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Development of a Simplified Flexible Pavement Design Protocol for New York State Department of Transportation Based on the AASHTO Mechanistic-Empirical Pavement Design Guide

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This report summarizes the local calibration of the distress models for the Northeast (NE) region of the United States and the development of new design tables for new flexible pavement structures. Design, performance, and traffic data collected on the Long-Term Pavement Performance (LTPP) sites in the NE region of the United States were used to calibrate the distress models. First, the AASHTOWare Pavement ME Design 2.1 with global calibration factors was used to compare the predicted and measured distress values. The local bias was assessed for all distress models except for the longitudinal cracking model; it was found the bias existed for this model even after calibration. The thermal cracking model was not calibrated because of inaccurate measured data. The calibration improved the prediction capability of the rutting, fatigue cracking, and smoothness prediction models.

The calibrated AASHTOWare software was used to run design cases for combinations of traffic volume and subgrade soil stiffness (resilient modulus, M_r) for 24 locations in the state of New York. The runs were performed for a road classified as Principal Arterial Interstate, 90% design reliability level, and 15- and 20-year design periods. State-wide average traffic volume parameters and axle load spectra were used to define the traffic. The configuration specified in the current design table used by NYSDOT, which is included in the Comprehensive Pavement Design Manual (CPDM), was followed for the pavement design solutions. The thicknesses for the select granular subgrade materials and the asphalt layer thicknesses were varied to include several values higher and lower than the thickness recommended by the CPDM. The thicknesses of asphalt surface and binder layers were kept constant; only the thickness of the asphalt base layer was changed. For each design combination, the design case with the thinnest asphalt layer for which the predicted distress was less than the performance criteria was selected as the design solution. The design solutions for each of the 24 locations were assembled in design tables.

The comparison of the design tables showed that some variation in the design thickness for the asphalt layers exists with thicker asphalt layers being needed for the locations in the upper part of the New York State. The comparison between the new design tables and the table included in the CPDM proved that the new design tables require thinner asphalt layers at low Annual Average Daily Truck Traffic (AADTT) and thicker asphalt layers at high AADTT than the corresponding designs in the CPDM table.

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Abstract

The New York State Department of Transportation (NYSDOT) has used the AASHTO 1993 Design Guide for the design of new flexible pavement structures for more than two decades. The AASHTO 1993 Guide is based on the empirical design equations developed from the data collected in the AASHO Road Test in the early 1960s. A newer pavement design method, called the Mechanistic-Empirical Pavement Design Guide (MEPDG), was developed by the National Cooperative Highway Research Program (NCHRP) to provide a more efficient and accurate design method that is based on sound engineering principles. The MEPDG models have been incorporated in the AASHTOWare Pavement ME Design 2.1 software program. Due to the advanced principles and design capabilities of the AASHTOWare program, NYSDOT decided to implement the MEPDG and calibrate the distress models included in the software for the conditions in the state.

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The calibrated AASHTOWare software was used to run design cases for combinations of traffic volume and subgrade soil stiffness (resilient modulus, M_r) for 24 locations in the state of New York. The runs were performed for a road classified as Principal Arterial Interstate, 90% design reliability level, and 15- and 20-year design periods. State-wide average traffic volume parameters and axle load spectra were used to define the traffic. The configuration specified in the current design table used by NYSDOT, which is included in the Comprehensive Pavement

Design Manual (CPDM), was followed for the pavement design solutions. The thicknesses for the select granular subgrade materials and the asphalt layer thicknesses were varied to include several values higher and lower than the thickness recommended by the CPDM. The thicknesses of asphalt surface and binder layers were kept constant; only the thickness of the asphalt base layer was changed. For each design combination, the design case with the thinnest asphalt layer for which the predicted distress was less than the performance criteria was selected as the design solution. The design solutions for each of the 24 locations were assembled in design tables.

The comparison of the design tables showed that some variation in the design thickness for the asphalt layers exists with thicker asphalt layers being needed for the locations in the upper part of the New York State. The comparison between the new design tables and the table included in the CPDM proved that the new design tables require thinner asphalt layers at low Annual Average Daily Truck Traffic (AADTT) and thicker asphalt layers at high AADTT than the corresponding designs in the CPDM table.

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Chapter 1: Introduction

The American Association of State Highway Officials (AASHO) sponsored the AASHO Road Test in Ottawa, Illinois, in 1958 to study the performance of pavement structures under different traffic loads and to quantify the damage induced by truck axles on pavement structures. The data collected in the experiment was used to develop an empirical-statistical method to design flexible and rigid pavement structures, later included in the AASHTO Pavement Design Guide. The first interim version of the Guide was published in 1972. Subsequent versions, incorporating several improvements to the design procedure, were developed in 1986, 1993, and 1998. The 1993 *AASHTO Guide for Design of Pavement Structures* was adopted by the highway agencies in 48 U.S. states (AASHTO, 1993).

In the 1990s, the AASHTO Joint Task Force on Pavement (JTFP) initiated a National Cooperative Highway Research Program (NCHRP) Research Project 1-37A, entitled "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures." The project called for the development of a design guide that employs existing state-of-the-practice mechanistic-based models and design procedures.

In 2004, the Mechanistic-Empirical Pavement Design Guide (MEPDG) became available. It was released for public review and evaluation. NCHRP conducted a formal review of the MEPDG under Project 1-40A, which resulted in several improvements to the MEPDG software. In April 2007, MEPDG 1.1 was submitted to NCHRP, the Federal Highway Administration (FHWA), and AASHTO. Later, MEPDG 1.1 was released to the public for implementation and evaluation purposes. A new version of this software program, called AASHTOWare Pavement ME Design (AASHTO, 2013), formerly DARWin-ME, which incorporates all models from the MEPDG program, can now be purchased from AASHTO.

The design method in both programs is mechanistic-empirical in principle, since it calculates the response of pavement structures under vehicle loading and it estimates the accumulation of pavement distresses based on pavement response. The calculation of pavement response for flexible pavement structures models the layered pavement structure loaded by vertical loads distributed uniformly over circular areas at the pavement surface. All pavement materials are assumed to be linear elastic. Traffic data in terms of truck volumes and axle load spectra are used to estimate the number and the magnitude of the vertical surface loads. Climatic data is used to estimate the stiffness of the structural layers; temperature data is used to estimate the stiffness of bituminous materials, while rainfall data is used to adjust the moisture in the unbound layer and estimate their stiffness. This procedure is very sound and flexible, and it surpasses considerably any currently available pavement analysis tool. It was adopted by AASHTO as the new design method for pavement structures in 2008, in lieu of the earlier empirical procedure. In order to account for local pavement configuration, climatic conditions, highway materials, and traffic characteristics and to improve the accuracy of distress prediction, the MEPDG and the AASHTOWare Pavement ME models must be calibrated to local conditions. The *Mechanistic-Empirical Pavement Design Guide, A Manual of Practice* (AASHTO, 2008) was published in July 2008 and the official associated software "AASHTOWare Pavement ME Design" in 2013. They are both available to the public and the research community through AASHTO. However, they are very complex and require specific and advanced expertise. AASHTOWare Pavement ME Design Version 2.1 is the latest version of AASHTOWare at the time this report is written.

The New York State Department of Transportation (NYSDOT) decided to implement the MEPDG program, and then the AASHTO Pavement ME Design software once it became available, due to the superior design principles these tools employ. To overcome the complexity of using the software program and the difficulty in assembling the extensive and detailed input values required to run it, NYSDOT needs a simple design procedure, built based on the AASHTO Pavement ME, which can be used by the regional offices to design the new flexible pavement structure. It is desirable that the new simple design procedure be similar to the design table currently used by NYSDOT regional offices. However, the design models must be calibrated to the local conditions in the state of New York to be accurate.

1.1 Current Design Practice Used by NYSDOT

Currently NYSDOT performs the design of flexible pavement structures following the Comprehensive Pavement Design Manual (CPDM; NYSDOT, 2014a). The CPDM was first issued by NYSDOT on October 31, 1994, and is based on the 1993 AASHTO Pavement Design Guide. The CPDM includes two methods for the design of flexible pavements: the Conventional Pavement Design Method for road sections shorter than 1.5 km and the Equivalent Single Axle (ESAL) Pavement Design Method for road sections longer than 1.5 km.

NYSDOT uses Table 1.1 to design the flexible pavements based on the Conventional Method. The designer should obtain the Annual Average Daily Traffic (AADT) and the estimated percent of trucks to find the structural layers' thicknesses. Table 1.2 is used to design the flexible pavements based on the ESAL pavement design method. Table 1.2 had been developed for 90% design reliability and 50-year design life. To perform the structural pavement design, the designer needs to calculate the cumulative ESALs during the design life and to estimate the subgrade soil resilient modulus (M_r). Then the designer can use Table 1.2 to select the total thickness of hot mix asphalt (HMA) and of the select subgrade layer.

NYSDOT developed a typical design section of flexible pavement structure in order to be used as a guide for NYSDOT engineers; the typical pavement section configuration is shown in Figure 1.1. The same structural configuration was used for the simple design method developed in this research.

Table 1.1: Thickness Guide for Conventional Pavement Design

Annual Average Daily Traffic (AADT)	Percent Trucks	Subbase Course Thickness (all Pavements) (mm)	HMA Pavement Thickness	
			Base Course	Top & Binder Courses Combined
Over 10,000 Vehicles	10% or more	300	150	90
	Less than 10%		125	
6,000 to 10,000	10% or more	300	125	90
	Less than 10%		100	
4,000 to 5,999	All	300	75	90
Under 4,000 Vehicles	All	300	75	80

Table 1.2: CPDM Flexible Pavement Design Tables

Mr =28 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 2	165	0
2 < ESALs <= 4	175	0
4 < ESALs <= 8	200	0
8 < ESALs <= 13	225	0
13 < ESALs <= 23	250	0
23 < ESALs <= 45	250	150
45 < ESALs <= 80	250	300
80 < ESALs <= 140	250	450
140 < ESALs <= 300	250	600

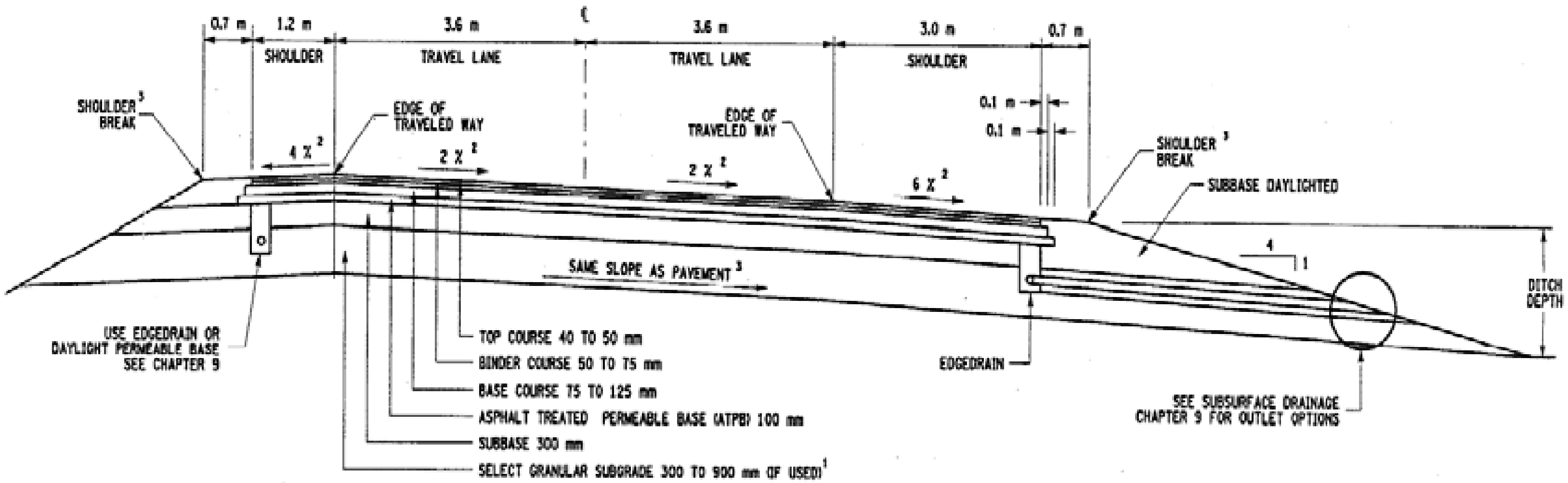
Mr =34 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 4	165	0
4 < ESALs <= 7	175	0
7 < ESALs <= 13	200	0
13 < ESALs <= 23	225	0
23 < ESALs <= 40	250	0
40 < ESALs <= 70	250	150
70 < ESALs <= 130	250	300
130 < ESALs <= 235	250	450
235 < ESALs <= 300	250	600

Mr =41 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 6	165	0
6 < ESALs <= 11	175	0
11 < ESALs <= 20	200	0
20 < ESALs <= 35	225	0
35 < ESALs <= 60	250	0
60 < ESALs <= 110	250	150
110 < ESALs <= 200	250	300
200 < ESALs <= 300	250	450

Mr =48 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 8	165	0
8 < ESALs <= 16	175	0
16 < ESALs <= 30	200	0
30 < ESALs <= 50	225	0
50 < ESALs <= 85	250	0
85 < ESALs <= 160	250	150
160 < ESALs <= 300	250	300

Mr =55 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 12	165	0
12 < ESALs <= 20	175	0
20 < ESALs <= 40	200	0
40 < ESALs <= 65	225	0
65 < ESALs <= 115	250	0
115 < ESALs <= 215	250	150
215 < ESALs <= 300	250	300

Mr =62 Mpa		
ESALs (million)	HMA Thickness (mm)	Select Granular Subgrade Thickness (mm)
ESALs <= 15	165	0
15 < ESALs <= 30	175	0
30 < ESALs <= 50	200	0
50 < ESALs <= 90	225	0
90 < ESALs <= 150	250	0
150 < ESALs <= 300	250	150



THE DIMENSIONS SHOWN ARE TYPICAL ONLY AND THE DETAILS ARE SHOWN TO GIVE THE DESIGNER AN IDEA OF WHAT OVERALL SECTION LOOKS LIKE.

1. SEE CHAPTER 6 FOR MORE INFORMATION OR CONTACT REGIONAL GEOTECHNICAL ENGINEER FOR NECESSITY AND THICKNESS OF SELECT GRANULAR SUBGRADE. DAYLIGHT SELECT GRANULAR SUBGRADE ONTO EMBANKMENT SLOPES OR AT THE EXTENSION OF THE DITCH BACKSLOPE IN CUT SECTIONS.
2. REFER TO THE HIGHWAY DESIGN MANUAL, CHAPTERS 2 AND 3, FOR THE PROPER CROSS SLOPE AND FURTHER DEVELOPMENT OF TYPICAL SECTIONS.
3. REFER TO THE HIGHWAY DESIGN MANUAL, CHAPTER 3, FOR SUBGRADE GRADE CHANGES AND EDGE OF SHOULDER DETAILS.

Figure 1.1: NYSDOT Design Typical Section

SUperior PERerforming Asphalt PAVEments (SUPERPAVE) asphalt mixtures are currently used on the majority of pavement construction done by NYSDOT since:

- Rehabilitation of these pavements is quick and easy,
- Life span of 15 to 20 years for thicker overlay and 8 to 10 years for single course overlay if proper maintenance is provided,
- Pavement foundation life is 50 years, and
- Construction cost is relatively low.

To extend the performance of HMA mixes, NYSDOT uses the performance-graded (PG) binder specifications for asphalt binders developed during the Strategic Highway Research Program (SHRP) in the early 1990s. The CPDM recommends specific PG grades for the asphalt binder depending on the geographic location of the pavement project, as given in Table 1.3.

Table 1.3: Performance Graded Binder Selection

Location	Location by Counties	Standard PG Binder Grades (Material Designation)	Polymer Modified PG Binder Grades (Material Designation)
Upstate	All Counties Not Listed under Downstate	64S-22 (702-64S22)	64V-221,2 (702-64V22)
Downstate	Orange, Putnam, Rockland, Westchester, Nassau, Suffolk Counties, and City of New York	64H-22 (702-64H22)	64E-22 (702-64E22)

NYSDOT recommends the aggregate in the asphalt mixes used for the top (surface) course to have the nominal maximum aggregate size (NMAS) of 12.5 mm and 9.5 mm. However, aggregates with NMAS of 9.5 mm are recommended where a lot of handwork is envisioned, if gravel aggregates are used, or for projects in urban areas.

For the binder course, the use of aggregates with NMAS of 19.0 mm and 25.0 mm are recommended. Normally, aggregates with NMAS of 19.0 mm are used for projects where the 20-year ESAL count is less than 10 million. In addition, NYSDOT recommends using aggregate with NMAS of 25 mm if the HMA pavement is thicker or if the 20-year ESAL count is over 10 million.

For the base course, aggregate gradation with NMAS of 25 mm, 37.5 mm, or 75 mm are recommended. Base course thickness and the estimated ESAL are considered in choosing the aggregate. NYSDOT recommends using NMAS of 75 mm or less if the base course is thick. However, aggregates with NMAS of 37.5 mm are the most common (Hall, 2012).

1.2 Study Objectives

The objectives of this research work are to:

- Calibrate the performance models for flexible pavements in the AASHTOWare Pavement ME Design 2.1 to the local conditions of Northeastern region of the United States.
- Develop design tables based on the AASHTO ME Pavement Design Guide to be used in several locations in the New York State, at least one location for each of the 11 NYSDOT regions.
- Compare the design tables built based on the AASHTO Pavement ME with the design table currently recommended in the NYSDOT CPDM.

Chapter 2: Traffic Data Collection, Assembly, and Analysis

2.1 Data Collection

The traffic data analyzed in this study was collected at the vehicle classification sites and weigh-in-motion (WIM) sites distributed over all 11 regional offices of NYSDOT. The TrafLoad software (NCHRP, 2005) was used to process the raw data, shown in Appendix A, for the period between 2007 and 2011. The data collected in 2010 by 52 vehicle classification and 19 WIM sites was selected for the initial statistical analysis (Romanoschi, Momin, Bethu, & Bendana, 2011). However, traffic data for other years was also analyzed to find if there was any statistical difference over time. It was assumed that proper data validation procedures were adopted by NYSDOT but an additional quality control of the data was done with the TrafLoad software. The software adopts a formatting validation algorithm to check each line of raw traffic data to find the acceptability of traffic data files.

2.2 Cluster Analysis

Ward's method of cluster analysis was carried out to differentiate the sites for Level 2 (regional) inputs. Semi-partial R-squared (SPR) value was used to determine the number of clusters to be selected for the analysis. The SPR value measures the loss of homogeneity due to merging of two clusters into a new cluster at a given step. High SPR values show that the two clusters are quite different. However, low values of SPR indicate more homogeneity among the clusters. Too low SPR values lead to too many clusters. On the other hand, too high SPR values would not represent the characteristics of each site properly, because it would result in too few clusters consisting of too many sites. Care must be taken to decide the number of clusters to be considered. In this analysis, an SPR below 0.05 indicates that the clusters can be merged.

2.3 MEPDG Runs

The cluster analysis helps grouping the traffic sites into clusters, based on similarity of data for a given parameter. However, it cannot indicate if the use of input data from different clusters changes the outcome of the pavement design process. Therefore, MEPDG runs were performed with the suggested average parameters obtained for each cluster. The runs were conducted for each type of traffic input separately; all other inputs were kept unchanged. The predicted distress values obtained from the MEPDG runs were used to study the effect of the variation in each traffic input parameter. The distresses predicted when cluster average values were used were compared to those predicted when statewide average values or MEPDG default values were used.

Since data from less than 20 WIM sites obtained for each year between 2007 and 2011 was insufficient to run the cluster analysis, only MEPDG runs were carried out to decide if the site-specific or statewide average values should be used for the Average Groups Per Vehicle (AGPV) and axle load spectra. The change in percentage of predicted distresses due to the use of site specific or statewide average values was used to differentiate the WIM sites. Similarly, MEPDG runs were carried out for the WIM data of other years in order to find if there is any variation over the years.

The MEPDG runs were performed for a typical primary road pavement structure used by NYSDOT. The studied asphalt pavement consisted of:

- A 4.0-inch asphalt concrete surface layer with a SM 9.5-mm mix;
- A 8.0-inch asphalt concrete base layer with a SM 19.0-mm mix;
- A 12.0-inch granular base layer, and
- An AASHTO A-7-6 soil for the infinite subgrade layer.

The material data used for the two HMA mixes were obtained from the construction records of an actual pavement project designed by NYSDOT. The following predicted distresses were used to determine the influence of variation in traffic parameters:

- Total rut depth (inches), and
- The difference between the initial IRI and terminal IRI, named delta IRI (inches/mile).

The predicted alligator and longitudinal cracking were not considered in the analysis since the cracking models incorporated in MEPDG are not considered reliable. Moreover, NYSDOT uses only total rut depth and IRI as trigger values for deciding when a distressed flexible pavement must be overlaid.

The MEPDG runs were also performed for a typical rigid pavement structure used by NYSDOT. The modeled jointed plain concrete pavement (JPCP) consisted of:

- A 12.0-inch JPCP slab,
- A 10.0-inch granular base layer, and
- An AASHTO A-7-6 soil for the infinite subgrade layer.

The traffic and climatic inputs were kept the same as MEPDG runs for asphalt pavement. This was done to compare the effects of traffic inputs on both types of pavements. The following predicted distresses were used to determine the influence of variation in traffic parameters:

- Mean joint faulting (inches), and
- The difference between the terminal IRI and initial IRI, named delta IRI (inches/mile).

Transverse cracking was not considered since the associated model is not considered reliable.

2.4 Identification of Traffic Inputs

The development of appropriate traffic inputs is necessary for the design and analysis of pavements. Level 1 data for all sites were not available. Moreover, site-specific traffic data may not be needed if statewide average values represent well the characteristics of most sites. Cluster analysis is used to differentiate the sites on the basis of different traffic parameters.

2.4.1 Analysis of Traffic Inputs for Flexible and Rigid Pavement

Cluster analysis was done on all traffic parameters for the vehicle classification data collected in 2010. This analysis could not be done on AGPV and axle load spectra due to unavailability of sufficient WIM data. MEPDG runs were carried out to find the significance of the results of cluster analysis for both asphalt and rigid pavements. In addition, the MEPDG runs also worked as a tool to differentiate WIM traffic inputs.

Typical asphalt and JPCP pavements were modeled for carrying out MEPDG runs to verify the effect of traffic inputs. The traffic and climate inputs were kept the same for the runs done for both pavement types. This helped compare the effects on both the rigid and asphalt pavements.

The distress values were found from the MEPDG runs for cluster and site-specific traffic inputs. These values were compared with the distress values found from the MEPDG runs for the statewide average values of traffic inputs. If the distress value for a site or cluster was less than 15% of the distress value corresponding to the statewide average value, the statewide average value was chosen as the appropriate traffic input.

2.4.2 Annual Average Daily Truck Traffic (AADTT)

Cluster analysis was not conducted for the Annual Average Daily Truck Traffic (AADTT) because this information is always available for every site. However, AADTT values can be categorized in three groups: low (0 to 299), medium (300 to 999), and high (>1,000). The majority of the sites showed low AADTT values.

2.4.3 Vehicle Class Distribution (VCD)

The cluster analysis of vehicle class distribution (VCD) in 2010 produced four distinct clusters; the average values of the proportion of each truck class in each cluster are shown in Figure 2.1. The two directions of traffic were considered separately for the cluster analysis of VCD. However, the direction of traffic showed effect only on four sites; the opposing directions of traffic for sites 5281, 8180, 9380, and 9381 belong to different clusters. These four sites were located on principal arterials.

The differences between clusters are mainly due to the variation of the proportion of Class 5 and Class 9 vehicles. Clusters 1 and 3 show higher proportion of Class 9 vehicles than Class 5 vehicles, with Class 9 vehicles being more dominant in Cluster 1. Cluster 2 shows almost equal proportion of Classes 5 and 9 vehicles, which is closer to the statewide average distribution. In Cluster 4, Class 5 vehicles are dominant compared with Class 9 vehicles. However, it may be mentioned that the proportion of Class 5 or Class 9 vehicles does not determine the cluster group for a site alone; it also depends on the other class of vehicles and the total number of sites being considered.

It was observed that the sites in Cluster 1 have high one-way AADTT values, with an average of 1,241. These sites were located on interstate routes: I-81, I-86, and I-87. The sites in Cluster 2 exhibited one-way average AADTT of 320. These sites were located on I-295, NY 30, NY 13, NY 11, NY 414, and NY 394. The sites of Cluster 3 had an average AADTT of 210 in one direction of traffic. Most of the sites were located on rural principal arterials. They were located on NY 5, NY 11, NY 104, NY 37, and NY 219. Finally, the sites of Cluster 4 have the lowest average one-way AADTT (105). The majority of them were located on NY 10, NY 96B, NY 364, NY 54A, NY 201, and NY 145.

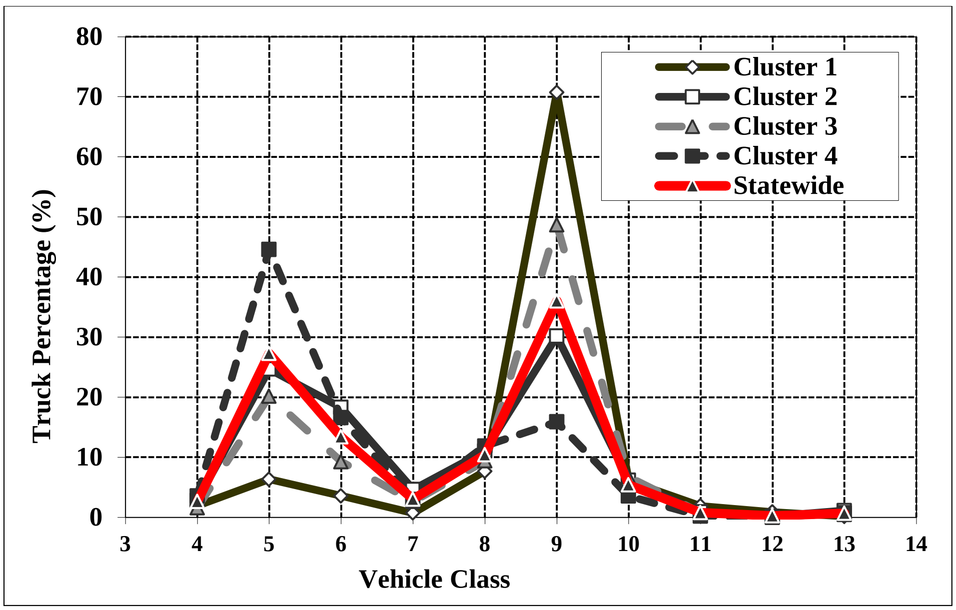


Figure 2.1: VCD Clusters (2010)

MEPDG runs were conducted for average values of VCD clusters for asphalt pavement. The percentage change in predicted distresses relative to those for statewide average VCD values are shown in Figure 2.2. The predicted distress values for total rutting and delta IRI are very close to the distress values for statewide average VCD values. However, Clusters 1 and 3 show higher predicted distresses in comparison to Cluster 2 and 4 and the state-wide average, due to the large proportion of Class 9 vehicles. The predicted distresses for Cluster 4 are the lowest due to the presence of higher proportion of Class 5 vehicles than Class 9 vehicles. Since NYSDOT uses only total rut depth and IRI as trigger values for deciding when a distressed flexible pavement must be overlaid, the insignificant effects on the predicted distresses for total rutting and delta IRI suggest that state-wide average VCD could be used effectively without affecting the outcome of the design.

MEPDG runs for the typical rigid pavement show that there is almost no difference in delta IRI in comparison to those for statewide average VCD values (Figure 2.3). The change in distress values for mean joint faulting may seem significant but the actual distress values are very small. For example, the statewide average value for mean joint faulting is 0.003 inch, while the average value for Cluster 1 is 0.004 inch. Therefore, the results of the MEPDG runs for rigid pavements also suggest the use of statewide average vehicle classification distribution values (Table 2.1).

