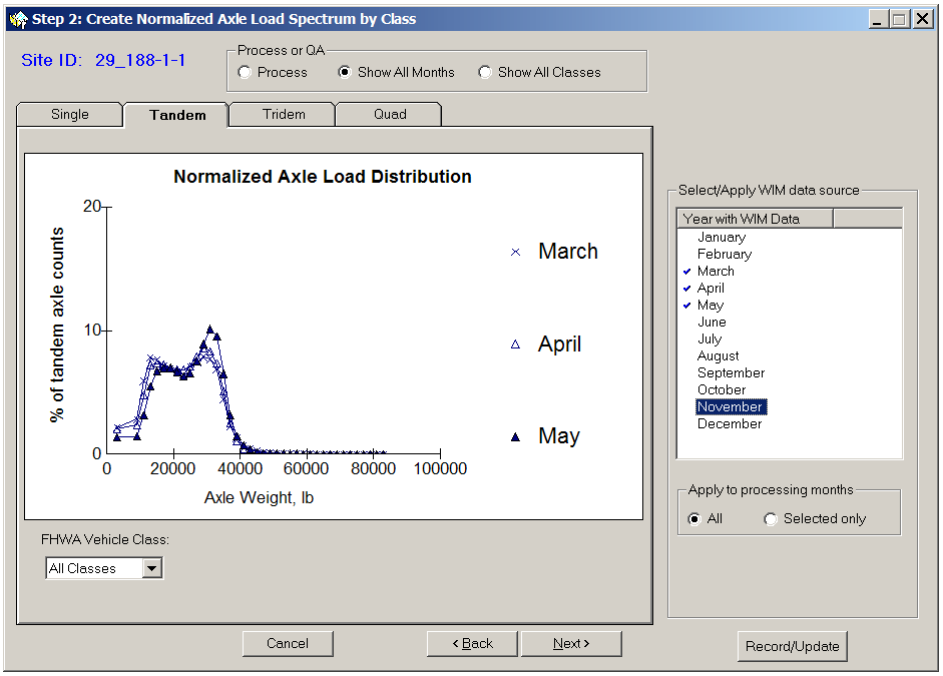


Figure A.29 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

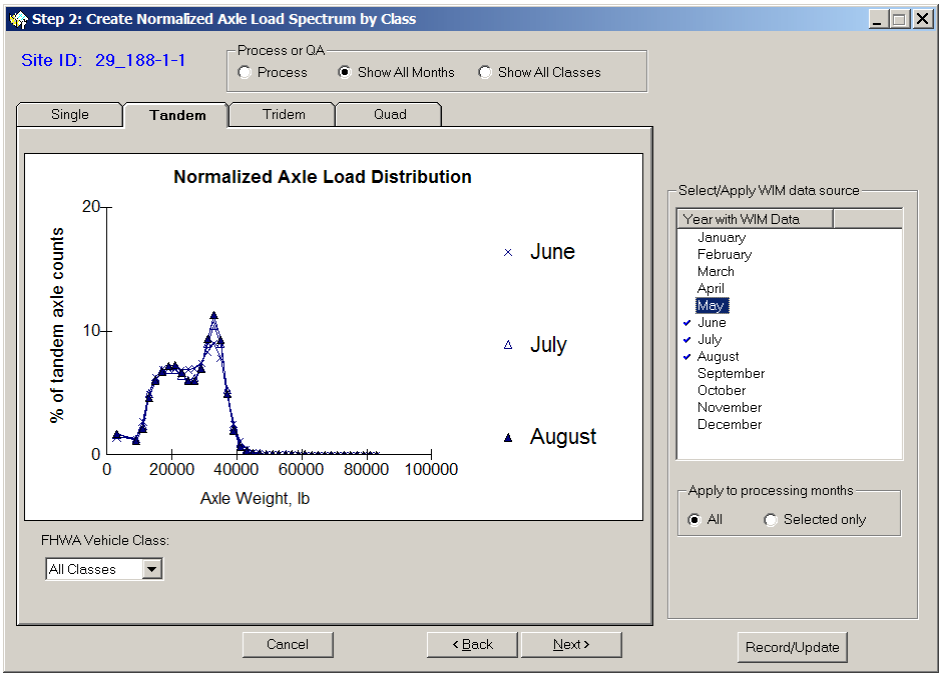


Figure A.30 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

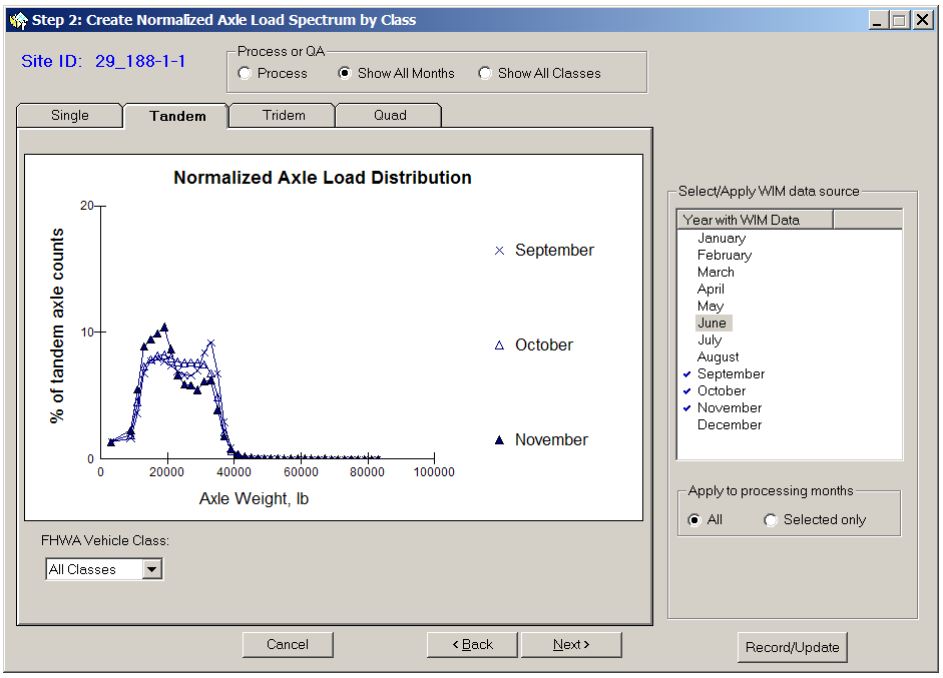


Figure A.31 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

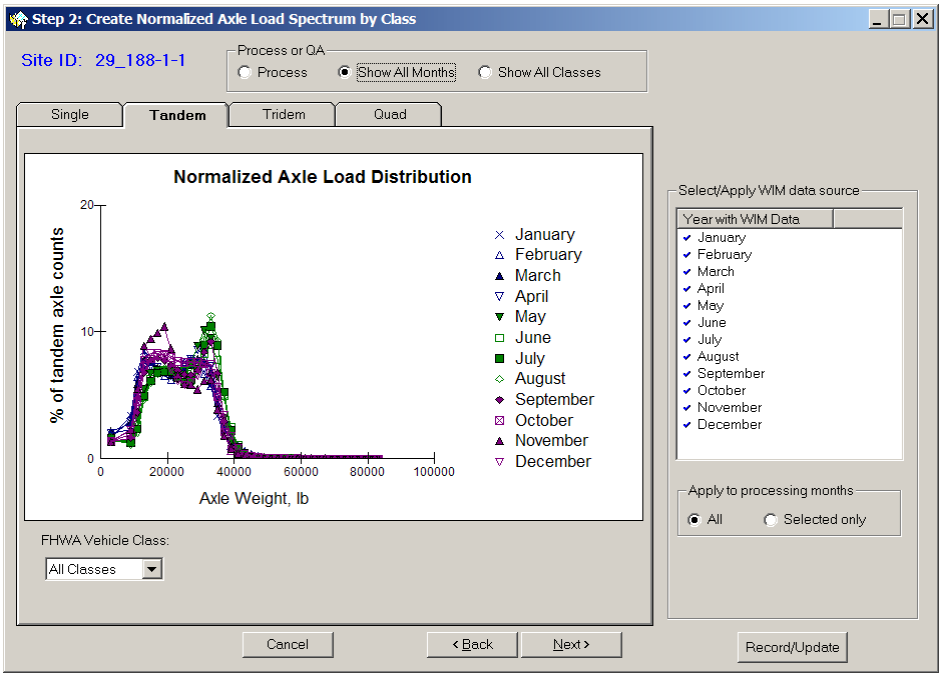


Figure A.32 Normalized tandem axle spectra for all classes for site 29_188-1-1 (SHRP_ID 6015)

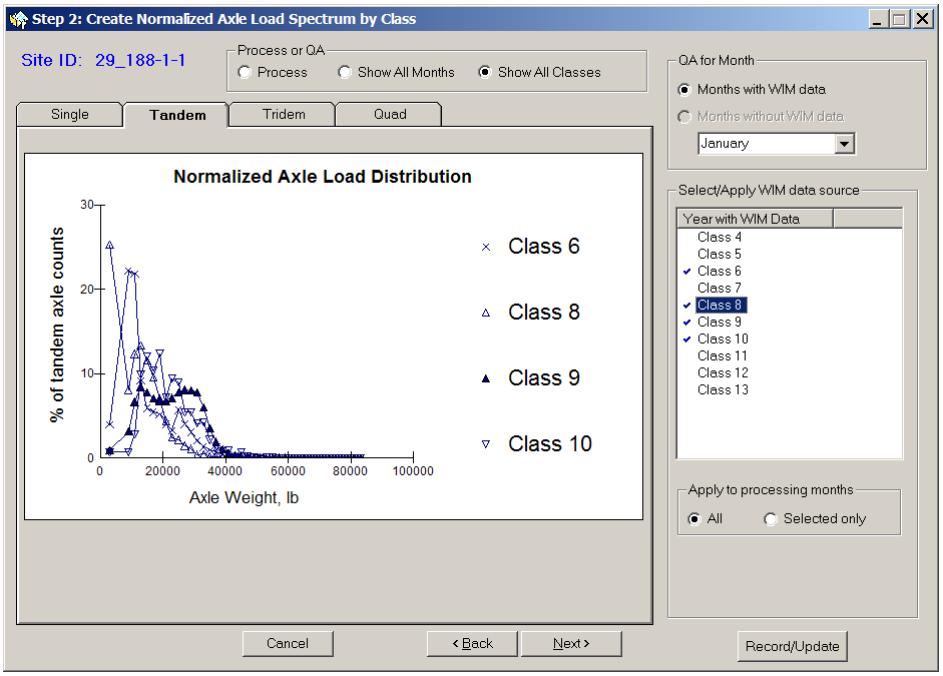


Figure A.33 Normalized tandem axle spectra for site 29_188-1-1 (SHRP_ID 6015)

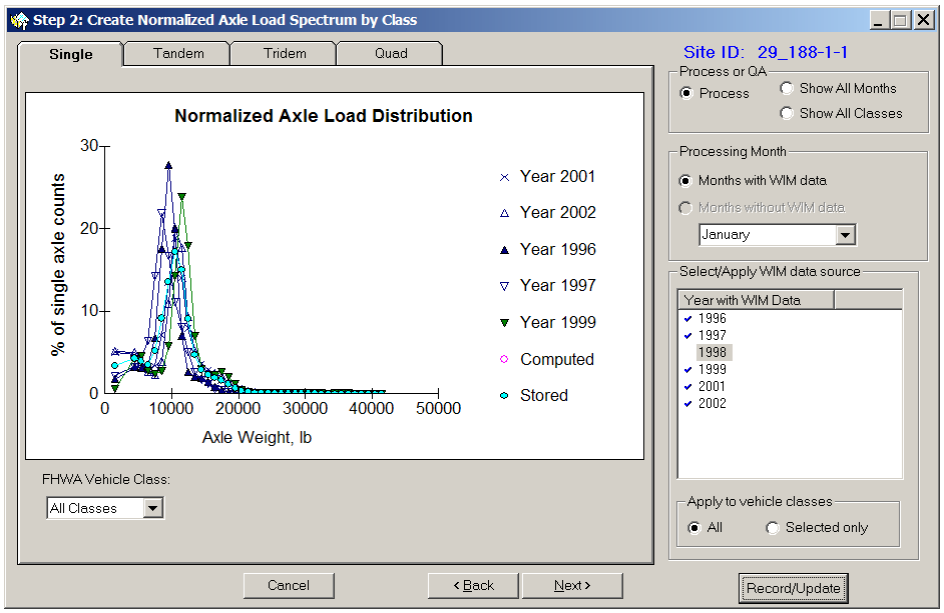
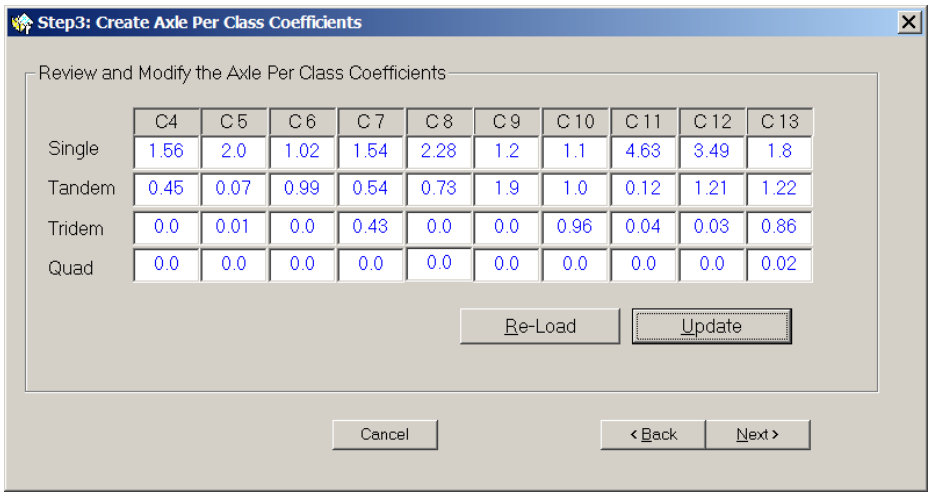


Figure A.34 Normalized single axle spectra for site 29_188_1-1 (SHRP_ID 6015)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.56	2.0	1.02	1.54	2.28	1.2	1.1	4.63	3.49	1.8
Tandem	0.45	0.07	0.99	0.54	0.73	1.9	1.0	0.12	1.21	1.22
Tridem	0.0	0.01	0.0	0.43	0.0	0.0	0.96	0.04	0.03	0.86
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.02

Figure A.35 Axle class coefficients for site 29_188-1-1 (SHRP_ID 6015)

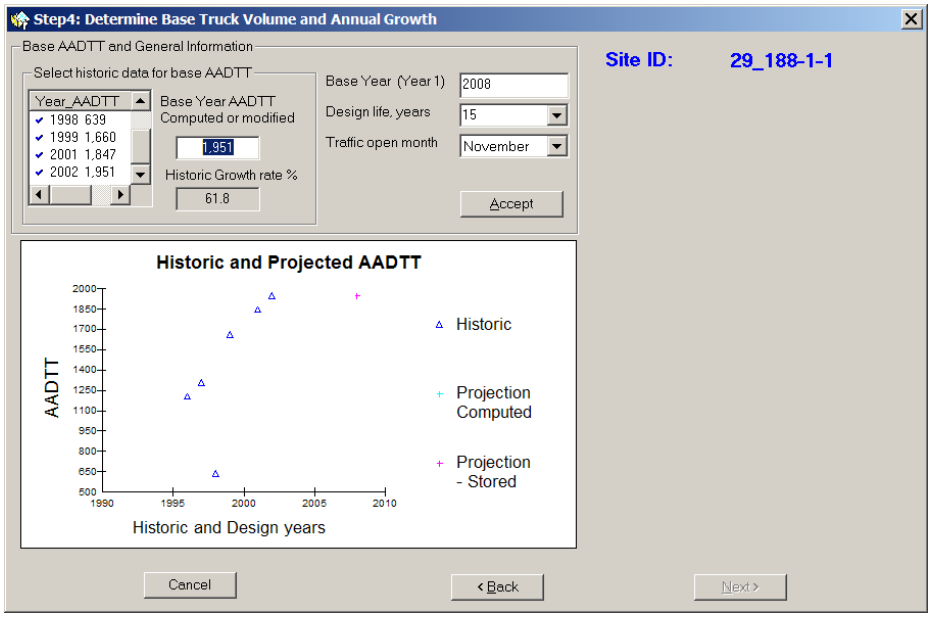


Figure A.36 Base truck volume and annual growth for site 29_188-1-1 (SHRP_ID 6015)

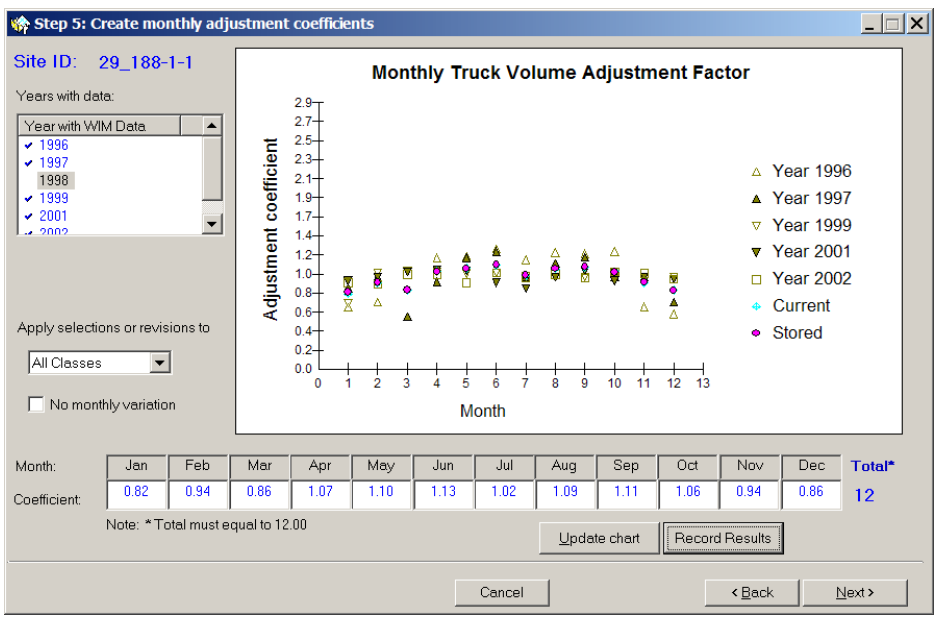
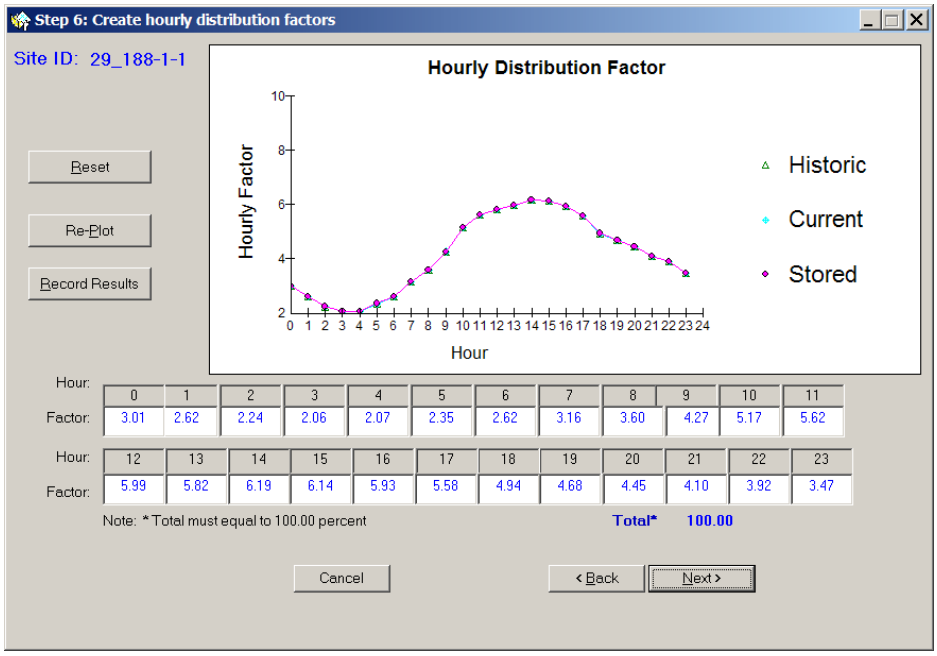


Figure A.37 Monthly truck volume adjustment coefficients for site 29_188-1-1 (SHRP_ID 6015)



Comment: Mid-day (10am—17pm)

Figure A.38 Hourly truck distribution factors for site 29_188-1-1 (SHRP_ID 6015)

SITE 29_188-5-1 (SHRP_ID 6015)

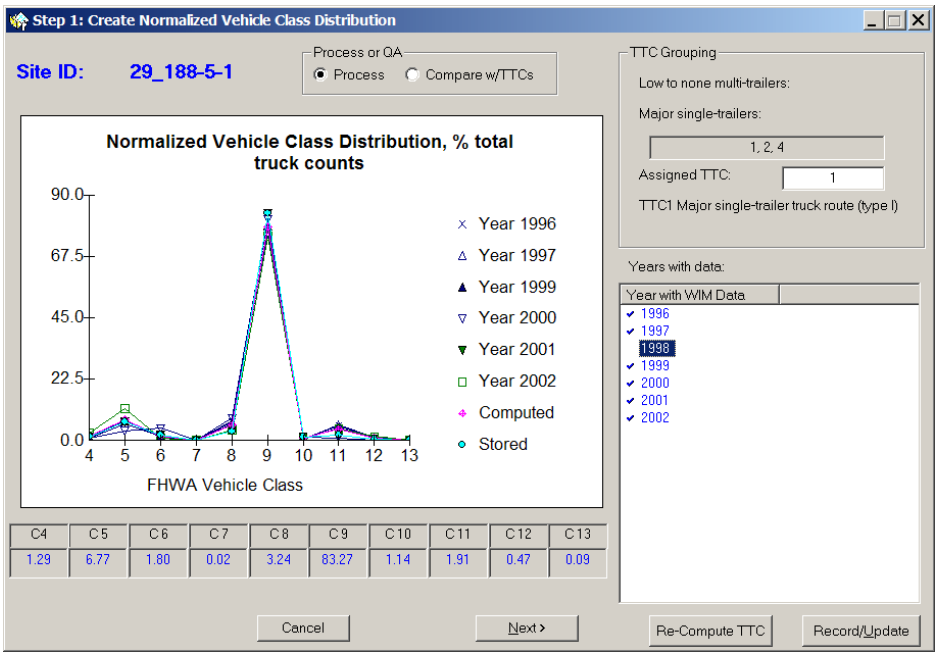


Figure A.39 Normalized vehicle class distribution for site 29_188-5-1 (SHRP_ID 6015)

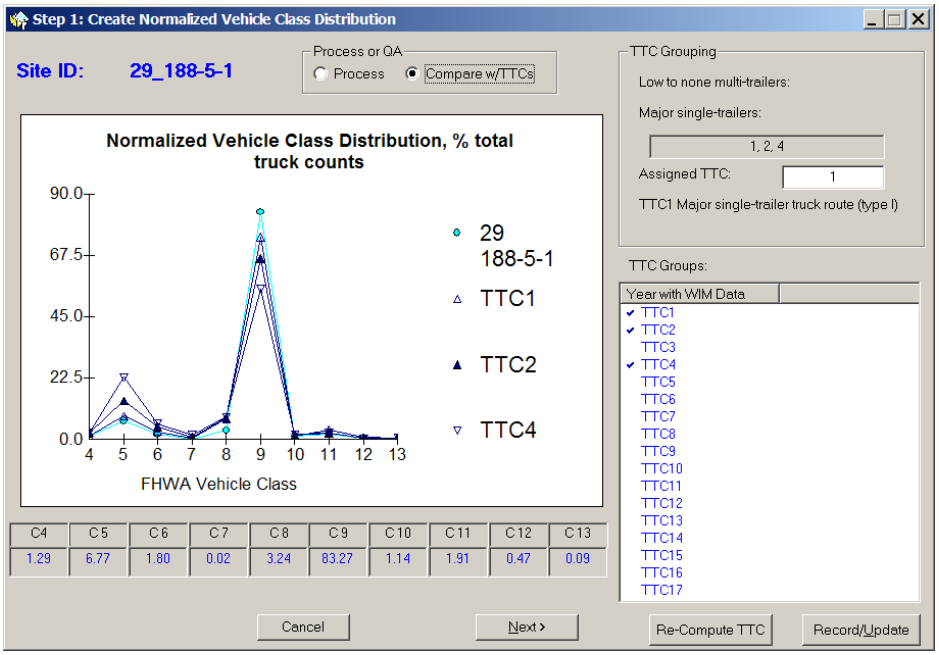


Figure A.40 Normalized vehicle class distribution compared with TTCs for site 29_188-5-1 (SHRP_ID 6015)

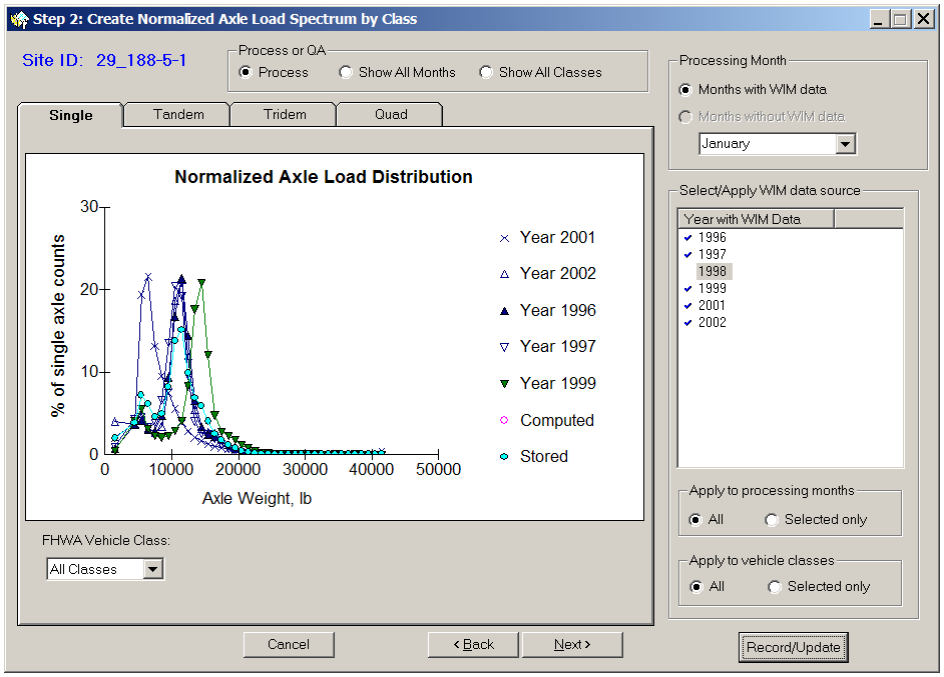
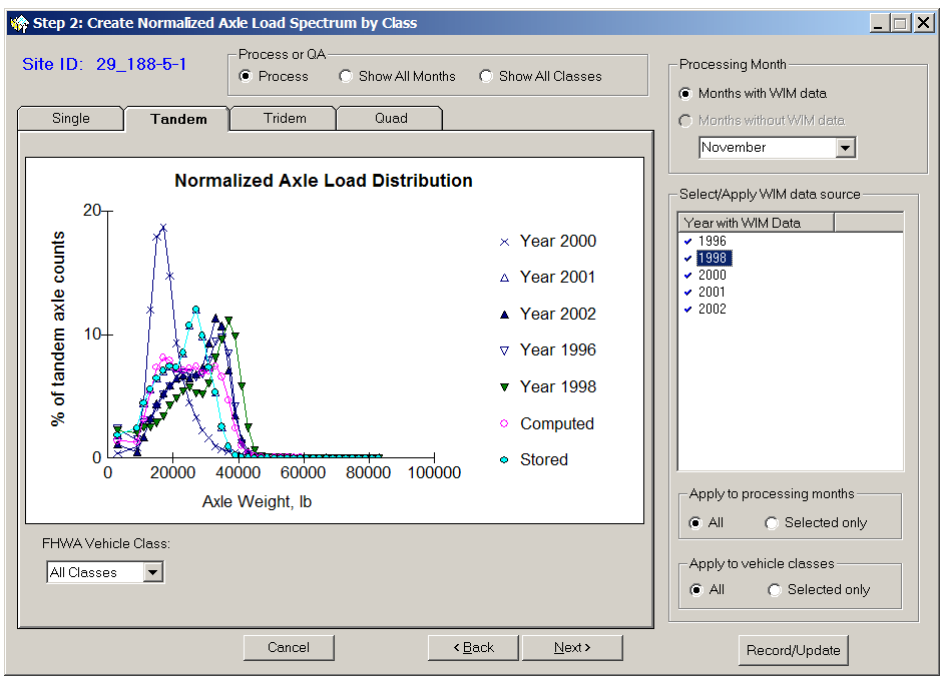


Figure A.41 Normalized single axle spectra for all classes for site 29_188_5-1 (SHRP_ID 6015)



Comment: Remove year 1998 and 2000:

Figure A.42 Normalized tandem axle spectra for all classes for site 29_188_5-1 (SHRP_ID 6015)