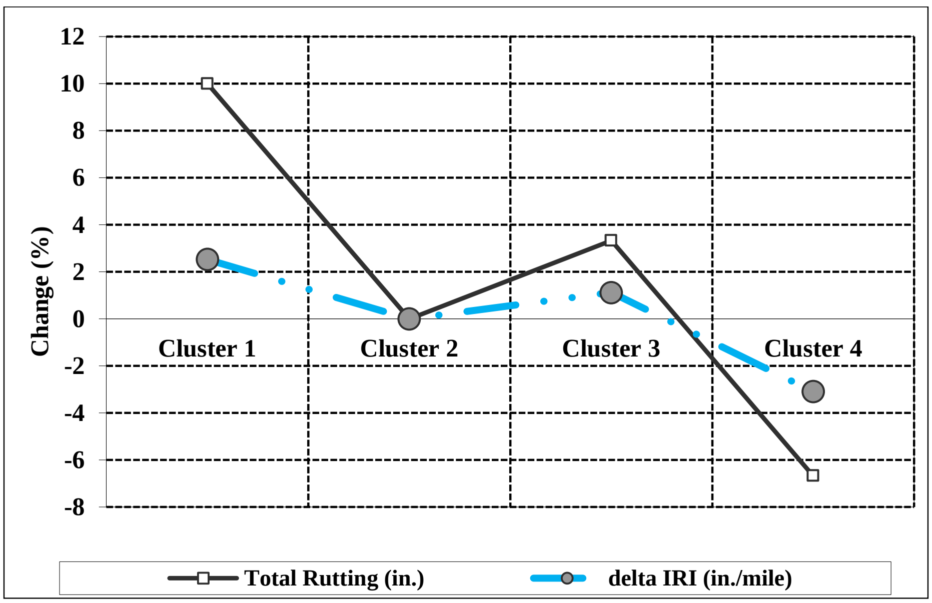


Figure 2.2: Change in Distress Values for Different VCD Clusters, 2010 – Flexible

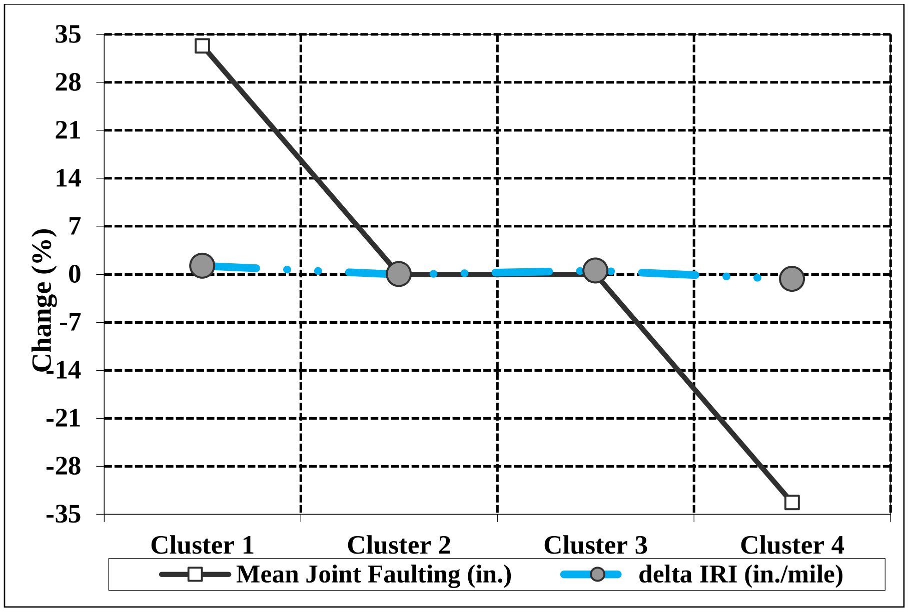


Figure 2.3: Change in Distress Values for Different VCD Clusters, 2010 – Rigid

Table 2.1: Statewide Average VCD for 2010

Vehicle Class	4	5	6	7	8	9	10	11	12	13
Statewide (%)	2.64	27.30	13.40	3.04	10.43	36.00	5.45	0.79	0.25	0.70

2.4.4 Monthly Distribution Factors (MDF)

For monthly distribution factors, a two dimensional analysis was done by considering the MDF for both Class 5 and Class 9 vehicles simultaneously, as they are the most predominant vehicle classes. The two-dimensional analysis was done to eliminate the shortcomings of one-dimensional analysis. However, careful attention was given so that the higher variability of one vehicle class does not affect the variabilty of other classes of vehicles. This was done to reduce the possibilty of bias in results.

Four distinctive patterns were observed from the cluster analysis of MDF for asphalt pavement (Figures 2.4 and 2.5). Site 6480 is an outlier because of its significantly high SPR. The site is located on an urban principal arterial – interstate with low AADTT of 110. The sites of Cluster 1 show similar pattern for both Class 5 and Class 9 vehicles. The member sites show low values of MDF in Fall and Spring and high values in Summer. The majority of the vehicle classification sites belonged to this cluster. These sites are distributed all over the state and have high variation in AADTT values. The sites in Cluster 2 show lower MDF values in Summer and higher values in Fall for Class 5 vehicles in comparison to the corresponding values for Cluster 9 vehicles. These sites have a medium average for AADTT (392) and are located mainly on I-86, I-88, NY 41, NY 104, and US 219. The sites of Cluster 3 show higher MDF values in Summer for Class 9 vehicles than for Class 5 vehicles. These sites have a low average AADTT (101). The majority of the sites were located on NY 3, NY 30, NY 64, and NY 96B. The sites of Cluster 4 show higher MDF values cluster and have a lower average AADTT (70) in Summer for Class 9 vehicles than for Class 5 vehicles. Sites 1281, 4482, and 9381 belong to this cluster and have a low average AADTT (70).

MEPDG runs were conducted to verify the results of cluster analysis of MDF for asphalt pavement. Figure 2.6 shows the percentage change in predicted distresses when average MDF values for each cluster are used instead of the statewide average MDF values. The figure suggests that the predicted distresses are not sensitive to the MDF values. Therefore, it is recommended that statewide average MDF values (Table 2.2) be used as MEPDG inputs.

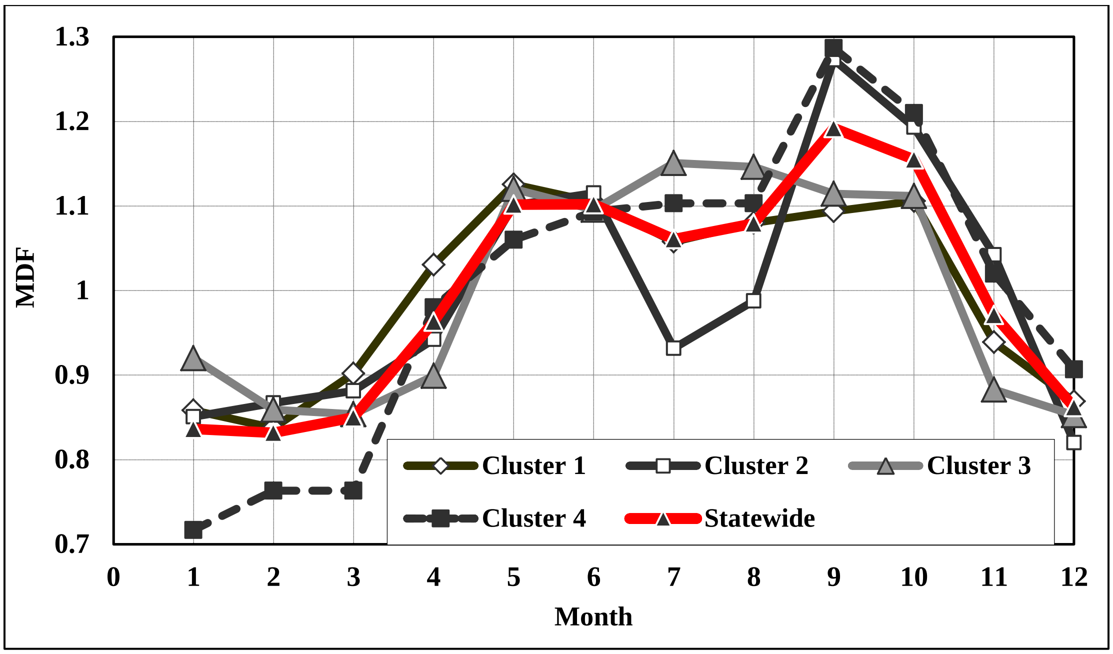


Figure 2.4: MDF Clusters (Class 5, 2010)

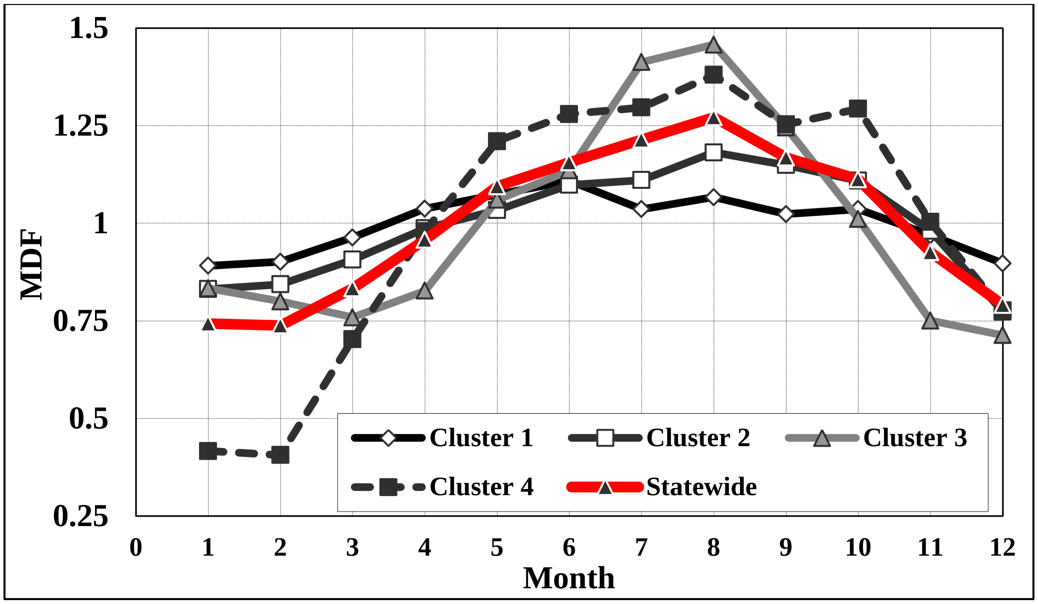


Figure 2.5: MDF Clusters (Class 9, 2010)

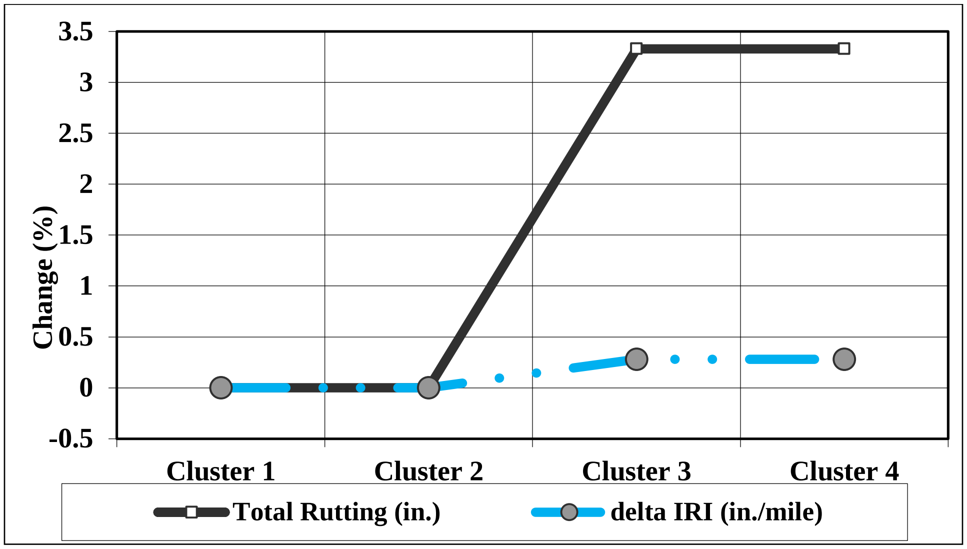


Figure 2.6: Change in Distress Values for Different MDF Clusters, 2010 – Flexible

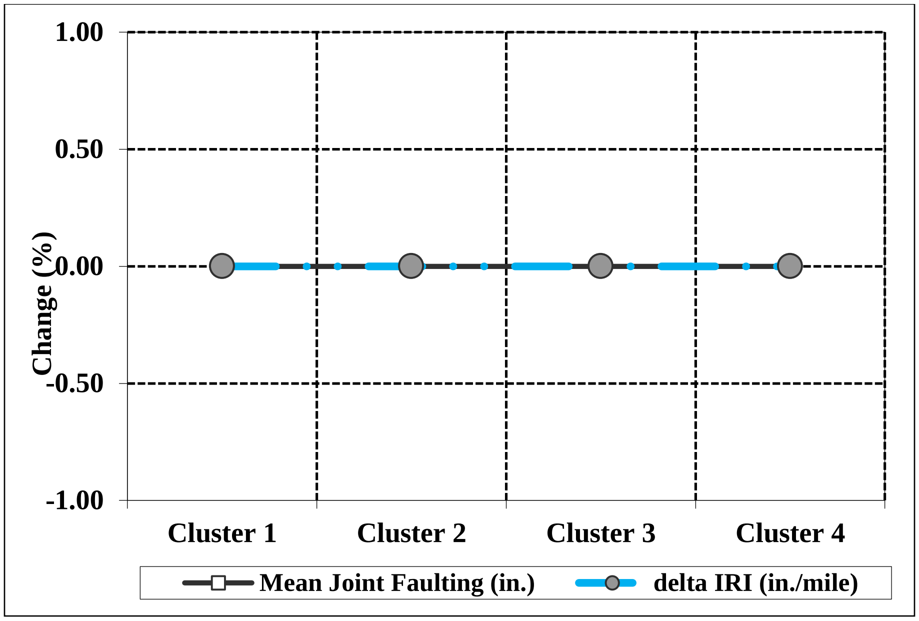


Figure 2.7: Change in Distress Values for Different MDF Clusters, 2010 – Rigid

MEPDG runs were also carried out to find the effect of monthly distribution factors on the rigid pavement (Figure 2.7). It shows no difference in distress values in comparison to those for statewide values. These statewide average values, given in Table 2.2, are suggested for both asphalt and rigid pavements.

Table 2.2: Statewide Average MDF for Class 5 and 9 Vehicles (2010)

Month	Class 5	Class 9
January	0.84	0.94
February	0.83	0.97
March	0.85	1.05
April	0.96	1.07
May	1.10	1.01
June	1.10	1.04
July	1.06	0.97
August	1.08	1.02
September	1.19	1.02
October	1.16	1.04
November	0.97	0.99
December	0.86	0.88

2.4.5 Hourly Distribution Factors (HDF)

The cluster analysis suggested four clusters for the HDF values, as shown in Figure 2.8. Cluster 1 shows similar pattern in comparison with the statewide average; more than half of the classification sites belong to Cluster 1. Almost all of the sites of Regions 8 and 9 of NYSDOT belong to Cluster 1. The sites of Cluster 4 show HDF values comparable to the MEPDG default values. These sites have high two-way AADTT (average of 2,300) and they are mainly located on interstate routes (I-81, I-84, I-86, and I-87). Five classification sites (6100, 7100, 7111, 7381, and 8280) also have high AADTT (average is 2,375) and are located on the same interstate routes but they constitute Cluster 4. Cluster 2 is characterized by high HDF values during morning and evening hours, about 25% higher than the statewide average values. The sites of Cluster 3 showed high AADTT (1,518). MEPDG runs were conducted to study the effect of HDF on rigid pavements; they are not included in the study of flexible pavements. Figure 2.9 suggests that HDF has no effect on the design of rigid pavements.

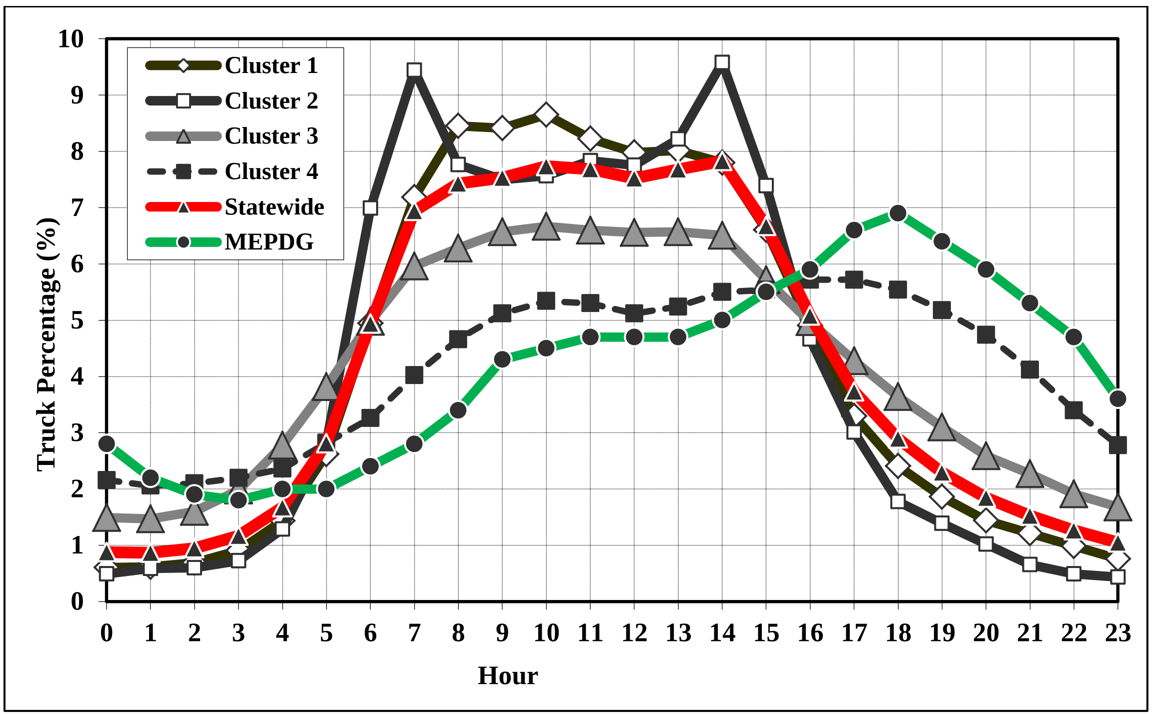


Figure 2.8: HDF Clusters (2010)

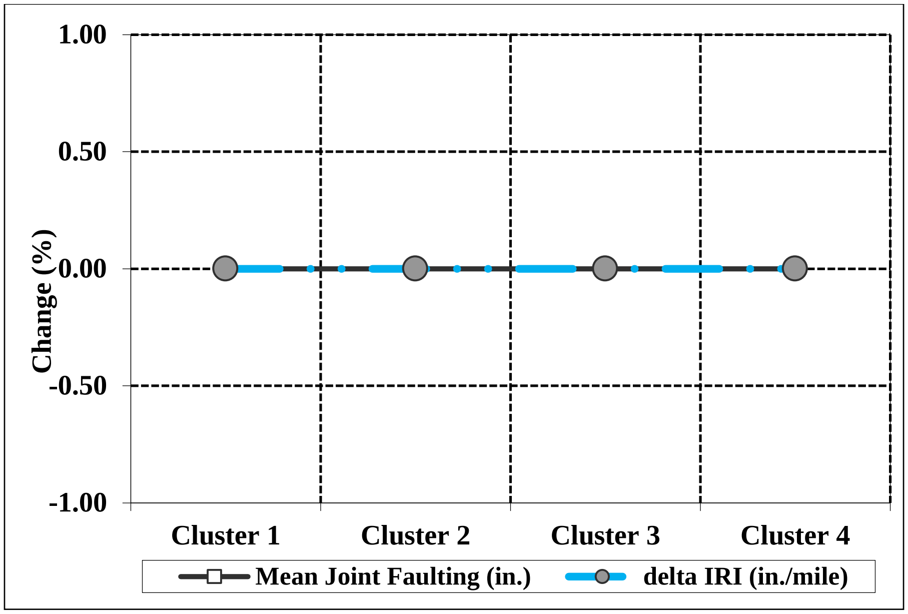


Figure 2.9: Change in Distress Values for Different HDF Clusters, 2010 – Rigid

2.4.6 Axle Groups Per Vehicle (AGPV)

MEPDG runs were carried out for site-specific AGPV values for all 19 WIM sites and for the statewide average values for a typical asphalt pavement. The percentage change in the predicted distress values in comparison with the distresses predicted when statewide average values were used are plotted in Figure 2.10. The figure indicates that the change in predicted distresses ranges between -3.23% and 0% for total rutting. For delta IRI, the change is in between -0.84% and +0.28%. Since no clear pattern could be found for the effect of the variation of AGPV in terms of traffic volume, location, or route functional classification, the use of statewide average values for AGPV is recommended.

The percentage change in distress values for AGPV also does not show any significant change when compared with those for statewide average values for rigid pavement (Figure 2.11). Therefore, statewide average values are recommended for both asphalt and rigid pavements. The statewide average AGPV values are given in Table 2.3.

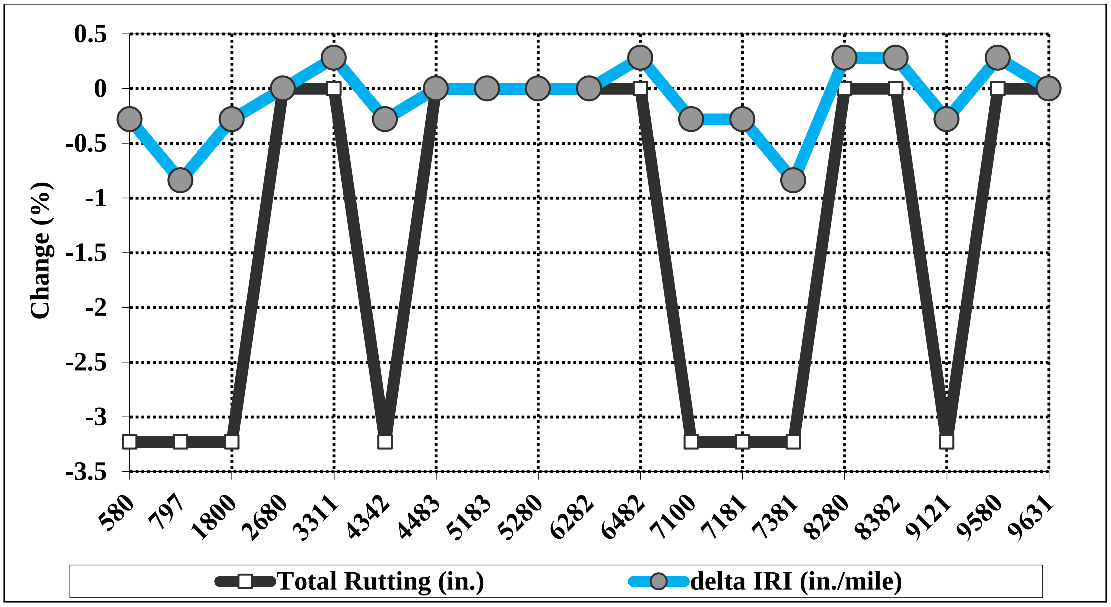


Figure 2.10: Change in Distress Values due to Variation of AGPV, 2010 – Flexible

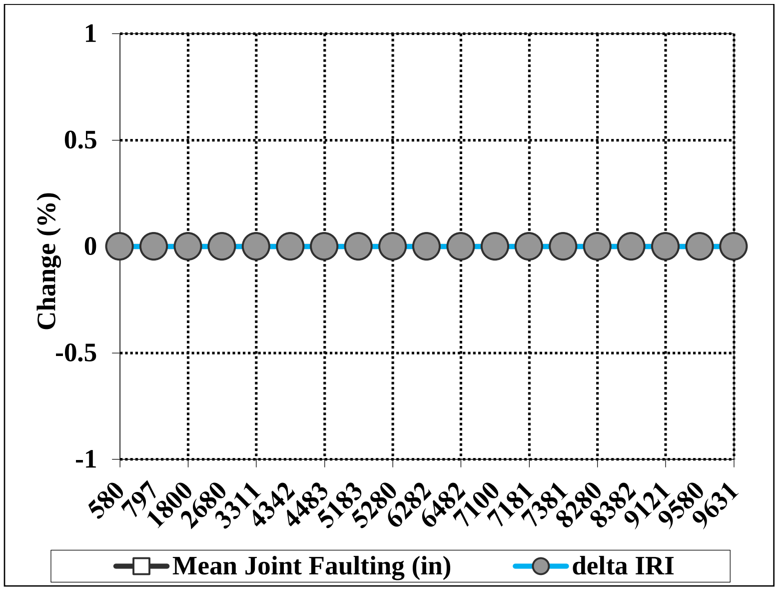


Figure 2.11: Change in Distress Values due to Variation of AGPV, 2010 – Rigid

Table 2.3: Statewide Average AGPV values (2010)

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.32	0.68	0.00	0.00
5	2.00	0.00	0.00	0.00
6	1.00	1.00	0.00	0.00
7	1.32	0.28	0.64	0.05
8	2.45	0.59	0.02	0.00
9	1.23	1.89	0.00	0.00
10	1.07	0.99	0.95	0.05
11	3.70	0.27	0.25	0.01
12	3.71	1.09	0.03	0.00
13	2.11	0.76	0.28	0.32

2.4.7 Axle Load Spectra

MEPDG runs were also performed to study the effect of axle load spectra on the predicted distresses of asphalt pavement. The change in distresses is shown in Figure 2.12. The percentage change in total rutting shows variation within a range of -22.9% to +42.9% (Figure 2.12). Most sites exhibit negative change in distress values which means that the use of statewide average load spectra would slightly overpredict total rutting for most projects. The percentage change in delta IRI is in between -9.1% and +15.5%. However, Site 797 exhibits high distress values. The VCD of this site shows that it has 0.6% Class 13 vehicles. This site can be considered as an outlier. Therefore, the use of statewide axle load spectra instead of site specific load spectra will not much affect the predicted IRI. As a result, the use of statewide average axle load spectra is recommended.

Similarly, the effect of axle load distribution factors was studied on rigid pavement (Figure 2.13). It shows significant change for the mean joint faulting, though actual predicted distress values are small. The percentage change in delta IRI is from -0.35% to 2.5% from the delta IRI corresponding to statewide average values; therefore, statewide average axle load spectra is suggested for the design of asphalt and rigid pavements. For the sake of brevity, the statewide average axle load spectra values are not included in this report; they can be obtained in electronic form from the authors and have been reported by Intaj (2012).

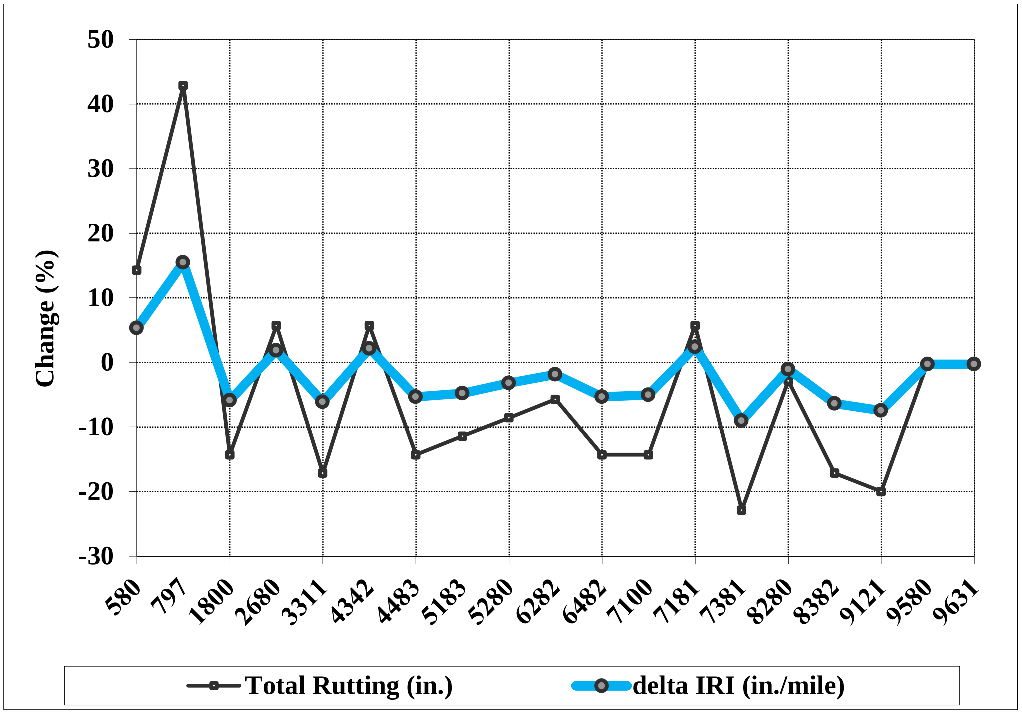


Figure 2.12: Change in Distress Values due to Variation of Axle Load Spectra, 2010 – Flexible

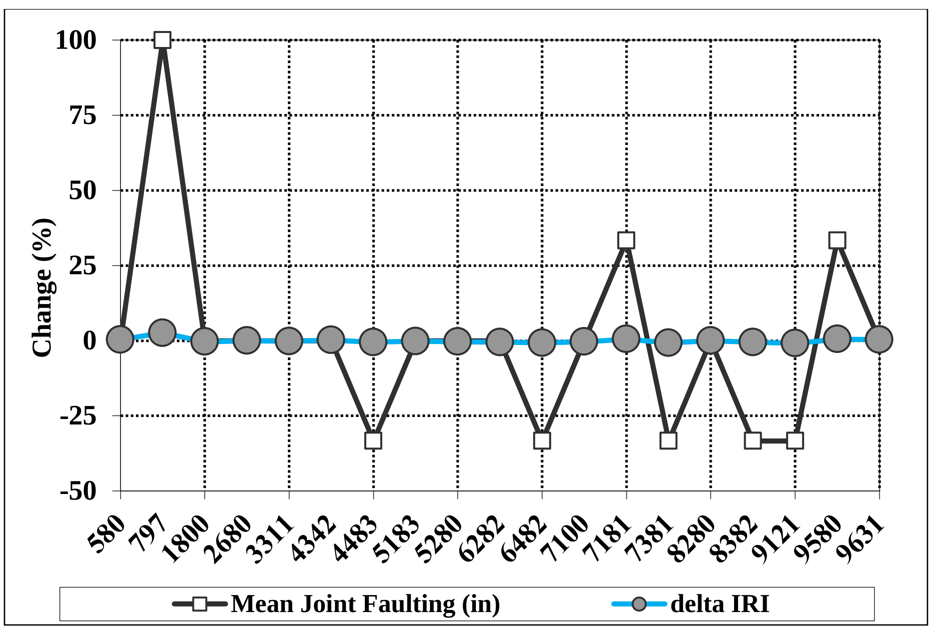


Figure 2.13: Change in Distress Values due to Variation of Axle Load Spectra, 2010 – Rigid

2.5 Change in Traffic Inputs for MEPDG with Years

The traffic data from the vehicle classification and WIM sites show variation with time. Moreover, the number of vehicle classification and WIM sites are not consistent over the years. Proper traffic inputs for the design of pavements were developed from the vehicle classification and WIM data of 2010 only. Therefore, the study of traffic inputs for different years would verify the observations recorded for 2010 data. Cluster analysis was performed for the traffic inputs of vehicle classification data from 2007 to 2011. MEPDG runs were carried out to verify the results of cluster analysis in terms of both flexible and rigid pavement performance for different years. In addition, MEPDG runs were also carried out for AGPV and axle load spectra data of other years.