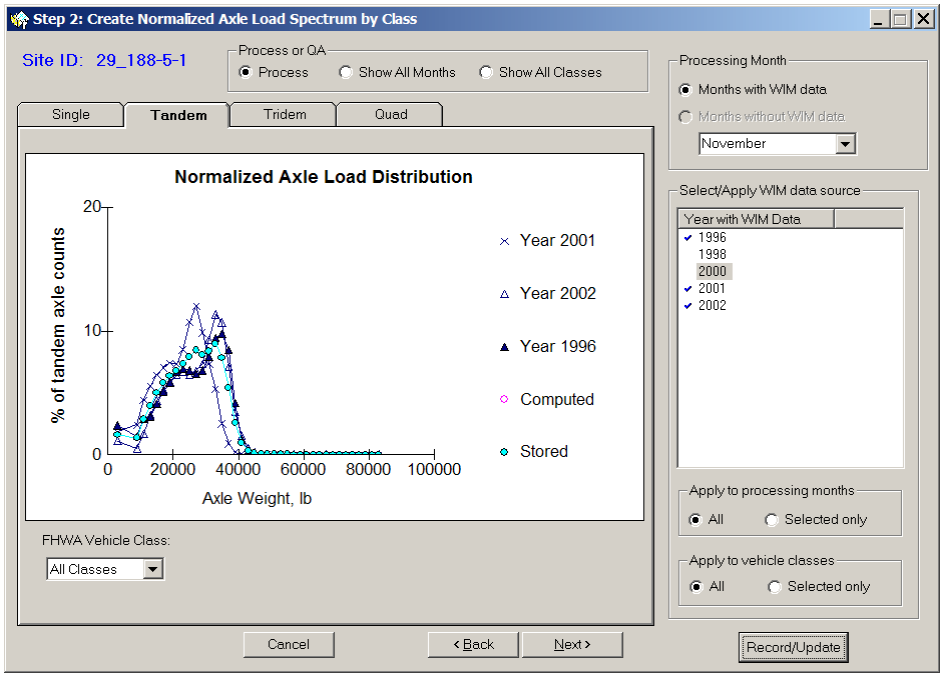


Figure A.43 Normalized tandem axle spectra for all classes for site 29_188_5-1 (SHRP_ID 6015)

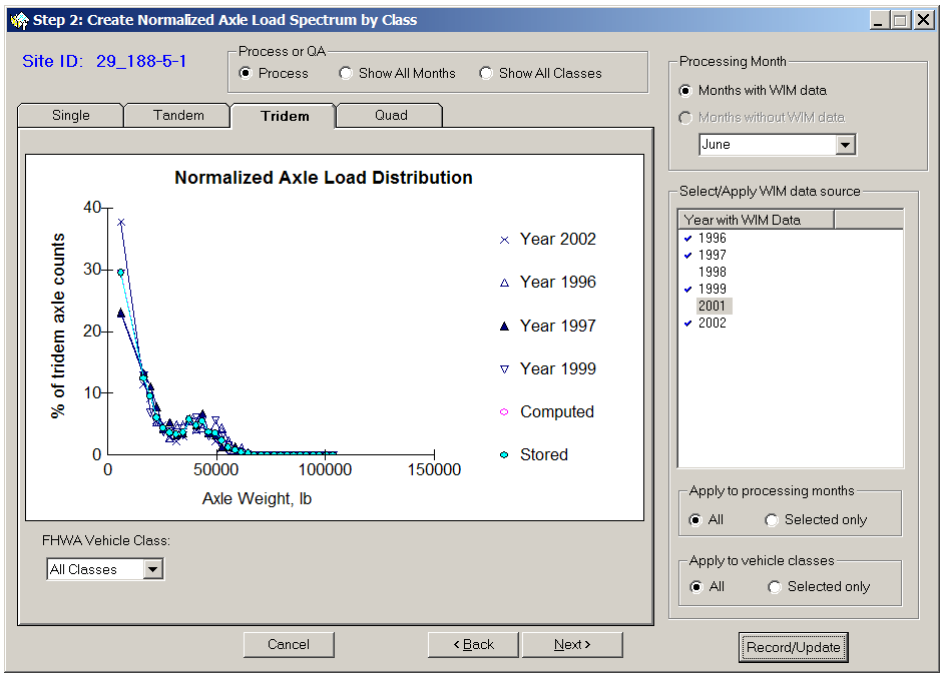


Figure A.44 Normalized tridem axle spectra for all classes for site 29_188-5-1 (SHRP_ID 6015)

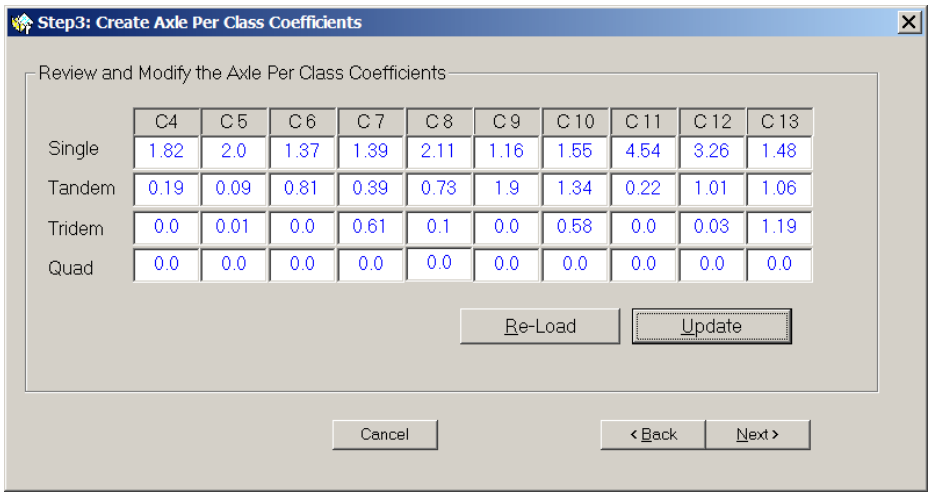


Figure A.45 Axle class coefficients for site 29_188-5-1 (SHRP_ID 6015)

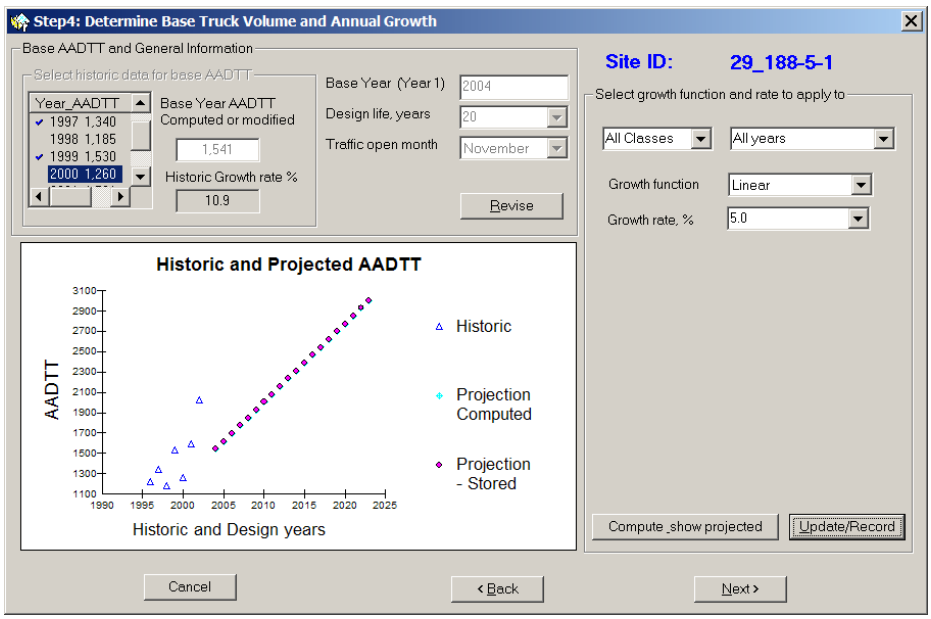


Figure A.46 Base truck volume and annual growth for site 29_188-5-1 (SHRP_ID 6015)

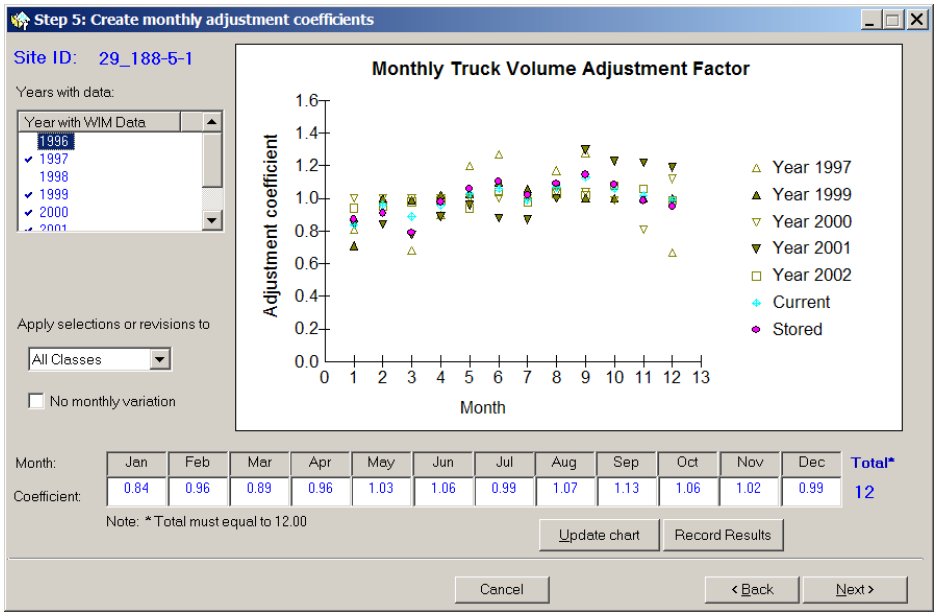
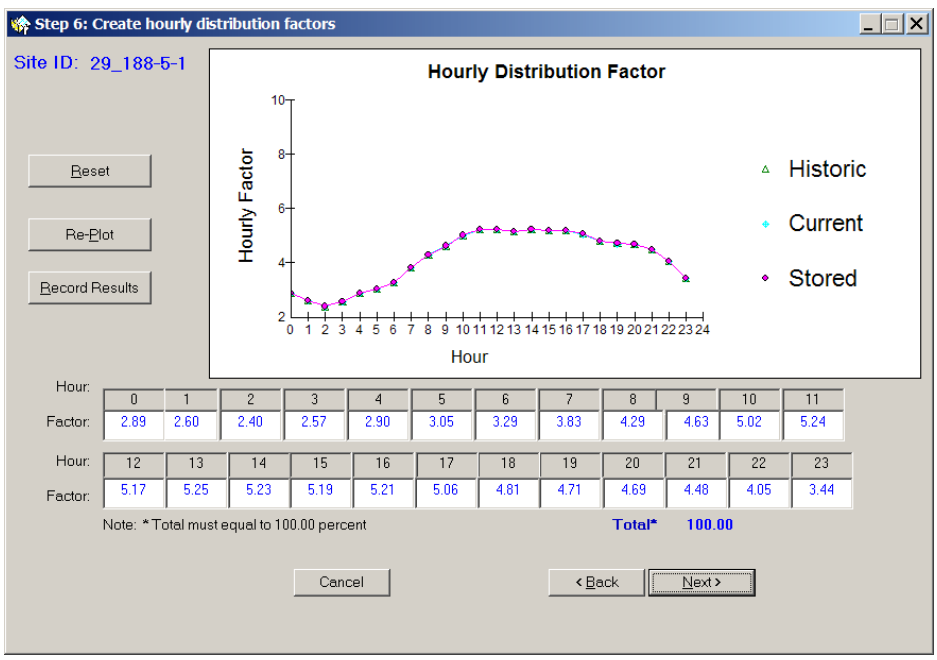


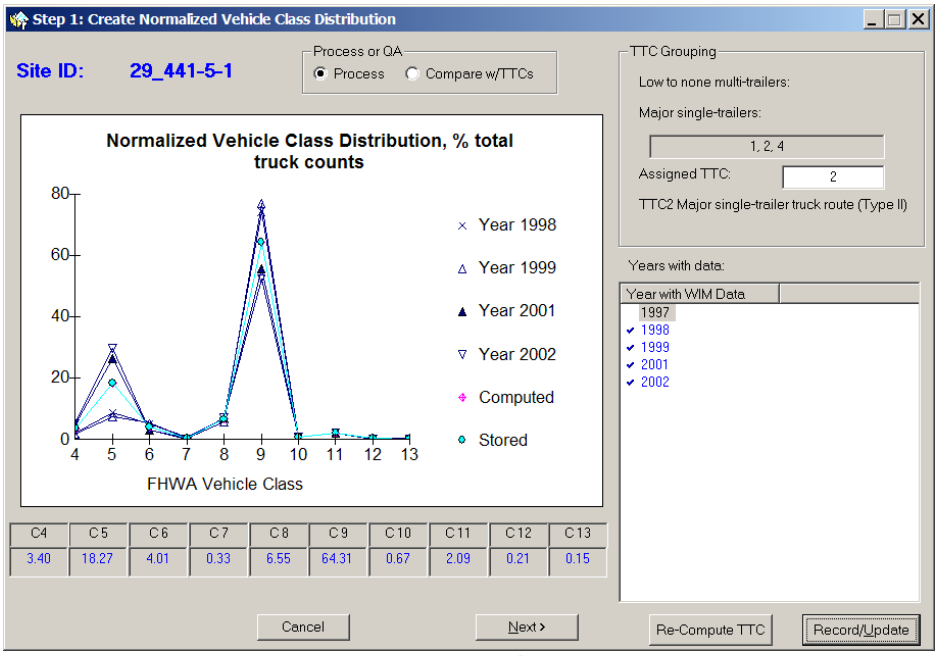
Figure A.47 Monthly truck volume adjustment coefficients for site 29_188-5-1 (SHRP_ID 6015)



Comment: Throughout the day (9am – 9pm peak)

Figure A.48 Hourly truck distribution factors for site 29_188-5-1 (SHRP_ID 6015)

SITE 29_441-5-1 (SHRP_ID 0963)



Comment: Took out 1997 data → TTC goes from 1 to 2

Figure A.49 Normalized vehicle class distribution for site 29_441-5-1 (SHRP_ID 0963)

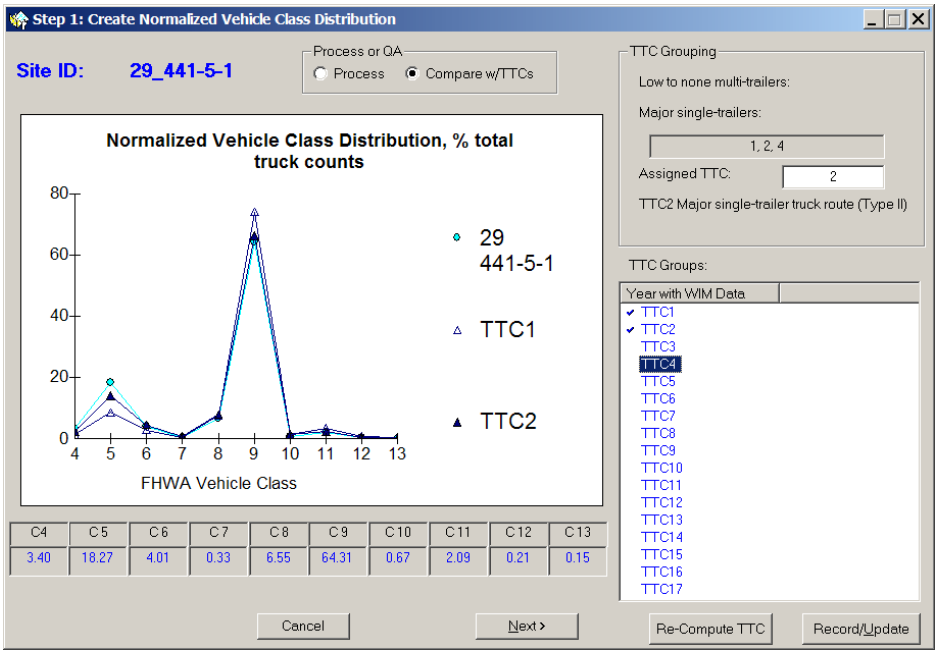
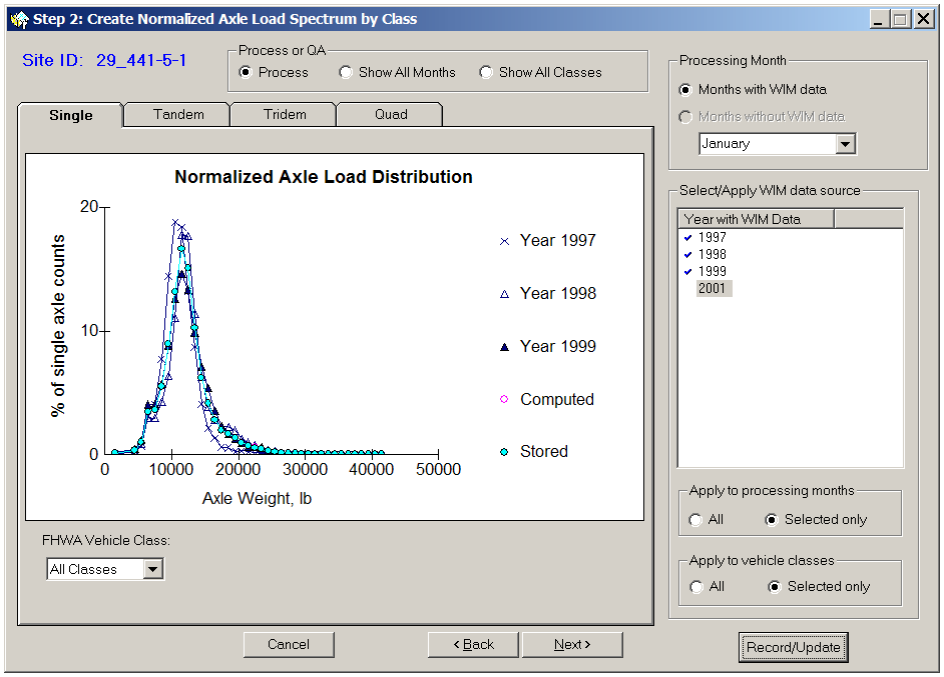


Figure A.50 Normalized vehicle class distribution for site 29_441-5-1 (SHRP_ID 0963)



Comment: 2001 shape not good.

Figure A.51 Normalized single axle spectra for all classes for site 29_441-5-1 (SHRP_ID 0963)

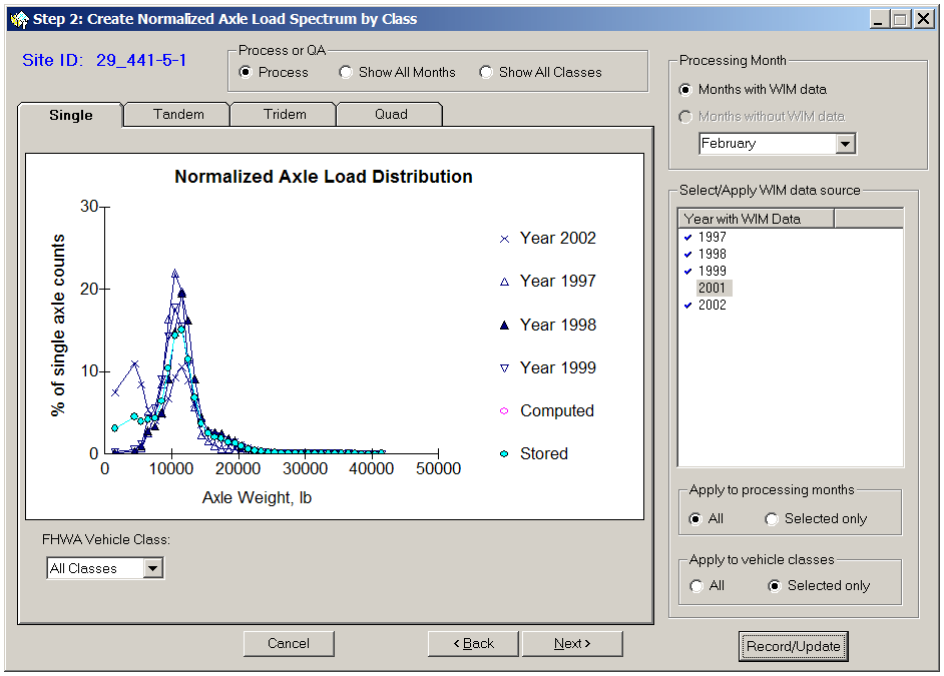


Figure A.52 Normalized single axle spectra for all classes for site 29_441-5-1 (SHRP_ID 0963)

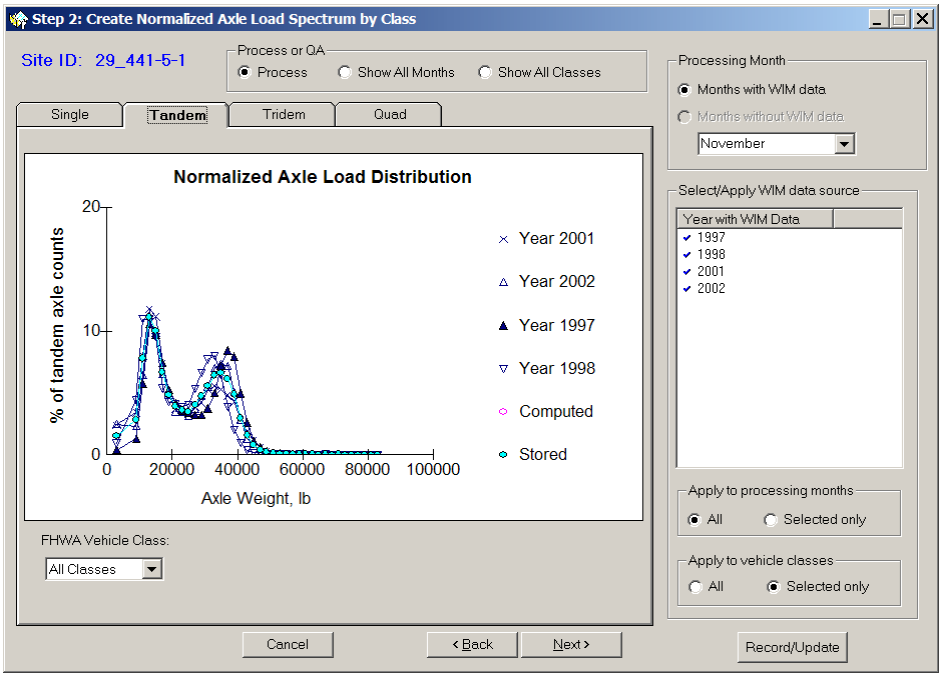


Figure A.53 Normalized tandem axle spectra for all classes for site 29_441-5-1 (SHRP_ID 0963)

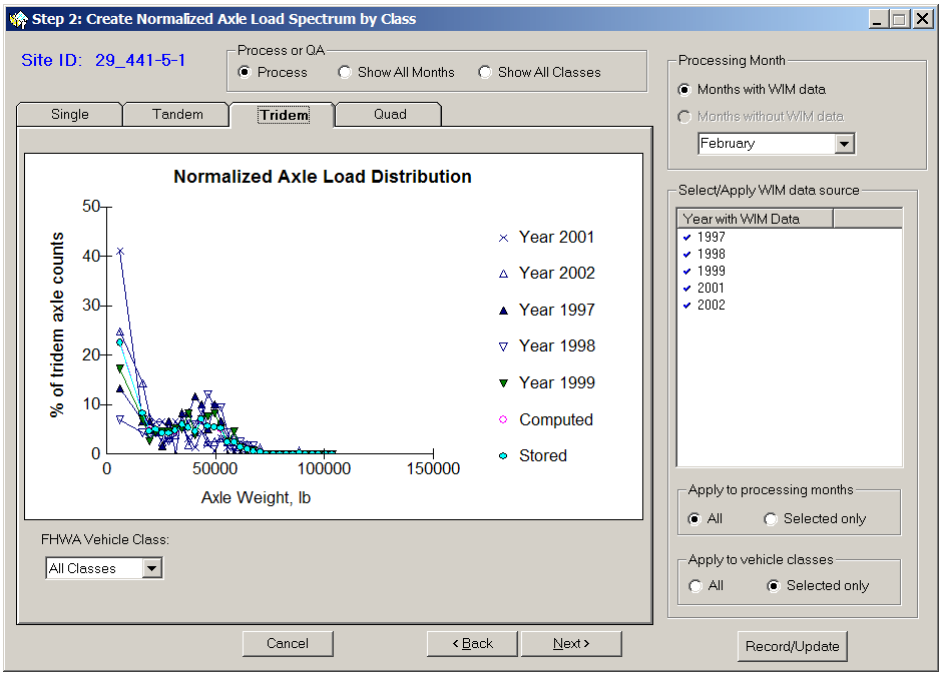


Figure A.54 Normalized tridem axle spectra for all classes for site 29_441-5-1 (SHRP_ID 0963)

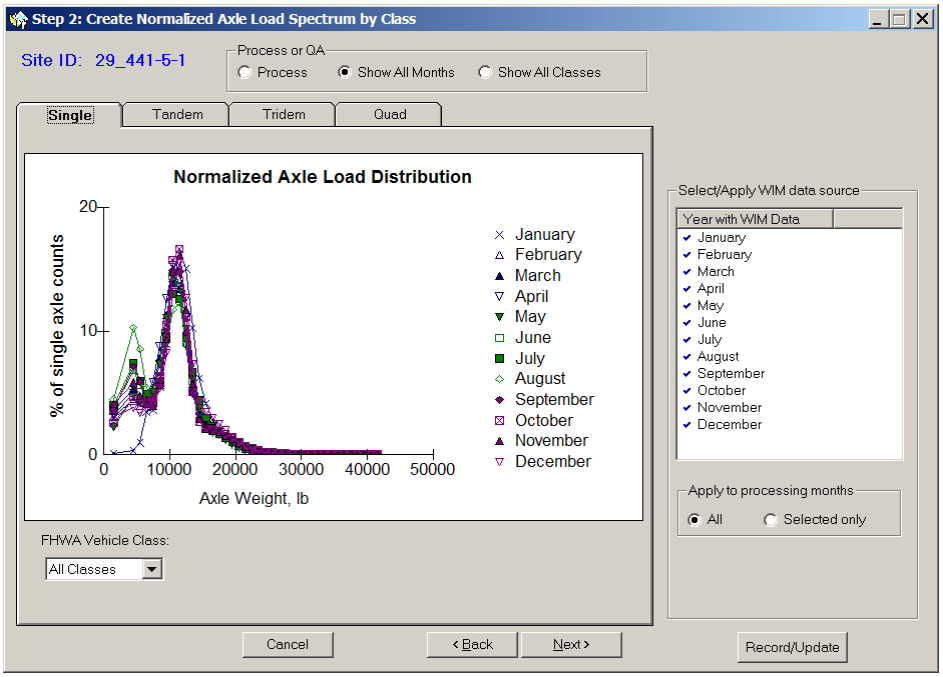


Figure A.55 Normalized single axle spectra for all classes for site 29_441-5-1 (SHRP_ID 0963)

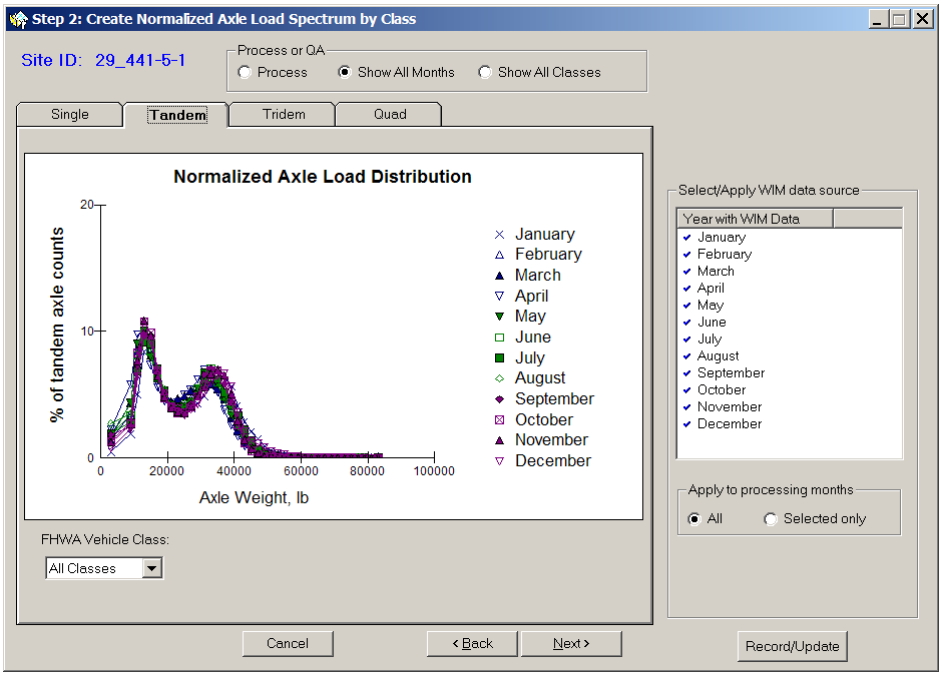


Figure A.56 Normalized tandem axle spectra for all classes for site 29_144-5-1 (SHRP_ID 0963)

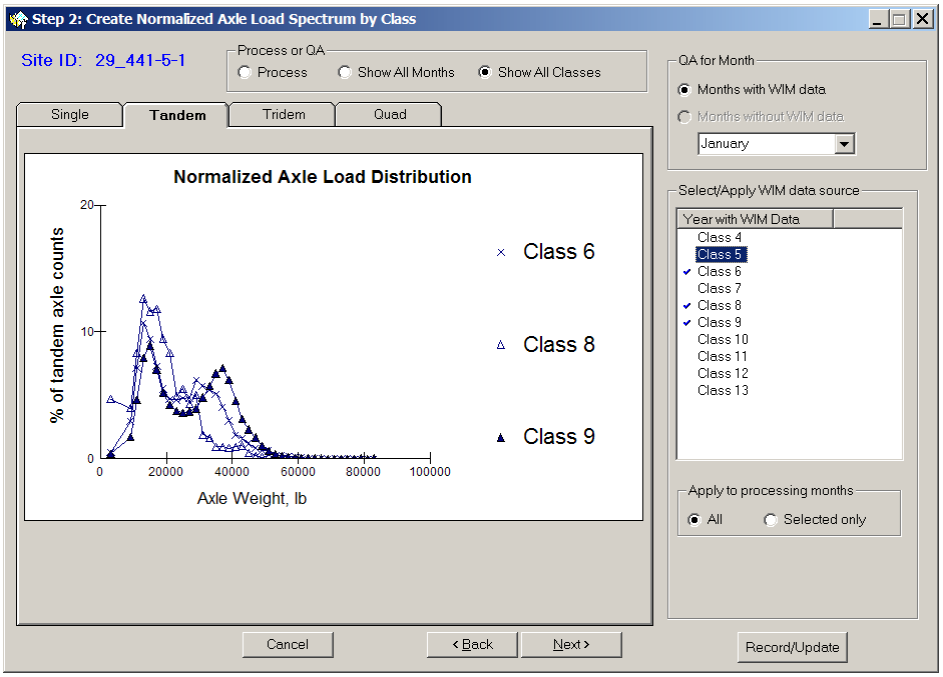


Figure A.57 Normalized tandem axle spectra by class for site 29_144-5-1 (SHRP_ID 0963)

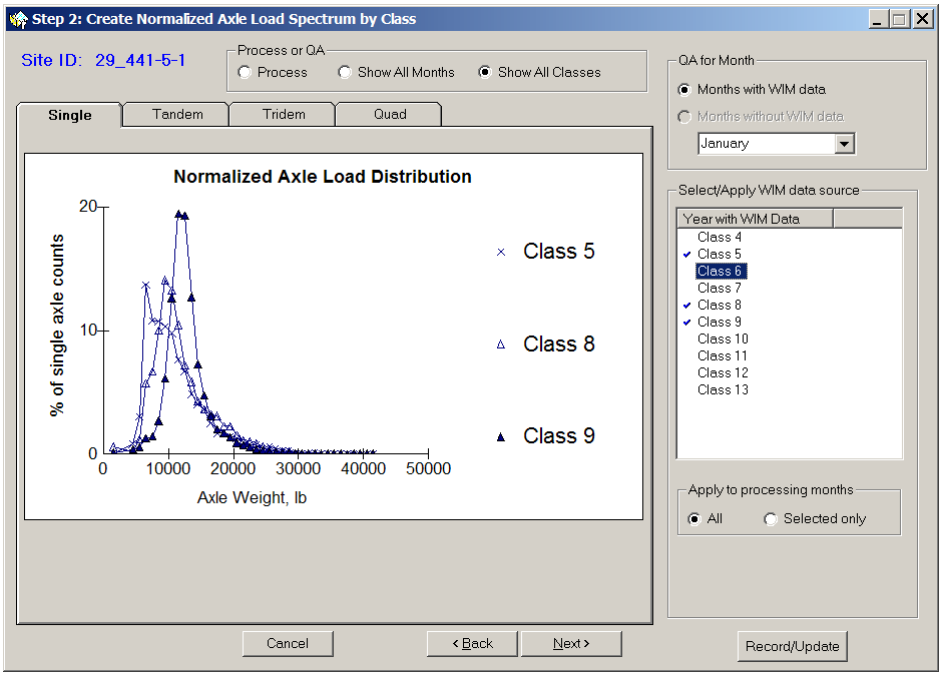
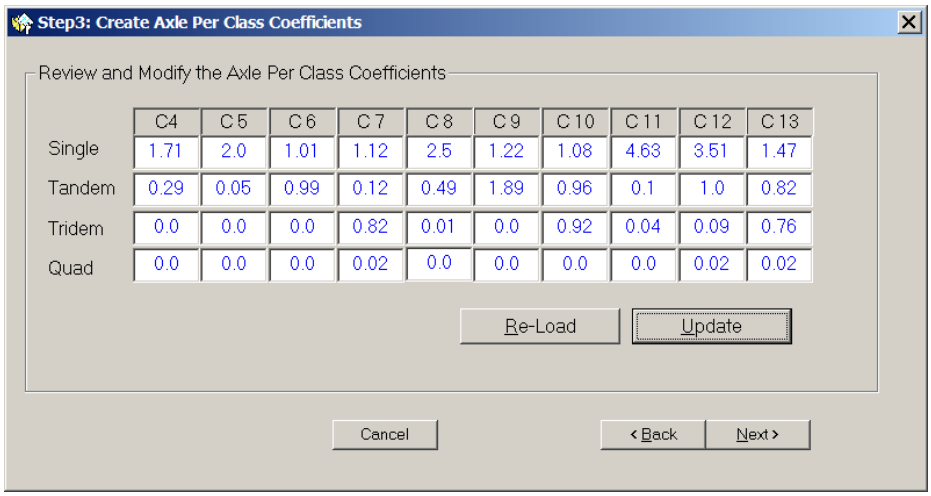


Figure A.58 Normalized single axle spectra by class for site 29_144-5-1 (SHRP_ID 0963)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.71	2.0	1.01	1.12	2.5	1.22	1.08	4.63	3.51	1.47
Tandem	0.29	0.05	0.99	0.12	0.49	1.89	0.96	0.1	1.0	0.82
Tridem	0.0	0.0	0.0	0.82	0.01	0.0	0.92	0.04	0.09	0.76
Quad	0.0	0.0	0.0	0.02	0.0	0.0	0.0	0.0	0.02	0.02

Figure A.59 Axle class coefficients for site 29_441-5-1 (SHRP_ID 0963)

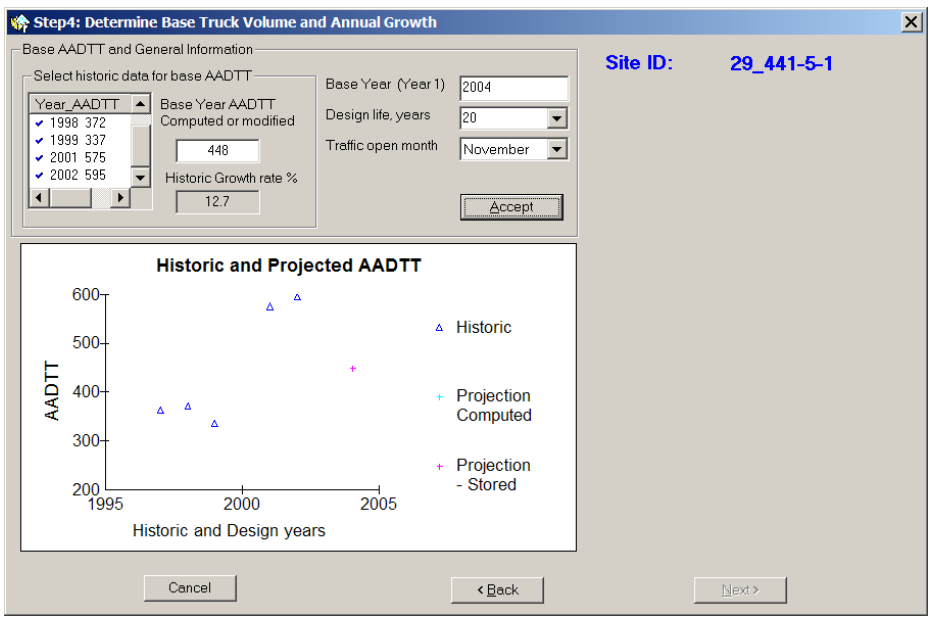


Figure A.60 Base truck volume and annual growth for site 29_441-5-1 (SHRP_ID 0963)

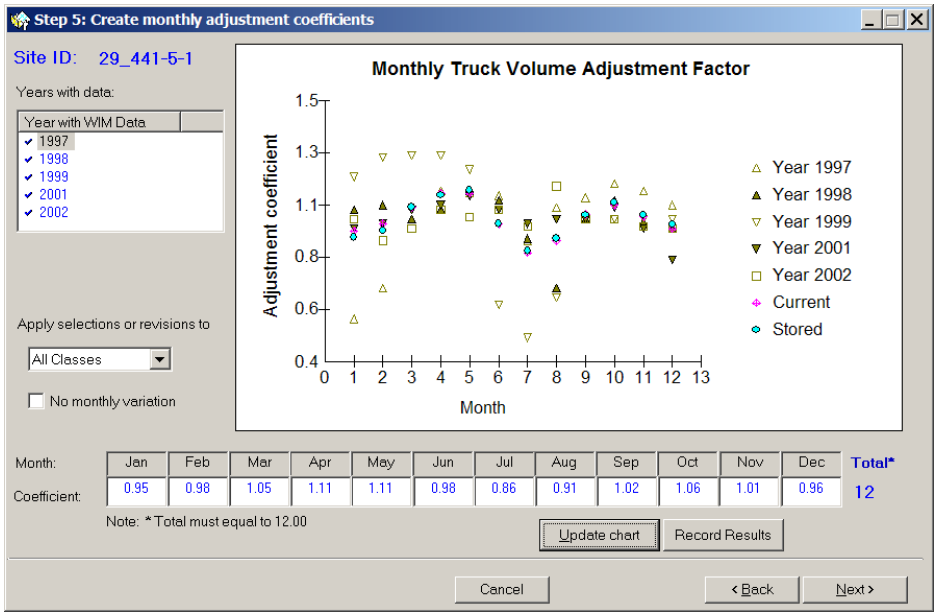


Figure A.61 Monthly truck volume adjustment coefficients for site 29_441-5-1 (SHRP_ID 0963)

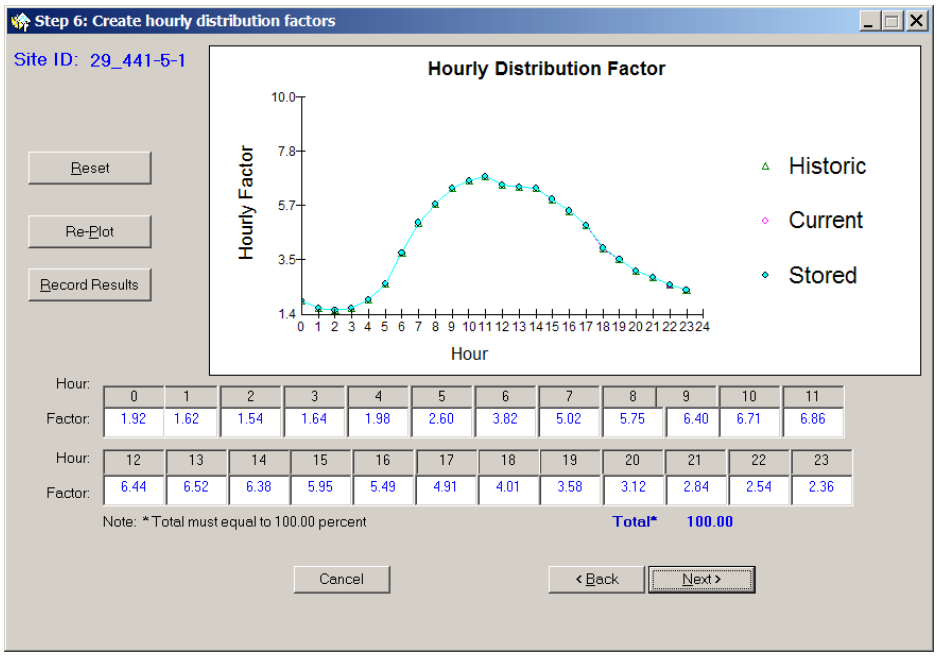
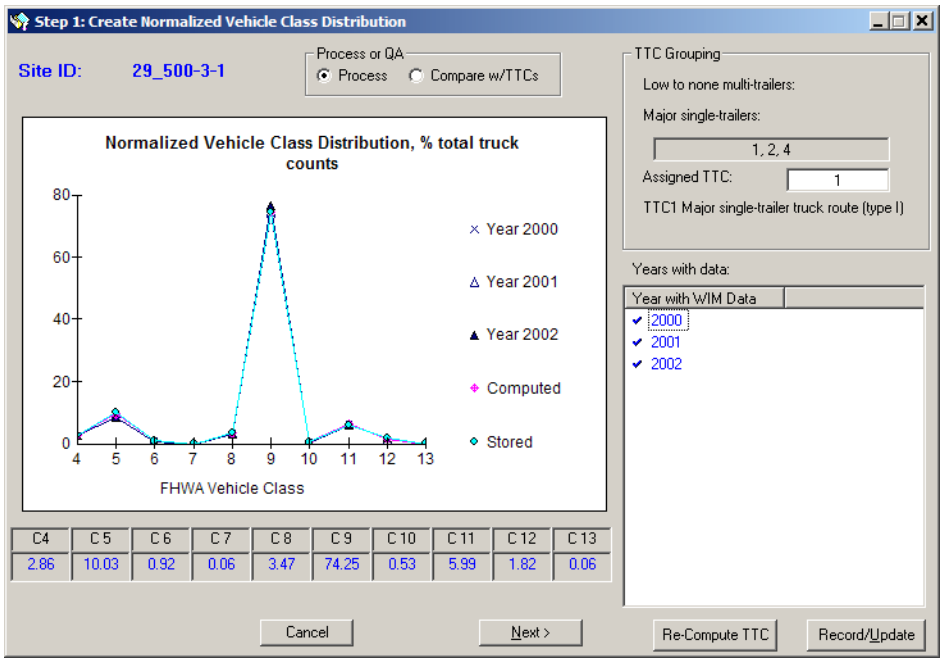


Figure A.62 Hourly truck distribution factors for site 29_441-5-1 (SHRP_ID 0963)

SITE 29_500-3-1 (SHRP_ID 5473)



Comment: Not much variation in vehicle class distribution from year to year.

Figure A.63 Normalized vehicle class distribution for site 29_500-3-1 (SHRP_ID 5473)

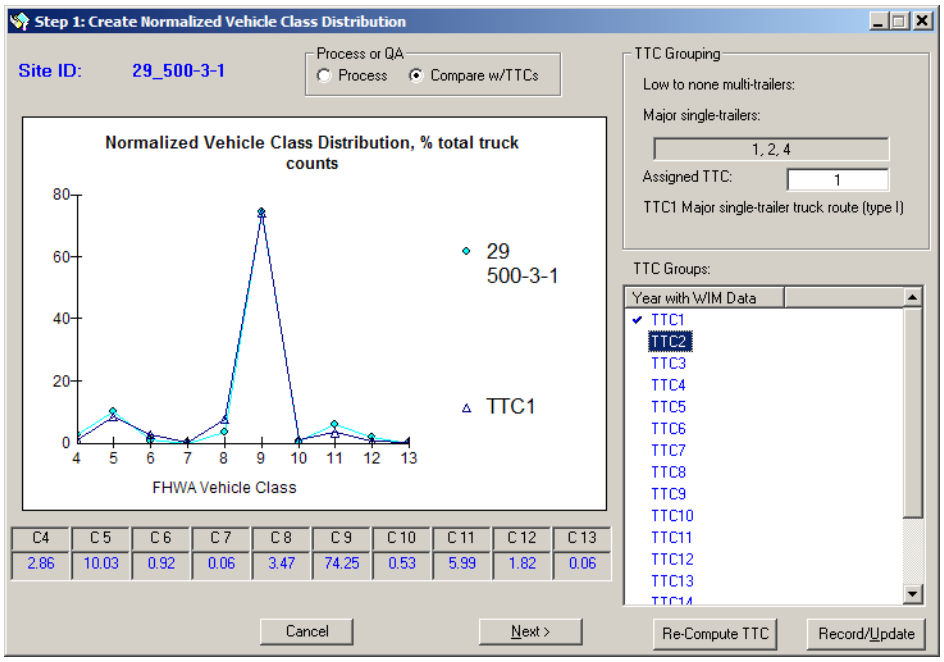


Figure A.64 Normalized vehicle class distribution for site 29_500-3-1 (SHRP_ID 5473)

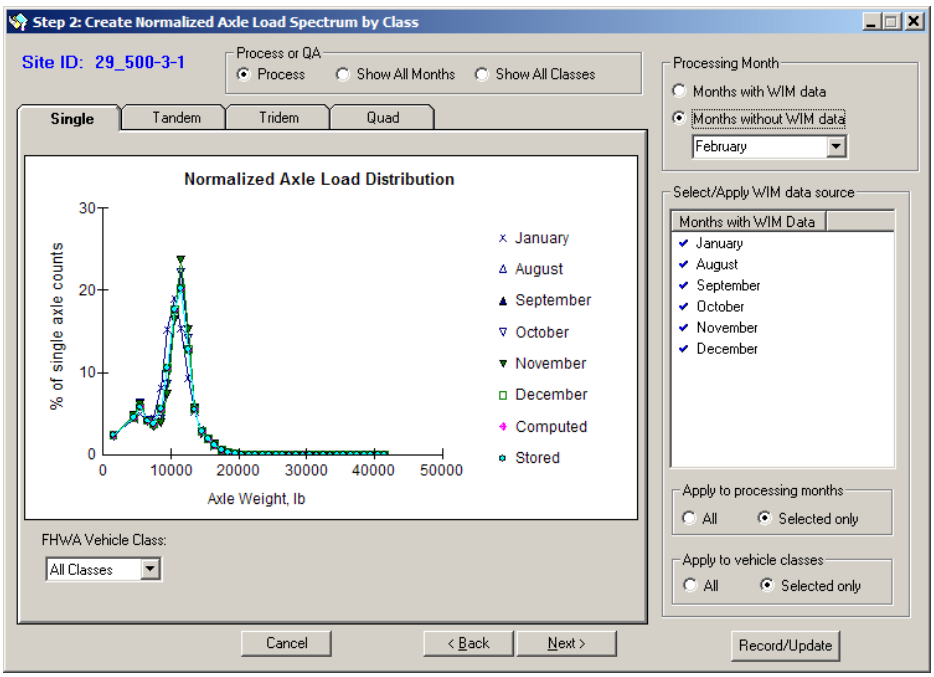


Figure A.65 Normalized single axle spectra for all classes for site 29_500-3-1 (SHRP_ID 5473)

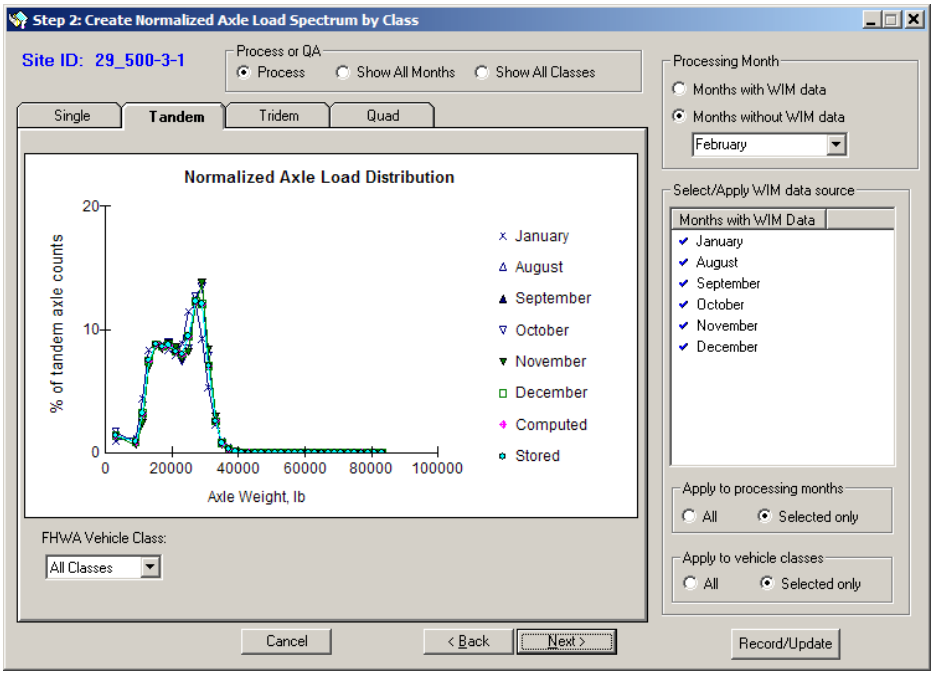
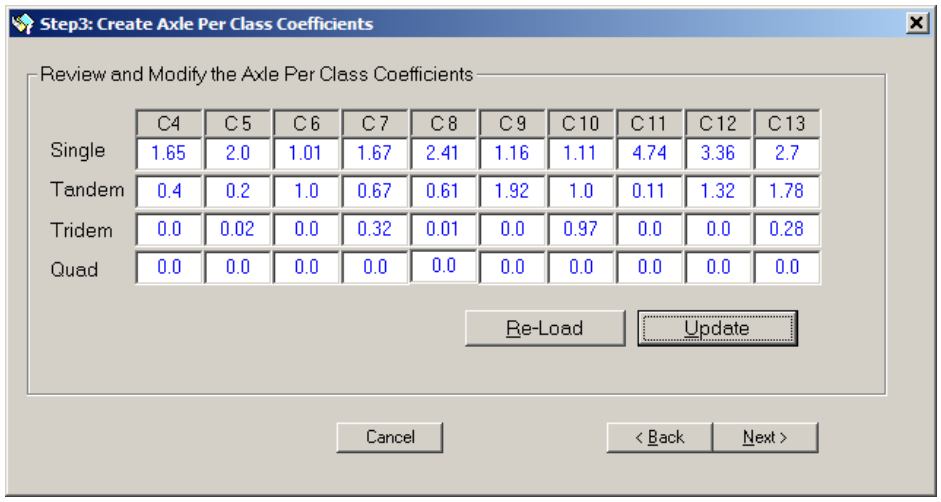


Figure A.66 Normalized tandem axle spectra for all classes for site 29_500-3-1 (SHRP_ID 5473)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.65	2.0	1.01	1.67	2.41	1.16	1.11	4.74	3.36	2.7
Tandem	0.4	0.2	1.0	0.67	0.61	1.92	1.0	0.11	1.32	1.78
Tridem	0.0	0.02	0.0	0.32	0.01	0.0	0.97	0.0	0.0	0.28
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.67 Axle class coefficients for site 29_500-3-1 (SHRP_ID 5473)

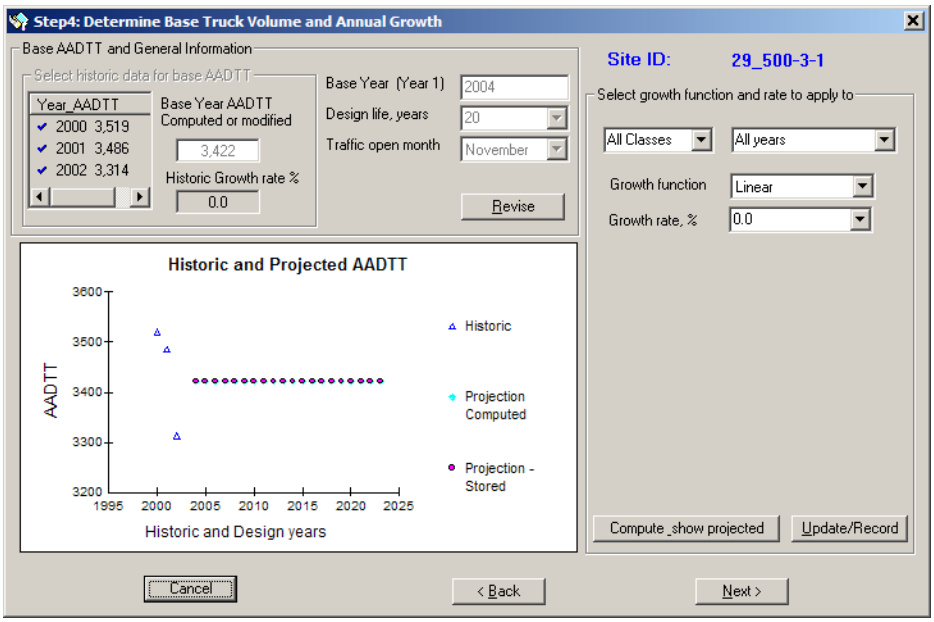


Figure A.68 Base truck volume and annual growth for site 29_500-3-1 (SHRP_ID 5473)

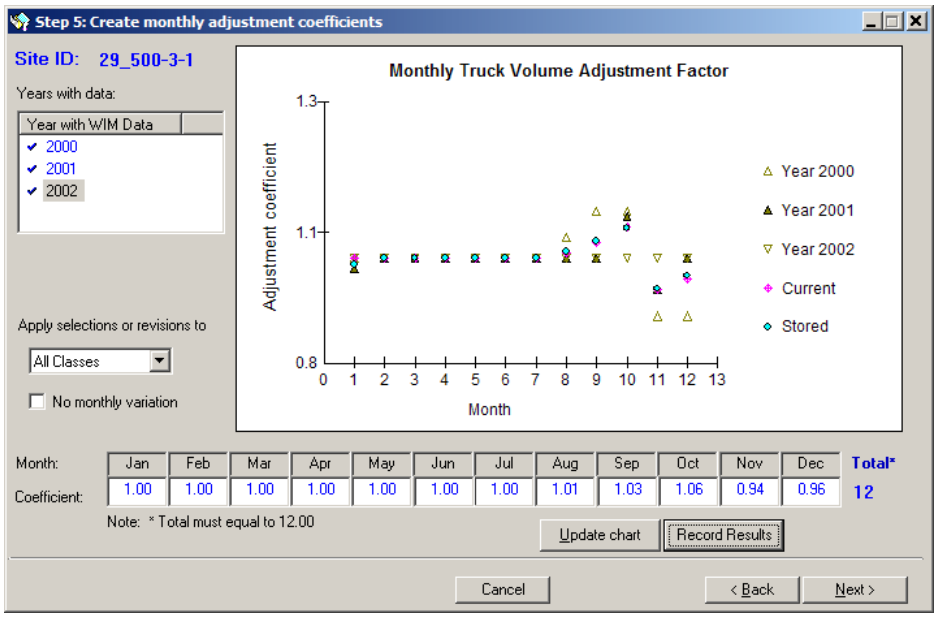


Figure A.69 Axle class coefficients for site 29_500-3-1 (SHRP_ID 5473)

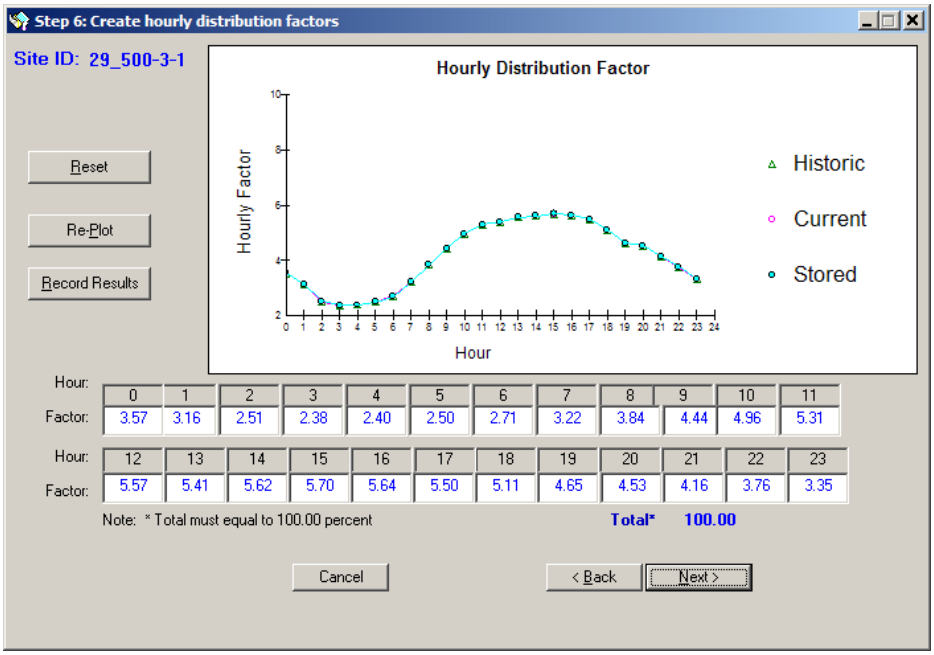
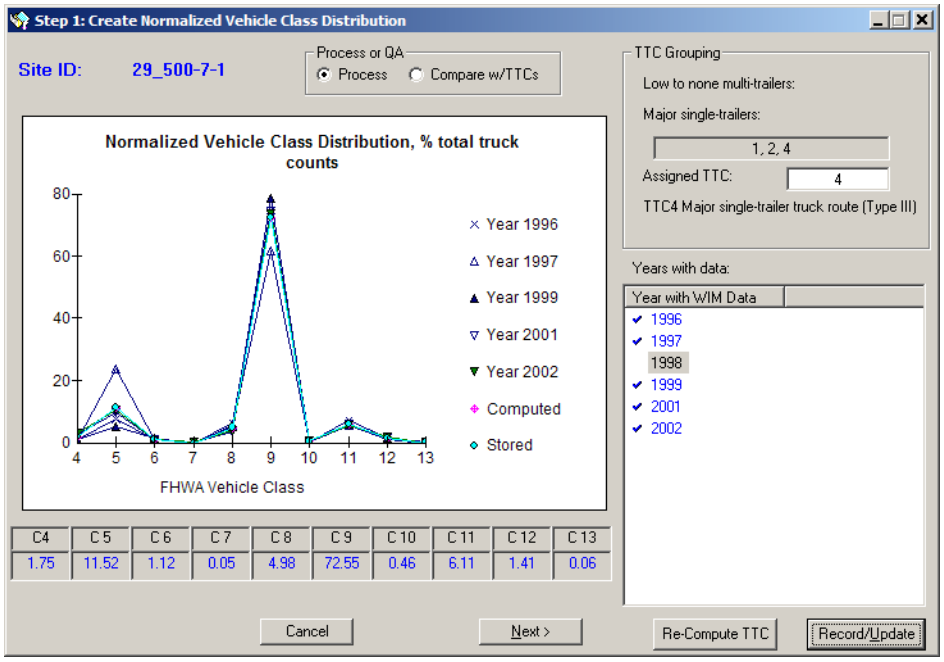