2.5.1 Vehicle Classification Distribution (VCD)

Both directions were considered for the cluster analysis of vehicle classification distribution data for 5 years (Table 2.4). The results produce the same number of clusters for all the years. No outlier is observed from the analysis of any year. However, 21 vehicle classification sites belonged to different types of clusters for different years. This variation is mostly observed for Clusters 2 and 4. This may be due to the change in proportion of Class 5 and Class 9 vehicles over the years. However, the direction of traffic has little impact on clusters for the years studied.

Table 2.4: Number of Sites Analyzed for Vehicle Class Distribution

Year	2007	2008	2009	2010	2011
Number of vehicle classification sites	55	75	57	52	45

The results of cluster analysis of vehicle class distribution for the years of 2007, 2008, 2009, and 2011 are shown in Figure 2.14, 2.17, 2.20, and 2.23. The results of the analysis are almost consistent for all the years. Therefore, it can be concluded that the proportion of Class 5 and 9 vehicles does not change significantly over time.

The results of MEPDG runs do not show any significant change in total rutting and delta IRI when compared with statewide values for flexible pavement (Figures 2.15, 2.18, 2.21, and

2.24). The distress values for rigid pavement show similar patterns for all the years (Figures 2.16, 2.19, 2.22, and 2.25). Even though the distress values for mean joint faulting are different from those corresponding to the statewide average VCD values, the differences are small. These results conform to the results of analysis for 2010, i.e., statewide average values are suggested for vehicle class distribution.

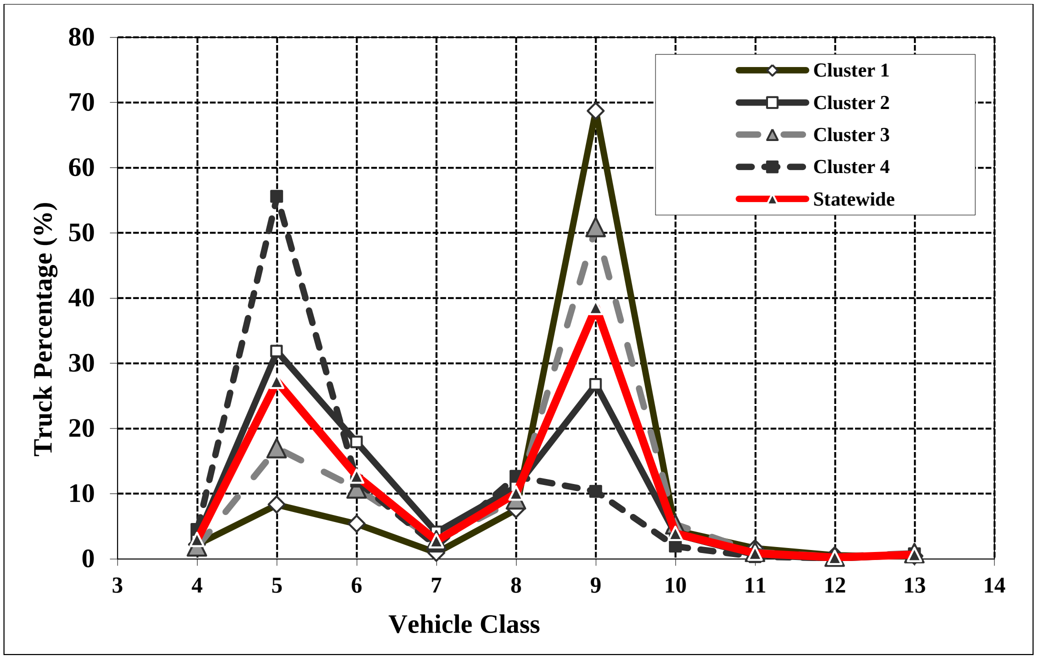


Figure 2.14: VCD Clusters (2007)

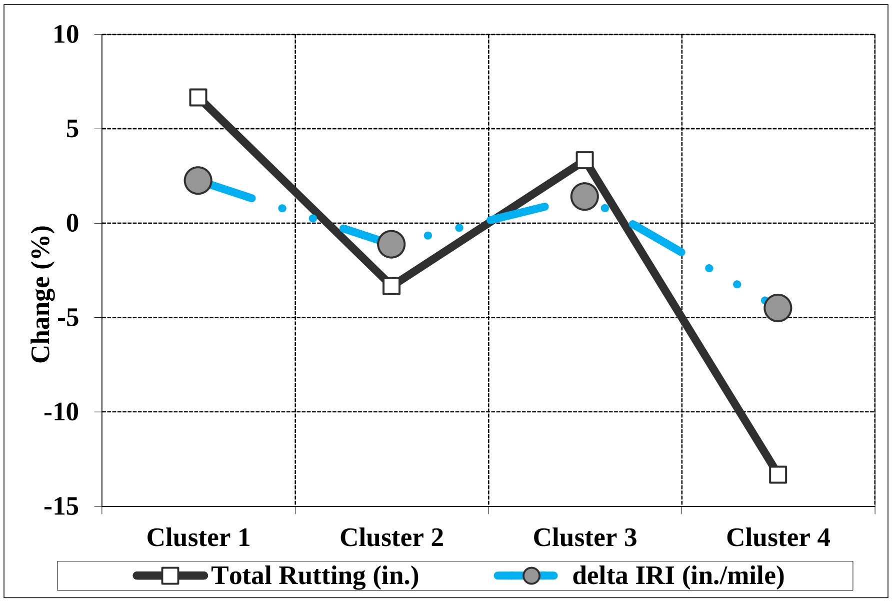


Figure 2.15: Change in Distress Values for Different VCD Clusters, 2007 – Flexible

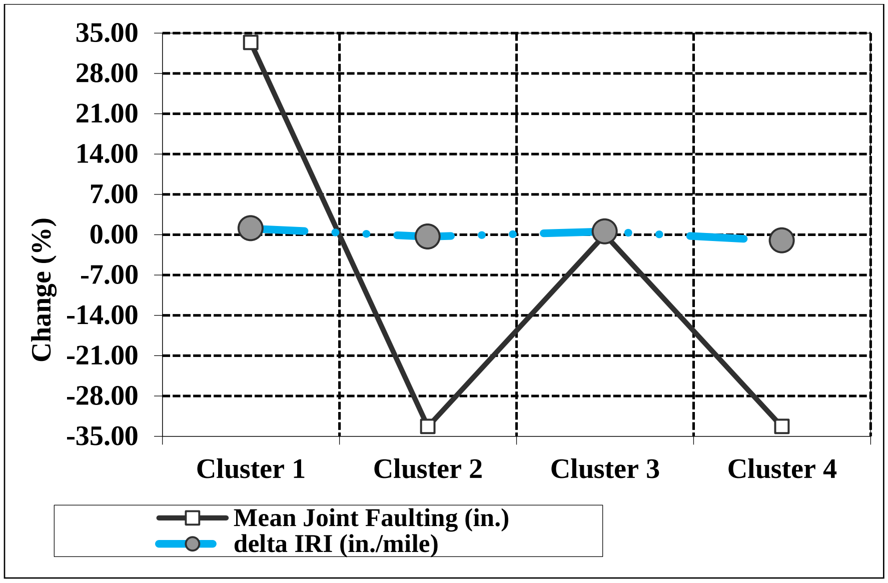


Figure 2.16: Change in Distress Values for Different VCD Clusters, 2007 – Rigid

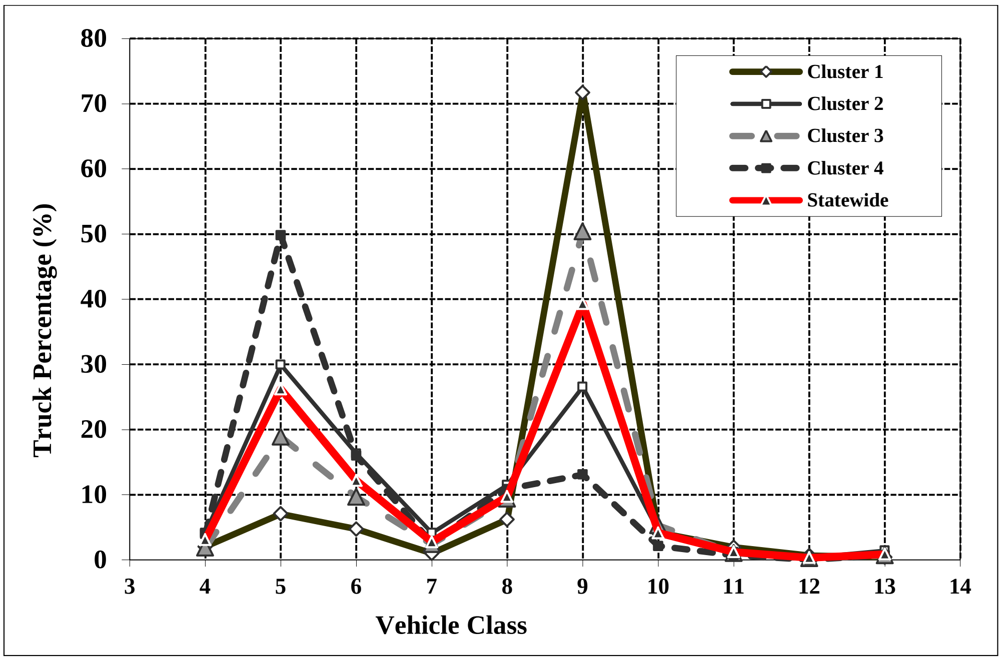


Figure 2.17: VCD Clusters (2008)

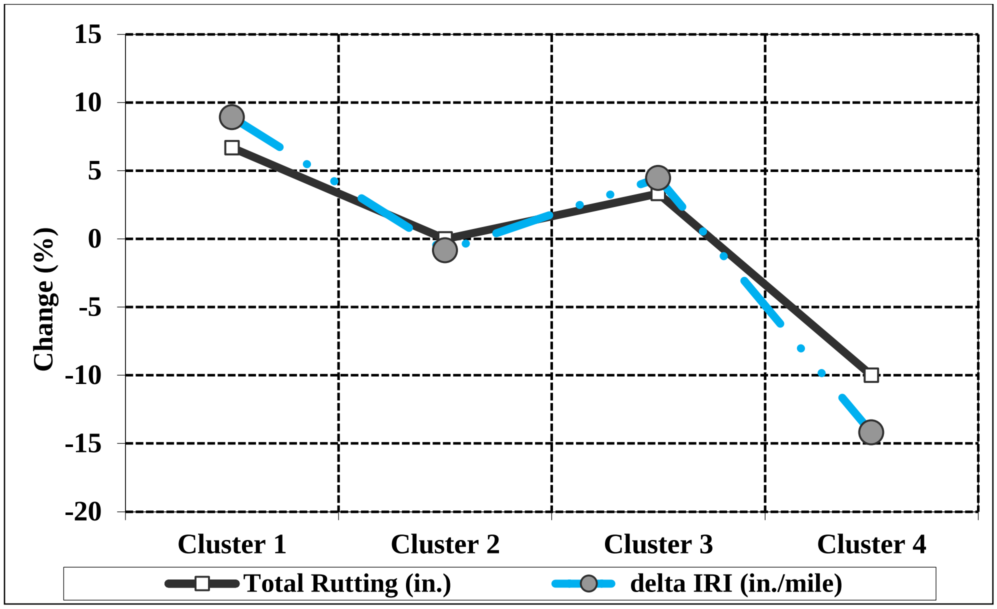


Figure 2.18: Change in Distress Values for Different VCD Clusters, 2008 – Flexible

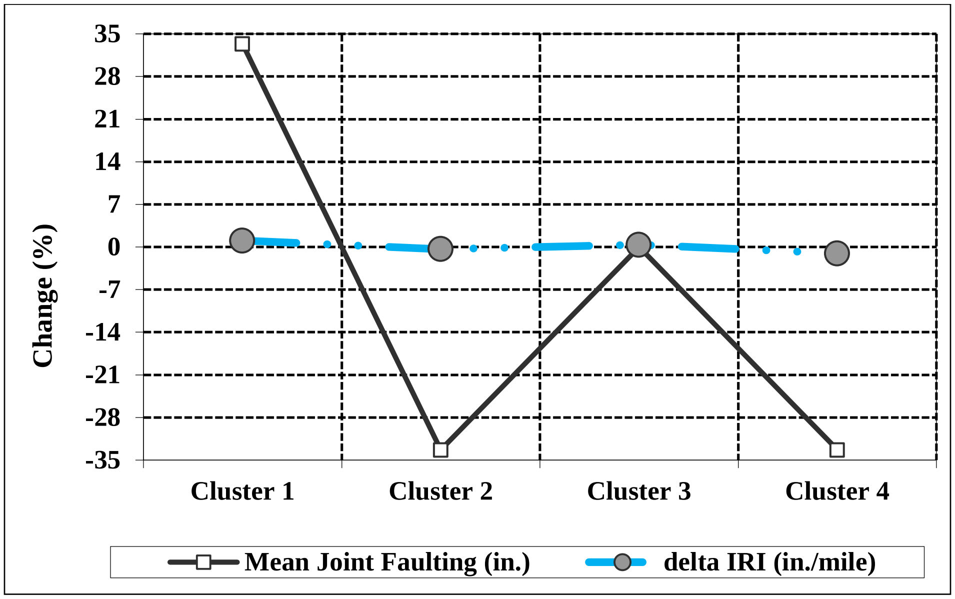


Figure 2.19: Change in Distress Values for Different VCD Clusters, 2008 – Rigid

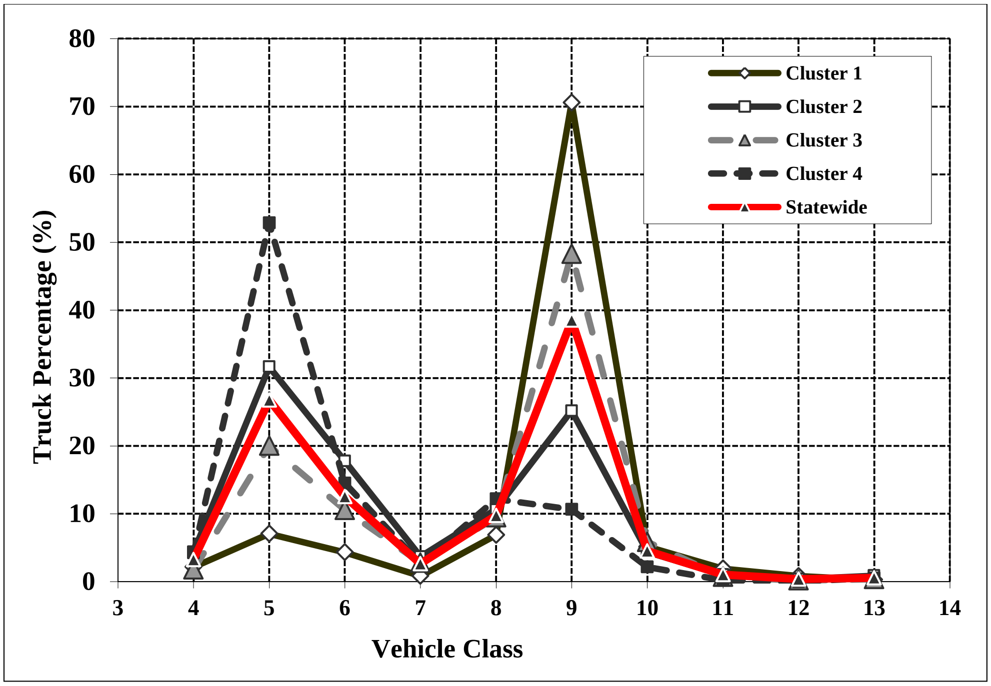


Figure 2.20: VCD Clusters (2009)

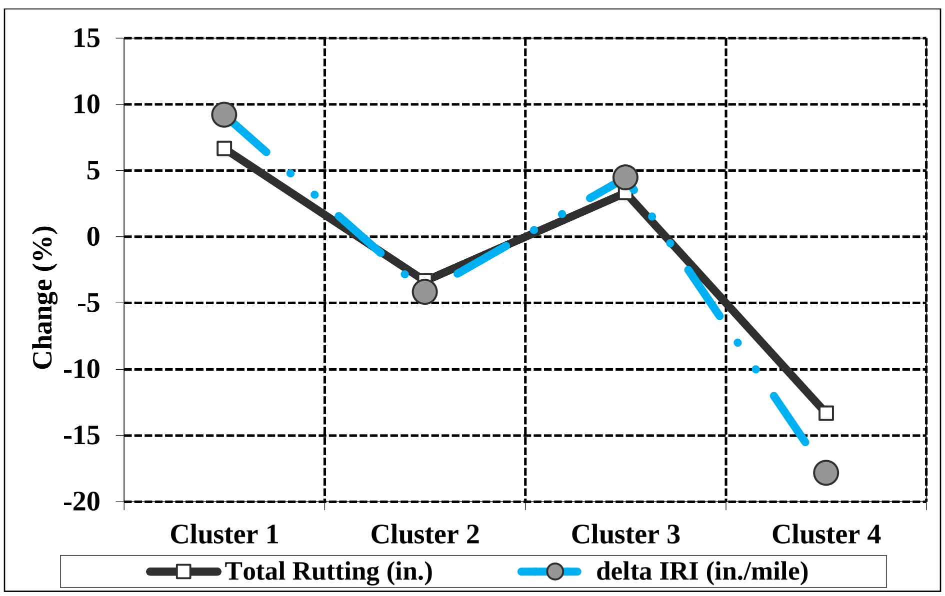


Figure 2.21: Change in Distress Values for Different VCD Clusters, 2009 – Flexible

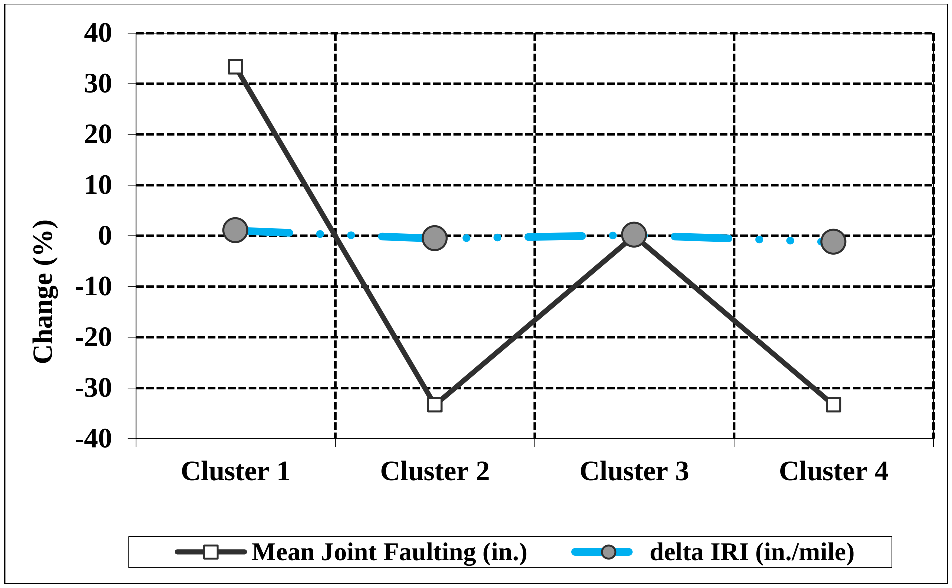


Figure 2.22: Change in Distress Values for Different VCD Clusters, 2009 – Rigid

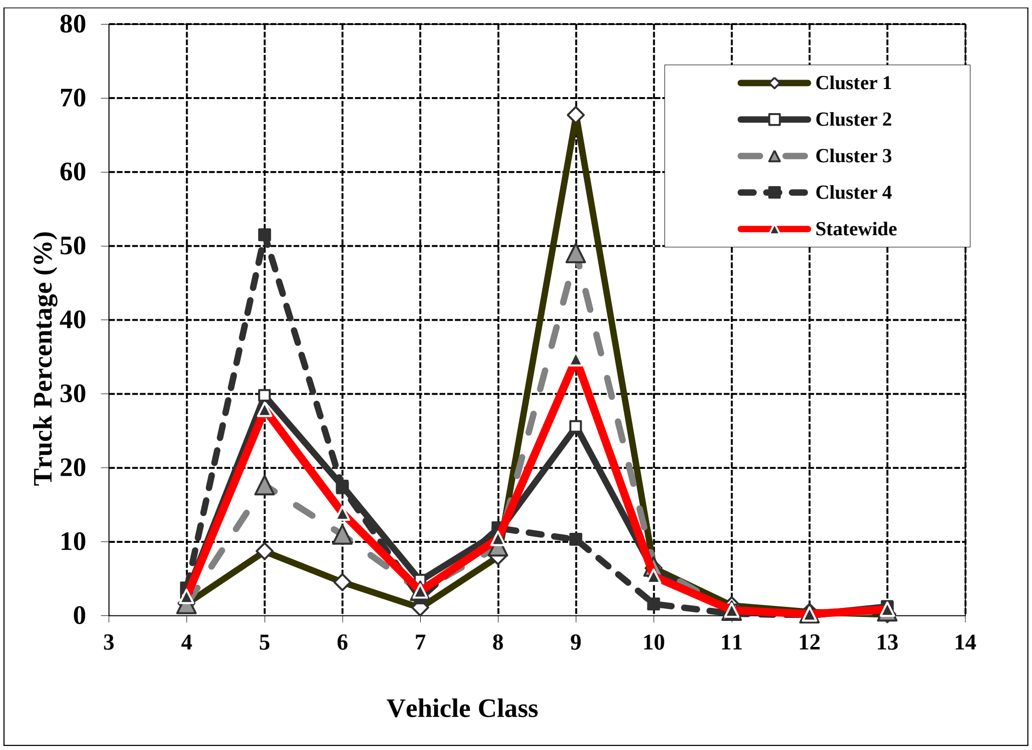


Figure 2.23: VCD Clusters (2011)

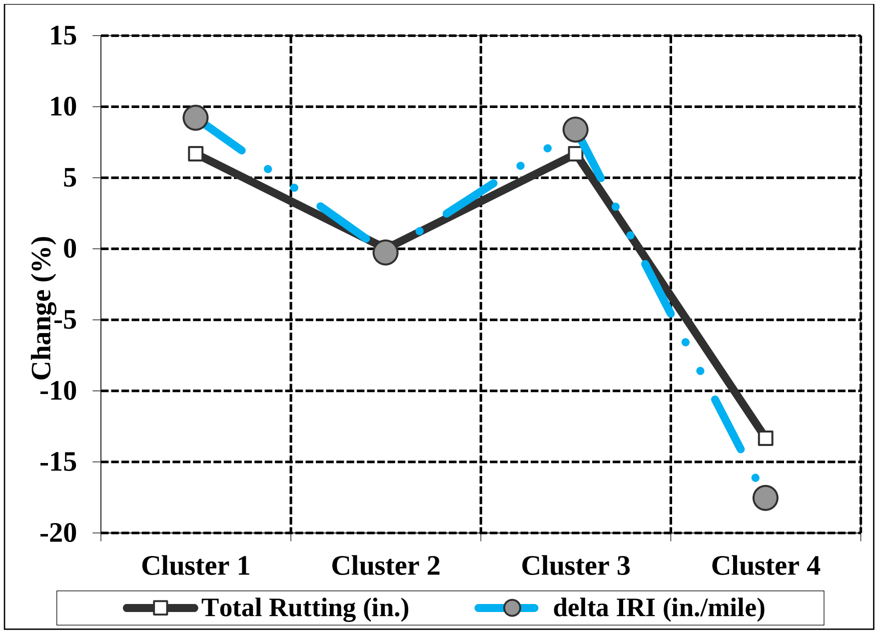


Figure 2.24: Change in Distress Values for Different VCD Clusters, 2011 – Flexible

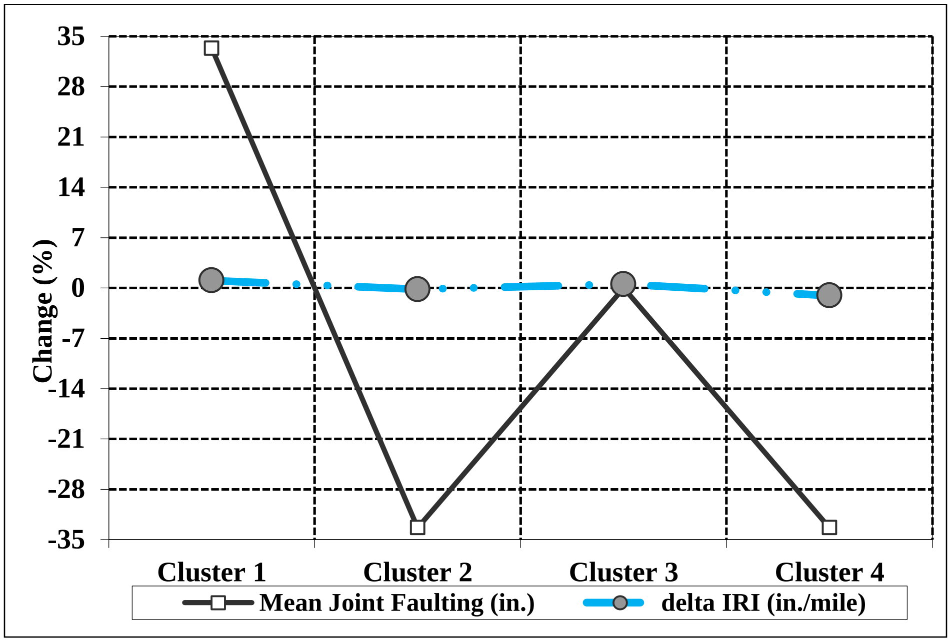


Figure 2.25: Change in Distress Values for Different VCD Clusters, 2011 – Rigid

Statewide average vehicle class distributions of different years are given in Table 2.5. The statewide average VCDs were compared in terms of pavement performance (Table 2.6). The predicted distress values of total rutting and delta IRI for the statewide average VCDs exhibit coefficients of variation of 6.1% and 2.1%, respectively, for flexible pavement, while the distress values of mean joint faulting and delta IRI show coefficients of variation of 0.0% and 0.1% for rigid pavement over the years studied. Therefore, using statewide average VCDs from different years has little impact on the predicted pavement performance.

Table 2.5: Statewide Average Vehicle Class Distributions for Different Years

Years	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
2007	2.97	27.23	12.67	2.77	10.11	38.57	3.91	0.87	0.23	0.65
2008	3.11	26.23	12.19	2.78	9.75	39.30	4.16	1.23	0.34	0.90
2009	3.26	26.74	12.52	2.66	9.76	38.56	4.53	1.03	0.38	0.57
2011	2.62	27.89	13.84	3.35	10.42	34.73	5.31	0.72	0.22	0.87

Table 2.6: Predicted Distress Values for Statewide Average VCDs for Different Years

Pavement Type	Predicted Distresses	Years					Mean	Standard Deviation	COV
		2007	2008	2009	2010	2011			
Flexible	Total Rutting (in.)	0.30	0.30	0.26	0.30	0.30	0.29	0.018	6.1%
	Delta IRI	35.5	35.6	33.9	35.6	35.5	35.2	0.740	2.1%
Rigid	Mean Joint Faulting (in.)	0.003	0.003	0.003	0.003	0.003	0.003	0.000	0.0%
	Delta IRI	57.2	57.3	57.3	57.2	57.2	57.2	0.055	0.1%

2.5.2 Monthly Distribution Factors (MDF)

Cluster analysis was conducted for monthly distribution factors of Class 5 and 9 vehicles (Table 2.7). The number of clusters is not consistent for the years; four clusters were obtained for 2007, 2008, and 2010, while three clusters were obtained for 2009 and five for 2011. One outlier site was recorded for 2007, 2008, and 2010, while three outlier sites were observed for 2009. No outlier is observed for 2011.