Figure A.70 Hourly truck distribution factors for site 29_500-3-1 (SHRP_ID 5473)

SITE 29_500-7-1 (SHRP_ID 5473)



Comment: 1998 outlier, left out

Figure A.71 Normalized vehicle class distribution for site 29_500-7-1 (SHRP_ID 5473)

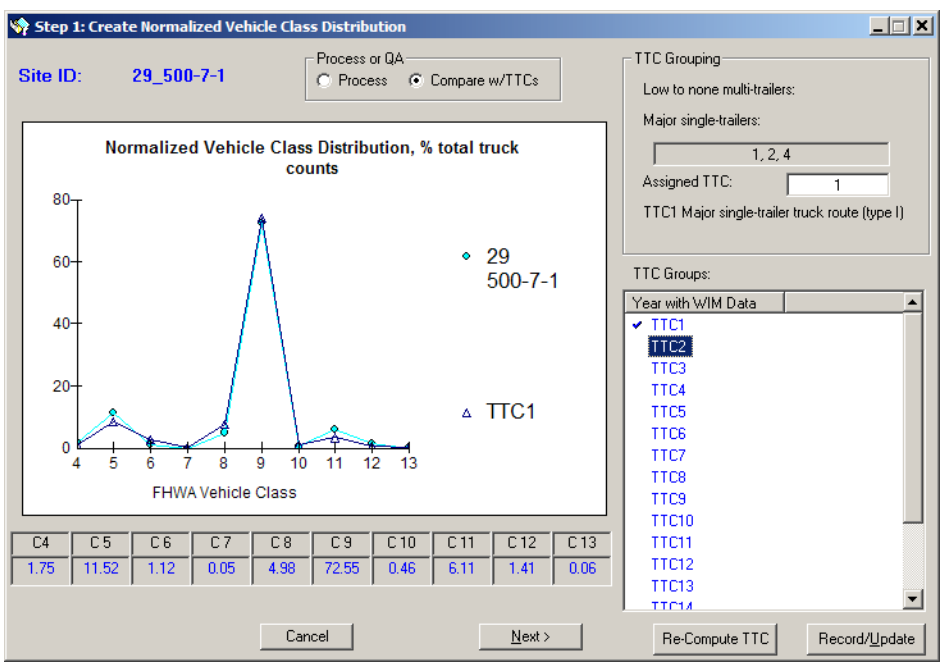


Figure A.72 Normalized vehicle class distribution (excluding 1998) for site 29_500-7-1 (SHRP_ID 5473)

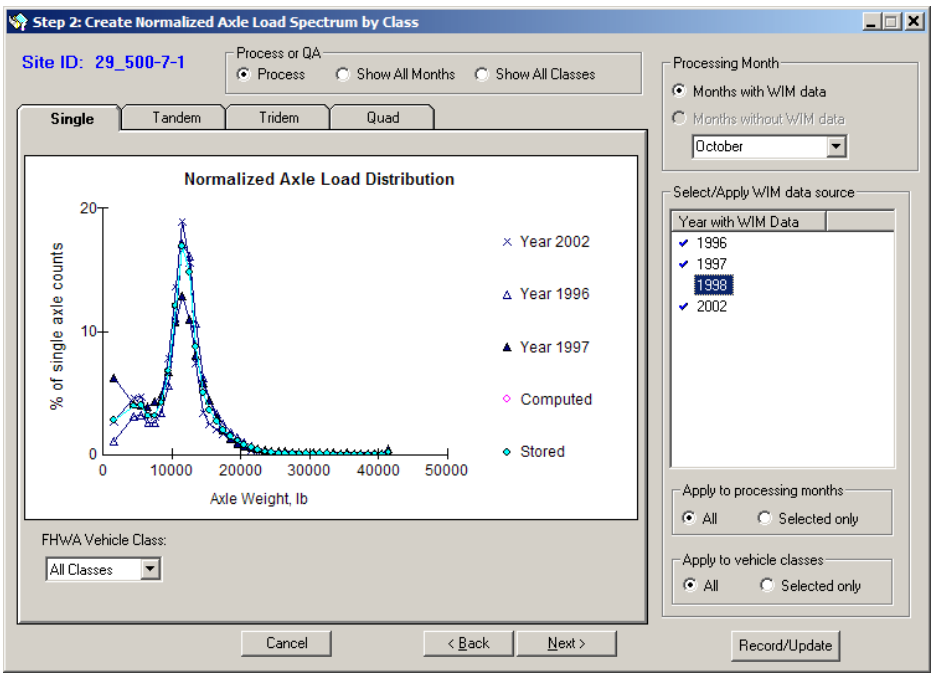


Figure A.73 Normalized single axle spectra for all classes for site 29_500-7-1 (SHRP_ID 5473)

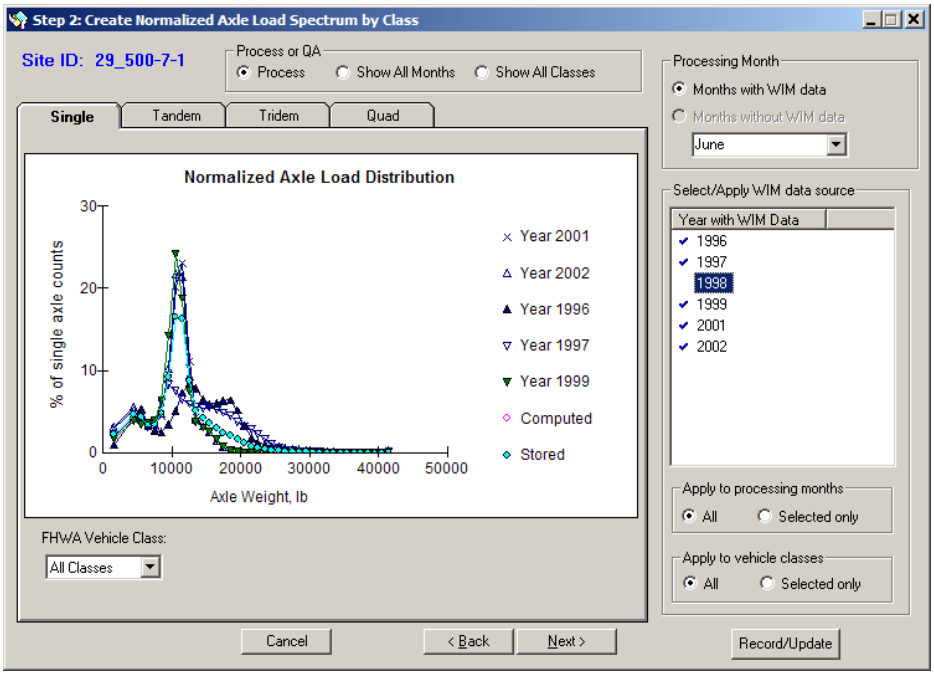


Figure A.74 Normalized single axle spectra for all classes for site 29_500-7-1 (SHRP_ID 5473)

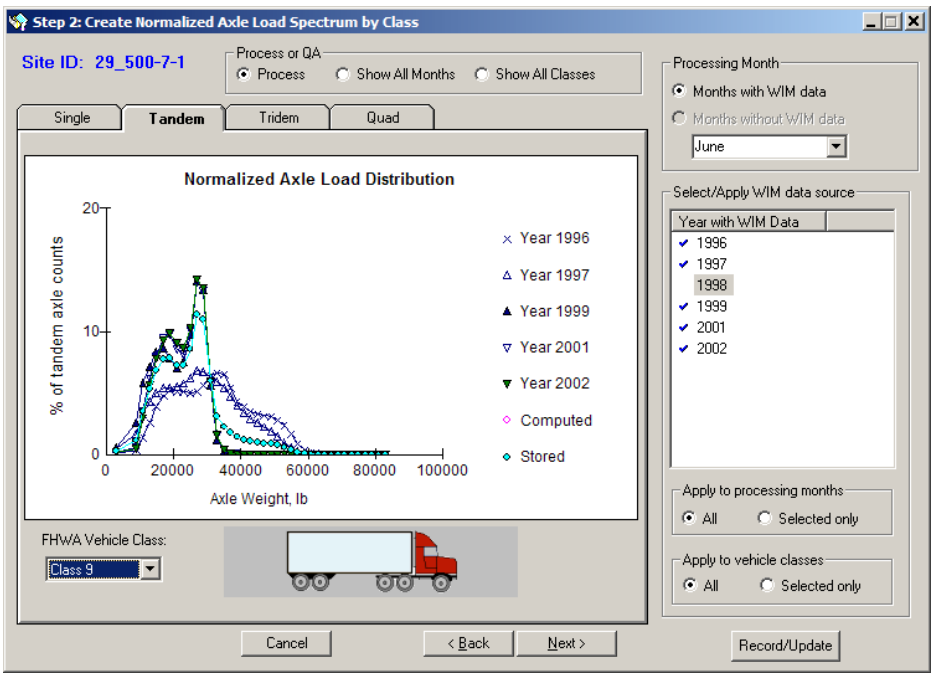


Figure A.75 Normalized tandem axle spectra for class 9 for site 29_500-7-1 (SHRP_ID 5473)

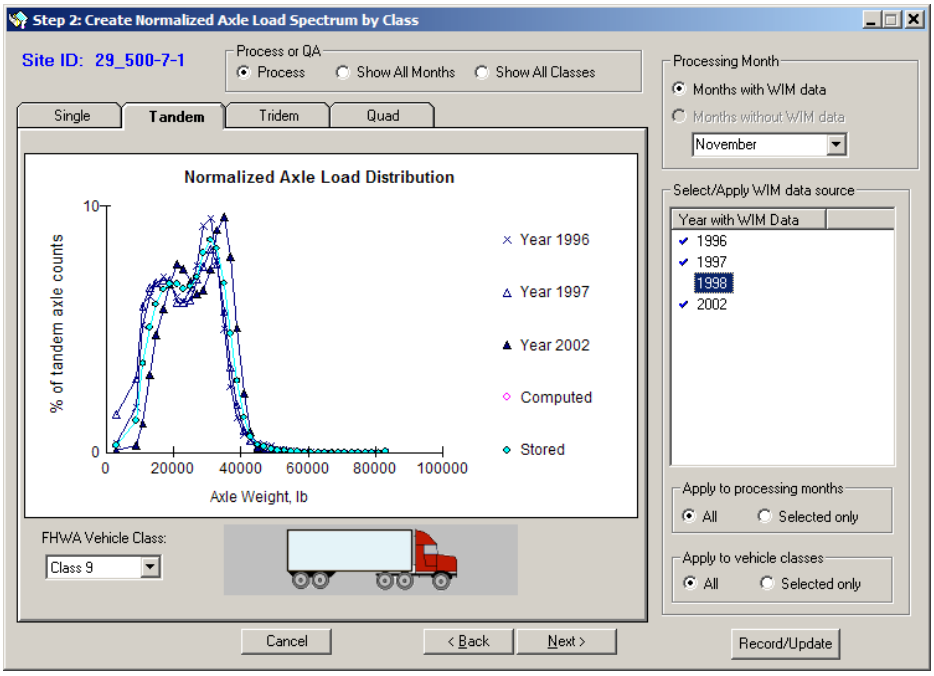


Figure A.76 Normalized tandem axle spectra for class 9 for site 29_500-7-1 (SHRP_ID 5473)

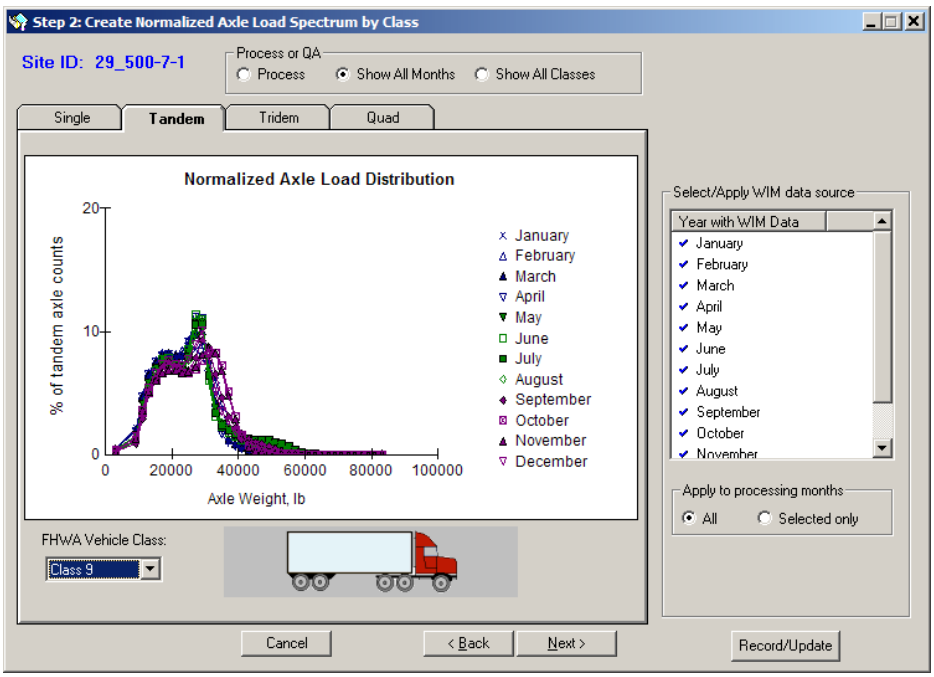


Figure A.77 Normalized tandem axle spectra for class 9 for site 29_500-7-1 (SHRP_ID 5473)

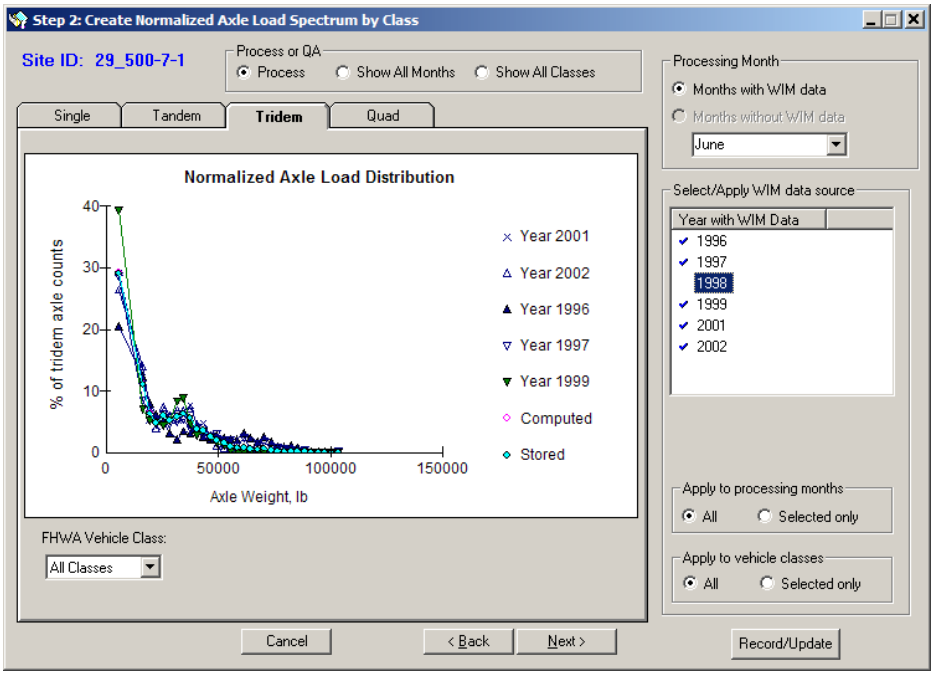
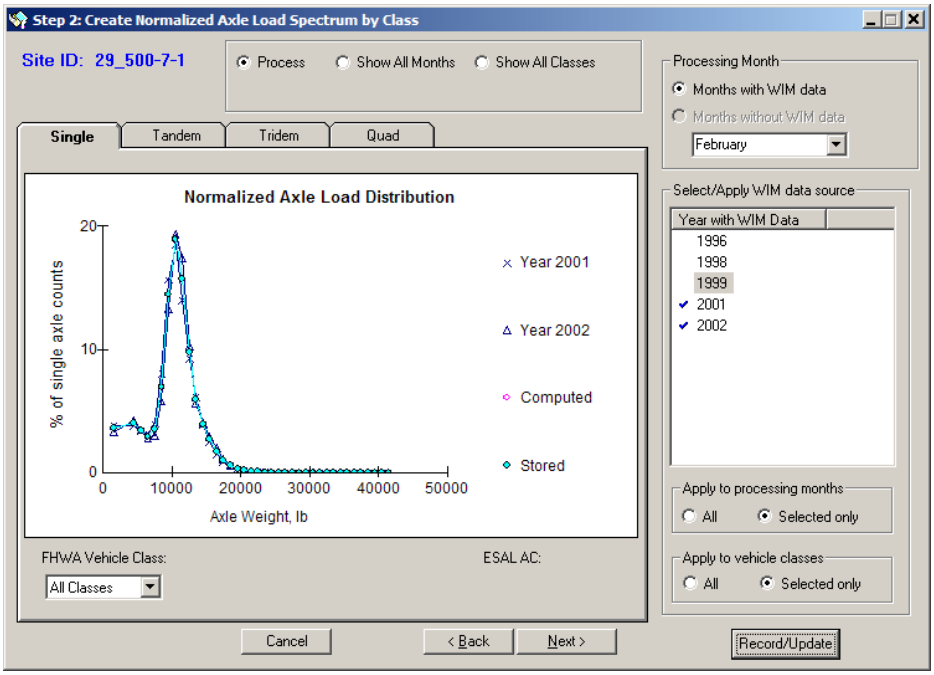


Figure A.78 Normalized tridem axle spectra for all classes for site 29_500-7-1 (SHRP_ID 5473)



Comment: Remove 96,97,98,99 data

Figure A.79 Normalized single axle spectra for all classes for site 29_500-7-1 (SHRP_ID 5473)

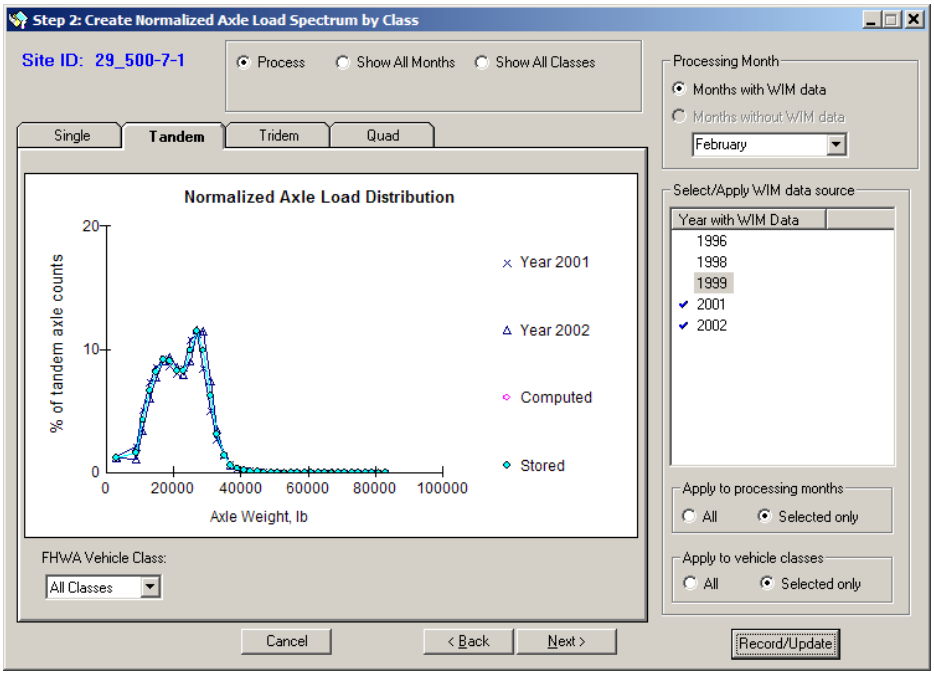
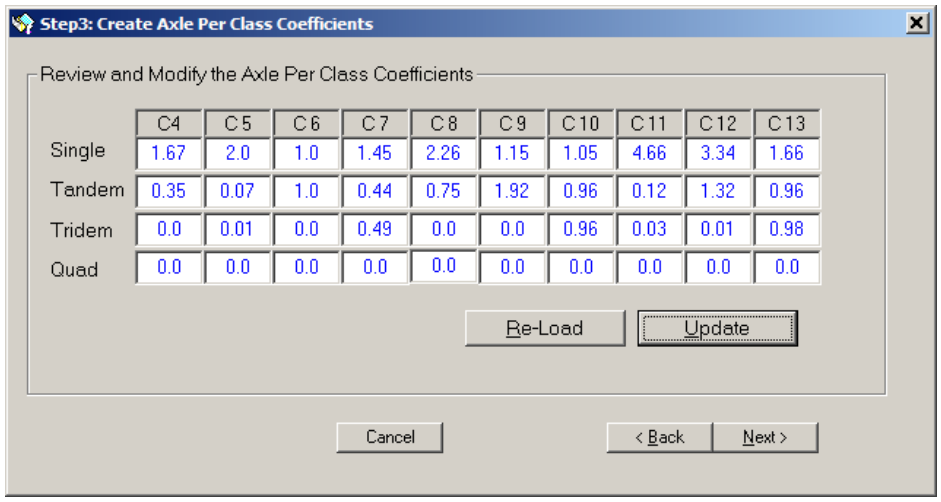


Figure A.80 Normalized tandem axle spectra for all classes for site 29_500-7-1 (SHRP_ID 5473)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.67	2.0	1.0	1.45	2.26	1.15	1.05	4.66	3.34	1.66
Tandem	0.35	0.07	1.0	0.44	0.75	1.92	0.96	0.12	1.32	0.96
Tridem	0.0	0.01	0.0	0.49	0.0	0.0	0.96	0.03	0.01	0.98
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.81 Axle class coefficients for site 29_500-7-1 (SHRP_ID 5473)

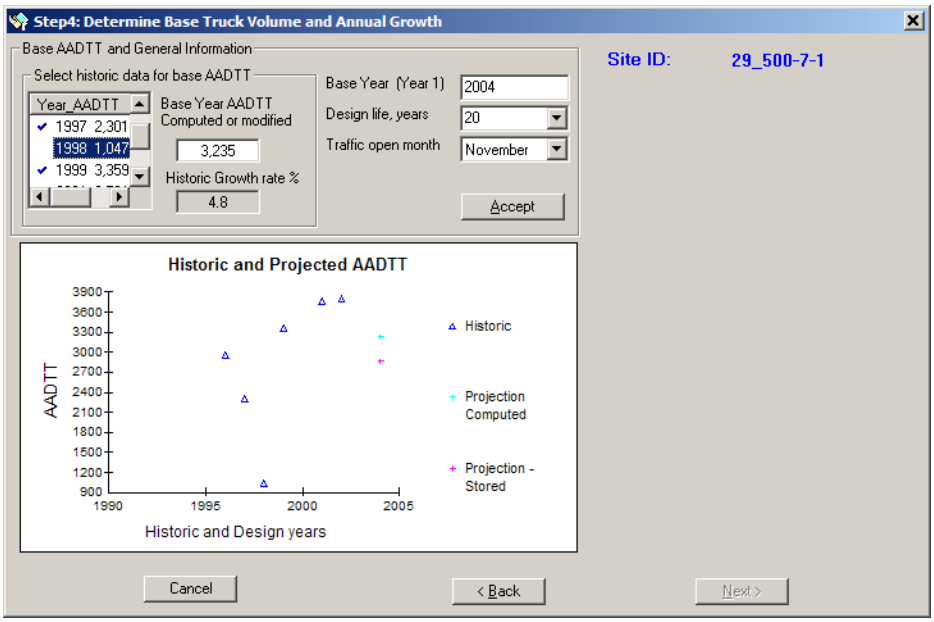


Figure A.82 Base truck volume and annual growth for site 29_500-7-1 (SHRP_ID 5473)

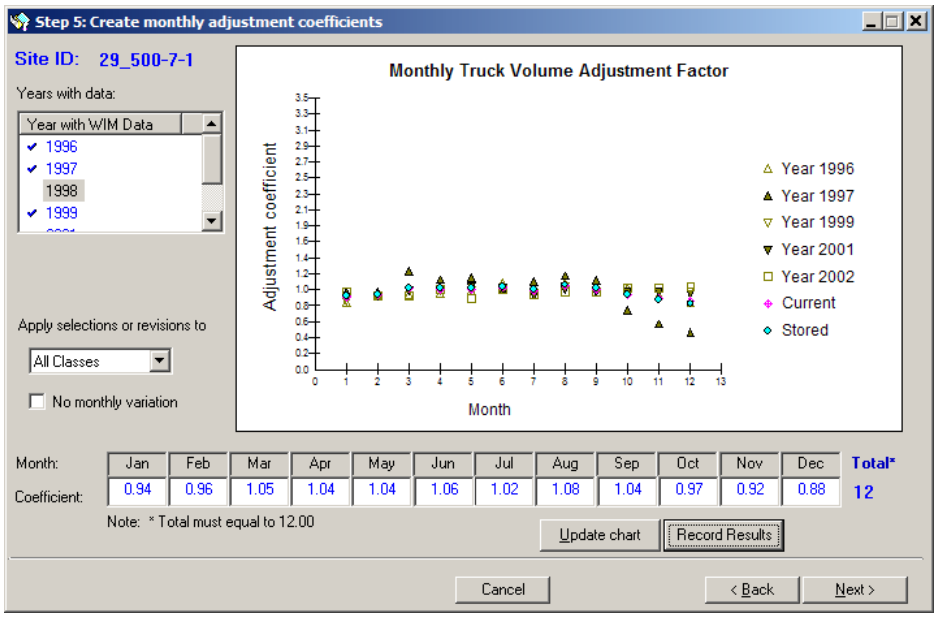
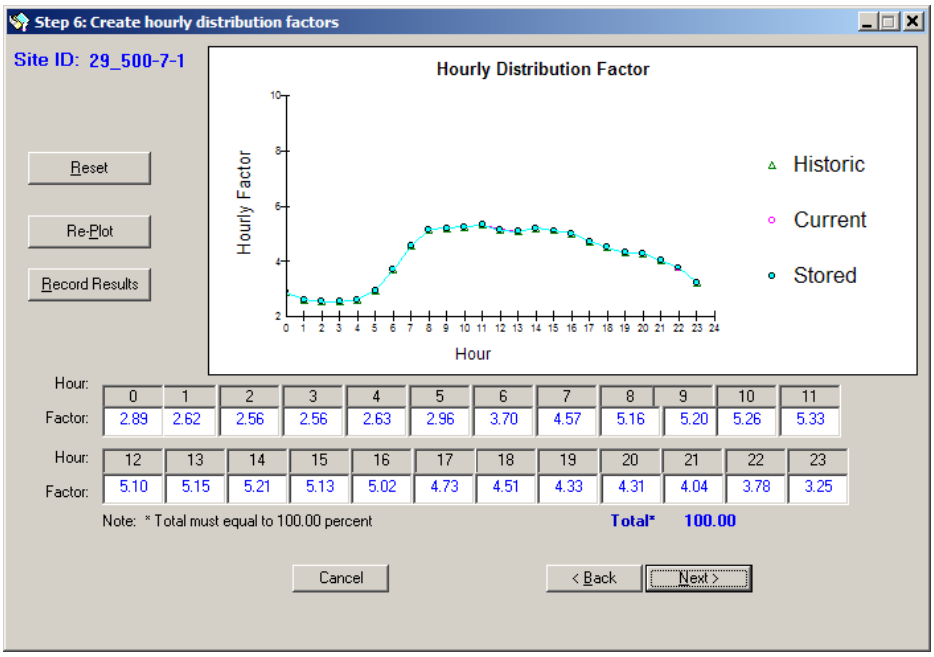