Table 2.7: Number of Sites Analyzed for Monthly Distribution Factors

Year	2007	2008	2009	2010	2011
Number of vehicle classification sites	38	38	34	52	45

The results of cluster analysis for monthly distribution factors are shown in Figures 2.26, 2.27, 2.30, 2.31, 2.34, 2.35, 2.38, and 2.39. The range of variation of the MDF values are almost the same for all the years. However, the peak and low values for different seasons of the years are not consistent. This may be due to the variation over the years in the number of vehicle classification and WIM sites and number of clusters.

The MEPDG runs for 2007 and 2008 show almost same results as for 2010, as they have same number of clusters (Figures 2.28, 2.29, 2.36, and 2.40). However, the distress values for total rutting and delta IRI do not have any significant variation when compared with the statewide values for flexible pavement over time. In addition, the distress values for mean joint faulting and delta IRI do not have much impact on pavement design in comparison with the statewide average values (Figures 2.29, 2.33, 2.37, and 2.41). The distress values for mean joint faulting may show high variation but the actual distress values are small. Therefore, the statewide average MDF values are recommended to be used as traffic inputs.

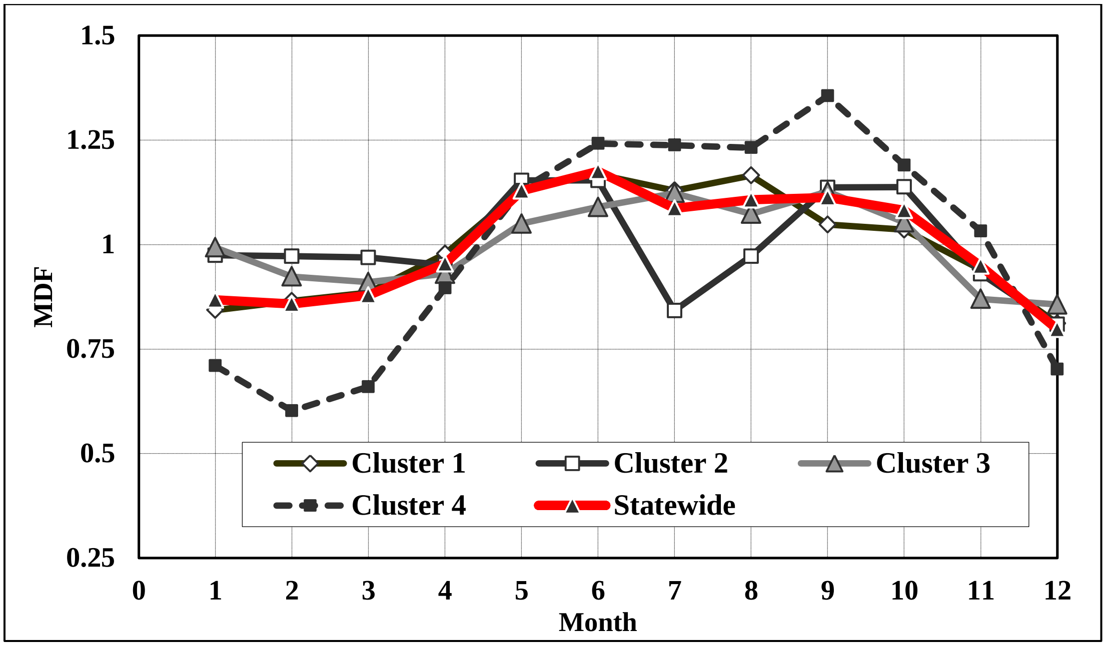


Figure 2.26: MDF Clusters (Class 5, 2007)

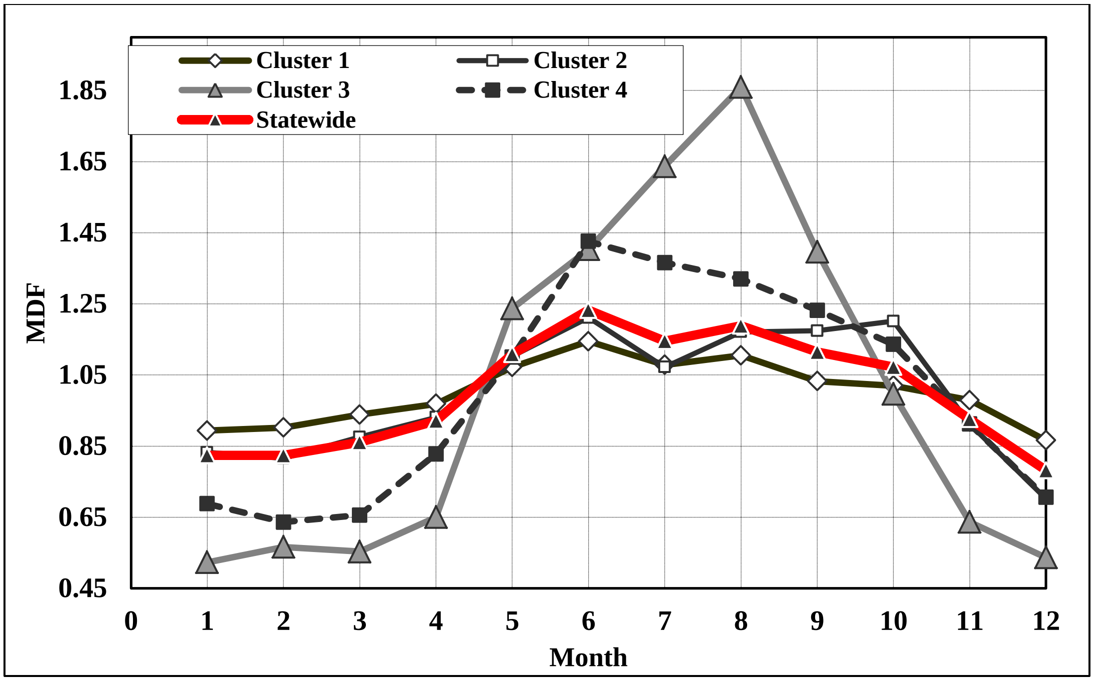


Figure 2.27: MDF Clusters (Class 9, 2007)

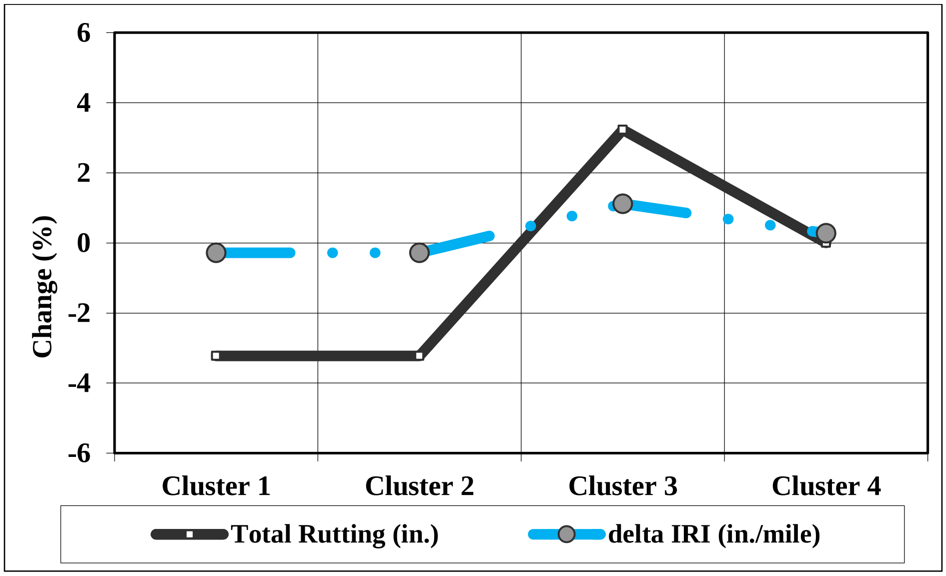


Figure 2.28: Change in Distress Values for Different MDF Clusters, 2007 – Flexible

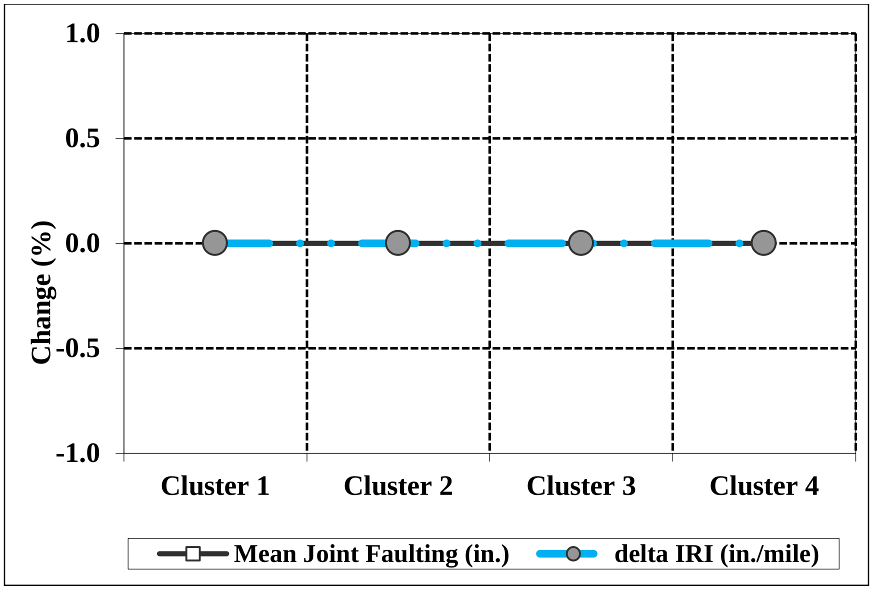


Figure 2.29: Change in Distress Values for Different MDF Clusters, 2007 – Rigid

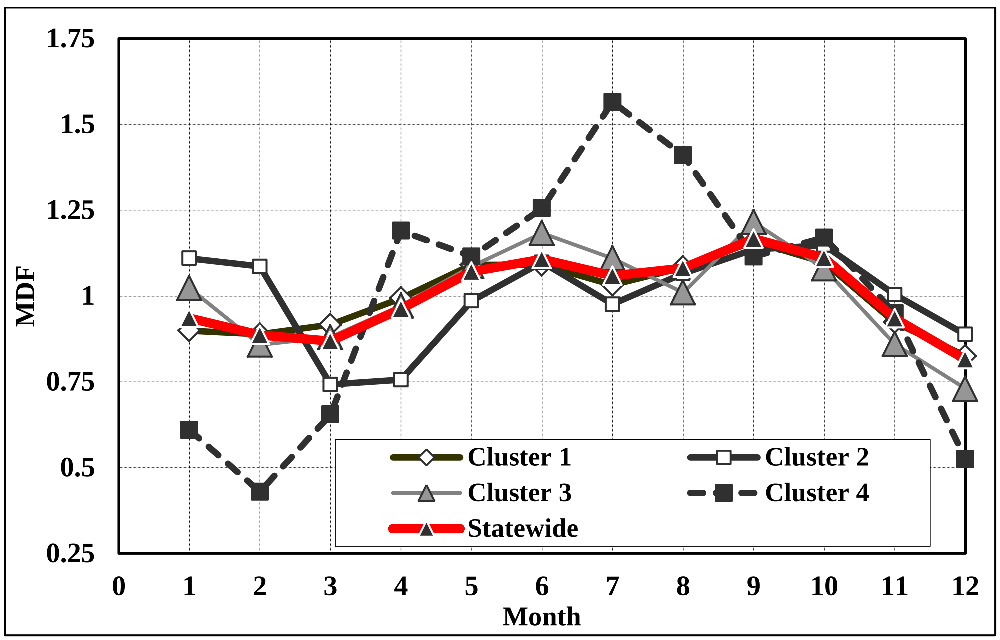


Figure 2.30: MDF Clusters (Class 5, 2008)

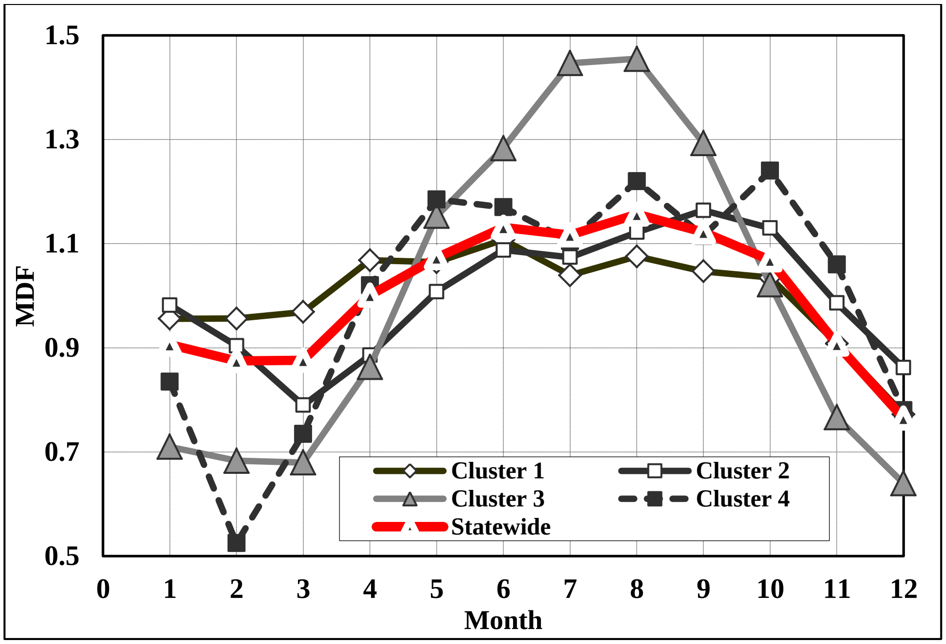


Figure 2.31: MDF Clusters (Class 9, 2008)

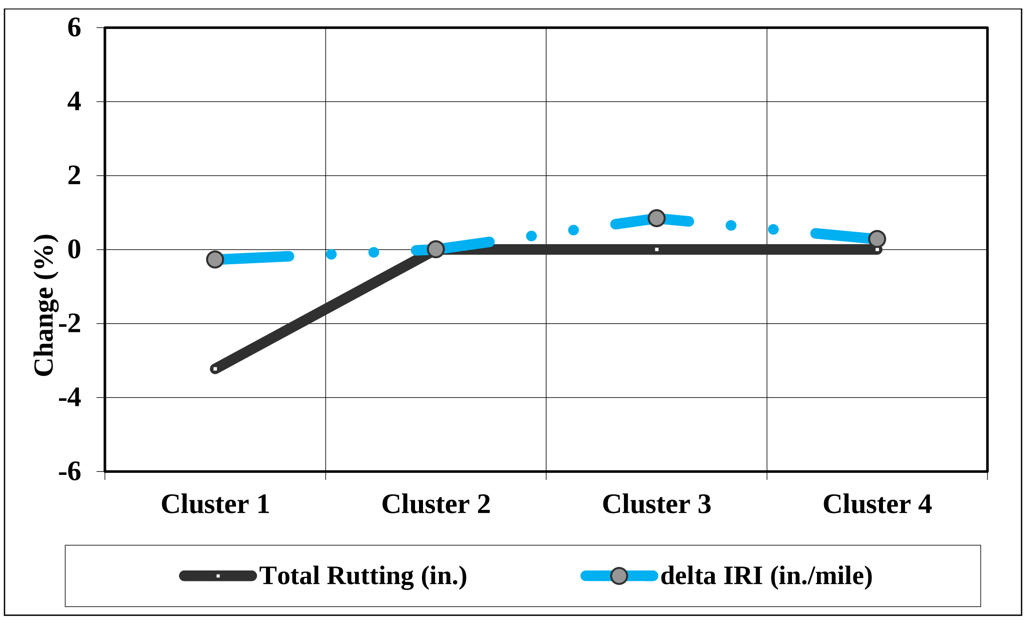


Figure 2.32: Change in Distress Values for Different MDF Clusters, 2008 – Flexible

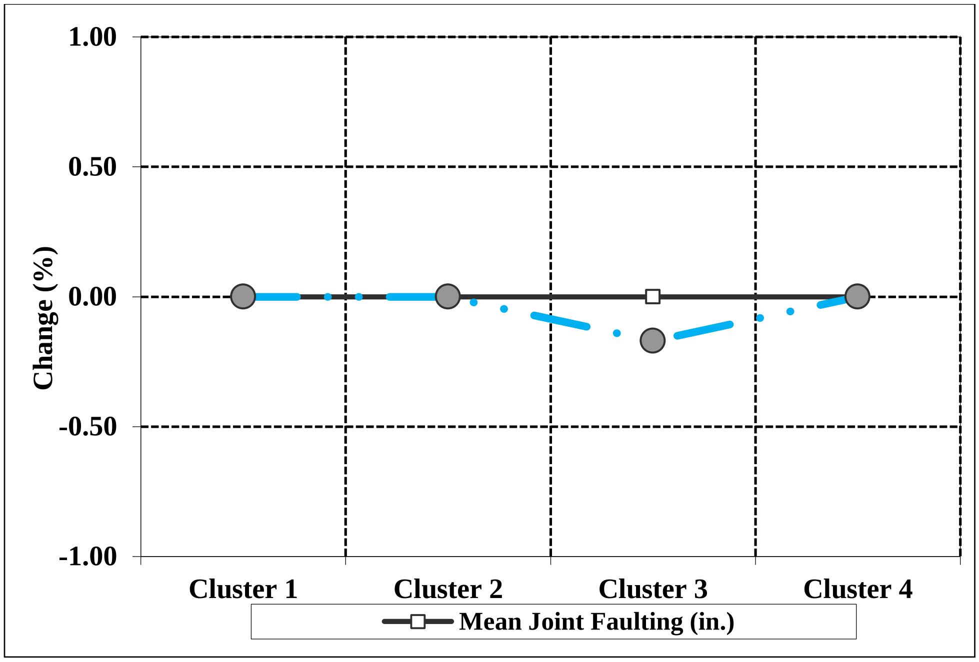


Figure 2.33: Change in Distress Values for Different MDF Clusters, 2008 – Rigid

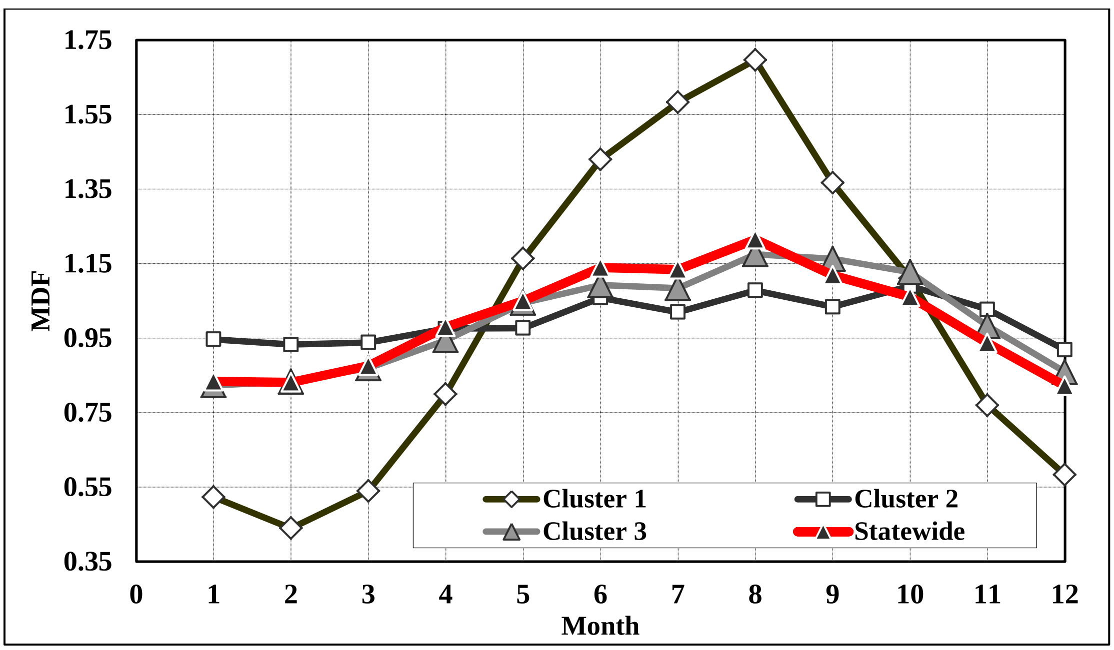


Figure 2.34: MDF Clusters (Class 5, 2009)

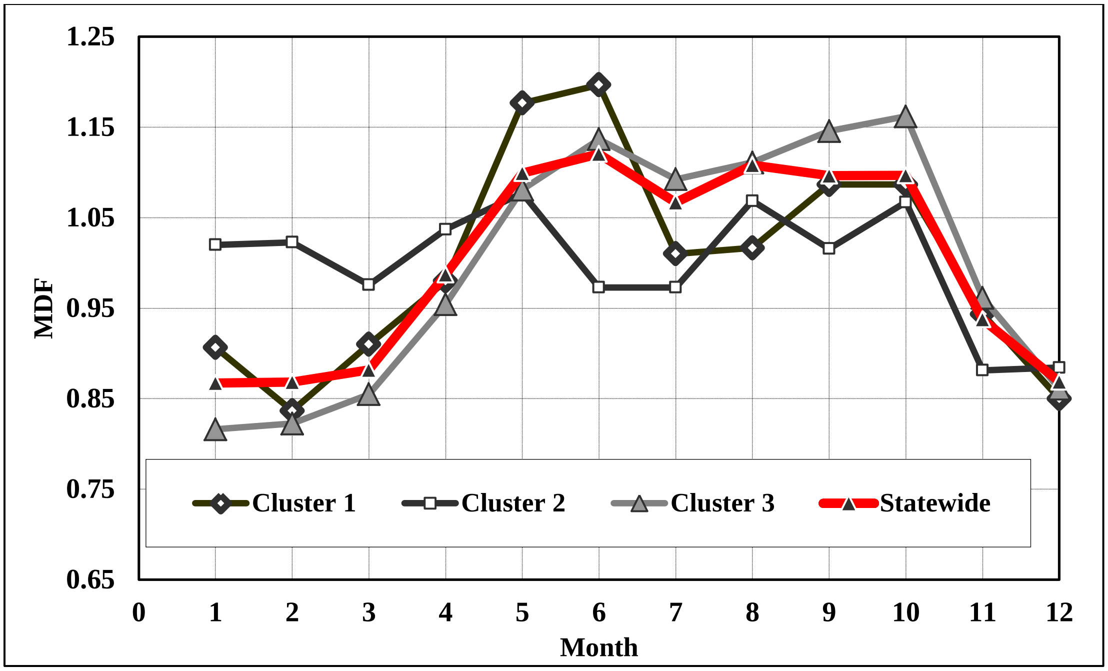


Figure 2.35: MDF Clusters (Class 9, 2009)

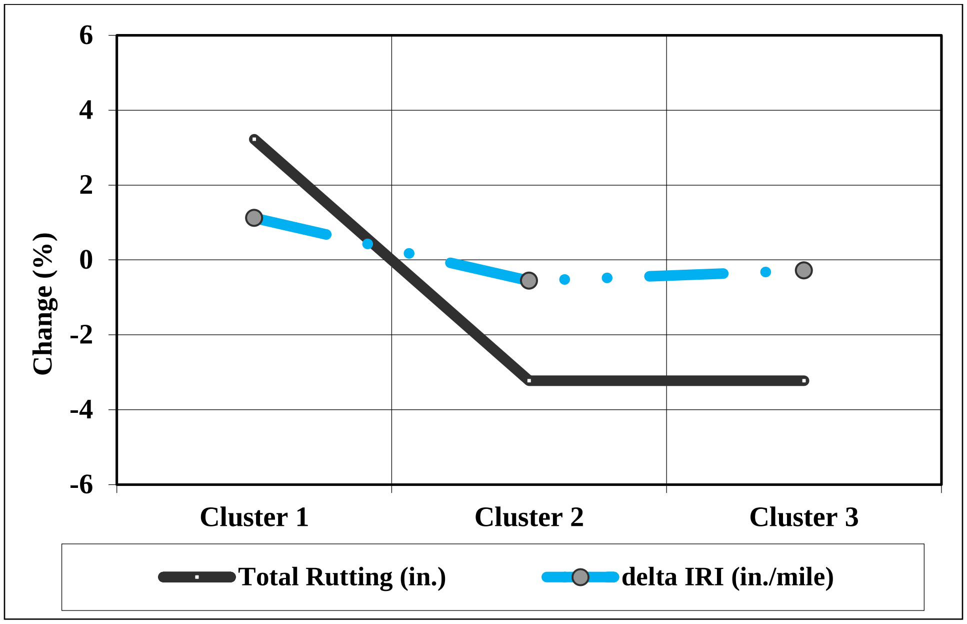


Figure 2.36: Change in Distress Values for Different MDF Clusters, 2009 – Flexible

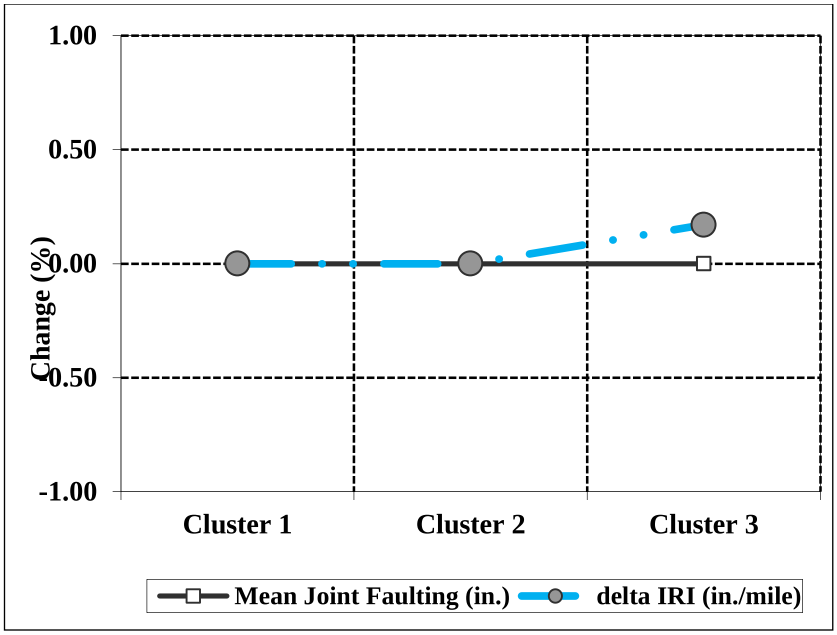


Figure 2.37: Change in Distress Values for Different MDF Clusters, 2009 – Rigid

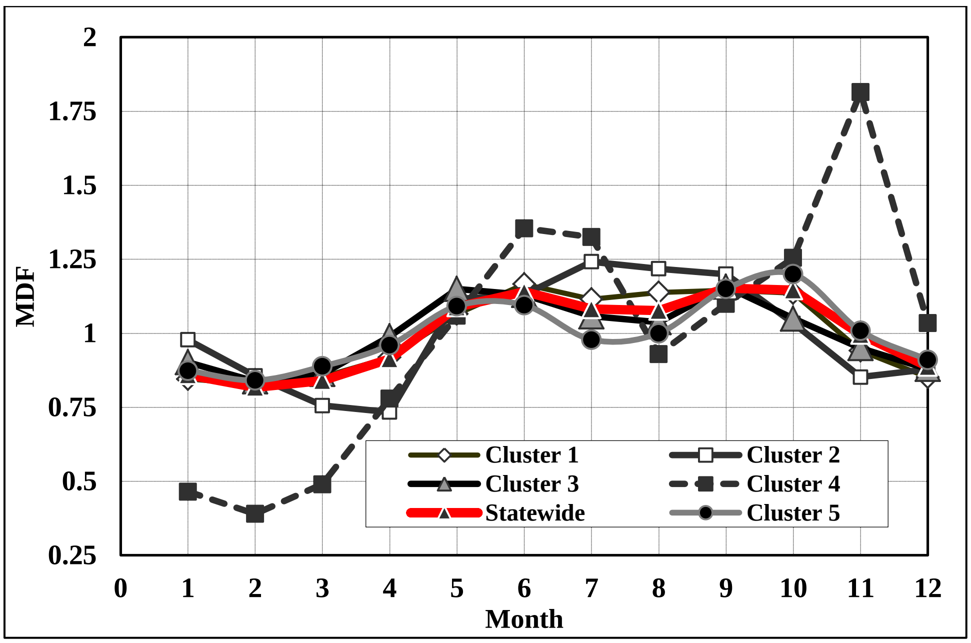


Figure 2.38: MDF Clusters (Class 5, 2011)

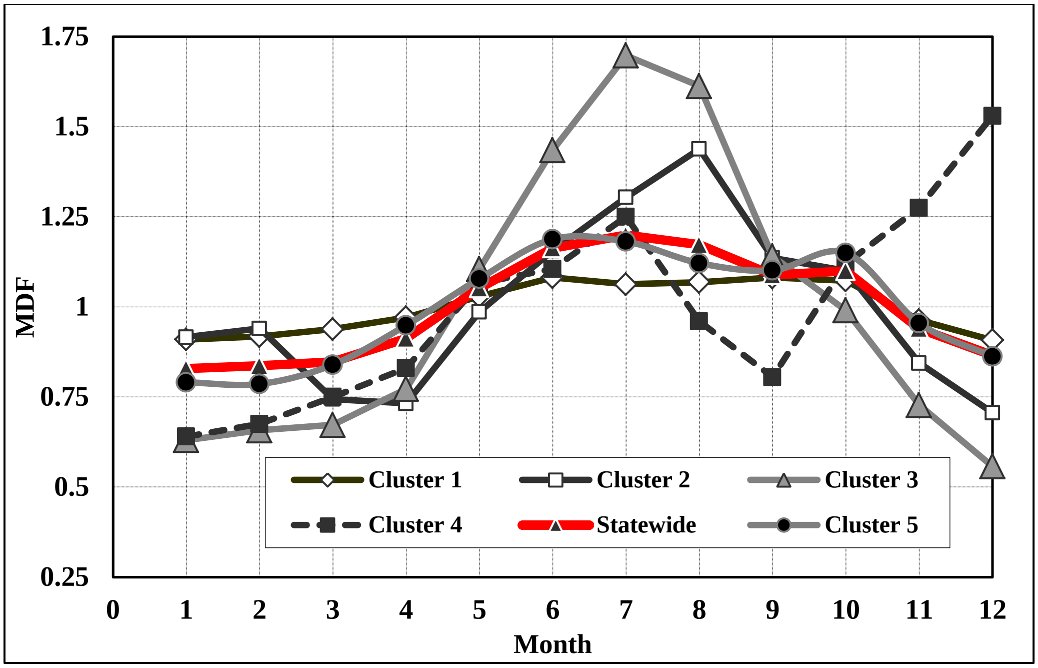


Figure 2.39: MDF Clusters (Class 9, 2011)

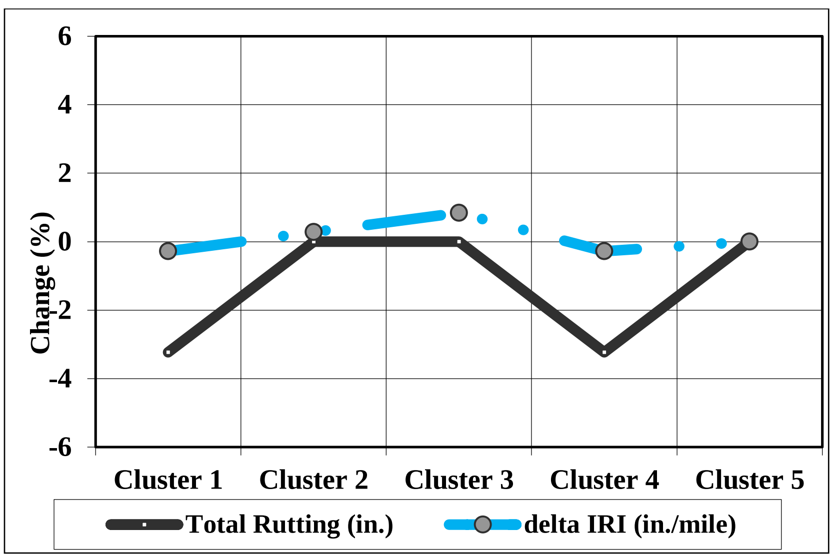


Figure 2.40: Change in Distress Values for Different MDF Clusters, 2011 – Flexible

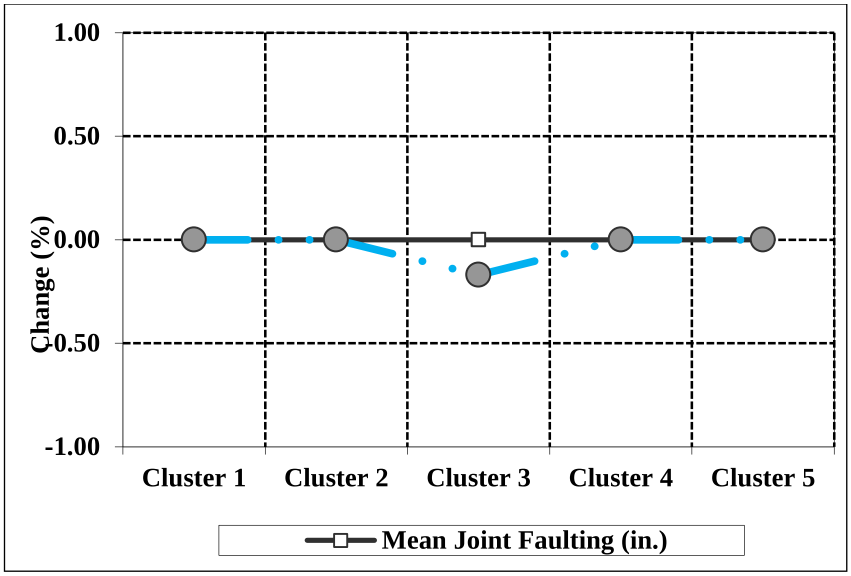


Figure 2.41: Change in Distress Values for Different MDF Clusters, 2011 – Rigid

The statewide average MDF values for Class 5 and 9 vehicles in different years are given in Table 2.8. They were compared in terms of the effect on pavement performance (Table 2.9). The predicted total rutting and delta IRI for the statewide average MDFs in different years have coefficients of variation of 1.5% and 0.1%, respectively. The coefficients of variation for the distress values of mean joint faulting and delta IRI are also very small. Therefore, the statewide average MDFs from different years have a very small impact on the predicted pavement performance.

Table 2.8: Statewide Average Monthly Distribution Factors for Different Years

Month	2007		2008		2009		2011	
	Class 5	Class 9	Class 5	Class 9	Class 5	Class 9	Class 5	Class 9
January	0.87	0.82	0.94	0.91	0.87	0.83	0.86	0.83
February	0.86	0.82	0.89	0.87	0.87	0.83	0.82	0.84
March	0.88	0.86	0.87	0.88	0.88	0.88	0.84	0.85
April	0.95	0.92	0.96	1.00	0.99	0.98	0.91	0.91
May	1.13	1.11	1.07	1.07	1.10	1.05	1.09	1.05
June	1.18	1.23	1.11	1.13	1.12	1.14	1.14	1.16
July	1.09	1.15	1.06	1.12	1.07	1.13	1.08	1.20
August	1.11	1.19	1.08	1.16	1.11	1.21	1.08	1.17
September	1.11	1.12	1.17	1.12	1.10	1.11	1.15	1.09
October	1.08	1.07	1.11	1.07	1.10	1.06	1.15	1.10
November	0.95	0.93	0.94	0.91	0.94	0.94	1.00	0.94
December	0.80	0.78	0.82	0.76	0.87	0.82	0.89	0.86

Table 2.9: Predicted Distress Values for Statewide Average MDFs for Different Years

Pavement Type	Predicted Distresses	Years					Mean	Standard Deviation	COV
		2007	2008	2009	2010	2011			
Flexible	Total Rutting (in.)	0.31	0.31	0.31	0.30	0.31	0.31	0.004	1.5%
	Delta IRI	35.7	35.7	35.7	35.6	35.7	35.68	0.045	0.1%
Rigid	Mean Joint Faulting (in.)	0.003	0.003	0.003	0.003	0.003	0.003	0.000	0.0%
	Delta IRI	57.2	57.3	57.2	57.2	57.2	57.2	0.045	0.1%

2.5.3 Hourly Distribution Factors (HDF)

The number of sites analyzed for hourly distribution factors are the same as those for vehicle classification distribution. The results of cluster analysis for hourly distribution factors is consistent for all the years. Four clusters were found and no outlier was observed for each of the years. However, most of the sites belong to different clusters in different years.

The results of cluster analysis of hourly distribution factors are shown in Figures 2.42, 2.44, 2.46, and 2.48. Variation of truck percentages for different hours of the day are observed over the years. However, the patterns of the graphs do not show much variation over the years. MEPDG runs were carried out considering cluster specific, statewide, and MEPDG default HDF values for rigid pavement for different years (Figures 2.43, 2.45, 2.47, and 2.49). No significant change in distress values was also found due to the variation in HDFs for rigid pavement over the years. As HDF is not considered for the design of flexible pavement, no MEPDG simulations were conducted for this type of pavement. Statewide average values of predicted distresses for different years show no variation over the 5 years (Table 2.10).

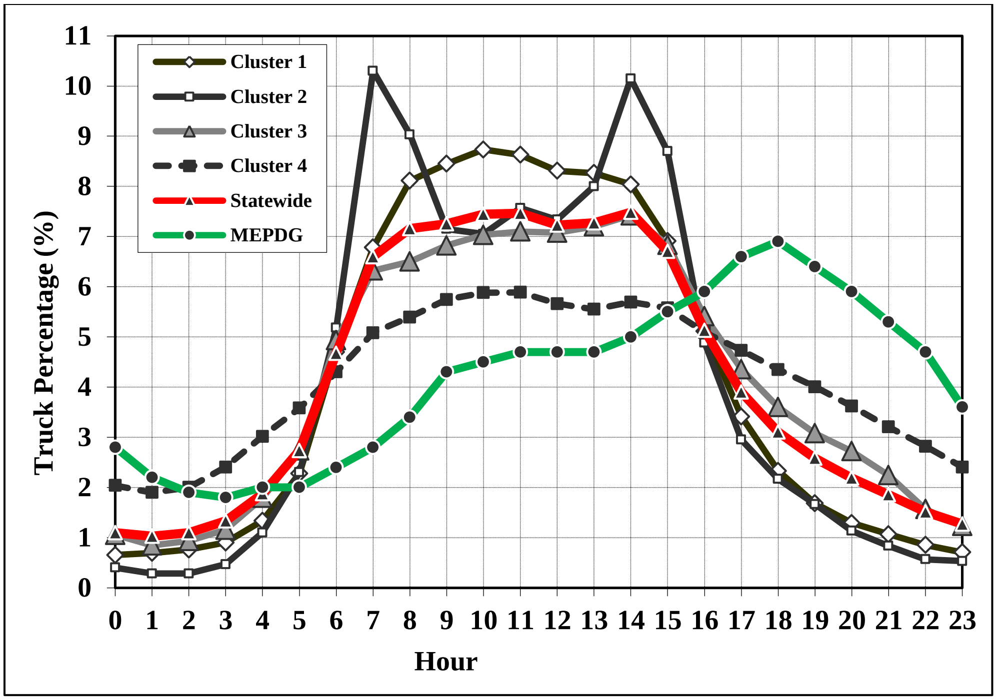


Figure 2.42: HDF Clusters (2007)

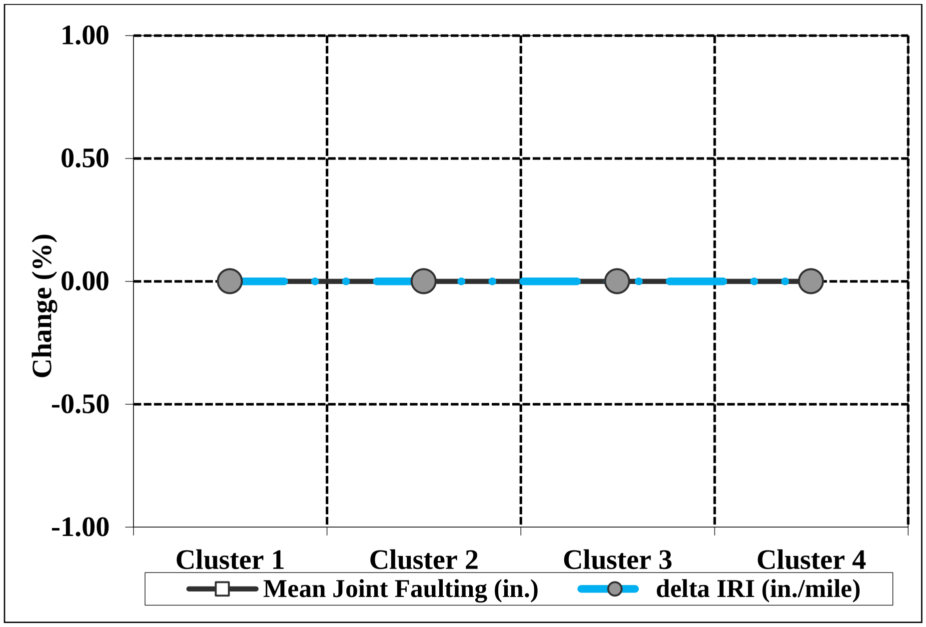


Figure 2.43: Change in Distress Values for Different HDF Clusters, 2007 – Rigid

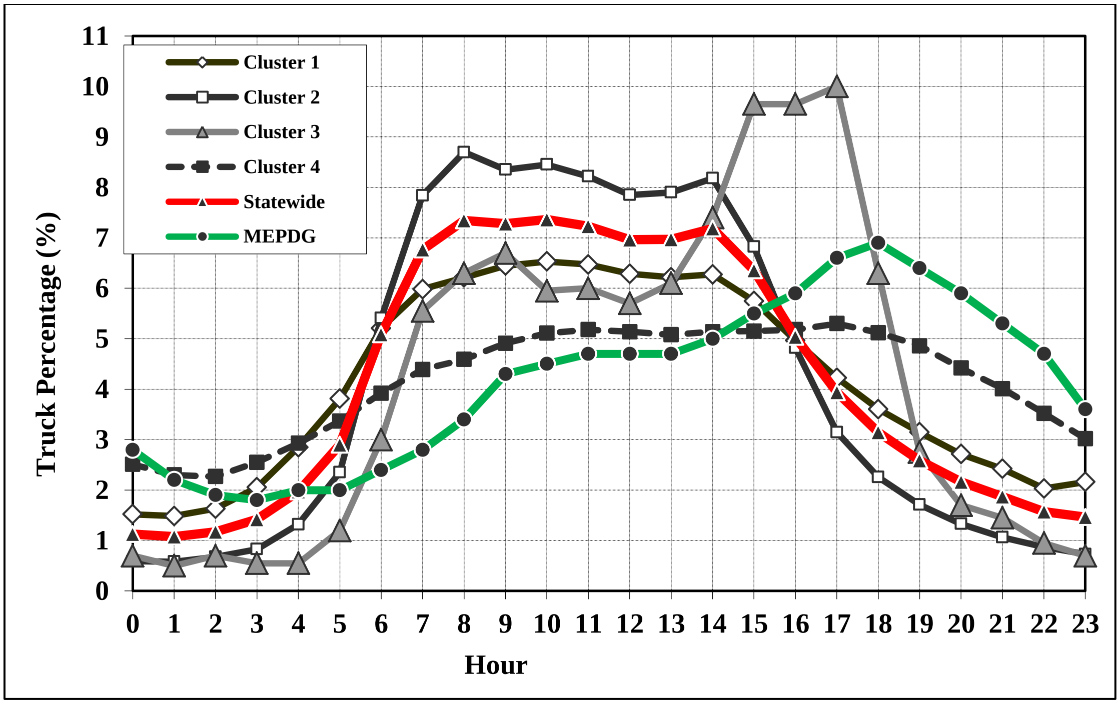


Figure 2.44: HDF Clusters (2008)

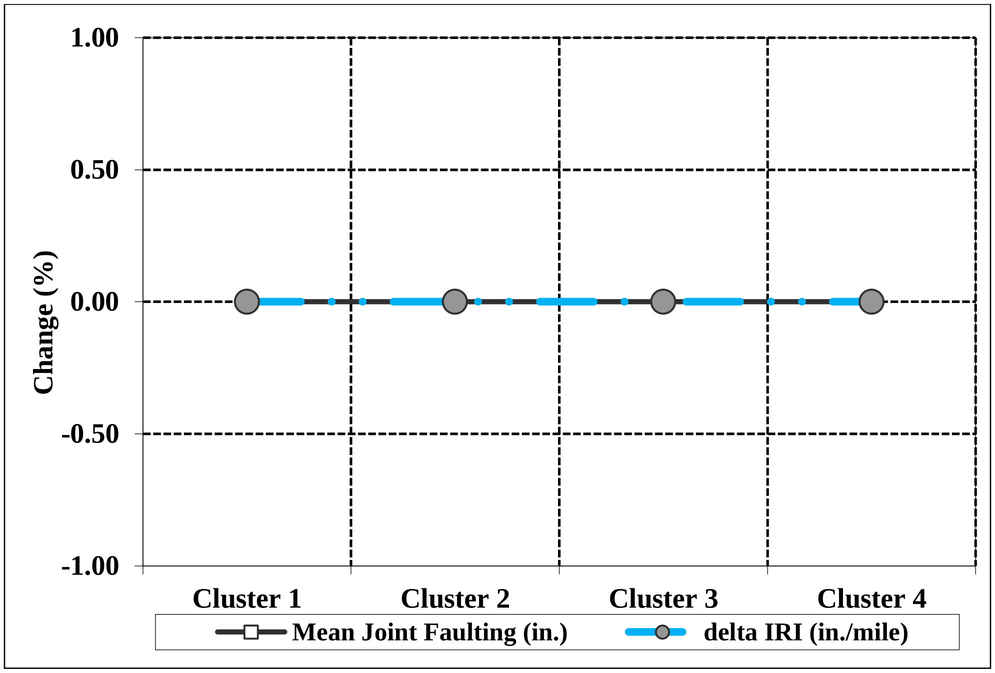


Figure 2.45: Change in Distress Values for Different HDF Clusters, 2008 – Rigid

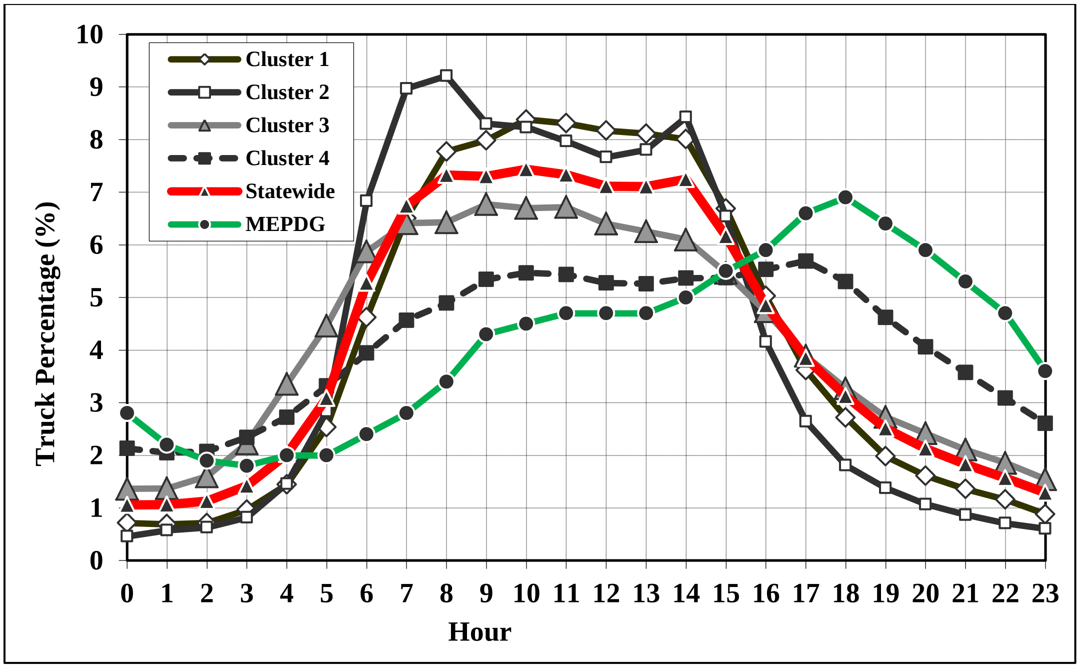


Figure 2.46: HDF Clusters (2009)

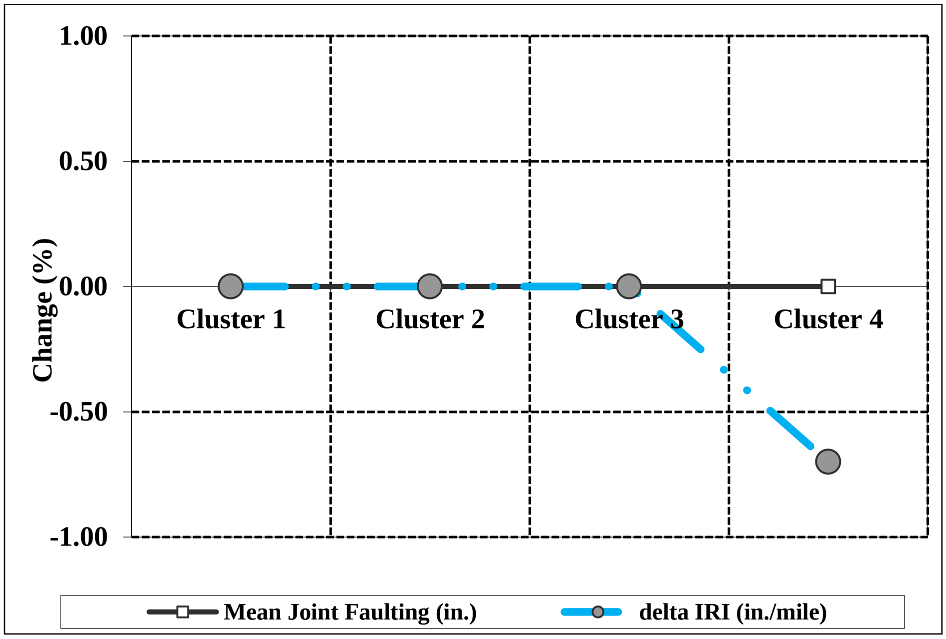


Figure 2.47: Change in Distress Values for Different HDF Clusters, 2009 – Rigid

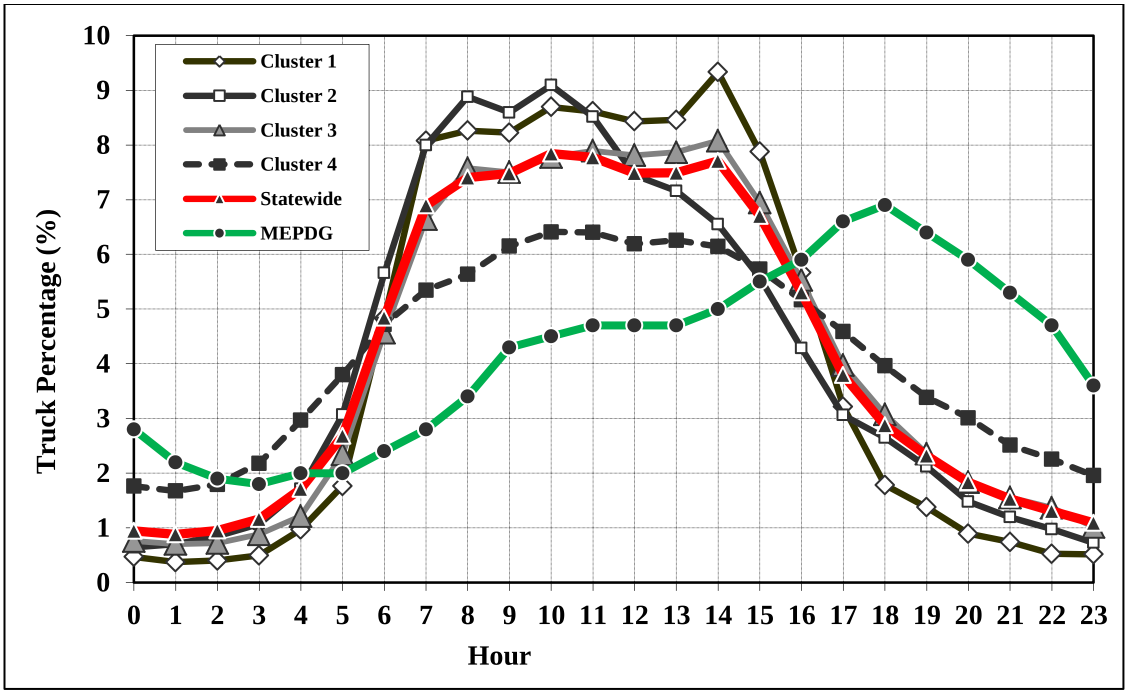


Figure 2.48: HDF Clusters (2011)

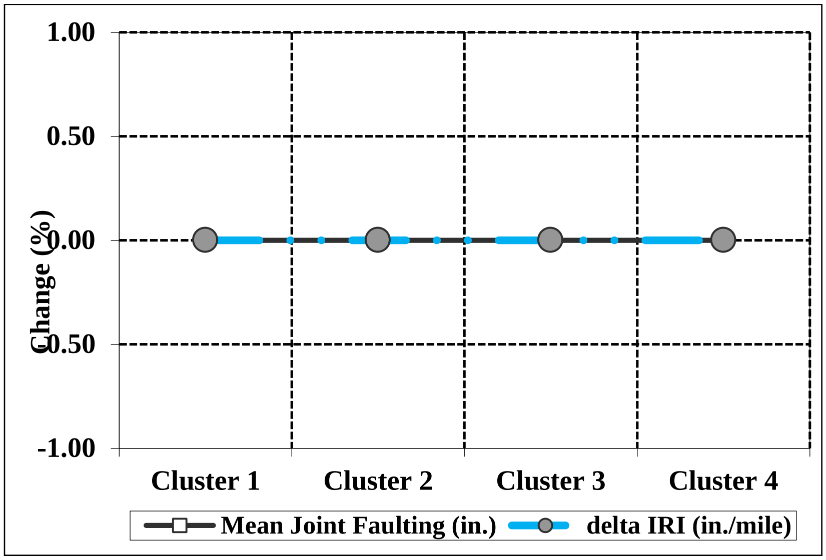


Figure 2.49: Change in Distress Values for Different HDF Clusters, 2011 – Rigid

Table 2.10: Predicted Distress Values for Statewide Average HDFs for Different Years

Predicted Distress	Years					Mean	Standard Deviation	COV
	2007	2008	2009	2010	2011			
Mean Joint Faulting (in.)	0.003	0.003	0.003	0.003	0.003	0.003	0.000	0.0%
Delta IRI	57.2	57.2	57.2	57.2	57.2	57.2	0.000	0.0%

2.5.4 Axle Groups per Vehicle (AGPV)

MEPDG runs were carried out to study the effects of AGPV values for different WIM sites for different years. Twelve sites were considered for this analysis for 2007 and 2008 and 14 sites were considered for 2009 and 2011. The predicted distress values show less variation for site specific AGPV values than for the statewide values for flexible pavements (Figures 2.50, 2.52, 2.54, and 2.56). No significant change in distresses for the MEPDG runs for rigid pavements except for Site 199 for 2007 (Figures 2.51, 2.53, 2.55, and 2.57). But the distress value for mean joint faulting for this site is only 0.007 inch and this site is no longer used by NYSDOT. Therefore, statewide average AGPV values are recommended for the design of pavement structures.