Figure A.83 Monthly truck volume adjustment coefficients for site 29_500-7-1 (SHRP_ID 5473)



Comment: peak 7AM – 9AM

Figure A.84 Hourly truck distribution factors for site 29_500-7-1 (SHRP_ID 5473)

SITE 29_930-3-1 (SHRP_ID 1010)

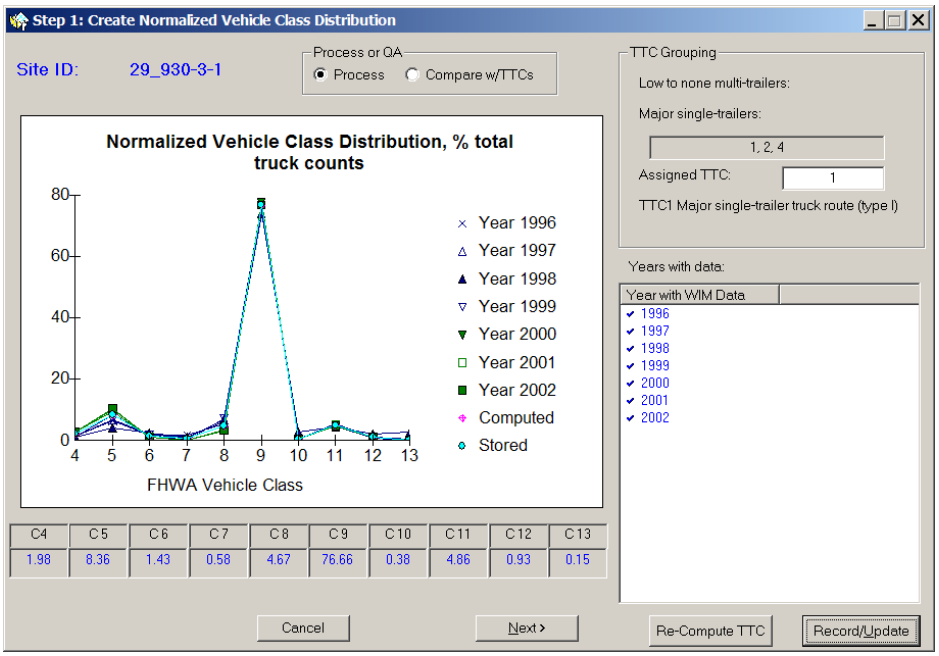


Figure A.85 Normalized vehicle class distribution for site 29_930-3-1 (SHRP_ID 1010)

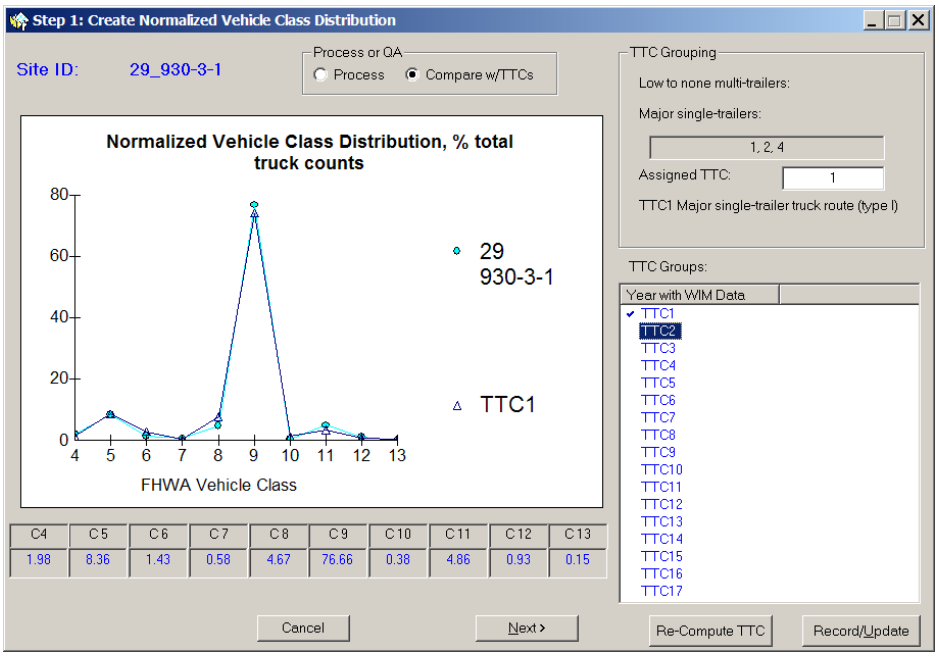


Figure A.86 Normalized vehicle class distribution compared with TTC for site 29_930-3-1 (SHRP_ID 1010)

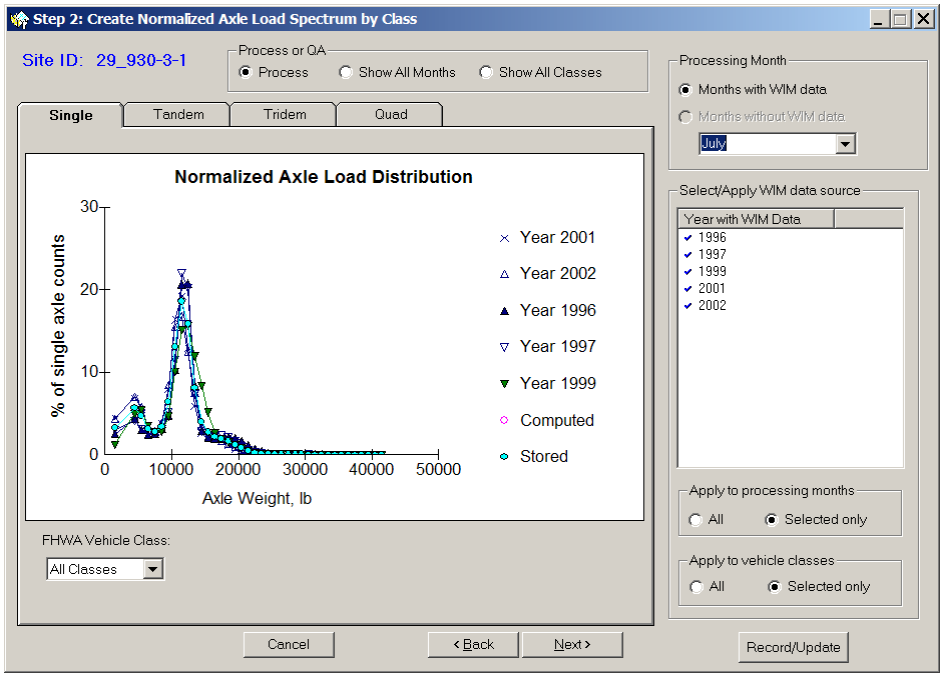


Figure A.87 Normalized single axle spectra for all classes for site 29_930-3-1 (SHRP_ID 1010)

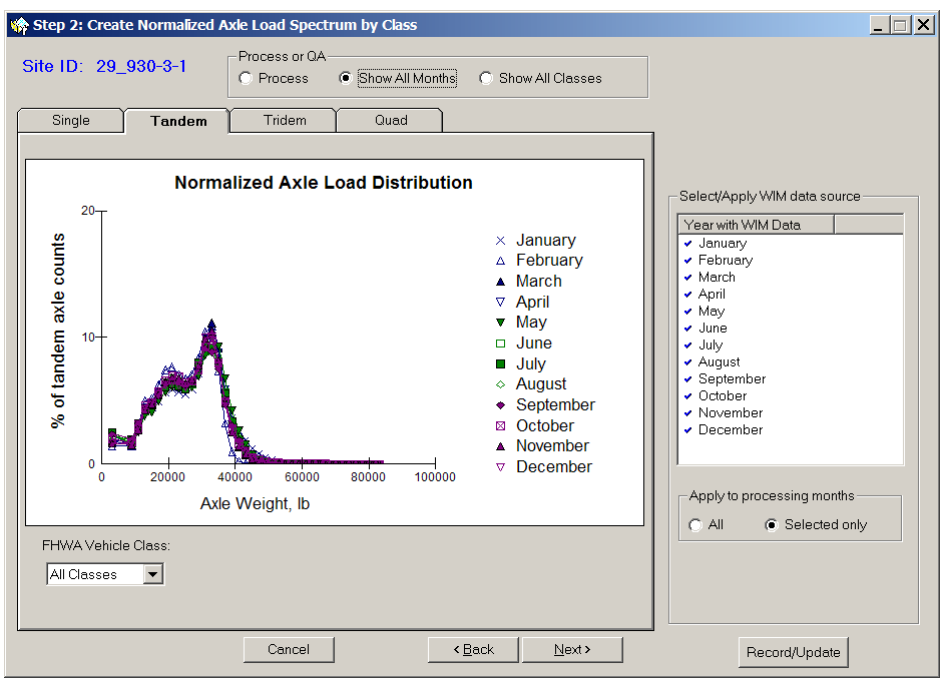
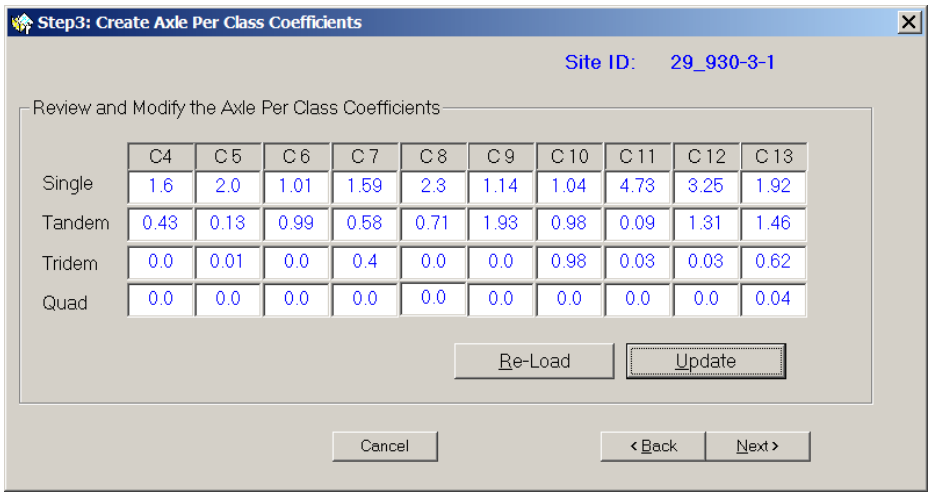


Figure A.88 Normalized tandem axle spectra for all classes for site 29_930-3-1 (SHRP_ID 1010)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.6	2.0	1.01	1.59	2.3	1.14	1.04	4.73	3.25	1.92
Tandem	0.43	0.13	0.99	0.58	0.71	1.93	0.98	0.09	1.31	1.46
Tridem	0.0	0.01	0.0	0.4	0.0	0.0	0.98	0.03	0.03	0.62
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.04

Figure A.89 Axle class coefficients for site 29_930-3-1 (SHRP_ID 1010)

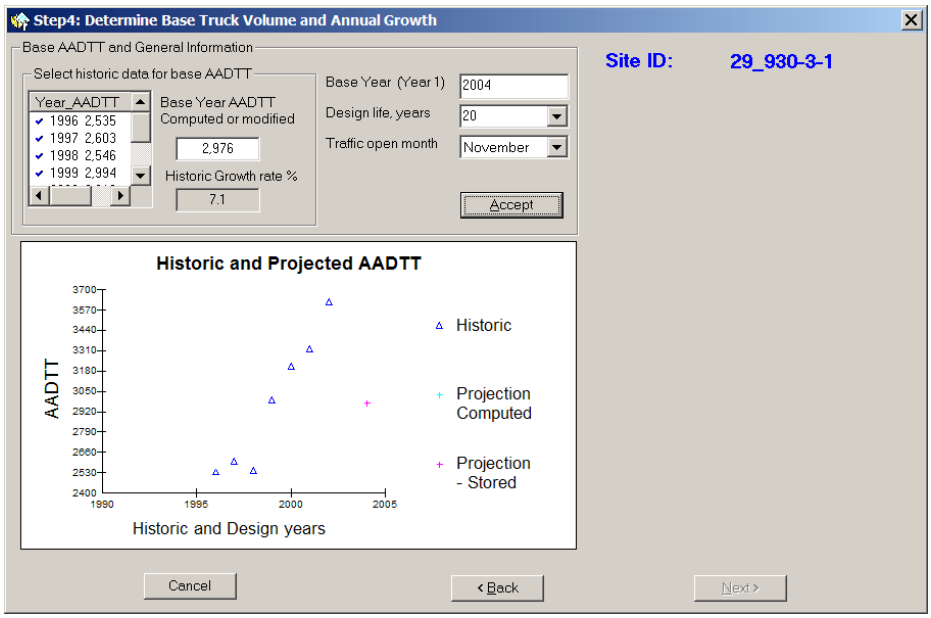


Figure A.90 Base truck volume and annual growth for site 29_930-3-1 (SHRP_ID 1010)

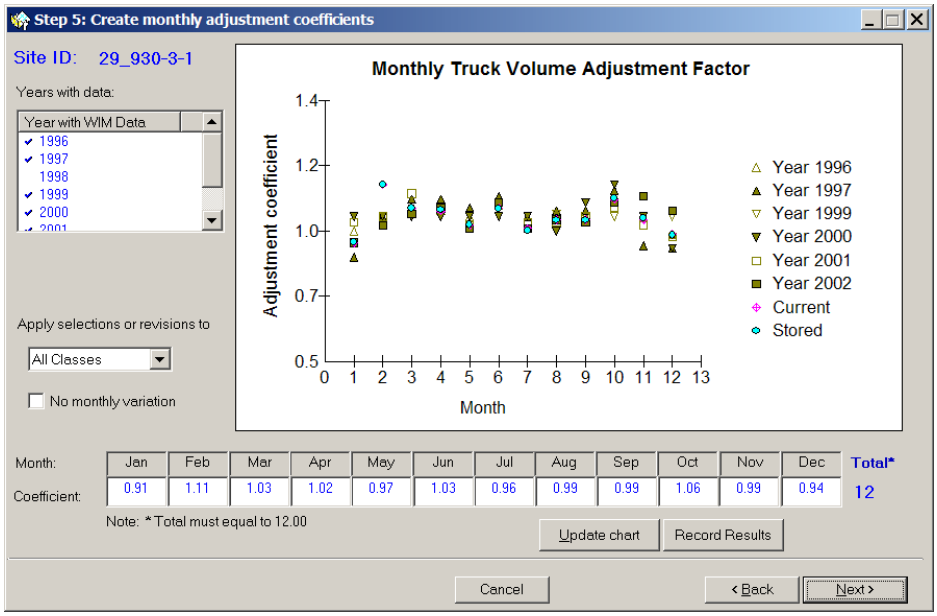
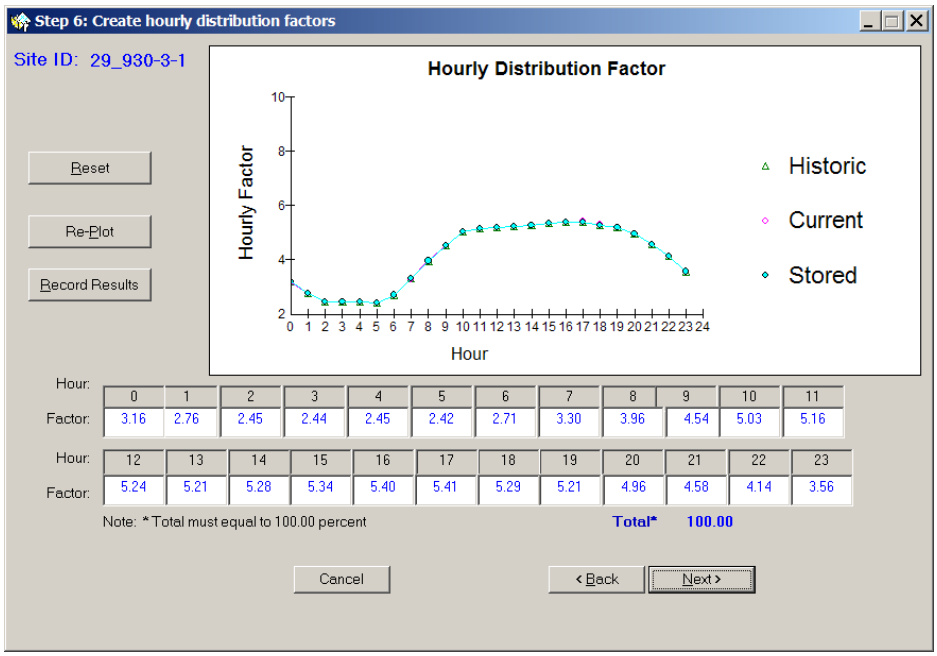


Figure A.91 Monthly truck volume adjustment coefficients for site 29_930-3-1 (SHRP_ID 1010)



Comment: flat throughout the day (9am-9pm)

Figure A.92 Hourly truck distribution factors for site 29_930-3-1 (SHRP_ID 1010)

SITE 29_420-1-1 (SHRP_ID 4069)

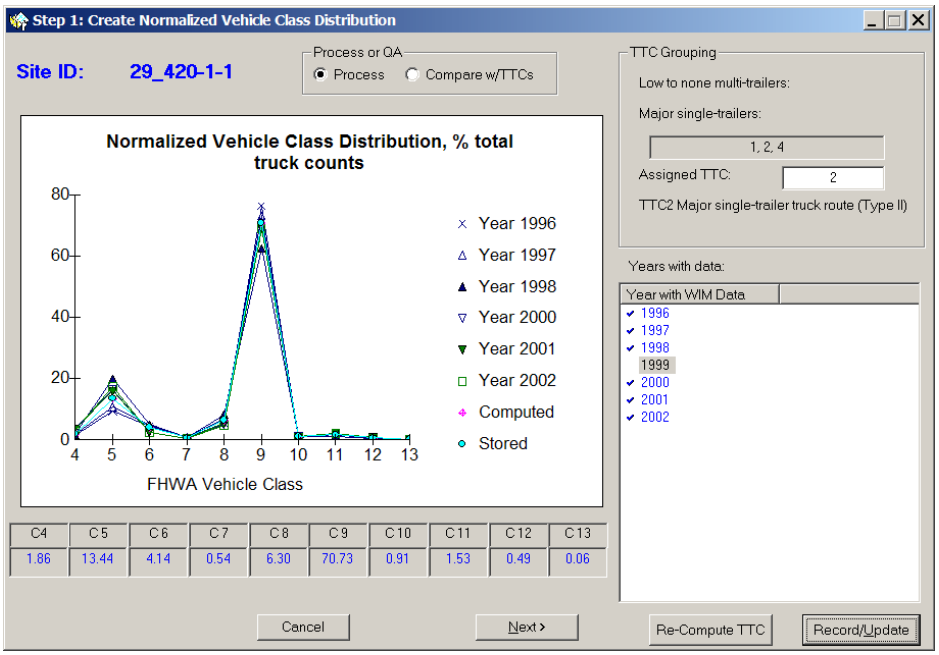


Figure A.93 Normalized vehicle class distribution for site 29_420-1-1 (SHRP_ID 4069)

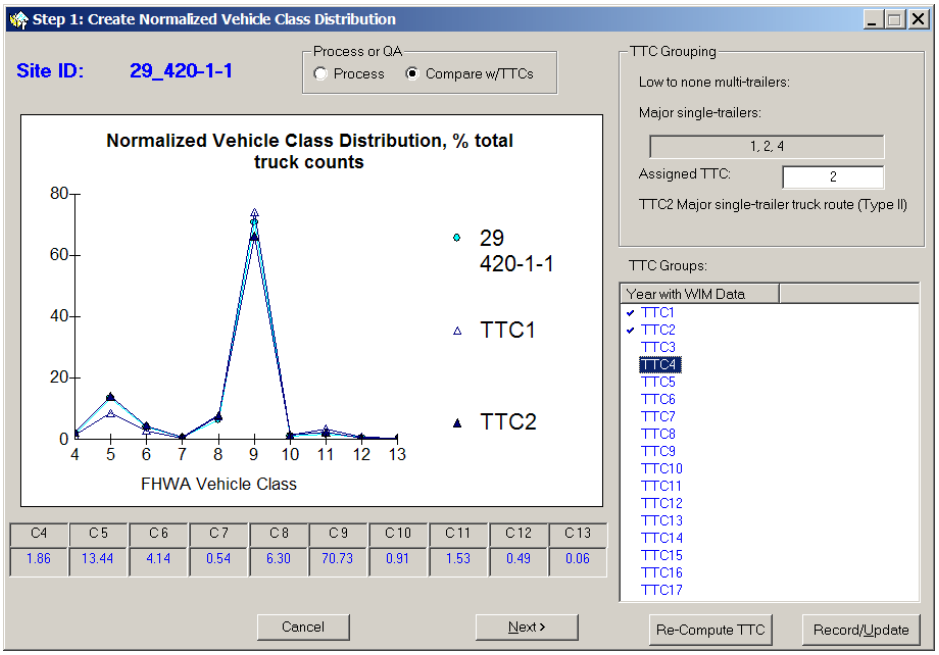


Figure A.94 Normalized vehicle class distribution compared with TTCs for site 29_420-1-1 (SHRP_ID 4069)

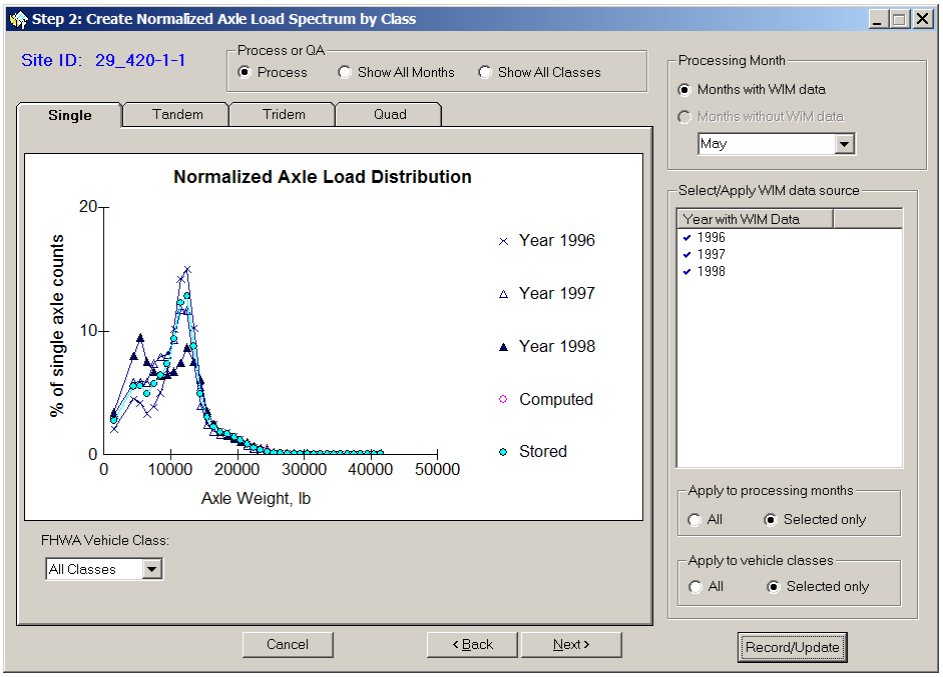


Figure A.95 Normalized single axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

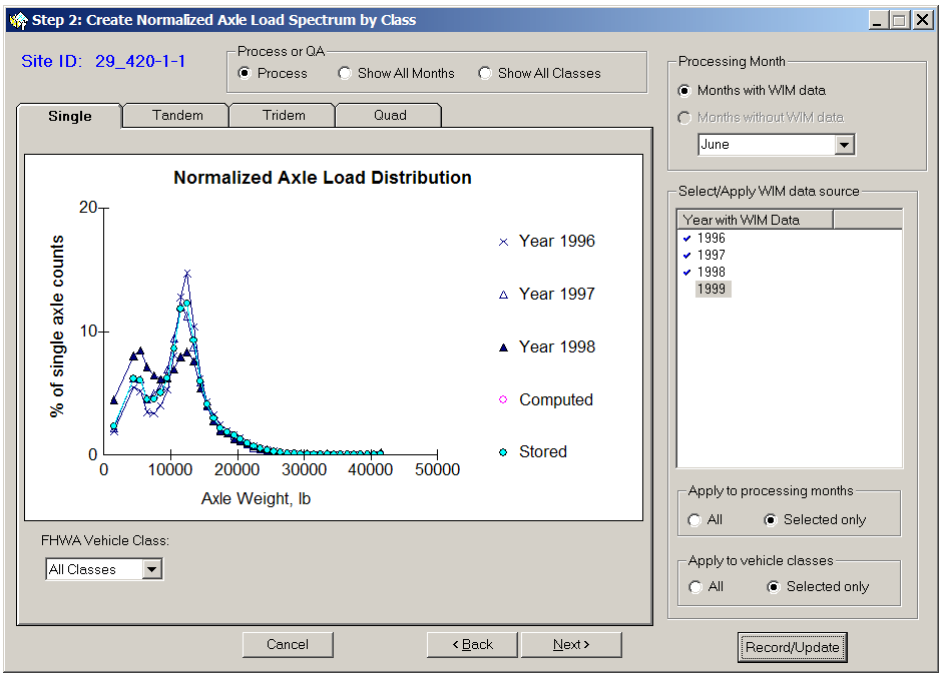


Figure A.96 Normalized single axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

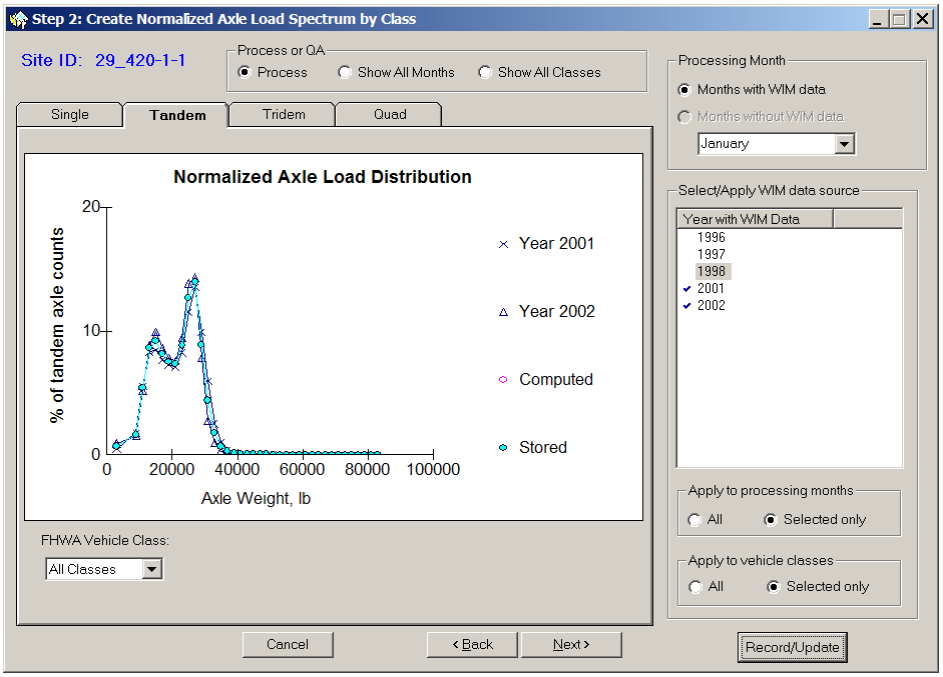
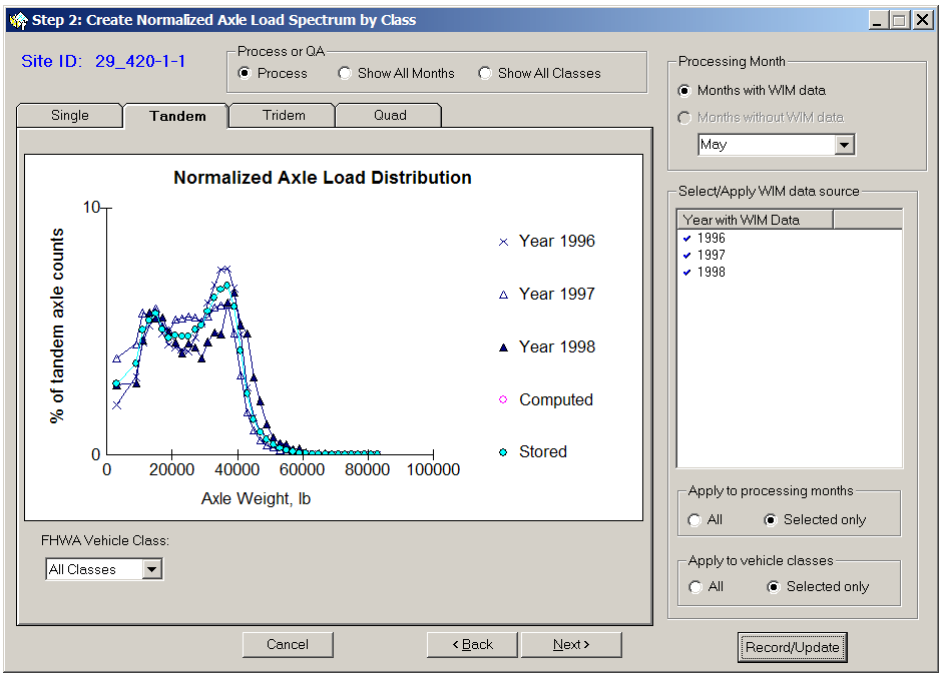