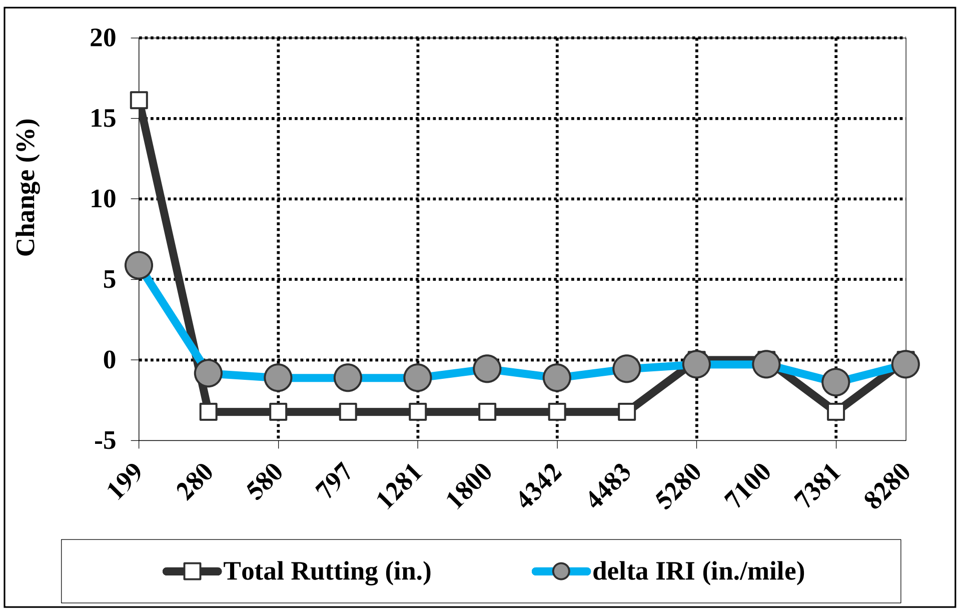


Figure 2.50: Change in Distress Values due to Variation of AGPV, 2007 – Flexible

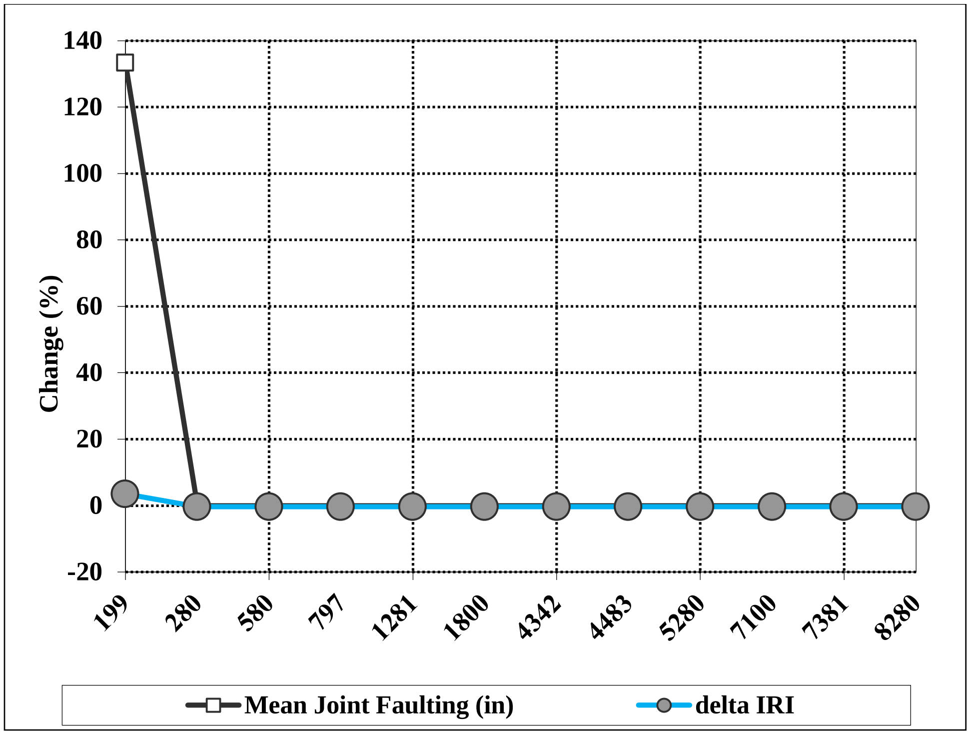


Figure 2.51: Change in Distress Values due to Variation of AGPV, 2007 – Rigid

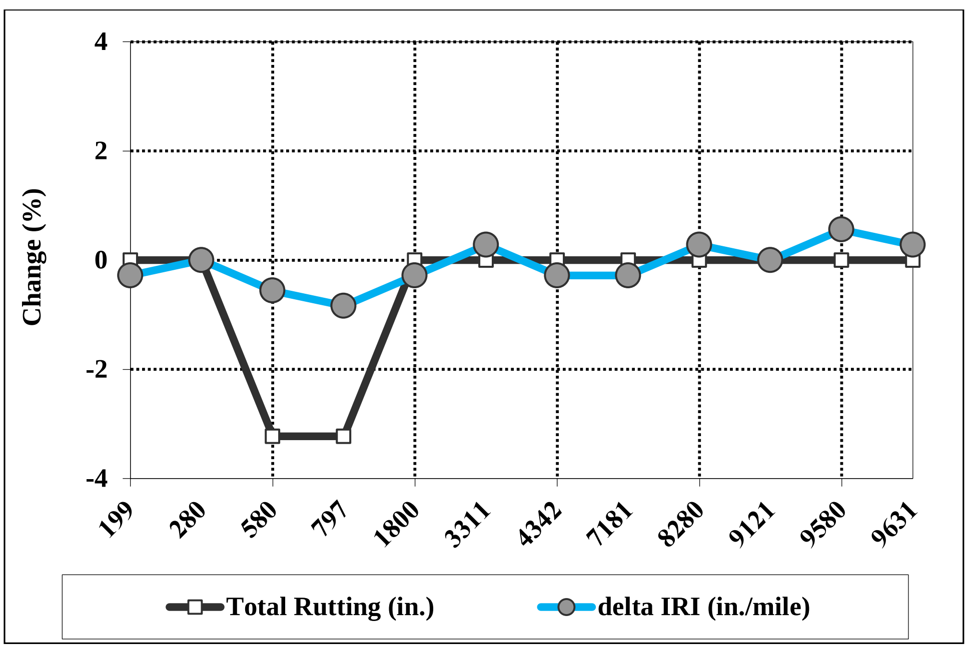


Figure 2.52: Change in Distress Values due to Variation of AGPV, 2008 – Flexible

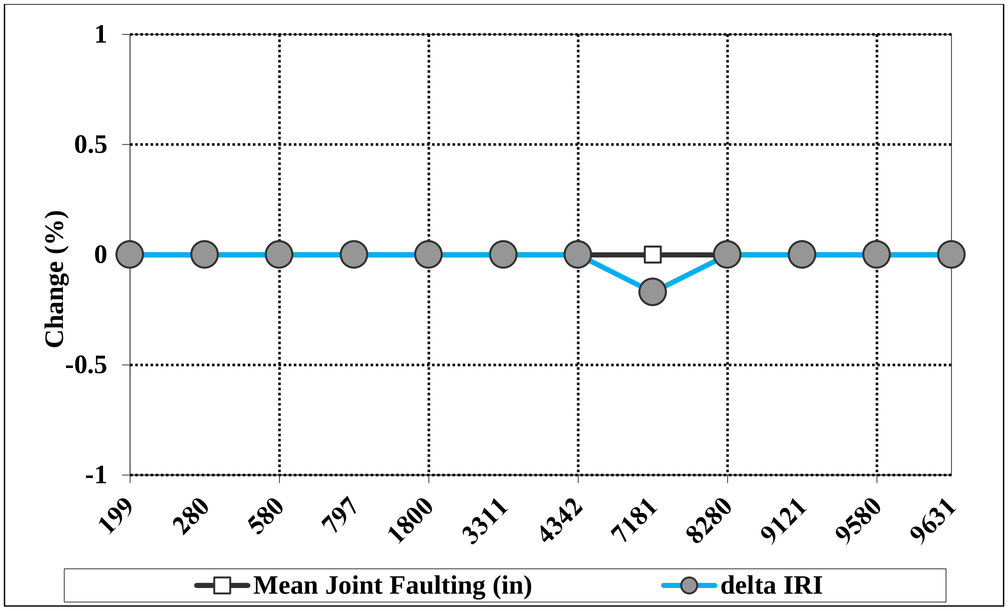


Figure 2.53: Change in Distress Values due to Variation of AGPV, 2008 – Rigid

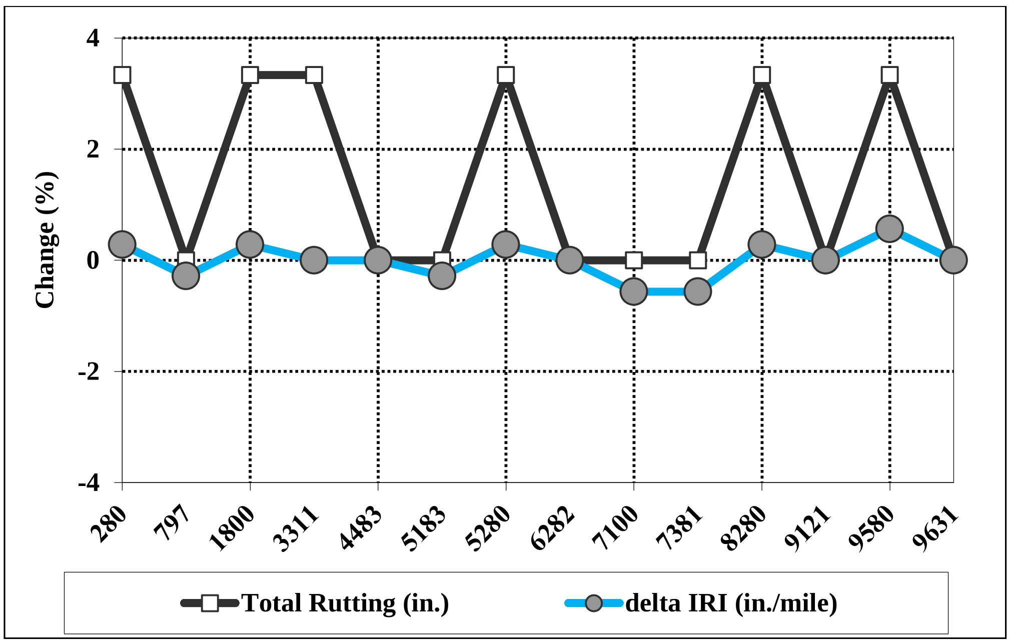


Figure 2.54: Change in Distress Values due to Variation of AGPV, 2009 – Flexible

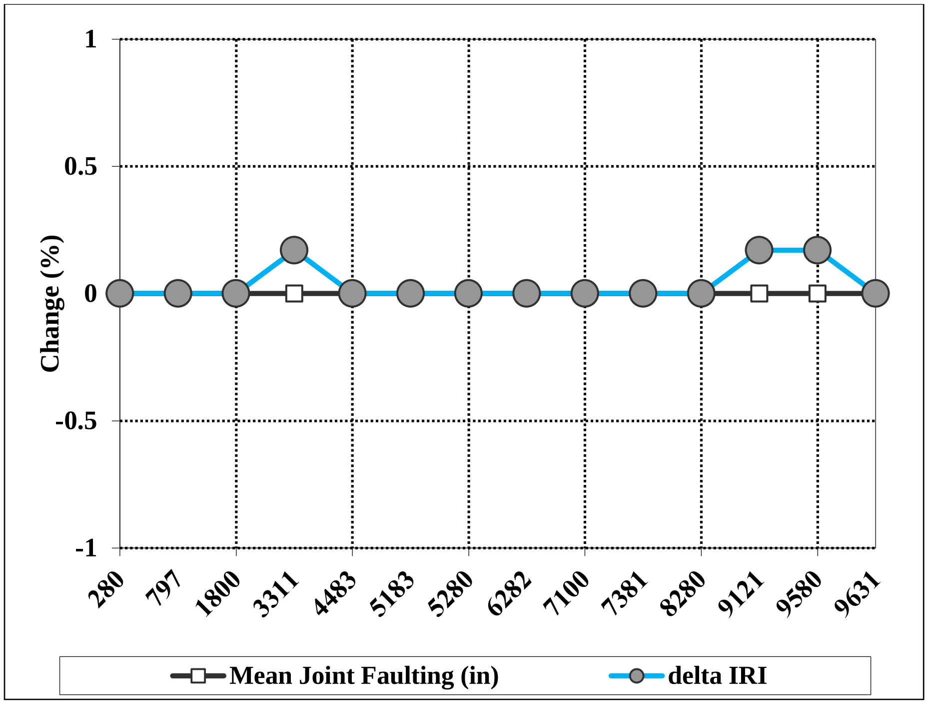


Figure 2.55: Change in Distress Values due to Variation of AGPV, 2009 – Rigid

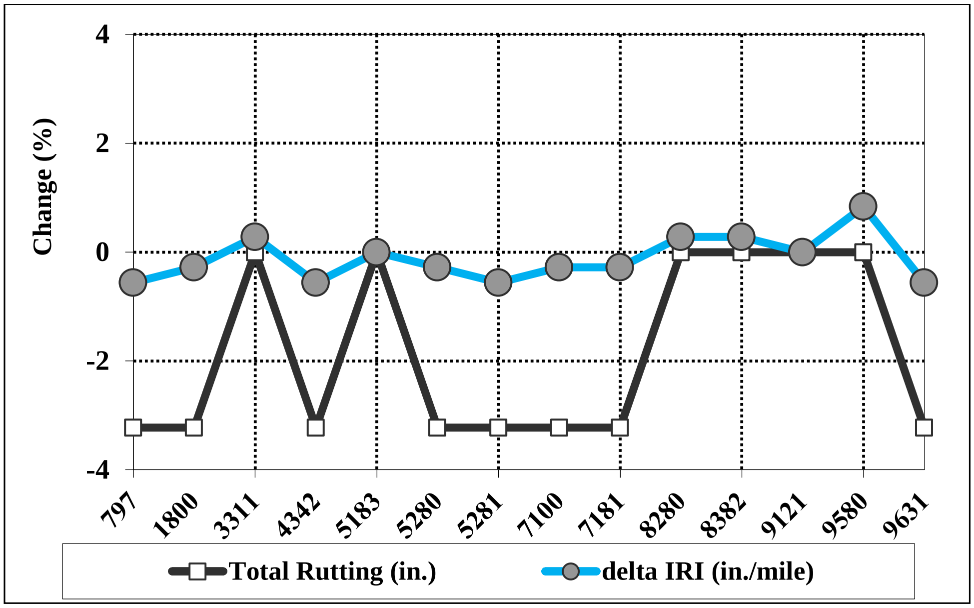


Figure 2.56: Change in Distress Values due to Variation of AGPV, 2011 – Flexible

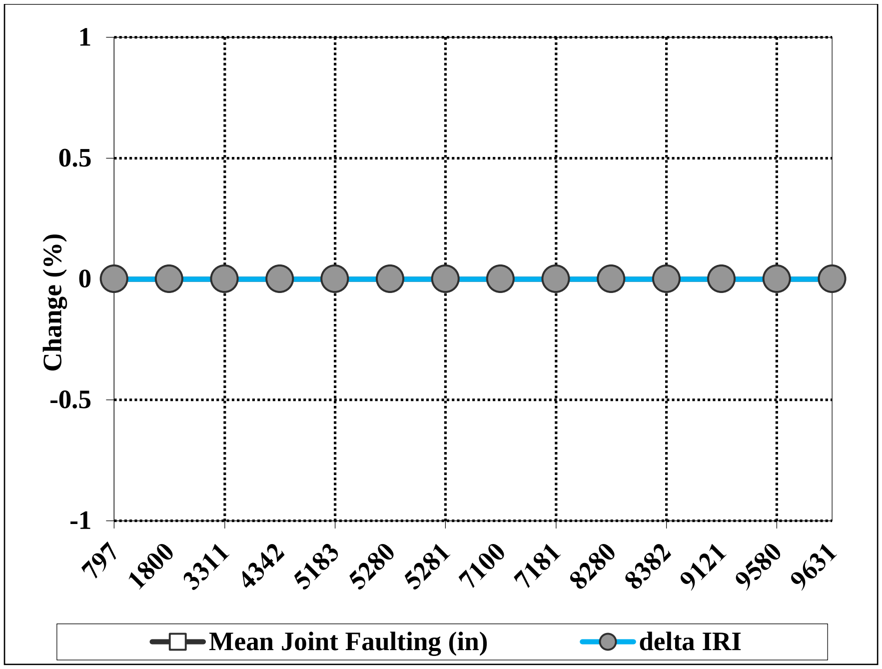


Figure 2.57: Change in Distress Values due to Variation of AGPV, 2011 – Rigid

Statewide average AGPV values for different years are given in Tables 2.11 to 2.14. The statewide average AGPVs were compared in terms of pavement performance (Table 2.15). The predicted distress values of total rutting and delta IRI for the statewide average AGPV values have the coefficients of variation of 1.5% and 0.2%, respectively. The predicted mean joint faulting was the same when statewide AGPV values from different years were used. However, the coefficient of variation for delta IRI is the same (0.2%) for both flexible and rigid pavements over the study period. Therefore, changing the statewide average AGPV values from one year to another has small effect on the predicted distresses.

Table 2.11: Statewide Average AGPV Values (2007)

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.46	0.71	0.00	0.00
5	2.16	0.02	0.00	0.00
6	1.07	1.07	0.01	0.00
7	1.30	0.26	0.73	0.09
8	2.52	0.70	0.02	0.01
9	1.34	2.04	0.00	0.00
10	1.11	1.05	1.06	0.06
11	3.38	0.29	0.35	0.09
12	3.66	1.29	0.09	0.00
13	1.88	0.87	0.47	0.30

Table 2.12: Statewide Average AGPV Values (2008)

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.34	0.67	0.00	0.00
5	2.00	0.01	0.00	0.00
6	1.00	1.00	0.00	0.00
7	1.17	0.19	0.75	0.06
8	2.33	0.67	0.01	0.01
9	1.20	1.90	0.00	0.00
10	1.06	1.01	0.95	0.04
11	3.68	0.25	0.24	0.02
12	3.76	1.09	0.01	0.00
13	1.81	1.41	0.26	0.51

Table 2.13: Statewide Average AGPV Values (2009)

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.27	0.73	0.00	0.00
5	2.00	0.01	0.00	0.00
6	1.00	1.00	0.00	0.00
7	1.30	0.28	0.67	0.04
8	2.44	0.61	0.01	0.00
9	1.24	1.88	0.00	0.00
10	1.08	0.99	0.95	0.04
11	3.77	0.22	0.25	0.01
12	4.00	0.98	0.00	0.01
13	1.99	0.83	0.29	0.31

Table 2.14: Statewide Average AGPV Values (2011)

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.30	0.71	0.00	0.00
5	2.00	0.01	0.00	0.00
6	0.99	0.99	0.01	0.00
7	1.35	0.30	0.62	0.05
8	2.40	0.64	0.01	0.00
9	1.22	1.89	0.00	0.00
10	1.09	0.99	0.95	0.05
11	3.74	0.31	0.21	0.01
12	3.74	1.06	0.03	0.01
13	2.13	0.86	0.37	0.31

Table 2.15: Predicted Distress Values for Statewide Average AGPV Values for Different Years

Pavement Type	Predicted Distresses	Years					Mean	Standard Deviation	COV
		2007	2008	2009	2010	2011			
Flexible	Total Rutting (in.)	0.31	0.31	0.30	0.31	0.31	0.31	0.004	1.5%
	Delta IRI	35.8	35.8	35.6	35.7	35.7	35.7	0.084	0.2%
Rigid	Mean Joint Faulting (in.)	0.003	0.003	0.003	0.003	0.003	0.003	0.000	0.0%
	Delta IRI	57.4	57.3	57.2	57.2	57.2	57.3	0.089	0.2%

2.5.5 Axle Load Spectra

MEPDG runs were carried out to study the effects of axle load spectra for different WIM sites in different years. Twelve sites were considered for this analysis for each of the years. The predicted distress values for total rutting show more variation than for delta IRI. However, the predicted distress values show little variation for site-specific axle load spectra than for the statewide average axle load spectra with a few exceptions (Figures 2.58, 2.60, 2.62, and 2.64). Site 280 shows high distress values for 2008 but this site is no longer used by NYSDOT. Site 797 shows high distress values comparing to statewide average values for 2008 and 2009. This site recorded 0.6% of Class 13 vehicles and this may be the cause for such high distresses. Site 5280 experiences almost 70% of Class 9 vehicles, which may be the reason of high distress values for 2011.

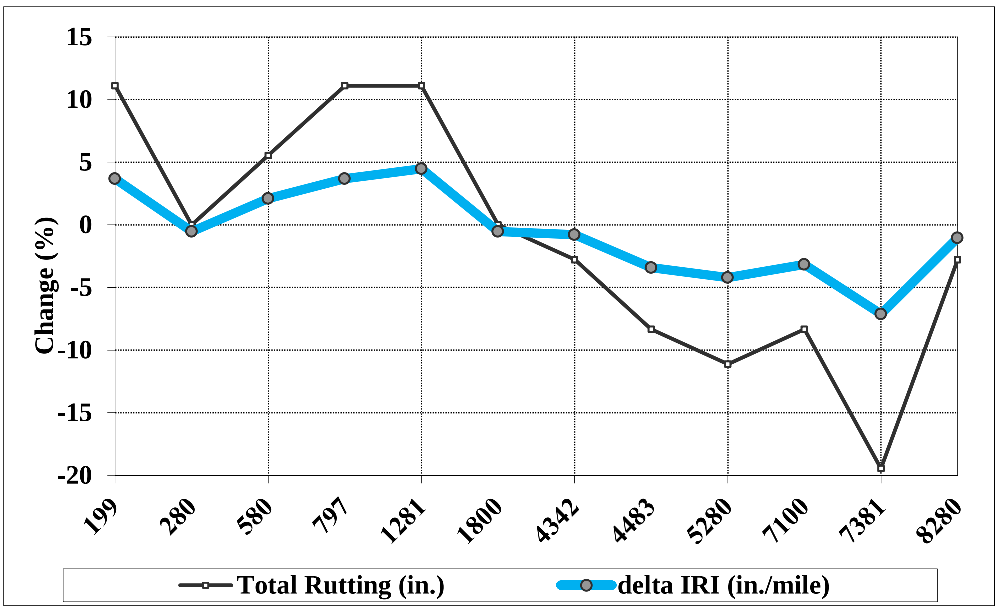


Figure 2.58: Change in Distress due to Variation of Axle Load Spectra, 2007 – Flexible

Site-specific distress values for the MEPDG runs for rigid pavements show significant variation for mean joint faulting comparing to the distress values for statewide average values in some cases (Figures 2.57, 2.61, 2.63, and 2.65). However, the actual distress values for mean joint faulting are small. For example, Site 199 shows a variation of 33.33% comparing to statewide average values but the predicted mean joint faulting is only 0.002 inch. Therefore, statewide average axle load spectra are recommended for the design of both rigid and flexible pavement structures.

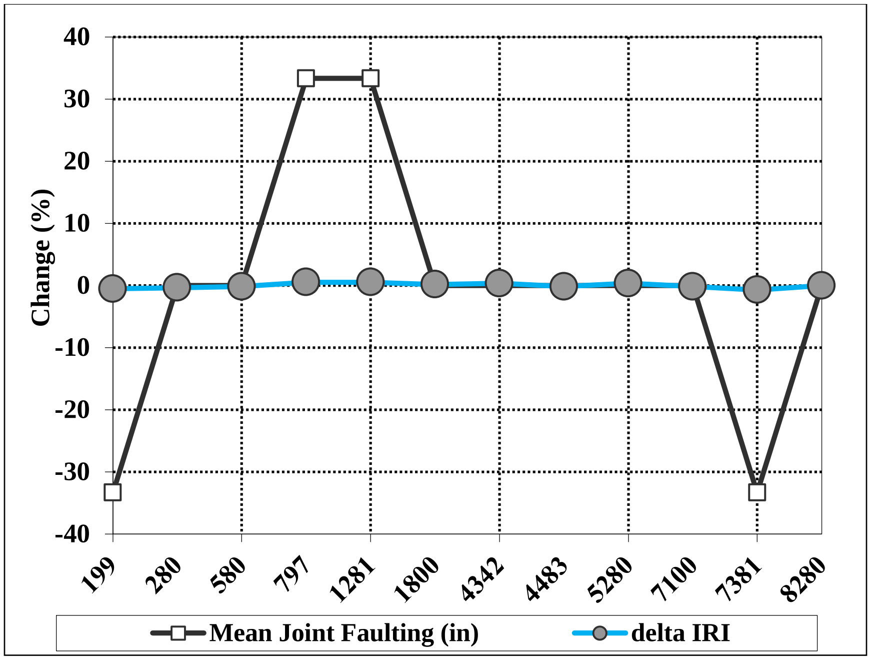


Figure 2.59: Change in Distress due to Variation of Axle Load Spectra, 2007 – Rigid

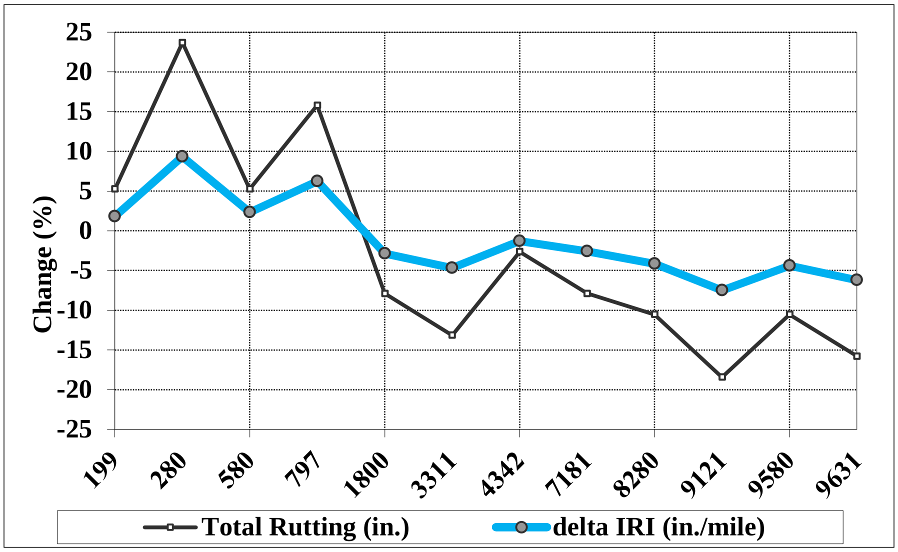


Figure 2.60: Change in Distress due to Variation of Axle Load Spectra, 2008 – Flexible

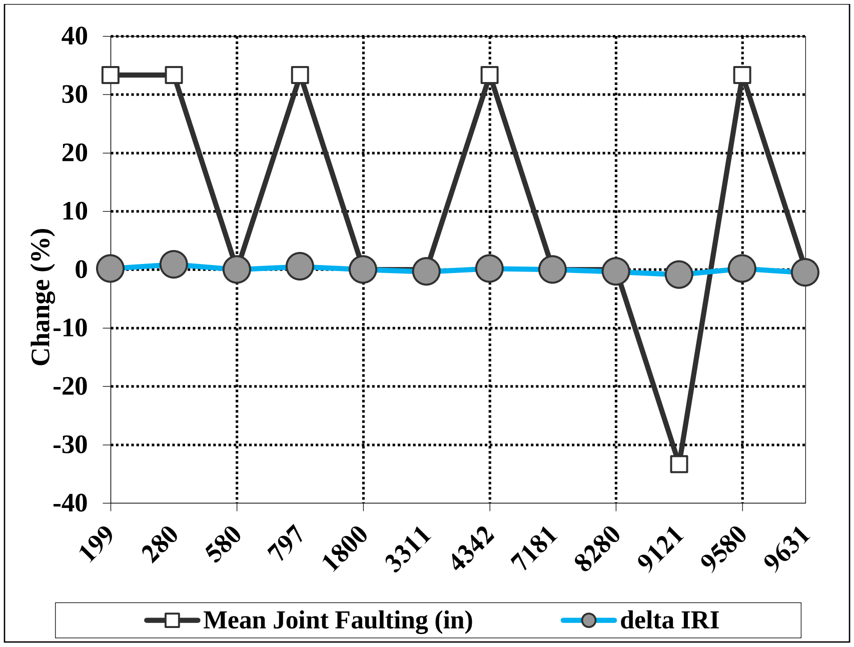


Figure 2.61: Change in Distress due to Variation of Axle Load Spectra, 2008 – Rigid

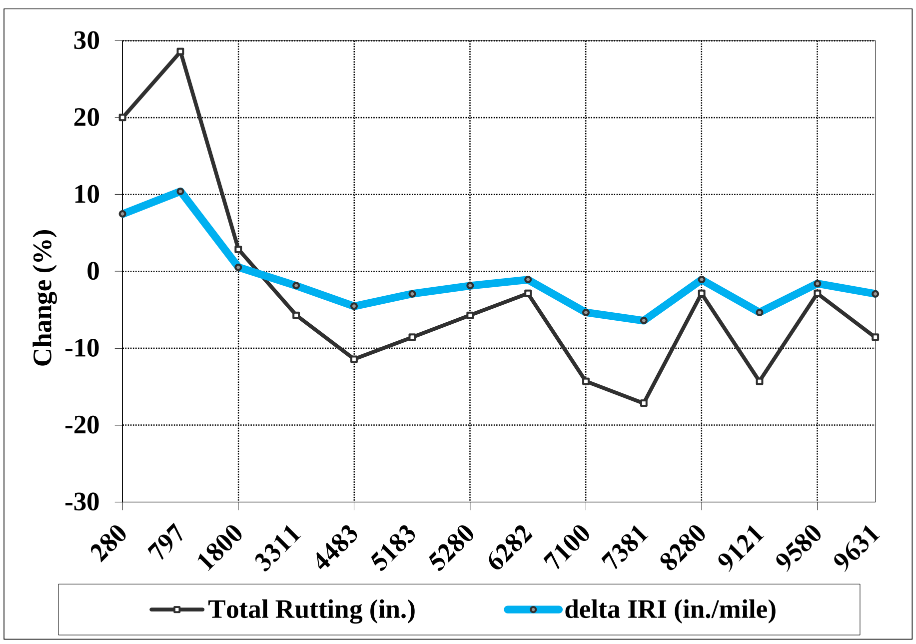


Figure 2.62: Change in Distress due to Variation of Axle Load Spectra, 2009 – Flexible