Figure A.97 Normalized tandem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)



Comment: Loaded using only the recent year's data

Figure A.98 Normalized tandem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

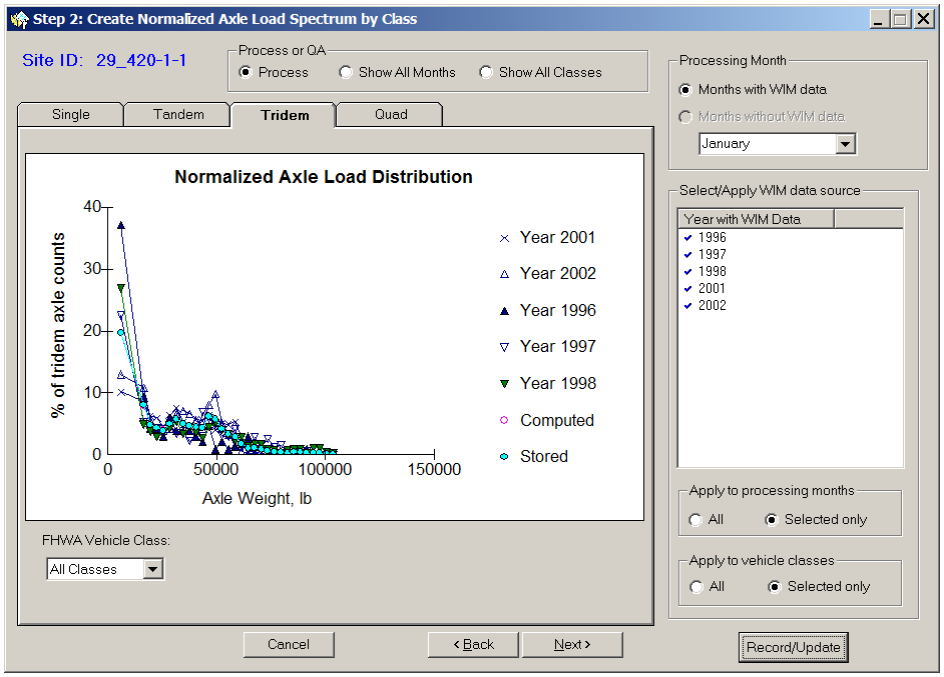
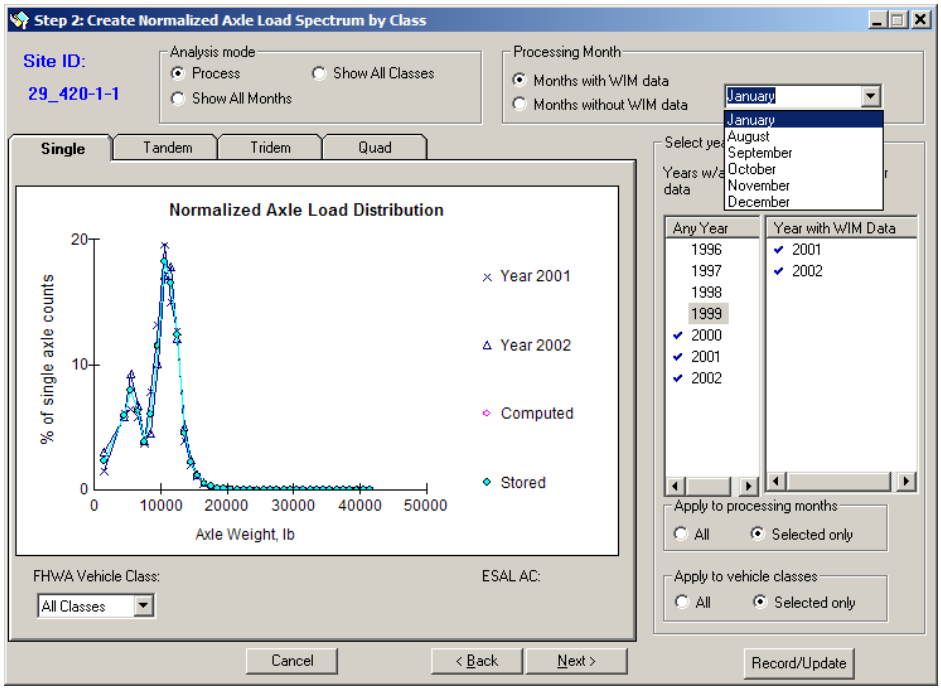


Figure A.99 Normalized tridem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)



Comment: remove suspect data (1996, 97, 98, 99) as a result, some months now do not have historic WIM data (2,3,4,5,6,7)

Figure A.100 Normalized single axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

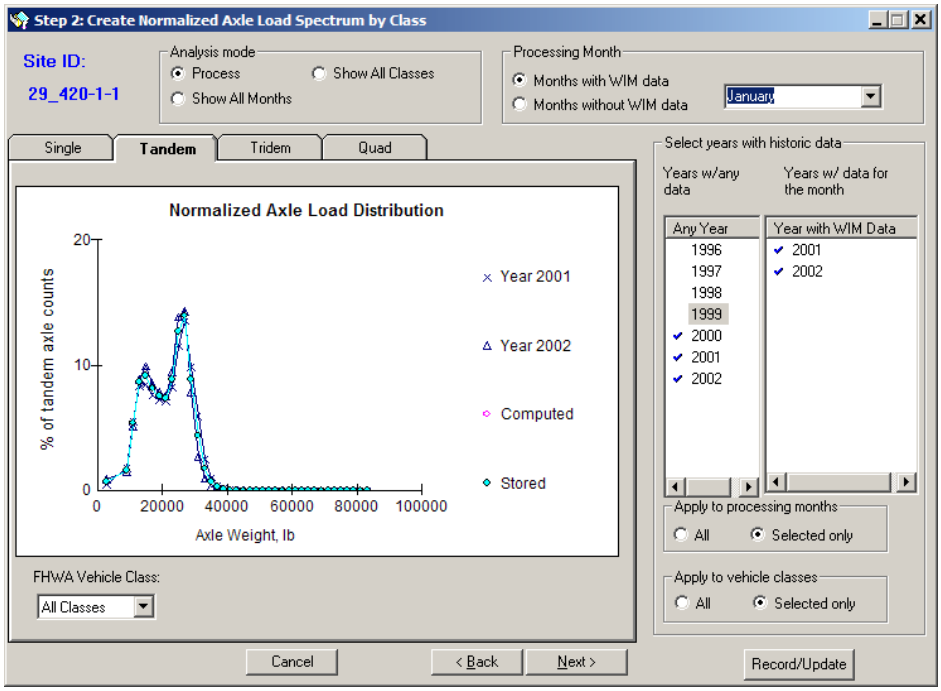


Figure A.101 Normalized tandem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

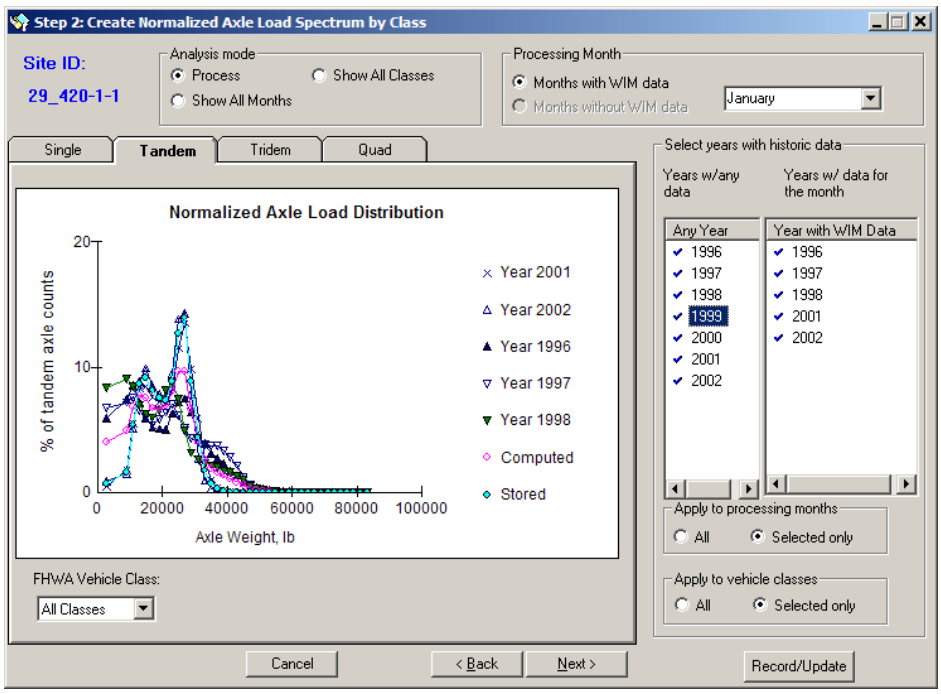


Figure A.102 Normalized tandem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)

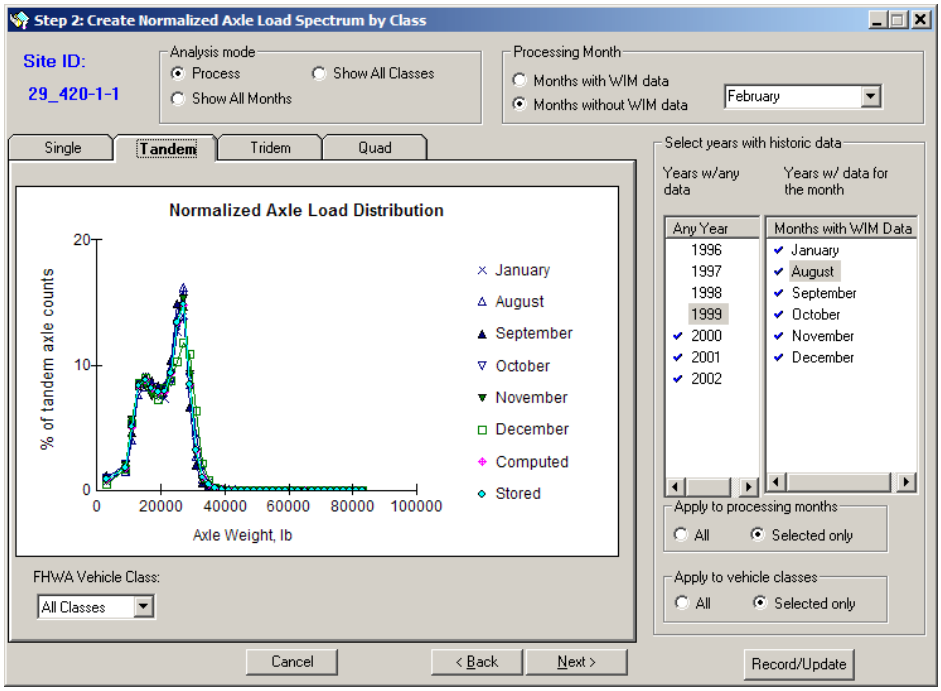
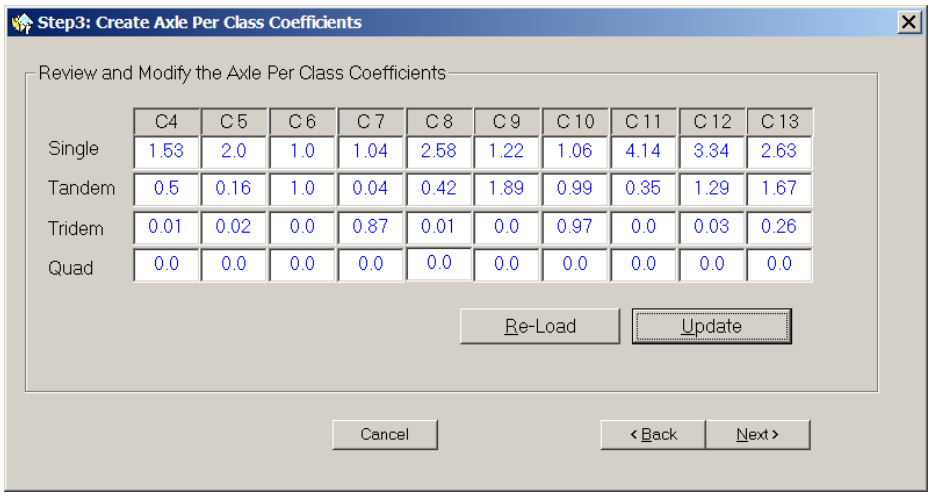


Figure A.103 Normalized tandem axle spectra for all classes for site 29_420-1-1 (SHRP_ID 4069)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.53	2.0	1.0	1.04	2.58	1.22	1.06	4.14	3.34	2.63
Tandem	0.5	0.16	1.0	0.04	0.42	1.89	0.99	0.35	1.29	1.67
Tridem	0.01	0.02	0.0	0.87	0.01	0.0	0.97	0.0	0.03	0.26
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.104 Axle class coefficients for site 29_420-1-1 (SHRP_ID 4069)

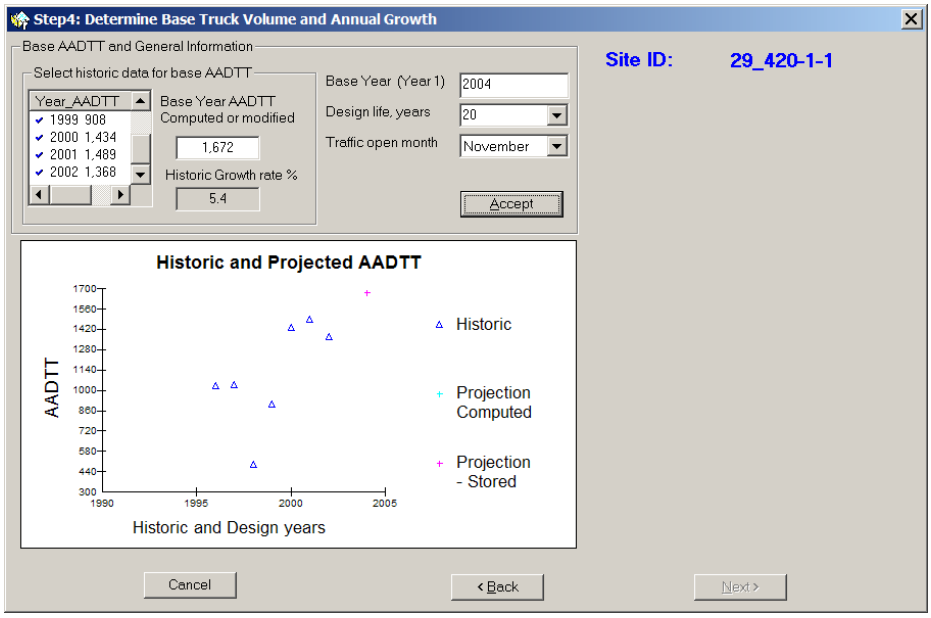


Figure A.105 Base truck volume and annual growth for site 29_420-1-1 (SHRP_ID 4069)

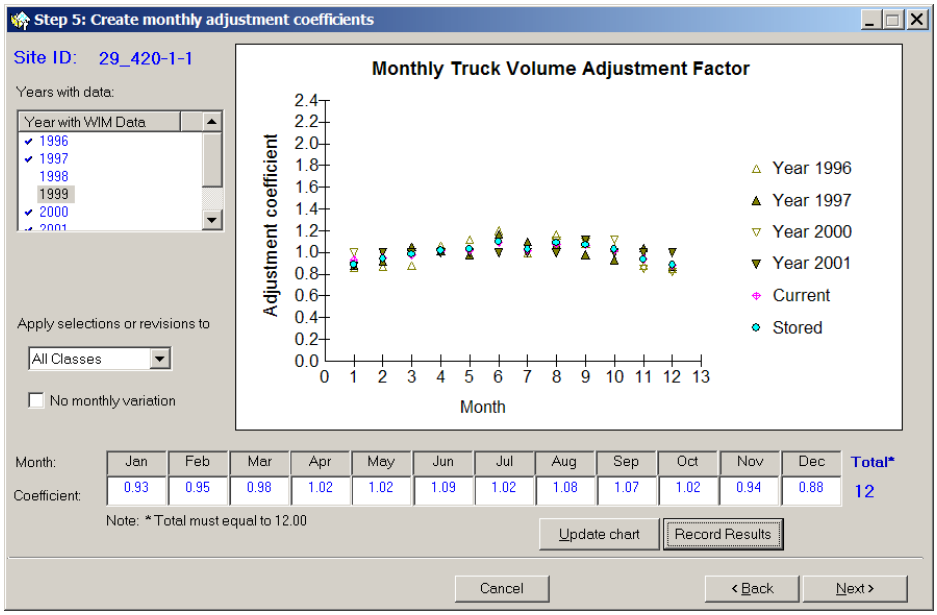


Figure A.106 Monthly truck volume adjustment coefficients for site 29_420-1-1 (SHRP_ID 4069)

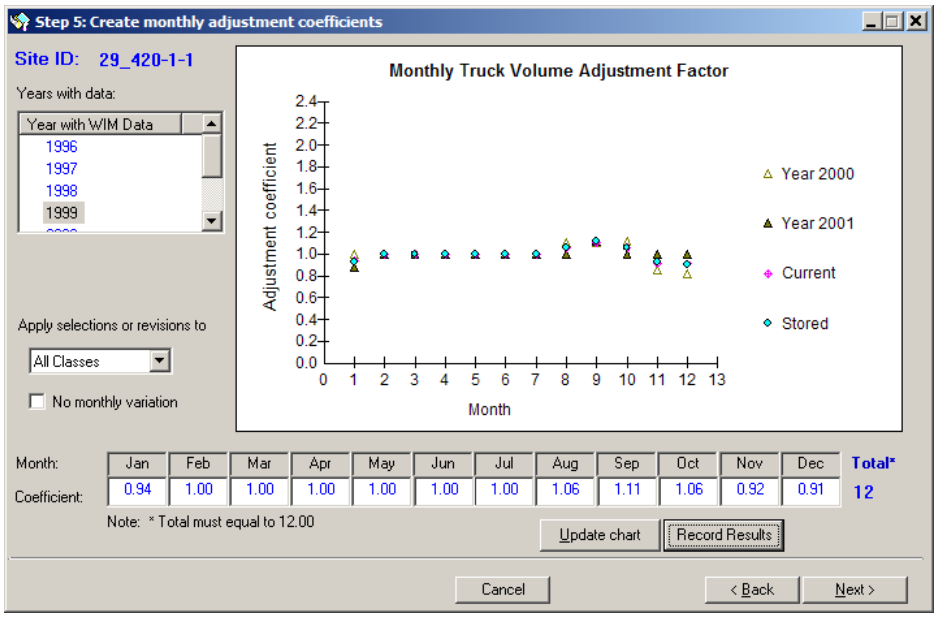
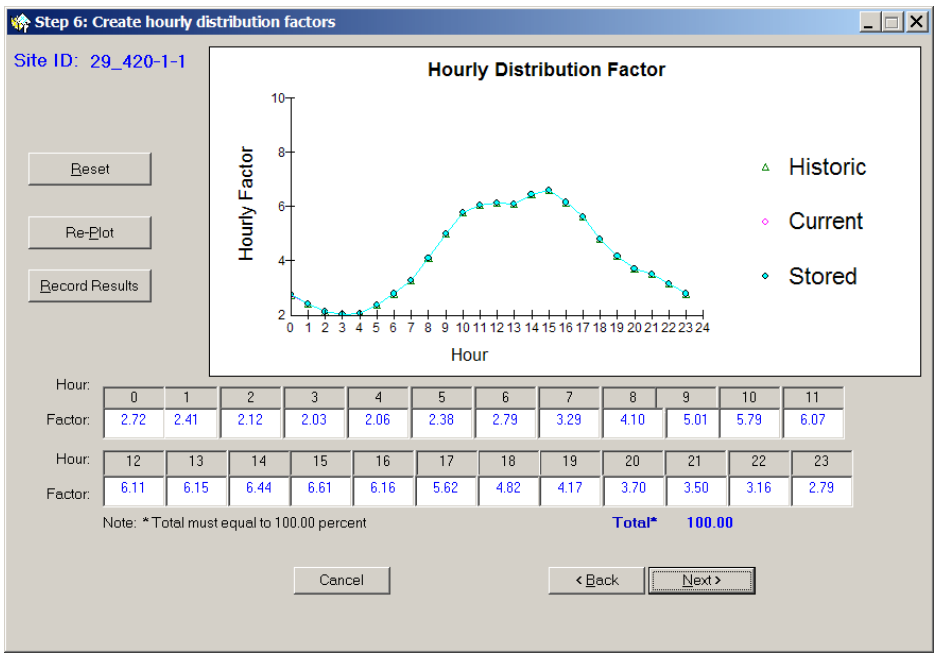


Figure A.107 Monthly truck volume adjustment coefficients for site 29_420-1-1 (SHRP_ID 4069)



Comment: peak mid-day 10AM- 4PM

Figure A.108 Hourly truck distribution factors for site 29_420-1-1 (SHRP_ID 4069)

SITE 29_760-3-1 (SHRP_ID 7054)

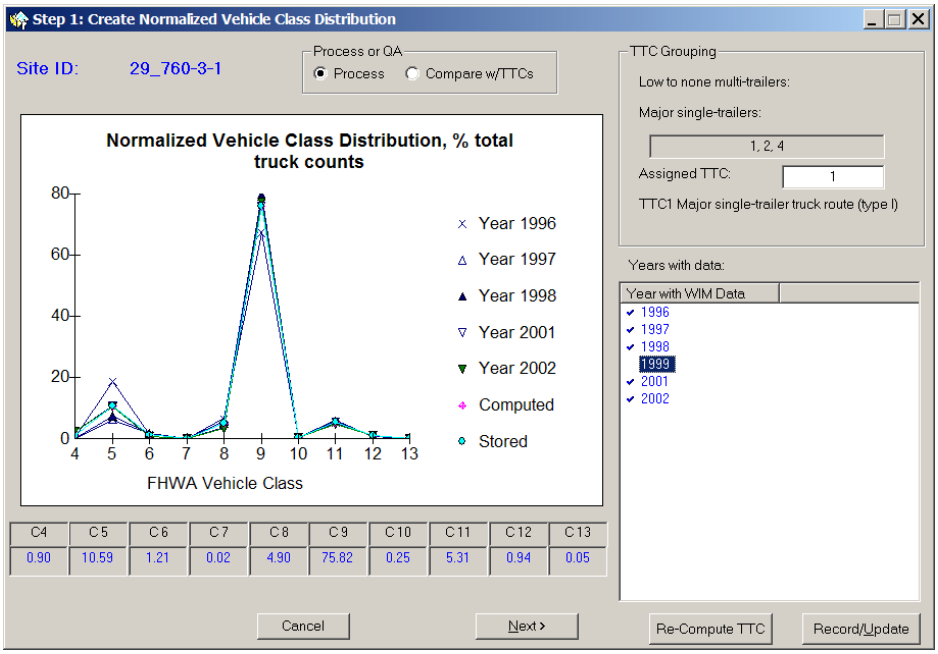


Figure A.109 Normalized vehicle class distribution for site 29_760-3-1 (SHRP_ID 7054)

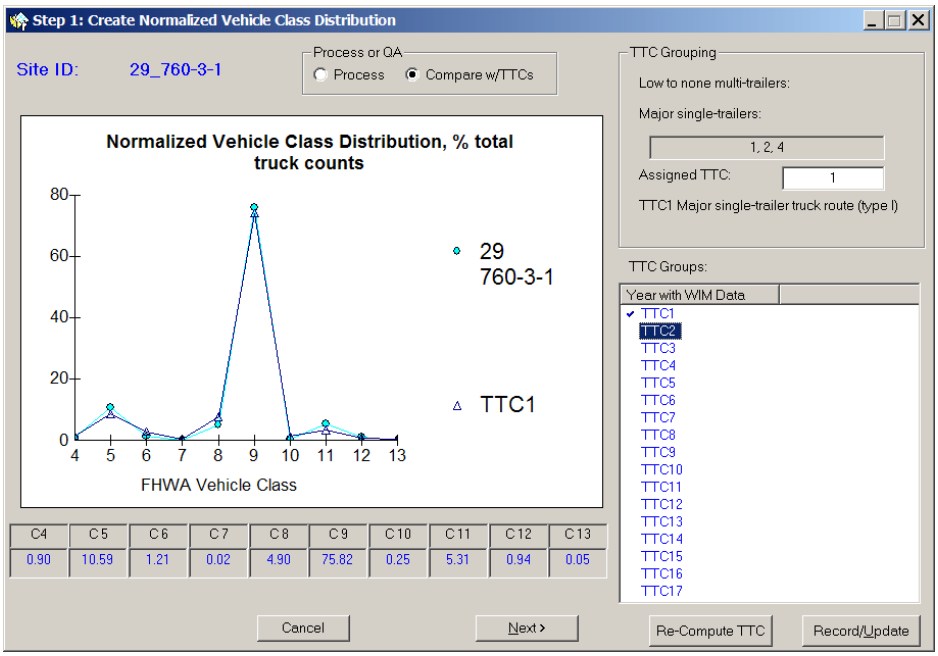


Figure A.110 Normalized vehicle class distribution compared with TTC for site 29_760-3-1 (SHRP_ID 7054)

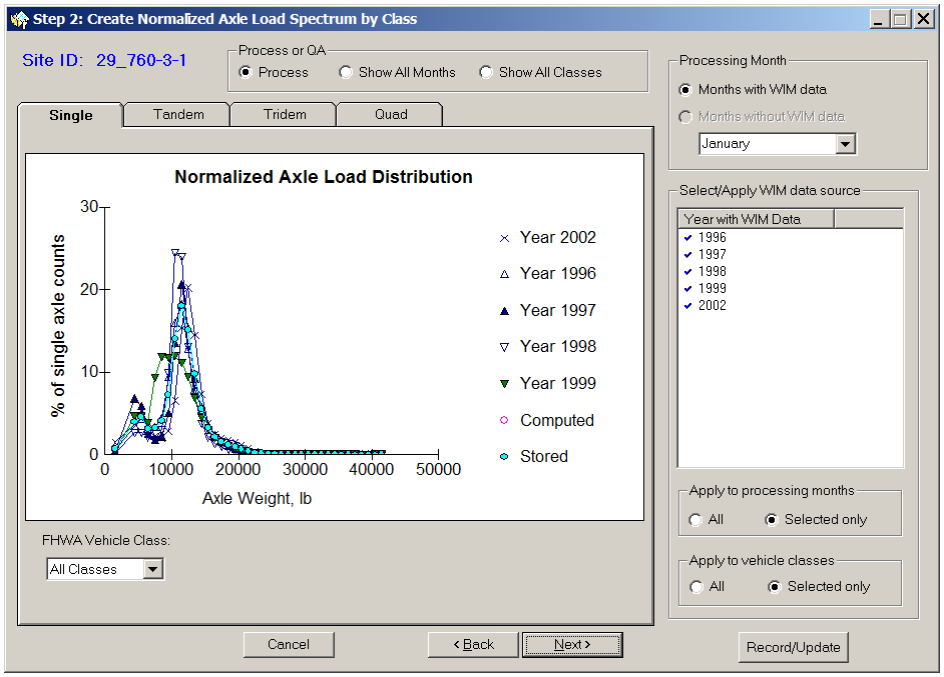
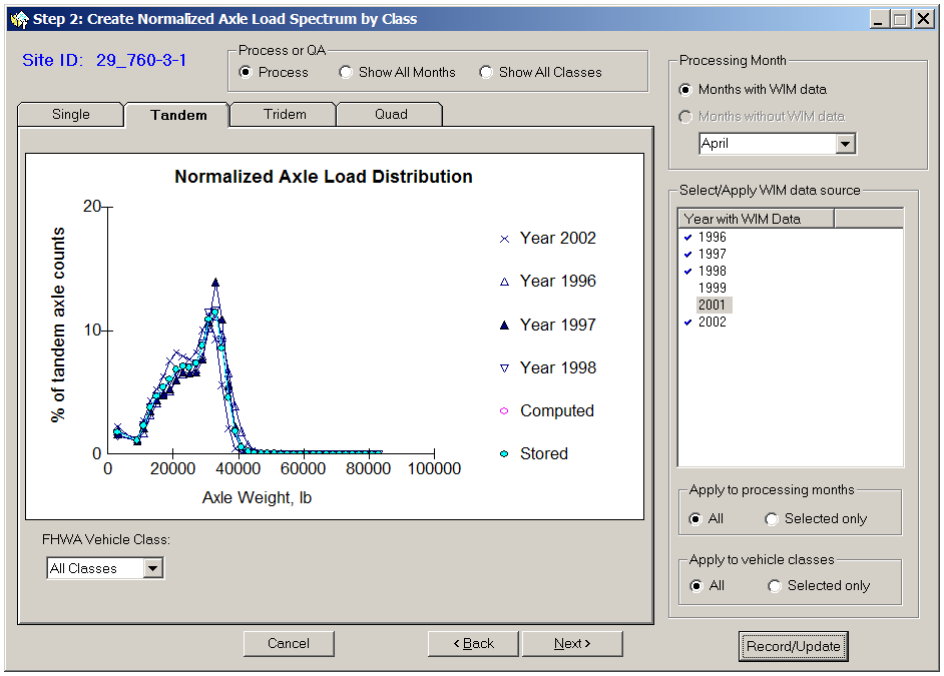


Figure A.111 Normalized single axle spectra for all classes for site 29_760-3-1 (SHRP_ID 7054)



Comment: excludes 1999 & 2001

Figure A.112 Normalized tandem axle spectra by class for site 29_760-3-1 (SHRP_ID 7054)

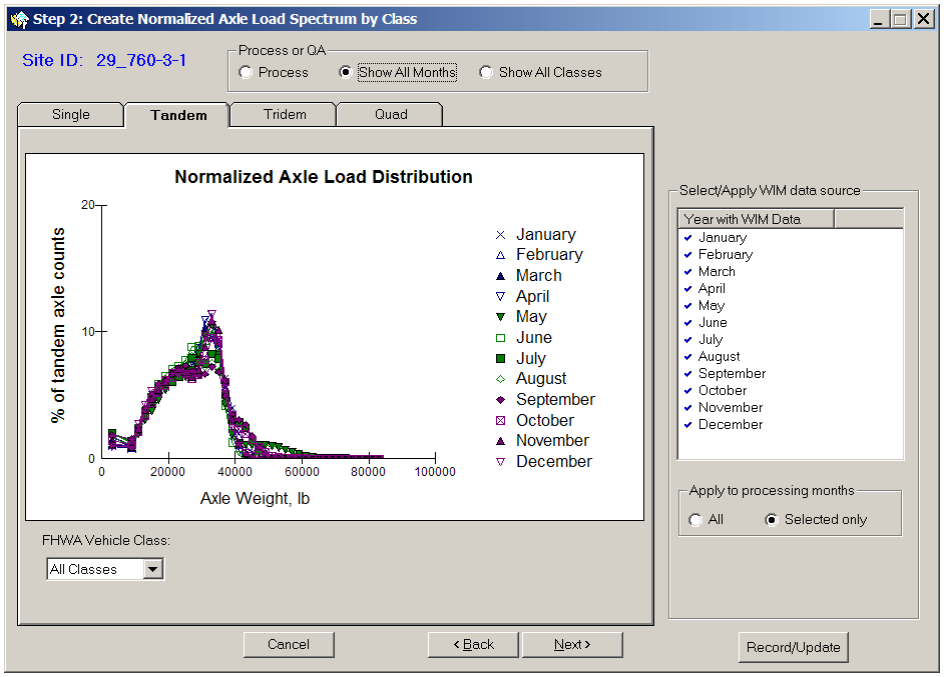
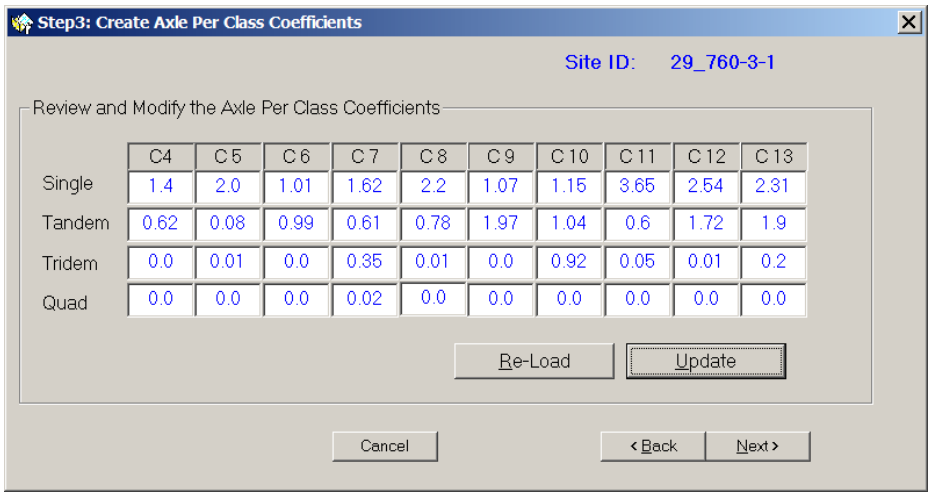


Figure A.113 Normalized tandem axle spectra by class for site 29_760-3-1 (SHRP_ID 7054)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.4	2.0	1.01	1.62	2.2	1.07	1.15	3.65	2.54	2.31
Tandem	0.62	0.08	0.99	0.61	0.78	1.97	1.04	0.6	1.72	1.9
Tridem	0.0	0.01	0.0	0.35	0.01	0.0	0.92	0.05	0.01	0.2
Quad	0.0	0.0	0.0	0.02	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.114 Axle class coefficients for site 29_760-3-1 (SHRP_ID 7054)

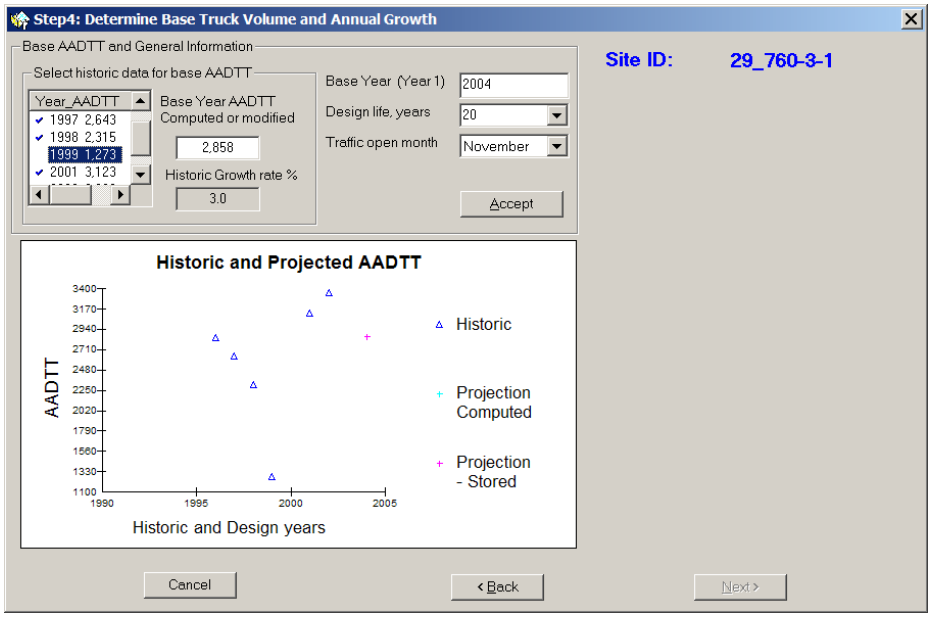


Figure A.115 Base truck volume and annual growth for site 29_760-3-1 (SHRP_ID 7054)

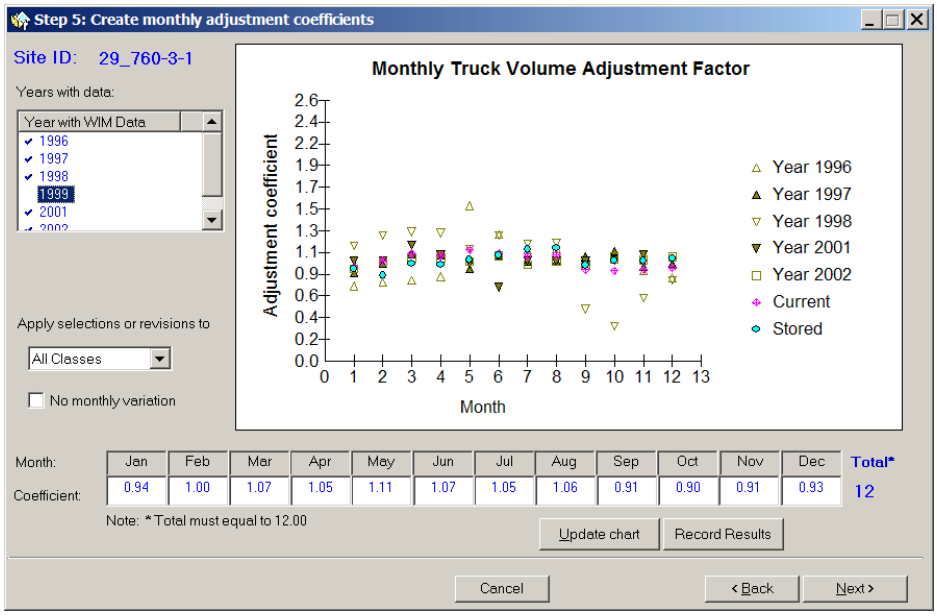
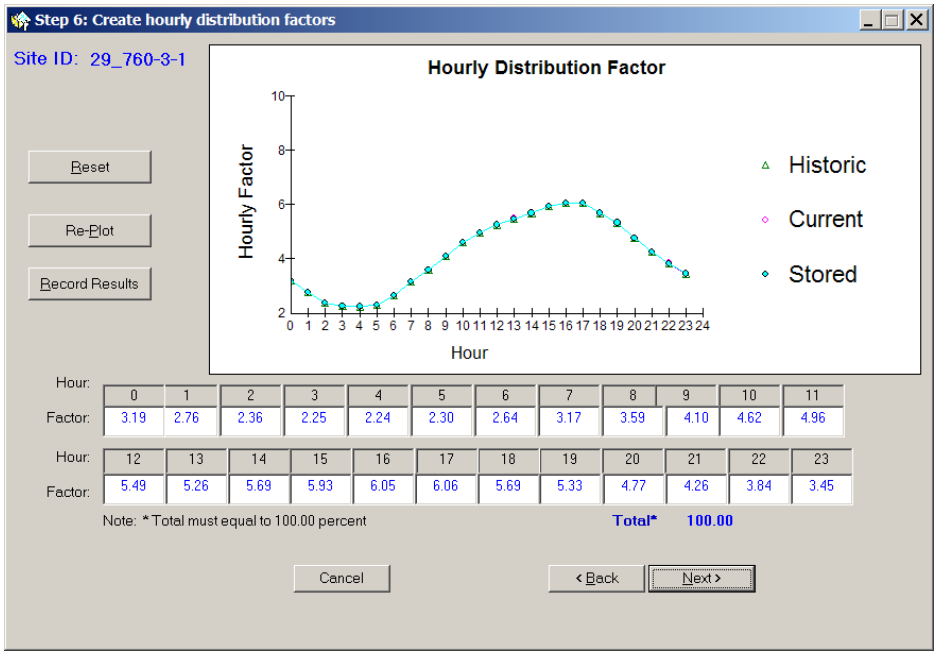


Figure A.116 Monthly truck volume adjustment coefficients for site 29_760-3-1 (SHRP_ID 7054)



Comment: peak values are in the PM

Figure A.117 Hourly truck distribution factors for site 29_760-3-1 (SHRP_ID 7054)

SITE 29_440-5-1 (SHRP_ID 4069)

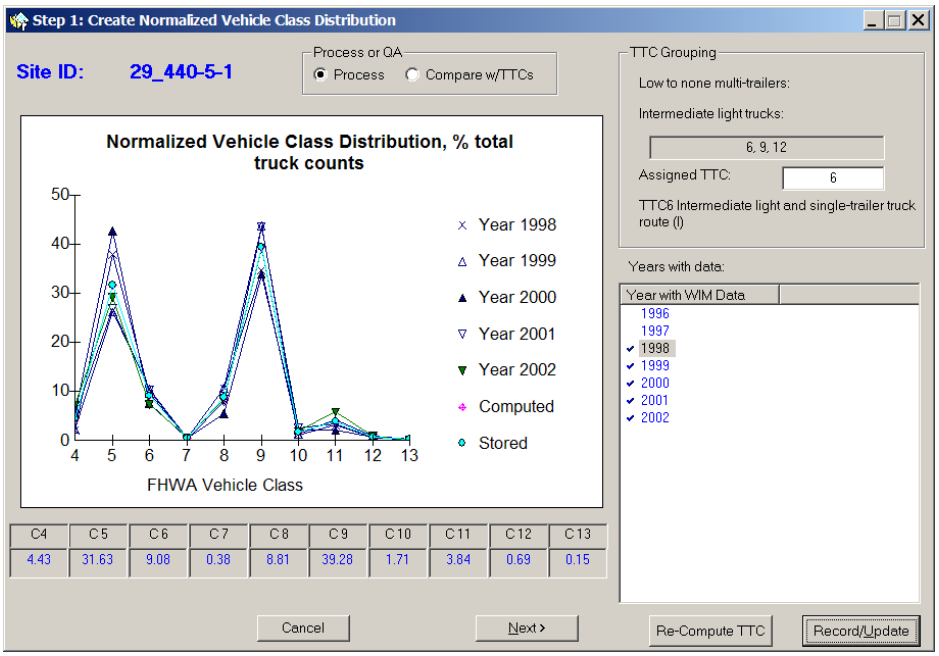


Figure A.118 Normalized vehicle class distribution for site 29_440-5-1 (SHRP_ID 4069)

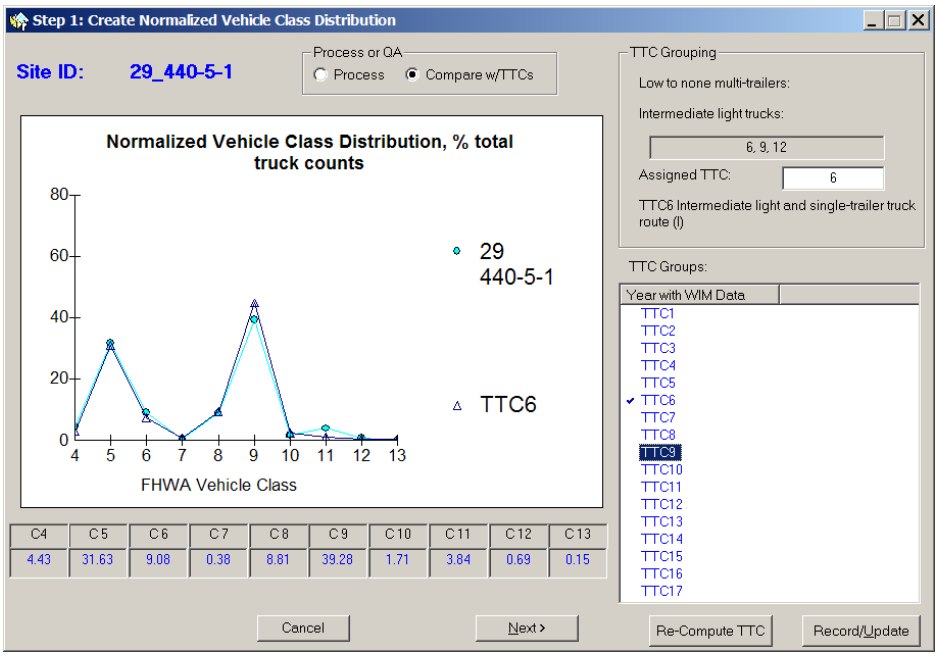


Figure A.119 Normalized vehicle class distribution compared with TTC for site 29_440-5-1 (SHRP_ID 4069)

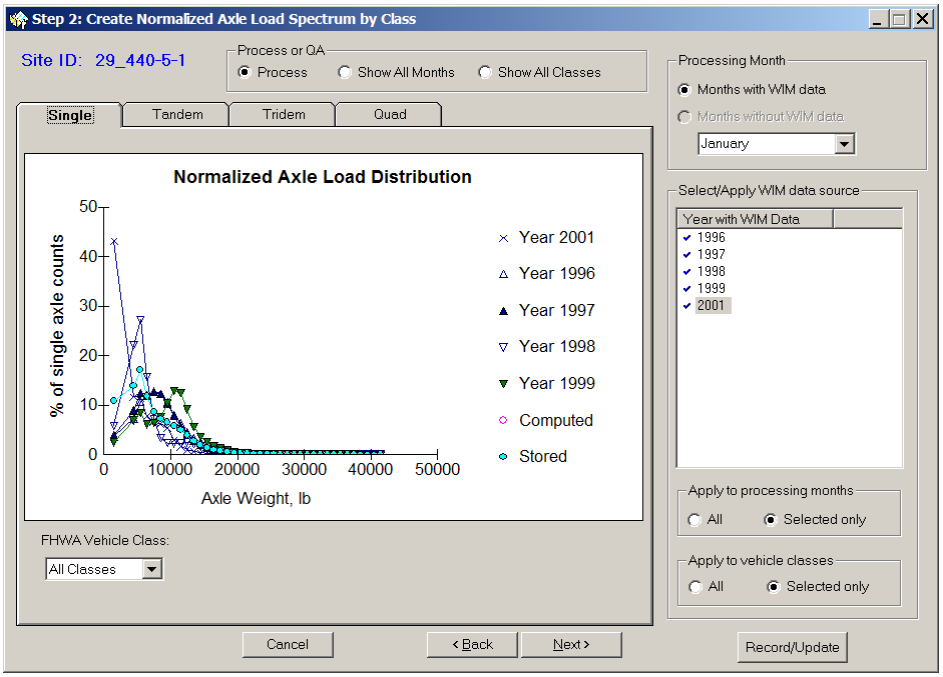


Figure A.120 Normalized single axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)

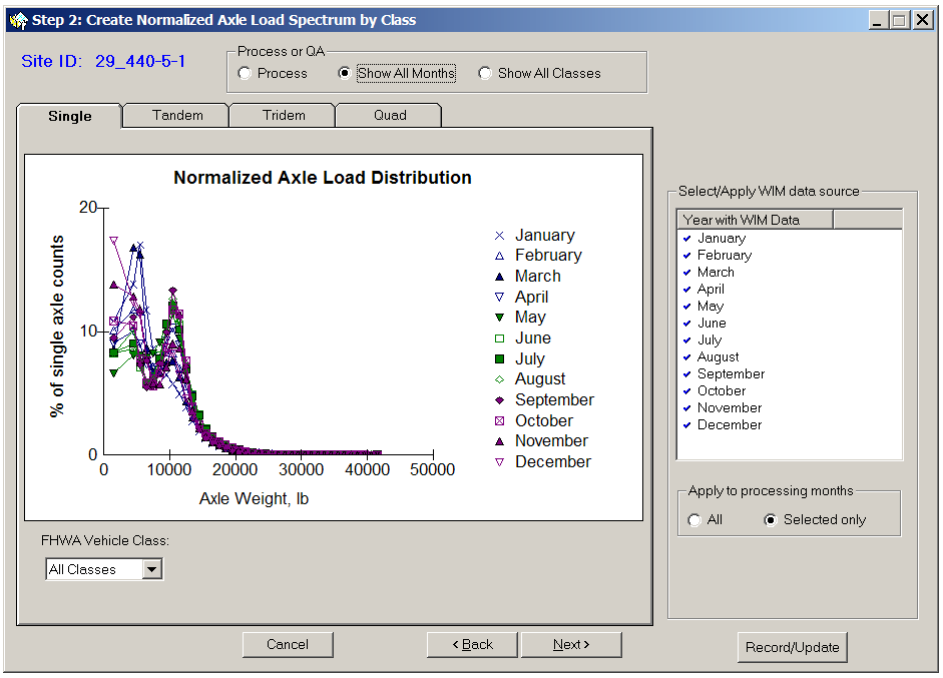


Figure A.121 Normalized single axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)

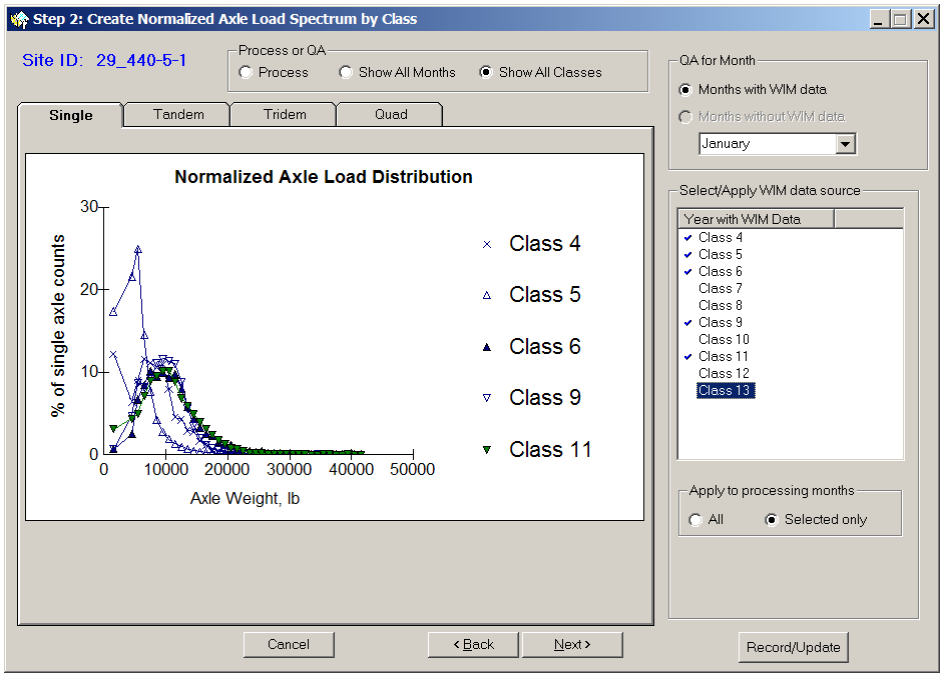


Figure A.122 Normalized single axle spectra by class for site 29_440-5-1 (SHRP_ID 4069)

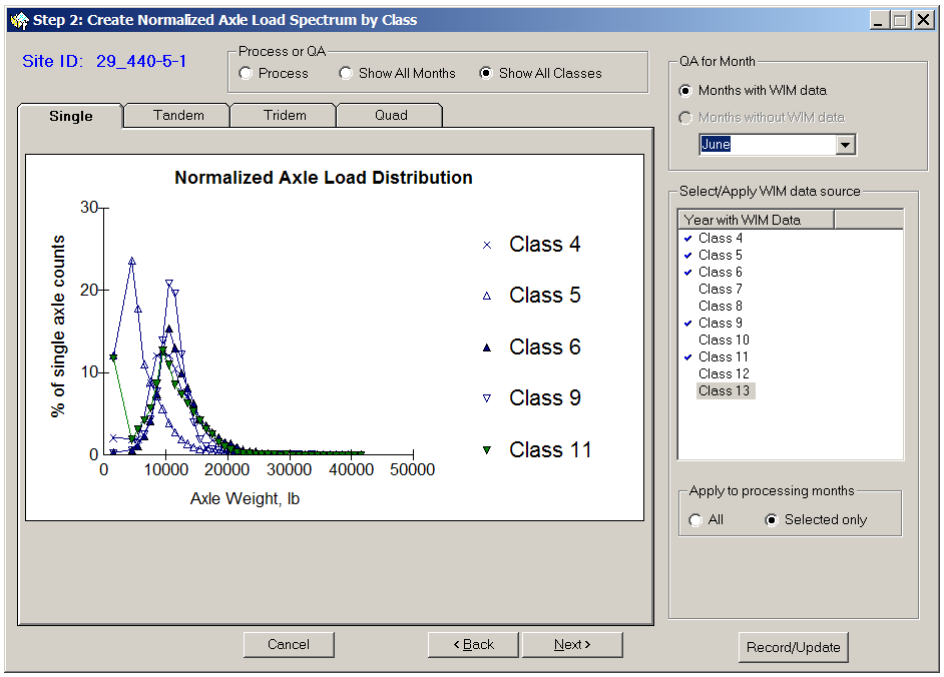


Figure A.123 Normalized single axle spectra by class for site 29_440-5-1 (SHRP_ID 4069)

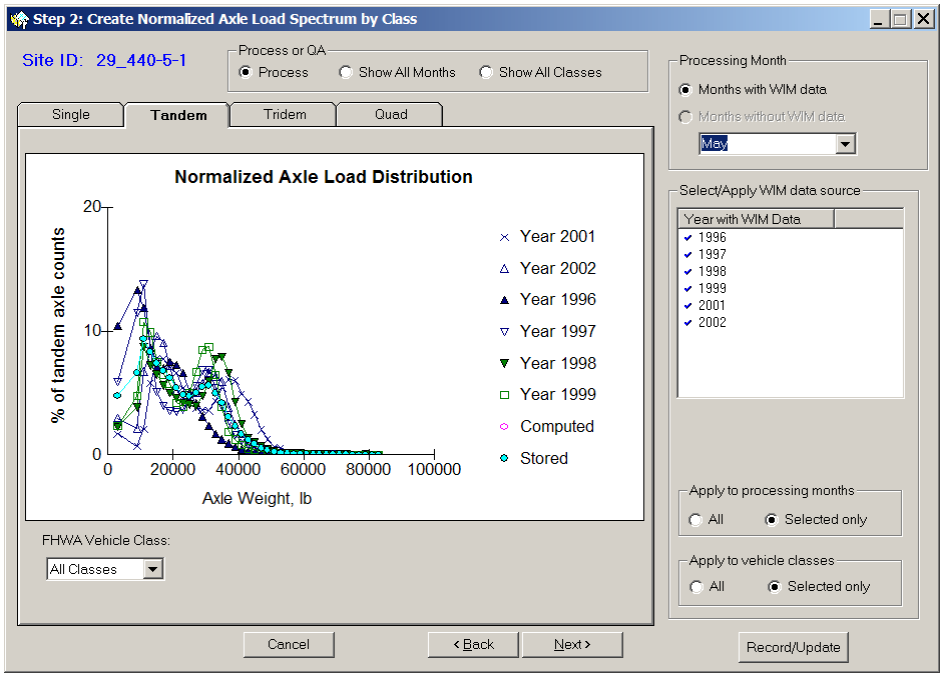


Figure A.124 Normalized tandem axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)

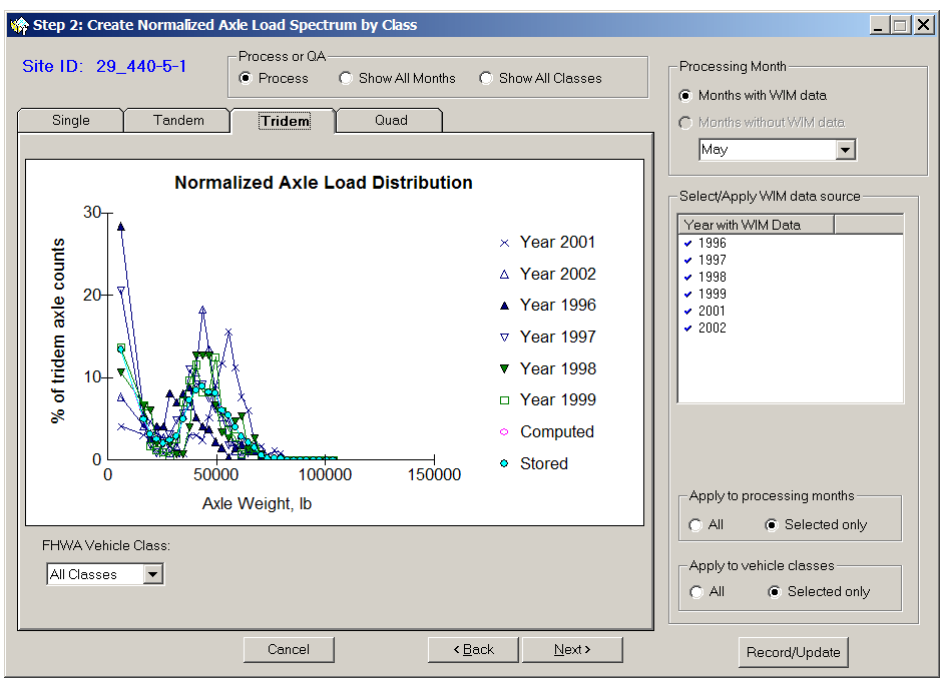
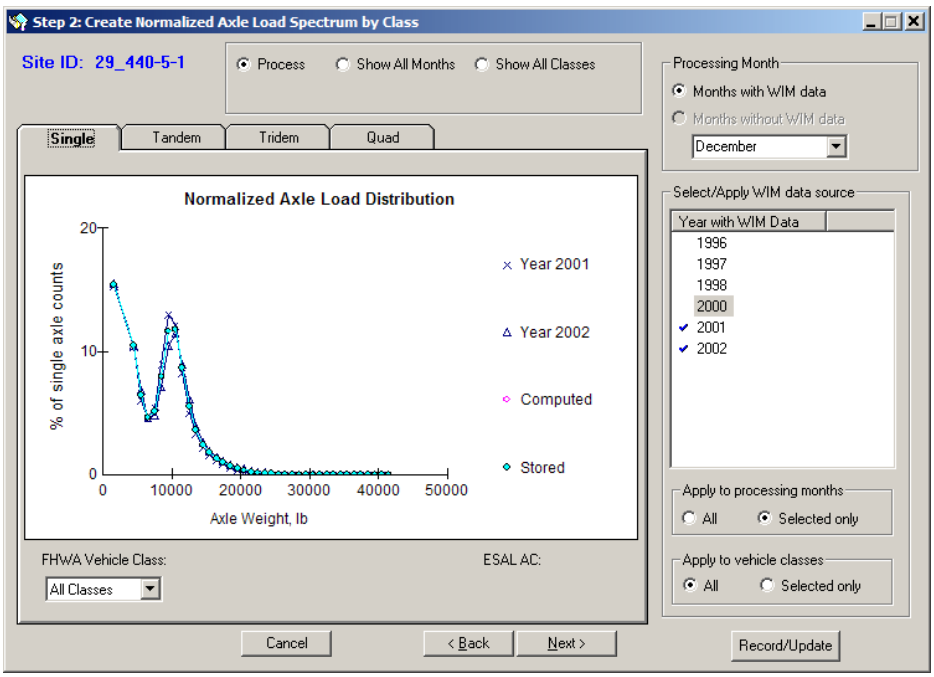