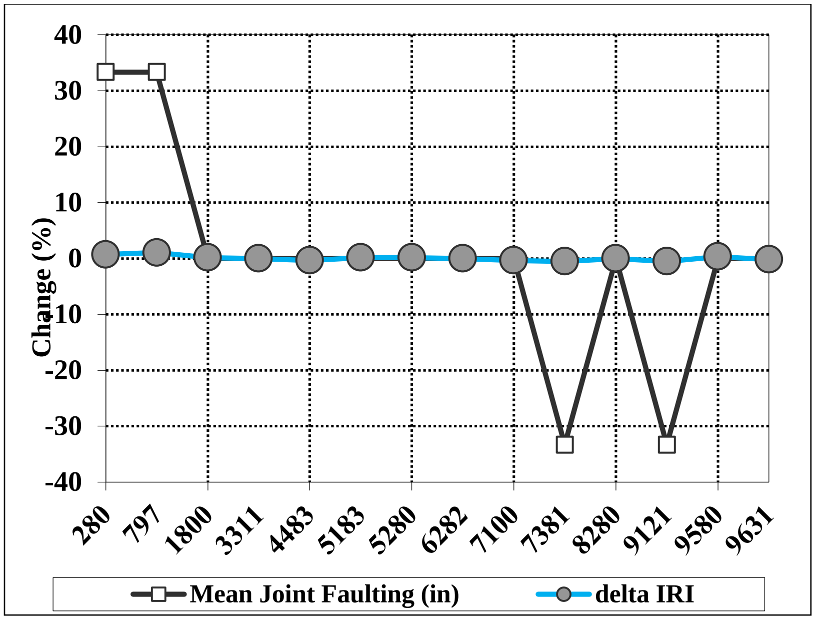


Figure 2.63: Change in Distress due to Variation of Axle Load Spectra, 2009 – Rigid

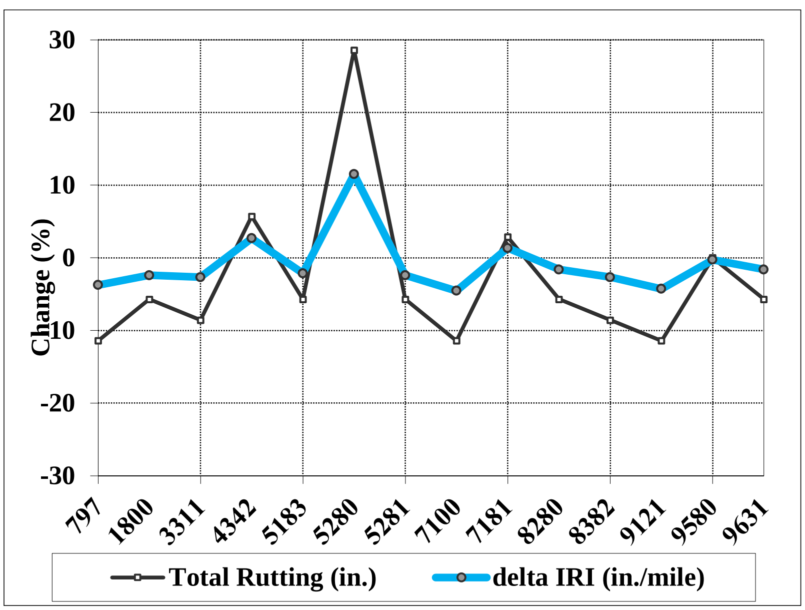


Figure 2.64: Change in Distress due to Variation of Axle Load Spectra, 2011 – Flexible

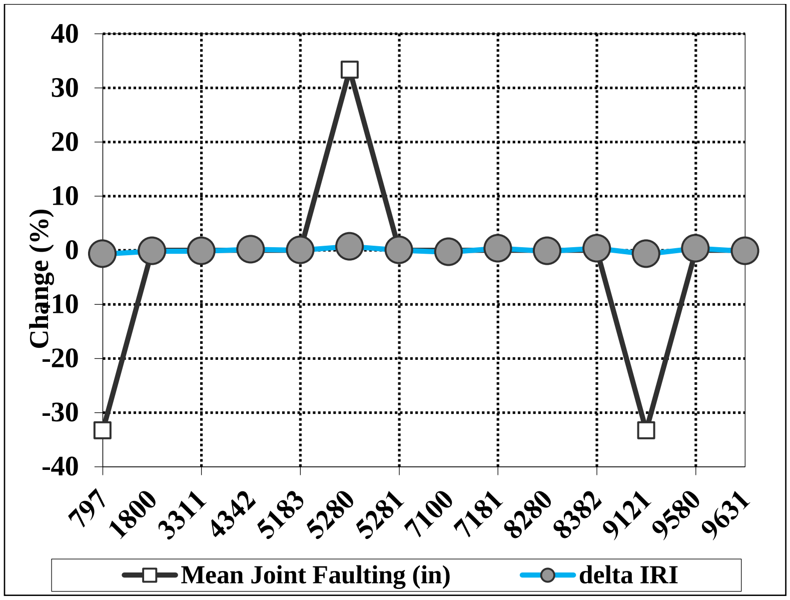


Figure 2.65: Change in Distress due to Variation of Axle Load Spectra, 2011 – Rigid

The statewide average axle load spectra were compared in terms of pavement performance (Table 2.16). The predicted distress values of total rutting and delta IRI for the statewide average axle load spectra have the coefficient of variation of 3.6% and 1.3%, respectively. Therefore, changing the statewide average axle load spectra from one year to another has small impact on the predicted pavement performance. Because of this, the statewide average values for VCD, MDF, AGPV, and axle load spectra recorded in 2010 are recommended to be used as traffic inputs to MEPDG and AASHTOWare Pavement ME Design. More classification and WIM stations with complete data were recorded in 2010 than for the other years.

Table 2.16: Predicted Distress Values for Statewide Average Axle Load Spectra for Different Years

Pavement Type	Predicted Distresses	Years					Mean	Standard Deviation	COV
		2007	2008	2009	2010	2011			
Flexible	Total Rutting (in.)	0.36	0.38	0.35	0.35	0.35	0.36	0.013	3.6%
	Delta IRI	37.9	38.6	37.5	37.5	37.4	37.8	0.497	1.3%
Rigid	Mean Joint Faulting (in.)	0.003	0.003	0.003	0.003	0.003	0.003	0.000	0.0%
	Delta IRI	57.4	57.6	57.4	57.4	57.4	57.4	0.089	0.2%

2.6 Conclusions and Recommendations from the Traffic Data Analysis

The proper implementation of MEPDG depends on the appropriate characterization of traffic data.

One of the efforts of this study was to characterize the traffic data and suggest the site-specific, regional, or state-wide average values for traffic inputs to MEPDG and AASHTOWare Pavement ME Design for New York State. Vehicle class distribution (VCD), monthly distribution factors (MDF), hourly distribution factors (HDF), average number of axle groups per vehicle (AGPV), and axle load spectra data were obtained from vehicle classification sites and WIM sites in New York State for the period of 2007–2011. These traffic data were processed with the TrafLoad software.

Cluster analysis was conducted on the VCD, MDF, and HDF data collected during the 5-year period. This statistical analysis could not be done for AGPV values and axle load spectra due to unavailability of data for a sufficient number of WIM sites over the time period. MEPDG runs were performed to study the effect of using site specific, cluster average, statewide average, and MEPDG default values on predicted distresses for typical new flexible and rigid pavement structures in New York State.

The main conclusions of the traffic data analysis are:

- Vehicle classification sites can be divided into three groups on the basis of one-way AADTT: low (0 to 299), medium (300 to 999), and high (>1000). The majority of the sites have low AADTT values.
- Four clusters are found for the vehicle classification distribution (VCD). They are differentiated on the basis of proportions of Class 5 and Class 9 vehicles. The direction of travel has little impact on the VCD. The results of cluster analysis are consistent during the entire period of analysis.
- Multidimensional clustering was adopted for monthly distribution factors (MDF) considering Class 5 and 9 vehicles simultaneously. Four clusters are found for 2007, 2008, and 2010. However, three and five clusters are found for 2009 and 2010, respectively.

- Four clusters were identified for the Hourly Distribution Factors (HDF) for each of the years. The results of cluster analysis are consistent over the years. HDF does not have any impact on the performances of flexible pavements since they are included only for the calculation of hourly traffic for the design of rigid pavements.
- Cluster analysis was not done on AGPV and axle load spectra due to limited availability of WIM data. MEPDG runs with AGPV and axle load spectra values recorded by individual WIM stations showed that the values lead to small changes of the predicted pavement distresses.
- Statewide average values are recommended for VCD, MDF, AGPV, and axle load spectra as traffic inputs to the MEPDG and AASHTOWare Pavement ME Design for both flexible and rigid pavements.
- The statewide average values do not show significant variation in terms of pavement performance over the years. The statewide average values for 2010 are recommended as the traffic inputs to MEPDG and AASHTOWare because the highest number of WIM sites are found for this year.

It is recommended that the analysis be repeated every 5 years to update the statewide average values to reflect any changes in traffic patterns and volumes. Moreover, since the top-down cracking models will be replaced in the future version of the software, further analysis should be carried out when the new versions of the AASHTO Pavement ME Design program with the new cracking models are available.

Chapter 3: Enhancing the Performance Models of AASHTOWare Pavement ME Design

3.1 Overall Concept for Enhancing the Performance Models

This chapter explains the procedure used to calibrate the performance models of AASHTOWare Pavement ME Design 2.1. The work of Momin (2011) was used as the primary source to assemble the required data. Then, the assembled data were used to calibrate the performance models so the bias between the predicted and the measured performance data was eliminated or reduced. The calibration was performed following the steps recommended in the AASHTO ME Local Calibration Guide developed under NCHRP Project 1-40B.

It is important to note that the local calibration deals only with the calibrated coefficients and exponents of the distresses models. The local calibration cannot change the form of the mathematical functions in the performance models.

3.2 Data Assembly

The process of assembling the essential data for calibration was done in two stages. The first stage evaluated the available data in the Pavement Management System (PMS) database of NYSDOT to determine if such data can be used for the local calibration (NYSDOT, 2002). It was found that complete calibration data were not available even for a single flexible pavement section. Therefore, NYSDOT PMS data was no longer used.

Then the Long-Term Pavement Performance Database (LTPP) was reviewed to assure the following data are available for new flexible pavements in Northeastern (NE) region of the United States:

- Traffic Data,
- Structural Data and Materials Properties Data,
- Climatic Data, and
- Distress Data.

Unfortunately, no GPS 1 and GPS 2 pavements sections were built in the New York State as part of the LTPP program (Abdullah, Romanoschi, Bendana, & Nyamuhokya, 2014). GPS pavement sections are monitored pavement sections that had been built 15 years prior to the implementation of the LTPP program (Elkins, Schmalzer, Thompson, & Simpson, 2003). GPS 1 sections are flexible pavements with unbound granular base, while GPS 2 are flexible pavement sections with bound base layers.

Since no complete calibration data was available for new flexible pavement structures in New York State, it was decided to use collected data from the LTPP program on flexible pavement sections in the neighboring states. This approach is reasonable since the states in the NE region of the United States have similar climatic conditions and use similar pavement structural configurations and materials in the construction of new flexible pavements.

It is important to mention that Momin (2011) conducted the regional calibration of the distress models embedded in MEPDG 1.1 by using data for 18 LTPP flexible pavement sites in the NE region of the United States. Momin performed an extensive effort to assemble the needed data and create input files for the MEPDG 1.1 software. His effort was considered for this work; the extracted data from Momin's effort are tabulated in the following appendices:

- Appendix B: Extracted Long-Term Pavement Performance (LTPP) Traffic Design Inputs
- Appendix C: Extracted Long-Term Pavement Performance (LTPP) Structural and Materials Properties Design
- Appendix D: Extracted Long-Term Pavement Performance (LTPP) Performance Data

3.2.1 Selection of the LTPP Sites

Twenty-nine LTPP monitored flexible pavement sections in the NE region of the United States have very similar conditions as the flexible pavements sections in New York State. Only 18 LTPP flexible pavement sections have the complete required data for calibration (Momin, 2011). Table 3.1 lists the LTPP pavement sections used in this research for the calibration of the AASHTO Pavement ME models.

Table 3.1: Selected LTPP Pavement Sections in Northeast

State Code	State	SHRP ID	Total Lanes	Structural Type	Construction Date 1	Construction Date 2
9	Connecticut	1803	2	Flexible	7/1/1988	1/17/1995
23	Maine	1001	4	Flexible	7/1/1988	6/6/1995
23	Maine	1009	2	Flexible	7/1/1988	8/22/1993
23	Maine	1028	2	Flexible	7/1/1988	5/12/1992
25	Massachusetts	1003	2	Flexible	6/1/1988	6/7/1988
34	New Jersey	1003	4	Flexible	8/1/1988	4/8/1994
34	New Jersey	1011	4	Flexible	7/1/1988	4/28/1998
34	New Jersey	1030	4	Flexible	12/1/1988	2/24/1991
34	New Jersey	1031	4	Flexible	7/1/1988	4/4/1996
34	New Jersey	1033	4	Flexible	7/1/1988	9/11/1997
34	New Jersey	1034	4	Flexible	12/1/1988	-
34	New Jersey	1638	4	Flexible	12/1/1988	-
42	Pennsylvania	1597	2	Flexible	8/1/1988	6/12/1990
42	Pennsylvania	1599	2	Flexible	8/1/1988	6/1/1999
50	Vermont	1002	2	Flexible	8/1/1988	-
50	Vermont	1004	2	Flexible	8/1/1988	10/6/1998
50	Vermont	1681	2	Flexible	6/1/1989	9/8/1991
50	Vermont	1683	2	Flexible	6/1/1989	9/23/1991
Sections with Missing Traffic Data and Unreliable Performance Data						
23	Maine	1012	4	Flexible	7/1/1988	-
23	Maine	1026	2	Flexible	7/1/1988	9/26/1996
25	Massachusetts	1002	6	Flexible	6/1/1988	6/5/1988
25	Massachusetts	1004	4	Flexible	8/1/1988	6/1/2001
33	New Hampshire	1001	4	Flexible	8/1/1988	8/1/2001
36	New York	1008	4	Flexible	5/1/1989	8/25/1989
36	New York	1011	4	Flexible	6/1/1988	9/14/1993
36	New York	1643	2	Flexible	5/1/1989	10/12/1989
36	New York	1644	2	Flexible	5/1/1989	6/19/1996
42	Pennsylvania	1605	2	Flexible	8/1/1988	6/14/1995
42	Pennsylvania	1618	2	Flexible	12/1/1988	8/27/1989

3.2.2 Traffic Data Assembly

Traffic data are necessary to run the AASHTOWare Pavement ME Design 2.1 so that the traffic loads during the design life can be used for distresses prediction. The traffic inputs of the AASHTOWare are summarized in Figure 3.1.

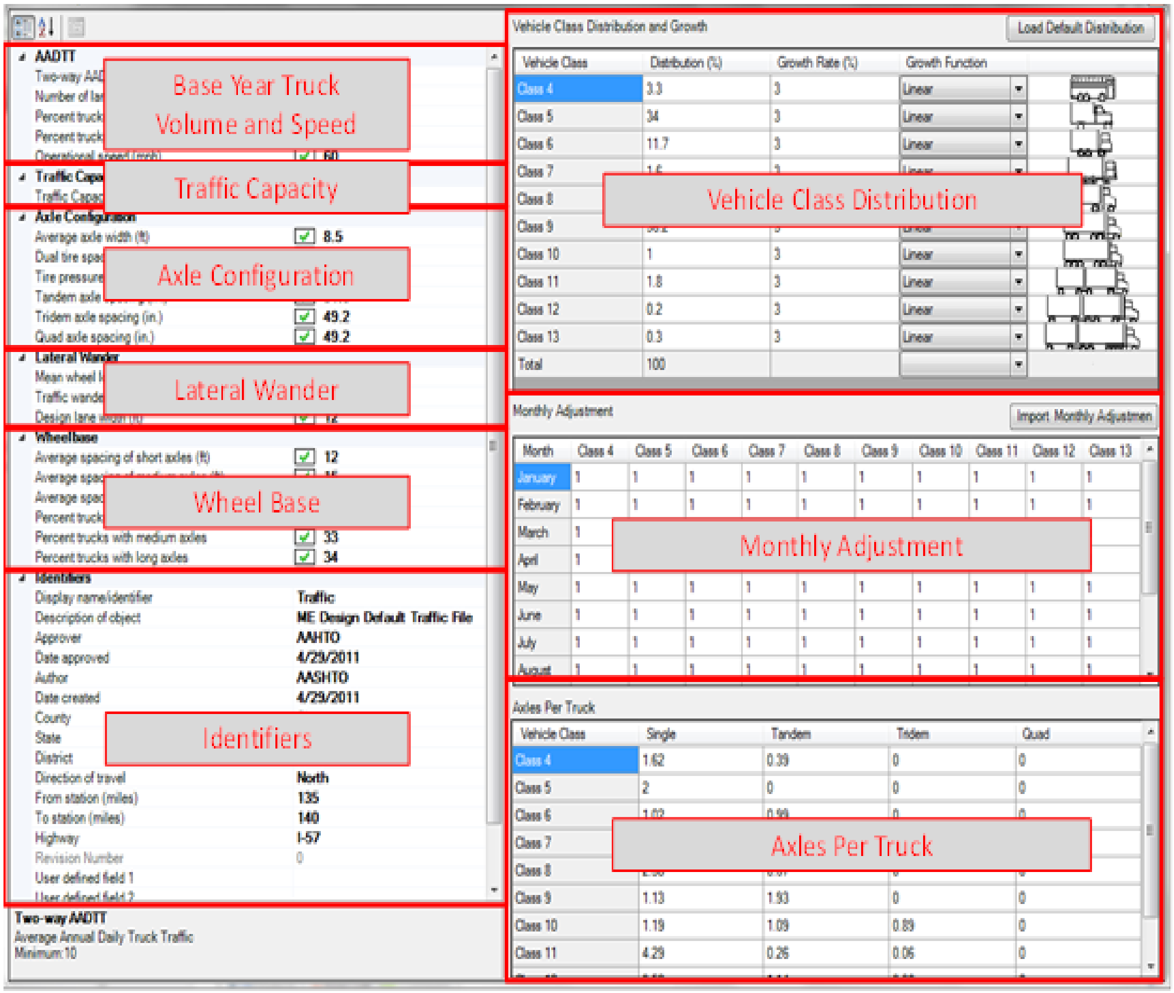


Figure 3.1: AASHTOWare Pavement ME Design 2.1 Traffic Inputs

For each selected LTPP site, the required traffic data during the base year were extracted from the traffic data tables assembled by Momin (2011). The extracted traffic data are tabulated in Appendix B. The traffic inputs are:

- *Average Annual Daily Truck Traffic (AADTT)*: It is the total volume of truck traffic recorded on a highway segment during an entire year, divided by the number of days in the year.
- *Vehicle Class Distribution (VCD):* It represents the percentage of each truck class (Class 4 to Class 13) in the total number of trucks. The Federal Highway Administration (FHWA) has classified the vehicles into 13 classes, out of which nine are truck classes, as illustrated in Figure 3.2.

- *Monthly Adjustment Factor (MAF):* Monthly adjustment factors represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month. The monthly distribution factor for a vehicle class, for a specific month, is computed by dividing for the monthly truck traffic from that class divided by the total truck traffic for the entire year (AASHTO, 2008).
- *Number of Axles per Truck:* It indicates the average number of axles for each truck class and for each axle type (Single, Tandem, Tridem, and Quad).
- *Axle Load Spectra:* It represents the axle load distribution for each axle type, for each month of the year and each vehicle class. It is the percentage of the total axle application within specified load intervals with respect to the axle type and vehicle class.
- *Growth Rate and Function:* The growth rate represents the annual rate of truck traffic growth over time in the exponential growth model. The extracted growth rate for each of the 18 LTPP selected sections, computed by Momin (2011), from the recorded truck traffic during the entire monitoring period is given in Table 3.2. AASHTOWare uses the same growth rate for all vehicle classes.

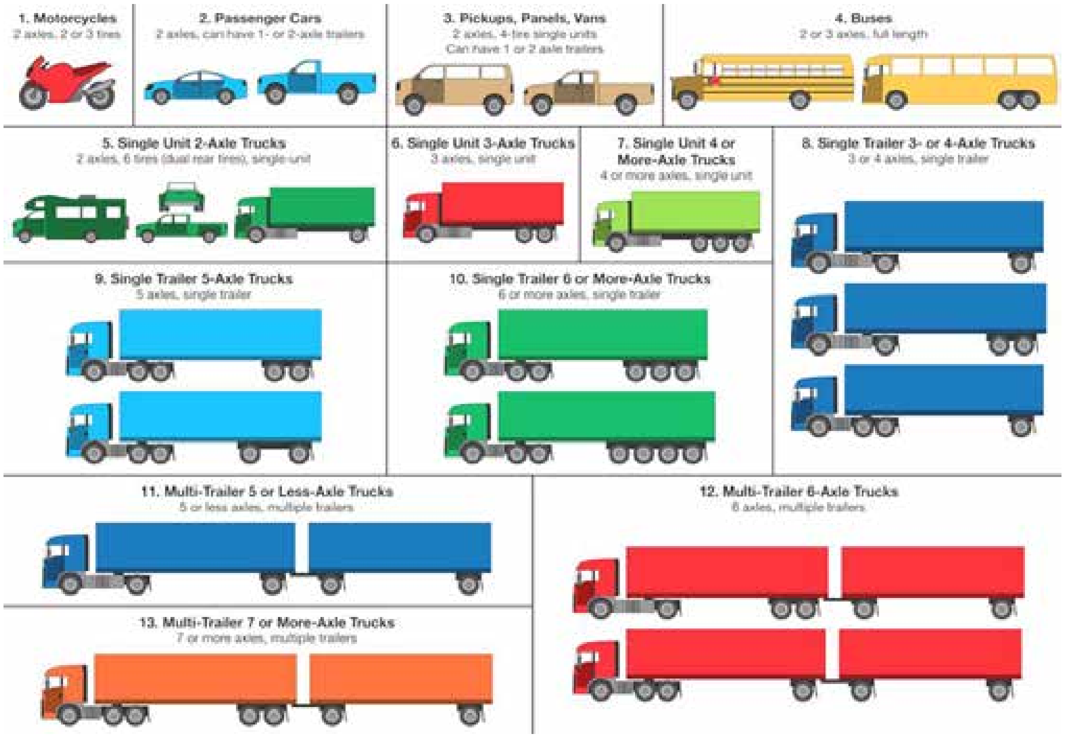


Figure 3.2: FHWA Vehicle Classification

Table 3.2: Exponential Traffic Growth Rate for the Selected LTPP Sections

SHRP ID	Traffic Growth Rate %	SHRP ID	Traffic Growth Rate %
091803	6.57	341033	-21.86
231001	1.15	341034	-0.83
231009	0.49	341638	-0.92
231028	5.9	421597	4.68
251003	-1.09	421599	-1.39
341003	-14.79	501002	3.33
341011	-6.5	501004	1.91
341030	0	501681	17.5
341031	9.59	501683	17.5

Additional traffic inputs are required by the AASHTOWare. Those traffic inputs are not readily available, so the default values suggested by the AASHTOWare software are normally used instead. These default values are Level 3 design inputs and are defined as:

- *Hourly Adjustment Factors*: It represents the ratio of the truck traffic in a given hour of the day divided by the total daily truck traffic. To reduce the computation time, it is not used for the design of flexible pavements.

- *Traffic Capacity:* Traffic capacity is an optional setting which allows a cap on forecasted traffic volume based on ME Design's internal capacity calculations which use the models included in the Highway Capacity Manual (HCM) 2000.
- *Axle Configuration*: It defines the average axle width, axle spacing, dual tire spacing, and tire inflation pressure.
- *Lateral Wander*: It includes the mean wheel location, traffic wander standard deviation, and design lane width.
- *Wheel Base*: It includes the average spacing of short, medium, and long axles. In addition, it includes the percentage of trucks having this axle spacing.
- *Identifiers:* It includes the source description of the traffic inputs.

3.2.3 Structural Layers and Materials Properties Data Assembly

Since the calibration of performance models relies on the runs of the AASHTOWare software for the LTPP pavement sections selected, it is imperative that the structural configuration and material properties of the LTPP sections are used in the runs. Therefore, the required inputs were extracted to be used in the design problems for the sections listed in Table 3.1. The inputs not available in the LTPP database were replaced with AASHTOWare default values. For example, default values were used for indirect tensile strength, reference temperature, creep compliance, etc. The extracted data from the LTPP database are:

- *Layer Thickness:* The database of the selected LTPP sites contains adequate information regarding the number of the layers, type of the materials, and the thicknesses. Therefore, they were extracted to create AASHTOWare design problems for the selected LTPP sites. The extracted structural data are tabulated in Appendix C.
- *Gradation Data of Aggregate in the HMA*: Since the LTPP database does not contain dynamic modulus data for HMA mixes, as required for Level 1 input in AASHTOWare software, only Level 3 input values (aggregate

gradation, binder grade, mix volumetric properties) could be extracted from the LTPP database. For Level 3 inputs, aggregate gradation data is used in the Witczak model to compute the dynamic modulus (E*) of the HMA layers. Figure 3.3 shows an example of the extracted aggregate gradation data for an HMA mix.

- *Penetration/Viscosity Grade for Asphalt Binders*: Since the LTPP database contains only viscosity or penetration grades for asphalt binders and not actual viscosity or penetration values, as required for Level 2 design, the viscosity grade of each asphalt mixture was obtained from the LTPP database. The AASHTOWare assigns default values for the viscosity of the asphalt binders if the viscosity or penetration grades are selected. Figure 3.4 shows the screen capture for Level 3 inputs for asphalt binder. The extracted viscosity grades of selected LTPP sites are listed in Table 3.3.

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	78
No.4 sieve	52
No.200 sieve	5

Figure 3.3: Aggregate Gradation of Surface HMA layer

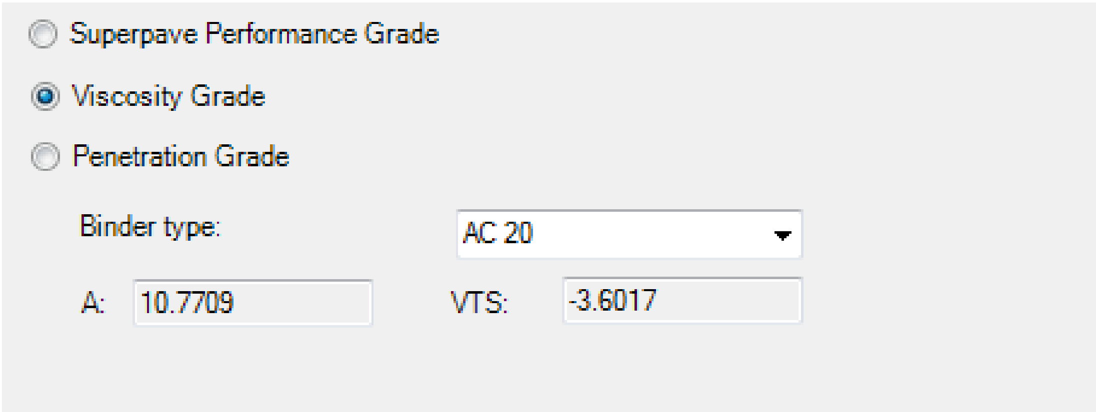


Figure 3.4: Level 3 Design Input Binder Grade

Table 3.3: Viscosity Grades for the Selected LTPP Sites

LTPP Site	Viscosity Grade
091803	AC-20
231001	AC-10
231009	Pen 85-100
231028	AC-10
251003	AC-20
341003	AC-20
341011	Pen 85-100
341030	AC-20
341031	AC-20
341033	AC-20
341034	AC-20
341638	AC-20
421597	AC-20
421599	AC-20
501002	Pen 85-100
501004	Pen 85-100
501681	Pen 85-100
501683	Pen 85-100

- *HMA Volumetric Properties*: AASHTOWare Pavement ME Design 2.1 defines the volumetric properties of the HMA mixture based on the following inputs:
  - Effective binder content (%)
  - Air voids (%)
  - Unit weight
  - Poisson’s ratio

  The effective binder content and air voids were extracted by Momin (2011) for each selected LTPP site. The values are tabulated in Appendix C. Since no data was found for the unit weight and the Poisson’s ratio of asphalt concrete, Level 3 design inputs were used.
- *Unbounded Layers Properties*: Selected LTPP sites have limited data for the unbounded layers. For this reason, Level 3 design inputs were used to cover the missing data. Only 10 sites have records about the base/subbase layers. The extracted data for the base/subbase layers are listed in Table 3.4. It is important to mention that the resilient modulus values recommended by AASHTO were used for the base and subbase layers; these values depend on the AASHTO classification of the soil (AASHTO, 2008).
- *Subgrade Soil Type and Properties*: The LTPP sites have adequate records regarding the soil types. However, there are no available gradation data. Therefore, Level 3 soil gradation data were used to substitute for the missing data. It was noticed that the LTPP Site #091803 has no information regarding the subgrade soil type. Therefore, it was assumed to be A-4 because it is the predominated soil in Connecticut (Malla & Joshi, 2006). The extracted data are listed in Table 3.5.