Figure A.125 Normalized tridem axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)



Comment: revised to show a much better correlation with only the 2001 and 2002 data

Figure A.126 Normalized single axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)

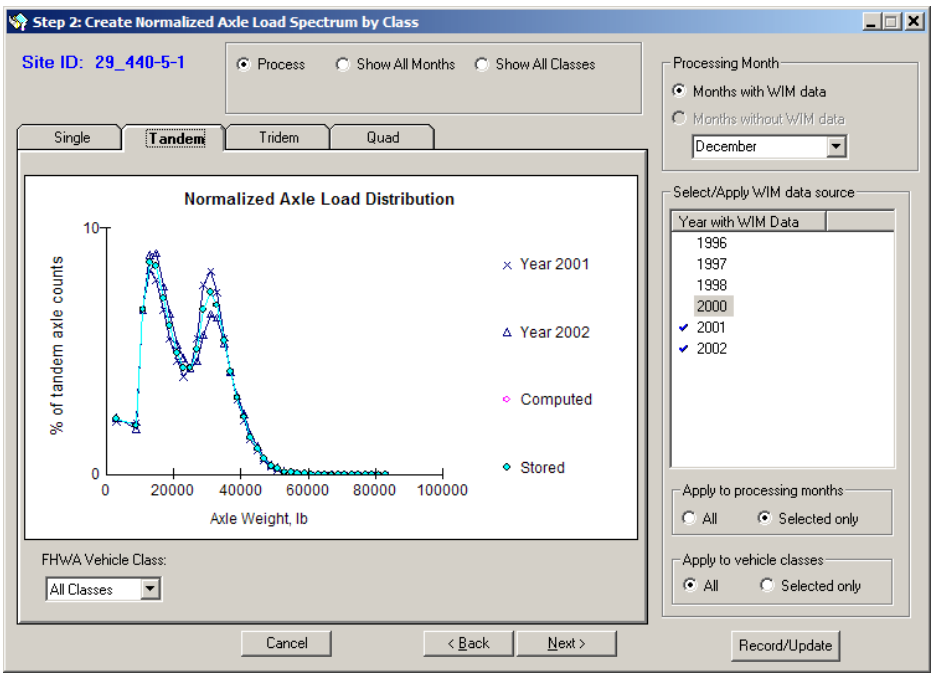


Figure A.127 Normalized tandem axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)

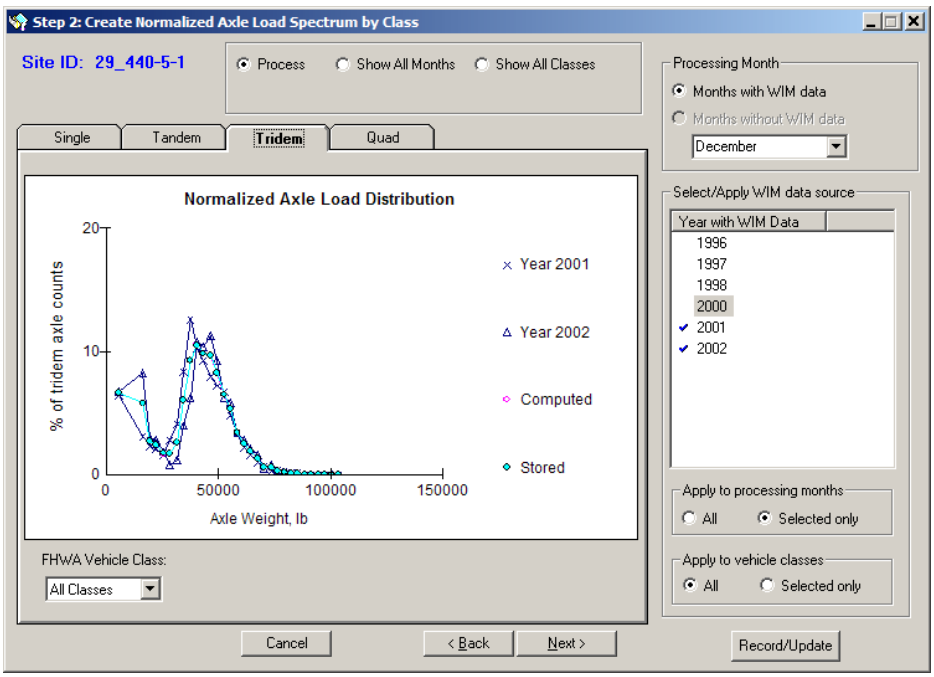
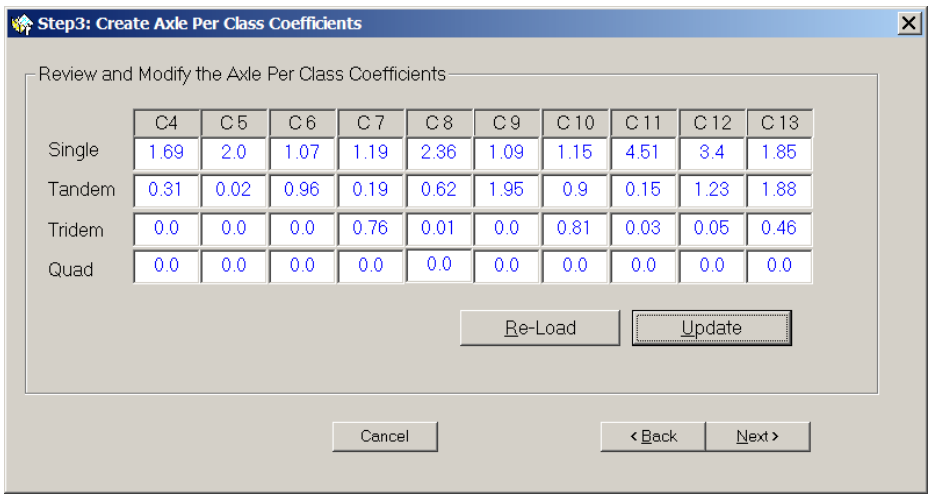


Figure A.128 Normalized tridem axle spectra for all classes for site 29_440-5-1 (SHRP_ID 4069)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.69	2.0	1.07	1.19	2.36	1.09	1.15	4.51	3.4	1.85
Tandem	0.31	0.02	0.96	0.19	0.62	1.95	0.9	0.15	1.23	1.88
Tridem	0.0	0.0	0.0	0.76	0.01	0.0	0.81	0.03	0.05	0.46
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.129 Axle class coefficients for site 29_440-5-1 (SHRP_ID 4069)

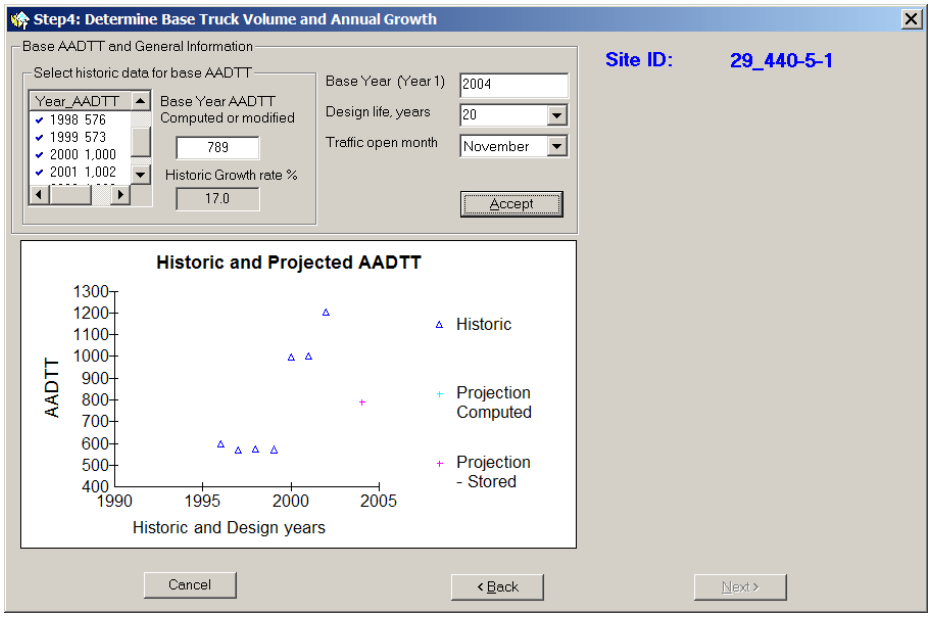


Figure A.130 Base truck volume and annual growth for site 29_440-5-1 (SHRP_ID 4069)

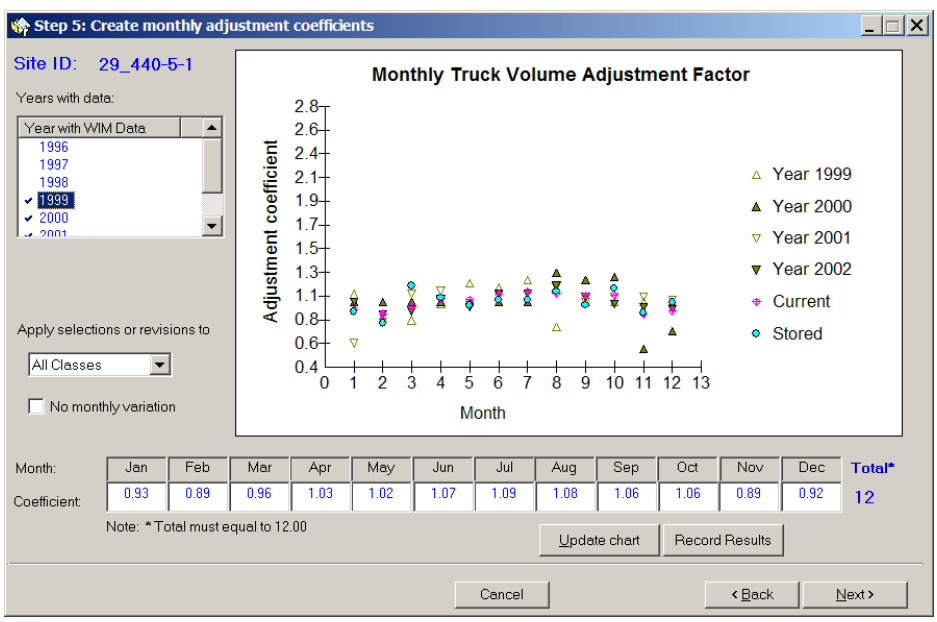
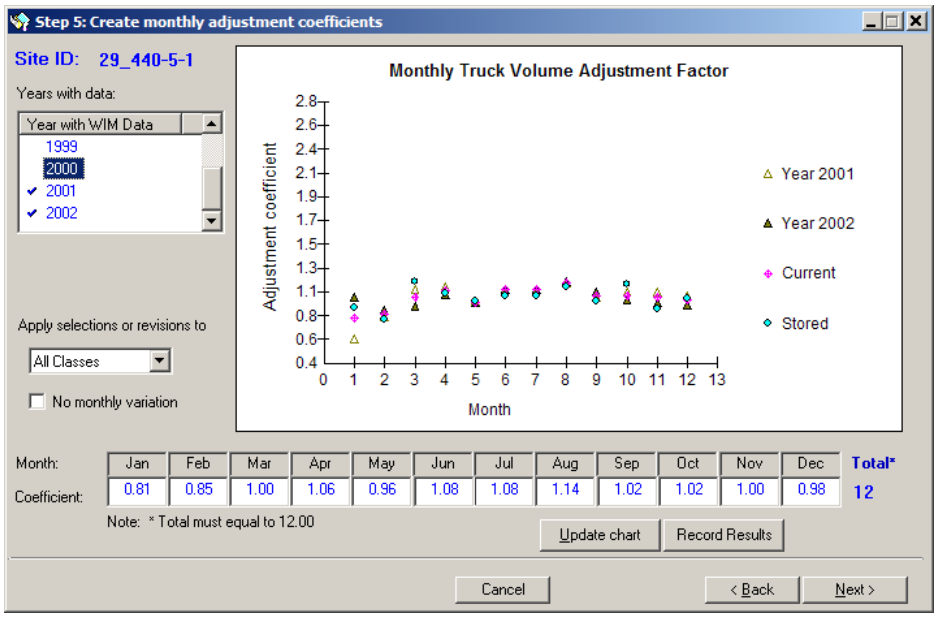
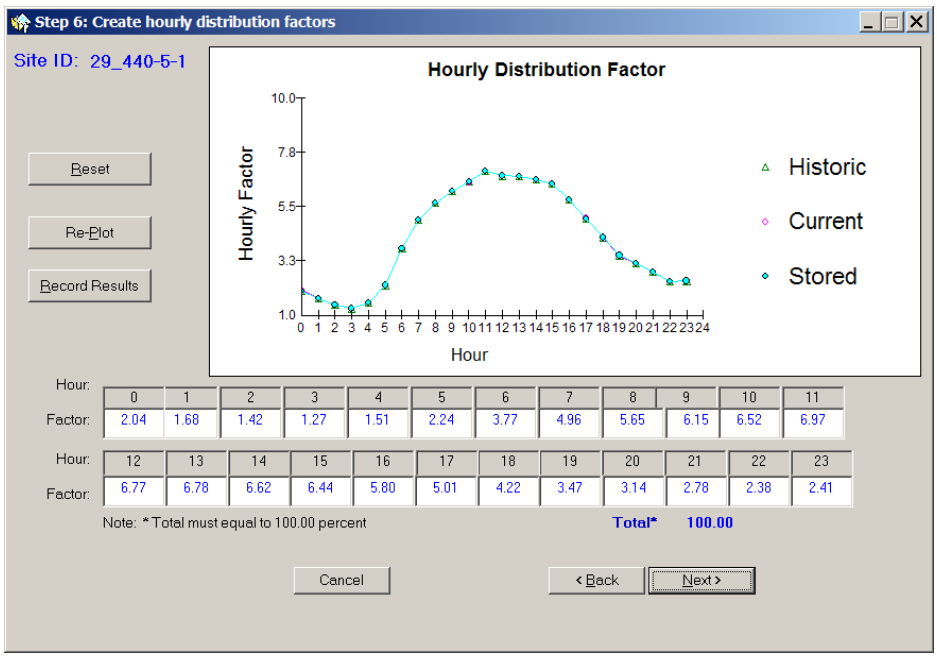


Figure A.131 Monthly truck volume adjustment coefficients for site 29_440-5-1 (SHRP_ID 4069)



Comment: reduced to 2001 and 2002

Figure A.132 Monthly truck volume adjustment coefficients for site 29_440-5-1 (SHRP_ID 4069)



Comment: peak values 9AM-4PM

Figure A.133 Hourly truck distribution factors for site 29_440-5-1 (SHRP_ID 4069)

SITE 29_740-5-1 (SHRP_ID 5503)

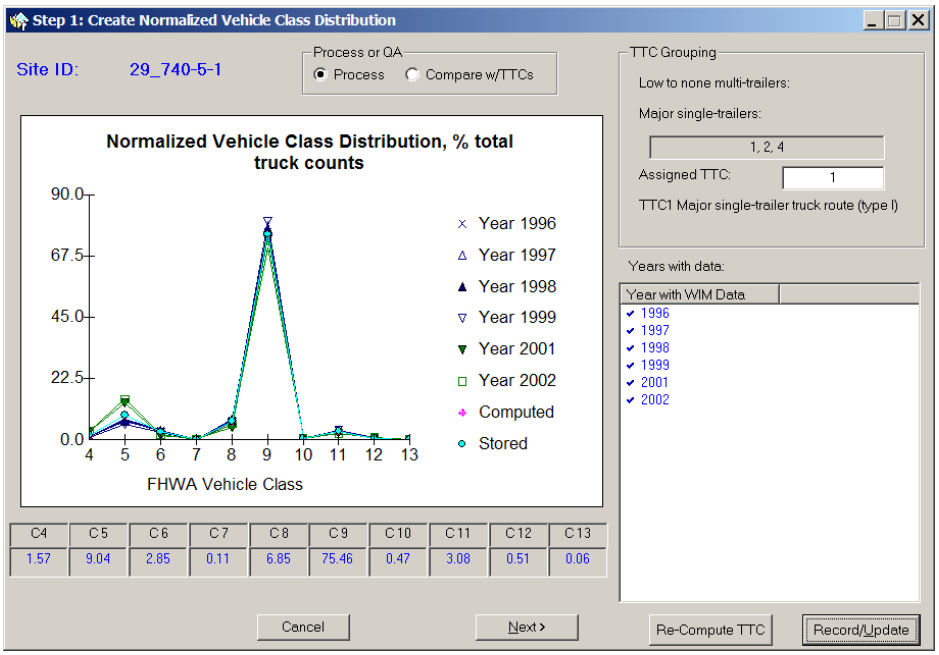


Figure A.134 Normalized vehicle class distribution for site 29_740-5-1 (SHRP_ID 5503)

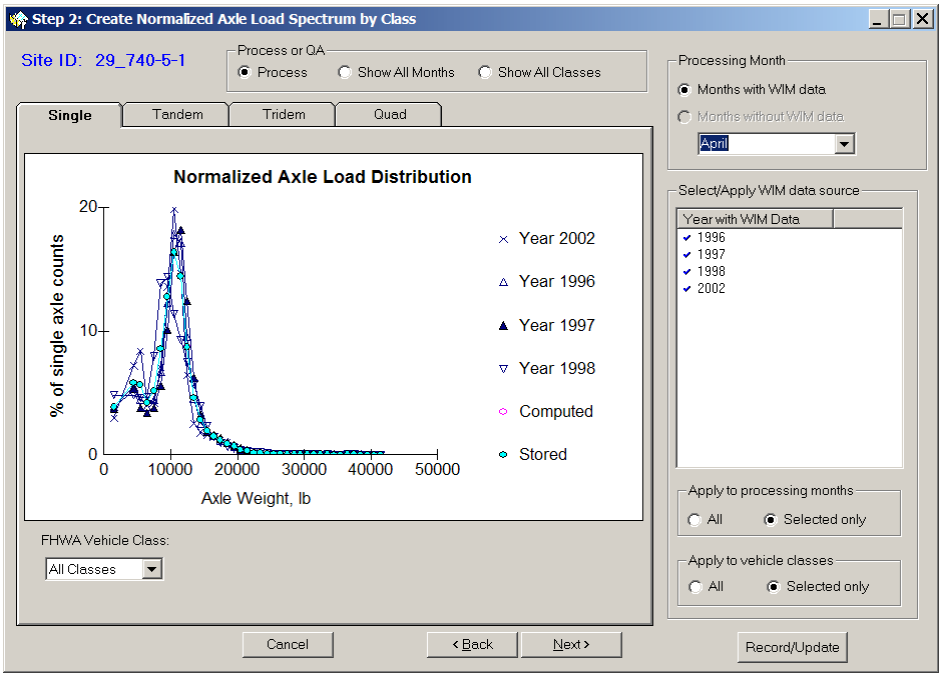


Figure A.135 Normalized single axle spectra for all classes for site 29_740-5-1 (SHRP_ID 5503)

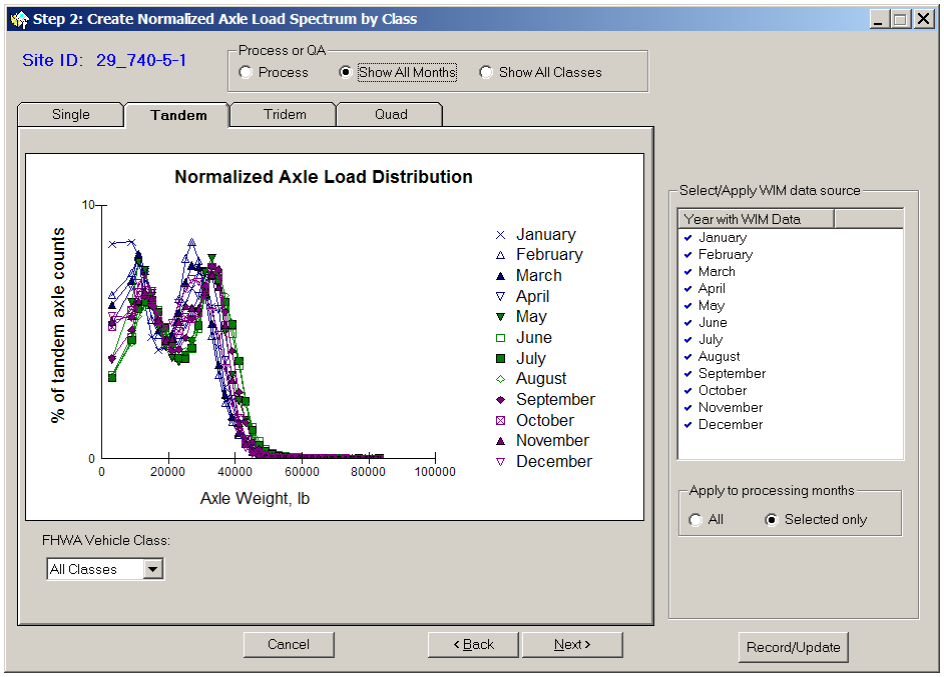
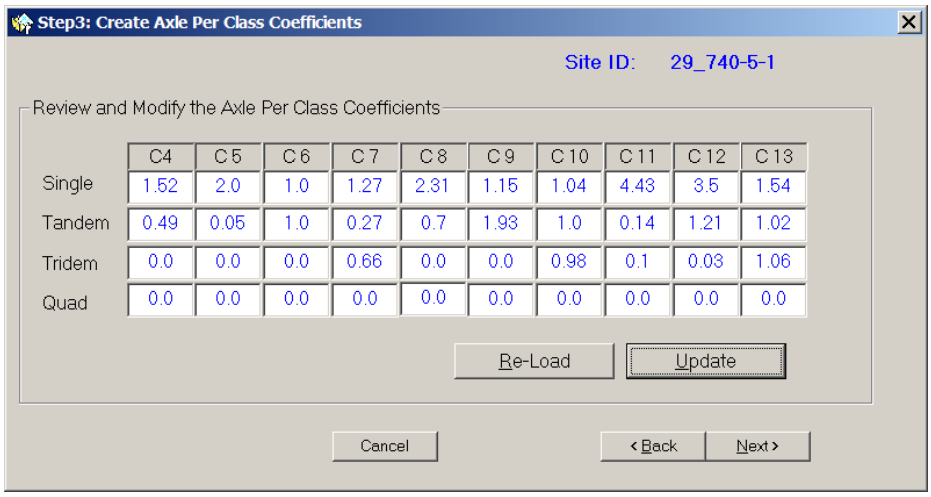


Figure A.136 Normalized tandem axle spectra for all classes for site 29_740-5-1 (SHRP_ID 5503)



	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
Single	1.52	2.0	1.0	1.27	2.31	1.15	1.04	4.43	3.5	1.54
Tandem	0.49	0.05	1.0	0.27	0.7	1.93	1.0	0.14	1.21	1.02
Tridem	0.0	0.0	0.0	0.66	0.0	0.0	0.98	0.1	0.03	1.06
Quad	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure A.137 Axle class coefficients for site 29_740-5-1 (SHRP_ID 5503)

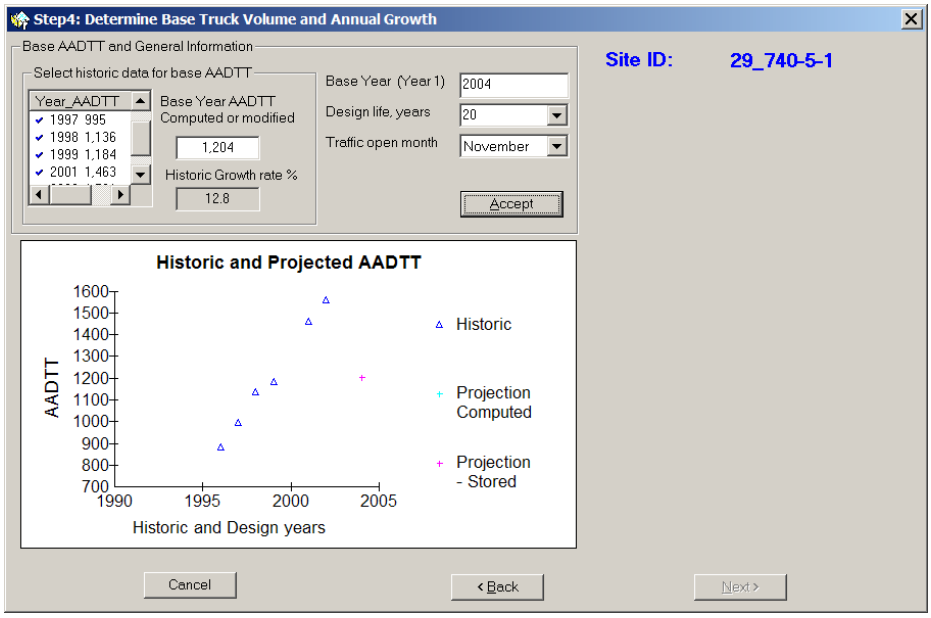
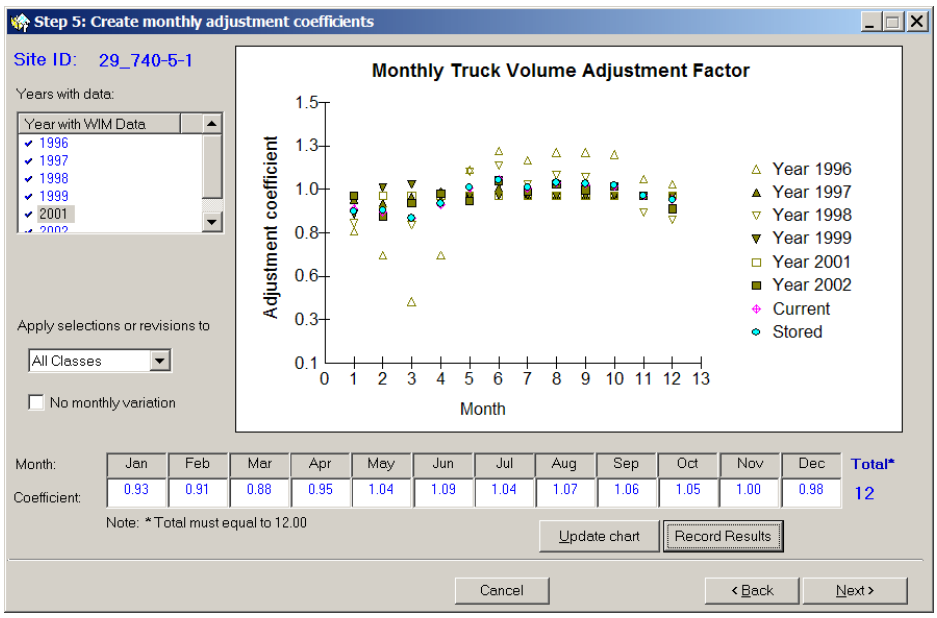
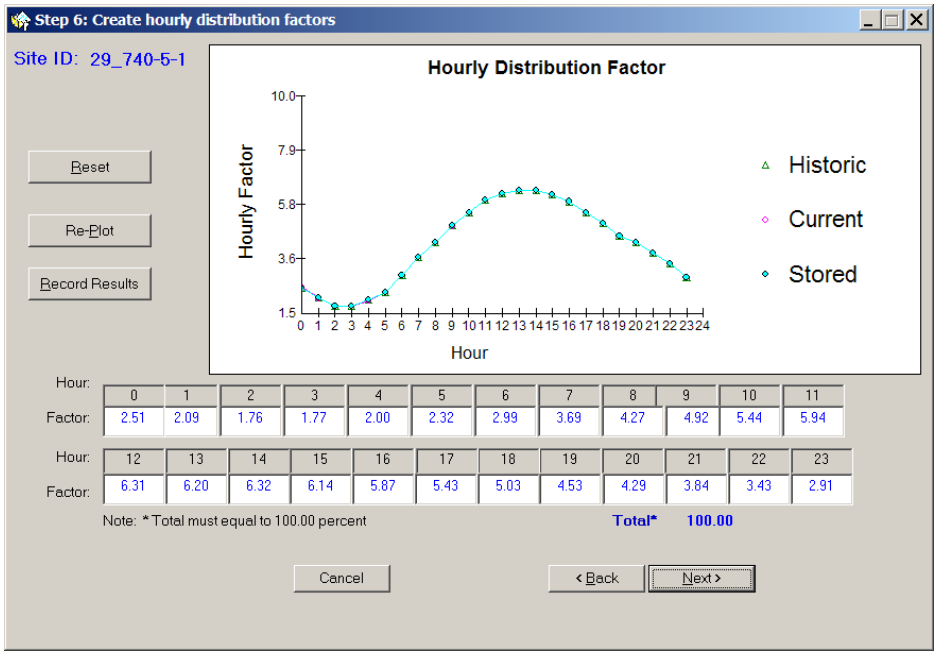


Figure A.138 Base truck volume and annual growth for site 29_740-5-1 (SHRP_ID 5503)



Comment: summer months are slightly higher

Figure A.139 Monthly truck volume adjustment coefficients for site 29_740-5-1 (SHRP_ID 5503)



Comment: bell shaped curve 10AM-5PM

Figure A.140 Hourly truck distribution factors for site 29_740-5-1 (SHRP_ID 5503)