Table 3.4: Extracted Data of the Base/Subbase Layers for the Selected LTPP Sites

LTPP Site	Construction #	Layer #	AASHTO Soil Classification	Plasticity Index	Max Dry Density in Lab	Optimum Moisture Content in the Lab	In Situ Dry Density (Mean)	In Situ Moisture Content (Mean)
231001	1	3	A-1-a		139	6.1		
251003	1	2	A-1-a		125	8.4		
231009	1	2	A-1-b	1	133	10	126	3
231009	1	3	A-1-a		139	7.9	139	3
231028	1	2	A-1-a		142	6.2	141	4
231028	1	3	A-1-a		143	7.4	137	3
341031	1	2	A-1-a					7
341033	1	2	A-1-a					5
091803	1	2	A-1-a		137	7.6	138	5

Table 3.5: Subgrade Soil Type and Properties for Selected LTPP Site

LTPP Site	Construction #	Layer #	AASHTO Soil Classification	CBR	Plasticity Index	Liquid Limit	Max Dry Density in Lab	Optimum Moisture Content in the Lab	In Situ Dry Density (Mean)	In Situ Moisture Content (Mean)
501002	1	1	A-7-6							
251003	1	1	A-2-4	10			114	12	106	
501004	1	1	A-6		0	0	112	12.6	102	82.1
231009	1	1	A-4							
231028	1	1	A-1-a		0	0	128	8.5		
501681	1	1	A-1-a		3	18				
501683	1	1	A-1-a		11	26				
091803	1	1	A-4				122	12.4		118.2
341003	1	1	A-7-6							
341011	1	1	A-7-6							
341030	1	1	A-4							
341031	1	1	A-7-6							
341033	1	1	A-2-4							
341034	1	1	A-1-a							
341638	1	1	A-1-b							
421597	1	1	A-7-5							
421599	1	1	A-7-5							

3.2.4 Selection of the Climatic Stations

The LTPP sites were monitored from 1986 to 1996. However, the stored climatic files in the AASHTOWare Pavement ME Design 2.1 contain the recorded weather data from 1996 to 2006. Momin (2011) generated the MEPDG 1.1 climatic files from 1986 to 1996 for the selected LTPP sites. The format of the generated climatic files is the same as the format of climatic files stored in the AASHTOWare. Therefore, the generated files by Momin were used in this study.

The AASHTOWare Pavement ME has two options for creating the climatic file for a given design project: (1) by selecting the closest weather station to the design project that is already included in the AASHTOWare software database, or (2) by creating a virtual weather station through interpolation of the data collected for up to five weather stations near the location of the design project. The inputs required for creating the climatic file are the latitude, longitude, elevation, and water table depth.

For all 18 LTPP pavement sections selected for local calibration in this study, the AASHTOWare software contains climatic data only from 1996 (partial) to 2006 (partial); the climatic data required for 1985 to 1996 when the 18 LTPP pavement sections were monitored is not available. The LTPP database contains climatic data for the 1985 to 1996 period, but only daily and monthly average values and not hourly data, as required by AASHTOWare Pavement ME. Therefore, the assembly of climatic data for the local calibration was done in the following steps:

Step 1: The annual average precipitation values from 1985 to 1996 were extracted from the LTPP database for each of the 18 projects listed in Table 3.1.

Step 2: From the AASHTOWare climatic files from 1996 to 2006 and for the weather stations corresponding to the 18 LTPP project locations, the annual average precipitation values for each year were calculated.

Step 3: The annual average precipitation from the AASHTOWare software database climatic files (covering the period from 1996 to 2006) and from the LTPP database (from 1985 to 1996) were compared for similarity.

Step 4: When such a pair of years from the two periods were found, the hourly precipitation data from the AASHTOWare climatic file (1996-2006) were copied to the corresponding year in the 1985 to 1996 period. The wind speed and percent sunshine data were also copied.

Step 5: The hourly temperature data for the period 1985 to 1996 was also taken from the corresponding AASHTOWare climatic file, but adjustments were made by subtracting for each day the difference between the average daily temperatures recorded in the AASHTOWare climatic file and in the LTPP climatic file. For example, if for a given pavement section, the average annual precipitation values recorded in LTPP in 1992 and in AASHTOWare database in 2001 were similar, 1992 and 2001 became paired years and the temperature on July 3 at 2:00PM in 1992 was computed as:

$$T_{1992\ [July3,\ 2;\ 00PM]} = \{T_{2001\ [July3,\ 2;\ 00PM]} + T_{1992\ [Avg.\ July\ 3]} - T_{2001\ [Avg.\ July\ 3]}\} \quad \textbf{Equation 3.1}$$

Where:

$T_{2001\ [July3,\ 2;\ 00PM]}$ – generated temperature data for July 3, 1992, at 2:00PM

$T_{1992\ [Avg.\ July\ 3]}$ – average daily temperature for July 3, 1992, in the LTPP database

$T_{2001\ [Avg.\ July\ 3]}$ – average daily temperature for July 3, 2001, in the AASHTOWare climatic database

Step 6: The hourly climatic database file was incorporated in the AASHTOWare parent folder and a new weather station for each SHRP ID was created.

3.2.5 Pavement Performance Data

The accuracy of measured distresses for the selected LTPP sites has a significant impact on enhancing the predictions of the embedded performance models in the AASHTOWare Pavement ME 2.1. The actual distresses values were extracted from the LTPP database. Then, they were tabulated in Appendix D. The extracted distresses data are:

- Total Rutting,
- Alligator Cracking,
- Longitudinal Cracking, and
- International Roughness Index (IRI).

3.3 Developing Calibrated Performance Models for NYSDOT

Although NYSDOT uses only IRI trigger value in deciding pavement overlay, the local calibration was performed for alligator cracking, total rutting, and IRI. During estimation of local bias, it was found that the measured thermal cracking data were unreliable, so the calibration of this model was not done. The calibration of longitudinal cracking was not conducted due to the lack of accuracy observed while developing the calibration coefficients. It is important to mention that the lack of accuracy in the predicted longitudinal cracking distresses was also observed by the Montana Department of Transportation in their effort to implement the MEPDG (Von Quintus & Moulthrop, 2007), as well as in Canada (Ahammed, Kass, & Hilderman, 2013).

3.3.1 Select Hierarchical Input Level

The design input level is selected by the designer based on the highway agency criteria. Nevertheless, the designer can use the design level inputs listed in Table 3.7, as recommended by AASHTO. NYSDOT has not developed a list of recommended design input levels yet; Table 3.7 was used to select the design input level. It is recommended to use the same design input levels in developing the design cases after calibrating the distresses models (Darter, Titus-Glover, & Von Quintus, 2009).

3.3.2 Sample Size Estimation for Distress Prediction Models

In this research, the minimum number of the required road segments needed to calibrate the performance models was determined based on the mean and variance. When employing both of them, a significant variation in the estimations of the sample size was found. At the end, the most reliable estimated sample size was adopted. The sample size was estimated for the following models:

- Rutting Model,
- Bottom Up Cracking Model,
- Thermal Cracking Model, and
- International Roughness Index Model.

To estimate the sample size, Equations 3.2 (for bias) and 3.3 (for precision) were employed (AASHTO, 2010):

$$N = \left(\frac{Z_{\propto/2} * \delta}{E_T}\right)^2$$ **Equation 3.2**

Table 3.6: Recommended Design Levels Inputs by AASHTO

Input Group		Input Parameter	Recalibration Input Level Used
Truck Traffic		Axle Load Distributions (Single, Tandem, Tridem) Truck Volume Distribution	Level 1
		Lane and Directional Truck Distributions	Level 1
		Tire Pressure	Level 3
		Axle Configuration, Tire Spacing	Level 3
		Truck Wander	Level 3
Climate		Temperature, Wind Speed, Cloud Cover, Precipitation, Relative Humidity	Level 1; Weather Stations
Materials Properties	Unbound Layers and Subgrade	Resilient Modulus-All Unbound Layers	Level 1; Backcalculation
		Classification and Volumetric Properties	Level 1
		Moisture-Density Relationships	Level 1
		Soil-Water Characteristic Relationships	Level 3
		Saturated Hydraulic Conductivity	Level 3
	HMA	HMA Dynamic Modulus	Level 3
		HMA Creep Compliance and Indirect Tensile Strength	Level 1, 2 and 3
		Volumetric Properties	Level 1
		HMA Coefficient of Thermal Expansion	Level 3
	PCC	PCC Elastic Modulus	Level 1
		PCC Flexture Strength	Level 1
		PCC Indirect Tensile Strength (CRCP Only)	Level 2
		PCC Coefficient of Thermal Expansion	Level 1
All Materials		Unit Weight	Level 1
		Poisson's Ratio	Level 1 and 3
		Other Thermal Properties; Conductivity, Heat Capacity, Surface Absorptivity	Level 3
Existing Pavement		Condition of Existing Layers	Level 1 and 2

The sample size estimation based on bias (or the mean) is summarized in Table 3.7. The following steps were employed to develop Table 3.7:

- The Level of Confidence was selected as 90%.
- The design reliability was selected as 90% based on the CPDM (NYSDOT, 2014a).
- The threshold value of each distress model was selected based on the recommended values by AASHTO (2008). However, the IRI trigger value was provided by the PMS unit of NYSDOT. NYSDOT used IRI trigger value ranges from 200 to 250 in/mile. Therefore, the mid-range value (225 in/mile) was used in this research.
- The Standard Error of Estimate (SEE) for each model was computed based on the trigger value of each distress model. For the IRI model, the SEE was selected to be 18.9 in/mile following AASHTO (2008).
- The tolerable Bias (E_T) was estimated at 90% confidence level.

As showed in Table 3.7, the estimated sample size satisfied the requirements for alligator cracking and rutting models. While the LTPP segments were the only segments that could be obtained, the estimated sample size for thermal cracking and IRI were not further considered, and it was assumed that the 18 LTPP sites were sufficient.

Table 3.7: Estimated Minimum Number of Sites Needed for Validation & Local Calibration Based on Bias

Pavement Type	HMA New Pavement			
Performance Model	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Performance Indicator Threshold (@ 90 Percent Reliability) (δ)	10%	0.4 in.	500 ft/mile	225 in/mile
Standard Error of Estimate (SEE)	5.30%	0.16 in.	83 ft/mile	18.9 in/mile
Tolerable Bias (E_T)	8.70%	0.27 in.	136 ft/mile	31 in/mile
Minimum No. of Sites Required for Validation and Local Calibration	4	6	36	142
Number of the LTPP Sections Used	17	18	17	17

$Z_{\propto/2} = 1.64$

$E_T = SEE * Z_{\propto/2}$

The minimum sample size was also estimated based on precision (or the variance), as shown in Table 3.8. Equation 3.3 was used for this purpose:

$$\frac{Se}{Sy} \geq \left[\frac{X\alpha^2}{n-1}\right]^{0.5}$$ **Equation 3.3**

Based on Equation 3.2, the sample size was estimated as follows:

- AASHTOWare Pavement ME 2.1 design problems were run with the global (national) calibration coefficients. The computed distresses were extracted and tabulated based on the site number, date, and distress type; they are listed in Appendix E.
- The maximum measured distresses for each site were tabulated with the corresponding maximum computed distresses. Then, the residuals were computed as the difference between the measured and the computed values (Devore & Farnum, 1999). This process was repeated for each distress model. Tables 3.8 to 3.12 summarize the outputs of this step.

- Then, the standard deviation of the maximum measured distresses (S_y) was computed for each model. The standard deviation of the residuals (S_e) was then computed for each distress model, as shown in Table 3.8.
- The same procedure was performed for the full set of measured distresses data instead of only for maximum distress values.
- Chi-Squared (X^2) values at 90% confidence level and (*n-1*) degree of freedom were computed. The parameter *n* represents the number of observations.

It should be noted that the ratio ($\frac{S_e}{S_y}$) compares the variability of the predicted performance to that of the measured performance. A ratio greater than 1.0 indicates that the variability of the residuals errors (the difference between the predicted and the measured values) is larger than that of the measured values. Therefore, a ratio less than 1.0 is preferable.

Table 3.8: Minimum Number of Sites Required for Validation and Local Calibration Based on Precision

Performance Models	Alligator Cracking	Rut Depth	Thermal Cracking	IRI
Based on Maximum Measured Values				
S_y	8.43%	0.23 in.	1,860 ft/mile	35 in/mile
S_e	8.54%	0.24 in.	1,996 ft/mile	54 in/mile
S_e/S_y	1.0	1.05	1.07	1.56
$(X^2\alpha/(n-1))$^0.5	1.0	1.05	1.07	1.28
Minimum Number of Sites for Validation and Local Calibration	325	225	249	17
Number of LTPP sections used	17	18	17	17
Based on Full Set of Measured Data				
S_y	6.99%	0.17 in.	1,662.7 ft/mile	40.98 in/mile
S_e	13.89%	0.24 in.	80.3 ft/mile	320.59 in/mile
S_e/S_y	1.99	1.37	0.05	7.82
$(X^2\alpha/(n-1))$^0.5	1.64	1.39	0.14	1.64
Minimum Number of Sites for Validation and Local Calibration	2	5	10,000,000.00	2
Number of LTPP sections Used	17	18	17	17

Table 3.9: Extracted Data for the Fatigue Model

Bottom-Up Cracking Model				
Number	**Segment ID**	**Max Measured**	**Computed**	**Residual**
1	231001	0.77	0.01	0.76
2	231009	1.60	0.02	1.58
3	231028	0.00	0.05	-0.05
4	251003	0.00	0.01	-0.01
5	341003	22.60	0.12	22.48
6	341011	22.60	0.04	22.57
7	341030	20.47	0.04	20.43
8	341031	10.15	0.06	10.10
9	341033	1.31	0.01	1.30
10	341034	0.16	0.02	0.14
11	341638	0.07	2.66	-2.59
12	421597	0.00	0.00	0.00
13	421599	0.00	0.00	0.00
14	501002	0.07	0.02	0.05
15	501004	4.11	0.00	4.11
16	501681	0.00	0.01	-0.01
17	501683	1.45	0.01	1.44

Table 3.10: Extracted Data for the Rutting Model

Rutting Model				
Number	**Segment ID**	**Max Measured**	**Computed**	**Residuals**
1	91803	0.24	0.20	-0.05
2	231001	0.25	0.51	0.26
3	231009	0.41	0.28	-0.14
4	231028	0.31	0.51	0.21
5	251003	0.30	0.18	-0.13
6	341003	0.42	0.83	0.40
7	341011	0.45	0.39	-0.05
8	341030	0.30	0.85	0.54
9	341031	0.48	0.55	0.07
10	341033	0.27	0.35	0.08
11	341034	0.35	0.28	-0.07
12	341638	0.39	0.32	-0.07
13	421597	0.17	0.22	0.05
14	421599	0.28	0.28	0.00
15	501002	0.33	0.62	0.29
16	501004	0.26	0.28	0.02
17	501681	0.19	0.49	0.30
18	501683	0.18	0.87	0.69

Table 3.11: Extracted Data for the Thermal Cracking Model

Thermal Cracking Model				
Number	**Segment ID**	**Max Measured**	**Computed**	**Residuals**
1	231001	2112	4010.94	1898.94
2	231009	2112	1780.33	-331.67
3	231028	1.14	1423.57	1422.43
4	251003	1594.43	3082.68	1488.25
5	341003	1461.87	3262.79	1800.92
6	341011	1552.86	6383.56	4830.70
7	341030	0.54	2895.64	2895.10
8	341031	24.99	6179.21	6154.22
9	341033	1908.68	2930.27	1021.59
10	341034	965.12	2885.25	1920.13
11	341638	1350.81	443.35	-907.46
12	421597	0.02	762.01	761.99
13	421597	0.02	762.01	761.99
14	501002	2112	4748.71	2636.71
15	501004	2112	2985.69	873.69
16	501681	2112	131.62	-1980.38
17	501683	2112	1517.09	-594.91

Table 3.12: Extracted IRI data

IRI Model				
Number	**Segment ID**	**Maximum Measured IRI**	**Computed IRI**	**Residuals**
1	231001	125.3	93.4	-31.88
2	231009	67.2	94.7	27.46
3	231028	91.7	88.1	-3.61
4	251003	122.6	83.6	-38.96
5	341003	124.5	85.9	-38.57
6	341011	115.7	101.3	-14.45
7	341030	252.9	75.7	-177.16
8	341031	144.7	90.2	-54.50
9	341033	199.1	96.1	-103.02
10	341034	96.3	98.7	2.38
11	341638	66.0	122.2	56.21
12	421597	107.0	67.9	-39.12
13	421599	93.8	90.9	-2.94
14	501002	93.5	114.3	20.79
15	501004	132.6	96.1	-36.50
16	501681	76.3	88	11.69
17	501683	142.6	87.3	-55.26

Based on Table 3.12, when the maximum measured distresses were used, the estimated sample size of the IRI model was the only one that equals to the number of obtained LTPP sites. Nevertheless, the LTPP sites were insufficient for other models. Thus, Table 3.7 was considered in this research since the estimated sample sizes of the performance models were less than or close to the LTPP sites. The estimated sample size based on the full set of measured data was abandoned due to unreliable estimation, such as the estimated sample size for the thermal cracking model.

It is obvious that there is contrast in the estimation process between the two estimation methods, although the sample size was estimated at the same confidence level. In Equation 3.2, the highway agency design criteria were used to compute SEE. Then, SEE was used to compute E_T; these two parameters have a great impact on the estimation process based on bias. However, the sample size was estimated in Equation 3.3 based on the distresses predicted by AASHTOWare with global calibration factors and the measured distresses. For this reason, Equation 3.2 is more reasonable than Equation 3.3. It is important to mention that both equations estimated the sample size at one-sided confidence level which makes the estimation process more precise. Statistically, the precision is defined for one-sided confidence level (Devore & Farnum, 1999).

3.3.3 Extraction, Evaluation, and Conversion of the Measured Data

Since Momin (2011) had extracted and converted the collected data given in Appendix D of this report, the data were checked and evaluated for use in this research. Since the resulting sample size is small, outliers were identified only when bias was found after the local calibration. The SAS computer software was used to identify the outliers.

3.3.4 Assess Local Bias and Standard Error of the Estimate (SEE) from Global Calibration Factors

The full set of measured data, also the computed distresses from the previous runs of the AASHTOWare, were used to assess the local bias and SEE. The null hypothesis used for this purpose was:

$$H_o: Measured\ Distresses = Computed\ Distresses$$
$$H_a: Measured\ Distresses \neq Computed\ Distresses$$

In addition, the plots of measured versus computed distresses were prepared for each model to investigate the location of the points versus the line of equality. The computed distresses for the LTPP sites are tabulated in Appendix E.

3.3.4.1 Determination of Local Bias for the Alligator Cracking Model

The null hypothesis was conducted to identify the existence of the local bias. Paired *t*-test at 95% confidence level was used to determine if there is a significant difference between the measured alligator cracking and the computed alligator cracking. After the test was performed, the null hypothesis was rejected. Therefore, at 95% confidence level there is a significant difference between the measured and computed distresses. The Sum Squared Errors (SSE), Bias, and Correlation Coefficient (R^2) are given in Table 3.13.

The plot of measured versus computed distresses (Figure 3.5) reveals that the alligator cracking model must be calibrated to local conditions.

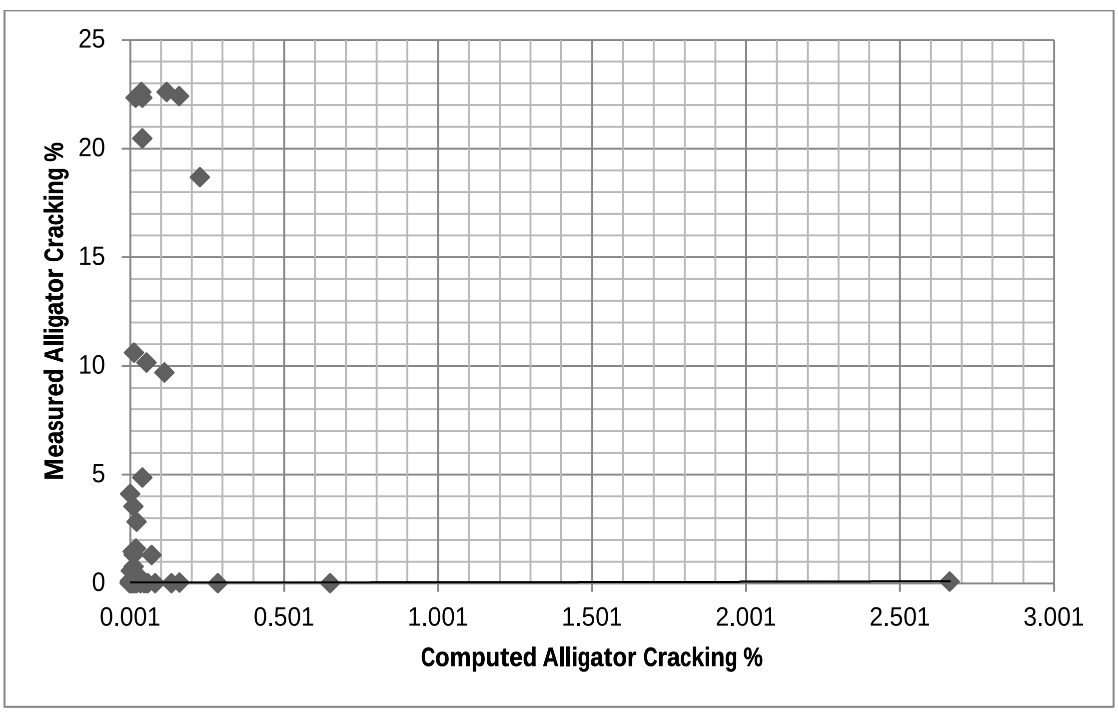


Figure 3.5: Measured vs. Computed Alligator Cracking (Global Calibration)

3.3.4.2 Determination of Local Bias for the Total Rutting Model

To determine the local bias, the computed total rutting was obtained by the summation the rutting for the asphalt concrete (AC), base, and subgrade layers. Then, a paired *t*-test was performed at 95% confidence level. The test concluded that, at 95% confidence level, there is a significant difference between the measured and computed total rutting. The results of the statistical analysis are given in Table 3.13.

Additionally, the plot of measured versus computed total rutting (Figure 3.6) shows that there is a poor linear relationship between the measured and the computed total rutting. Therefore, the local calibration must be performed for this model. The plotted data show a funnel shape which suggests that the variance is not constant because the embedded performance models in the AASHTOWare were globally calibrated. However, it is not possible to eliminate the non-constant variance by transformation techniques (Kutner, Nachtsheim, Neter, & Li, 2005), since only the calibration coefficients can be changed and not the variables themselves in the AASHTOWare models.

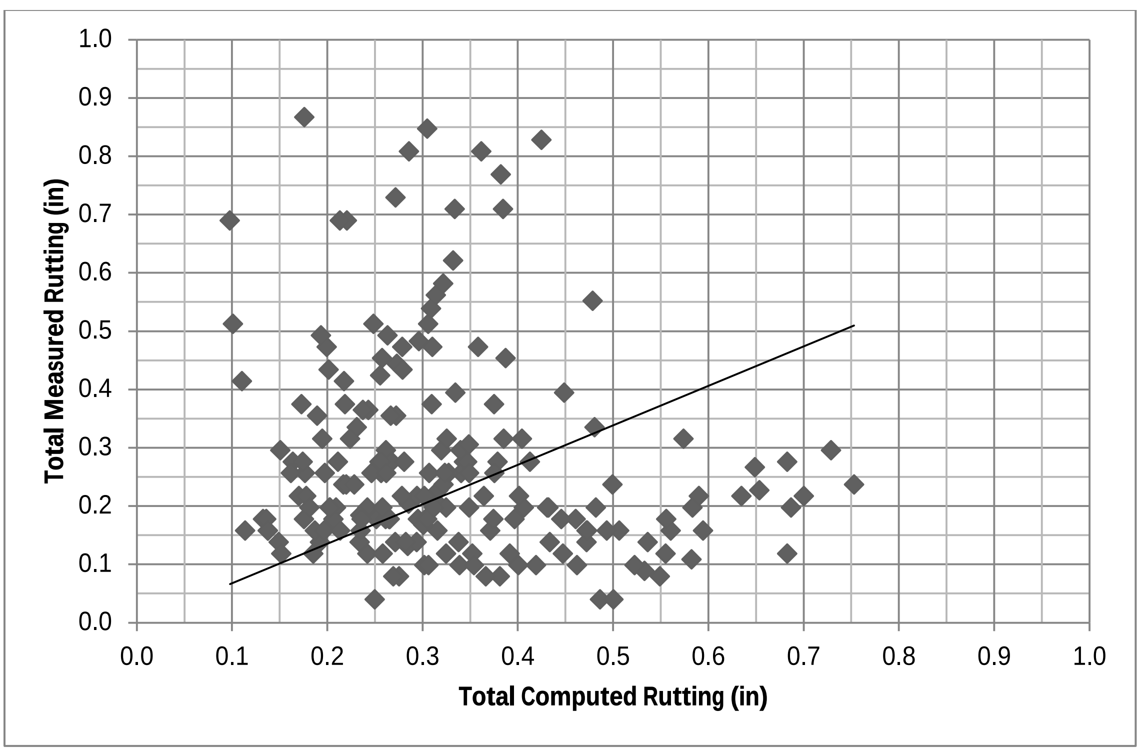


Figure 3.6: Measured vs. Computed Total Rutting (Global Calibration)

3.3.4.3 Determination of Local Bias for the Thermal Cracking Model

Local bias was determined by performing a paired *t*-test at 95% confidence level. The null hypothesis was rejected, so there is a significant difference between the measured and computed thermal cracking. This difference is reflected in Figure 3.7.

The points in Figure 3.7 are very highly scattered and the relationship between the measured and computed distress data is poor. Also, based on Table 3.13, the large value of the SSE and (R^2) indicate a large scatter of the measured data. Therefore, the local calibration could not be performed for the thermal cracking model.

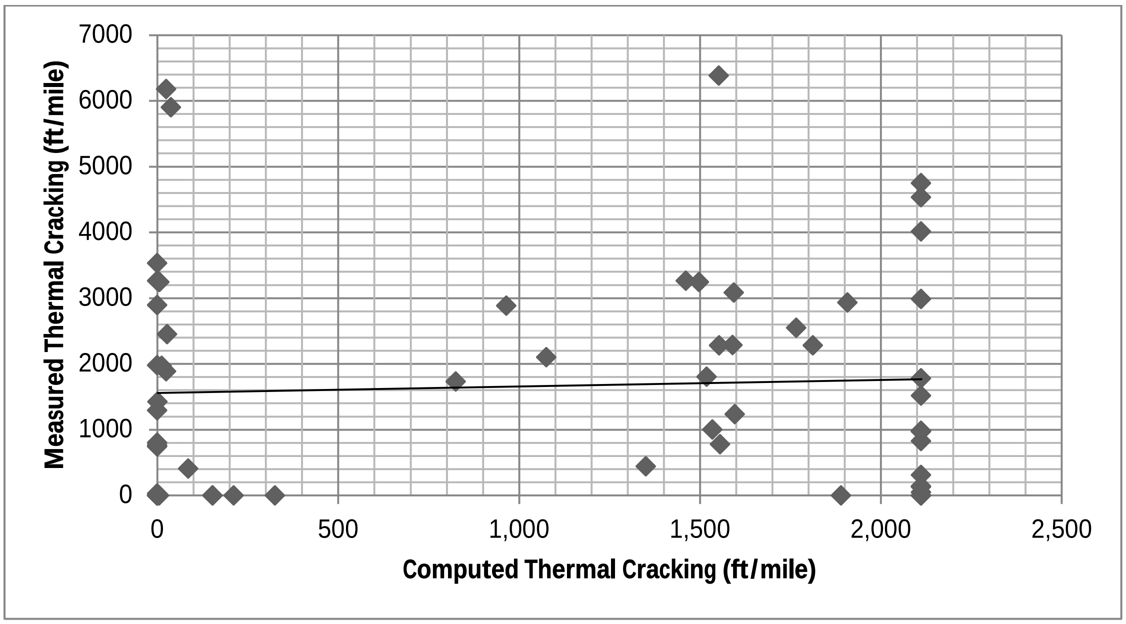


Figure 3.7: Measured vs. Computed Thermal Cracking (Global Calibration)

3.3.4.4 Determination of Local Bias for the IRI Model

The local bias was determined based on the null hypothesis. For this purpose, a paired t-test was employed. Based on the test, the null hypothesis was rejected. Therefore, at 95% confidence level there is significant difference between the measured and computed IRI. Then, SSE and R^2 were computed and they are listed in Table 3.13.

The plot of measured versus computed IRI is shown in Figure 3.8. The figure indicates a poor scatter of the points with respect to the line of equality. The computed SSE and R^2 values also indicate the need to perform the local calibration to reduce the SSE and increase the coefficient of determination. Hence, the local calibration for the IRI model was performed. The required outputs needed to assess the local bias are given in Table 3.13.

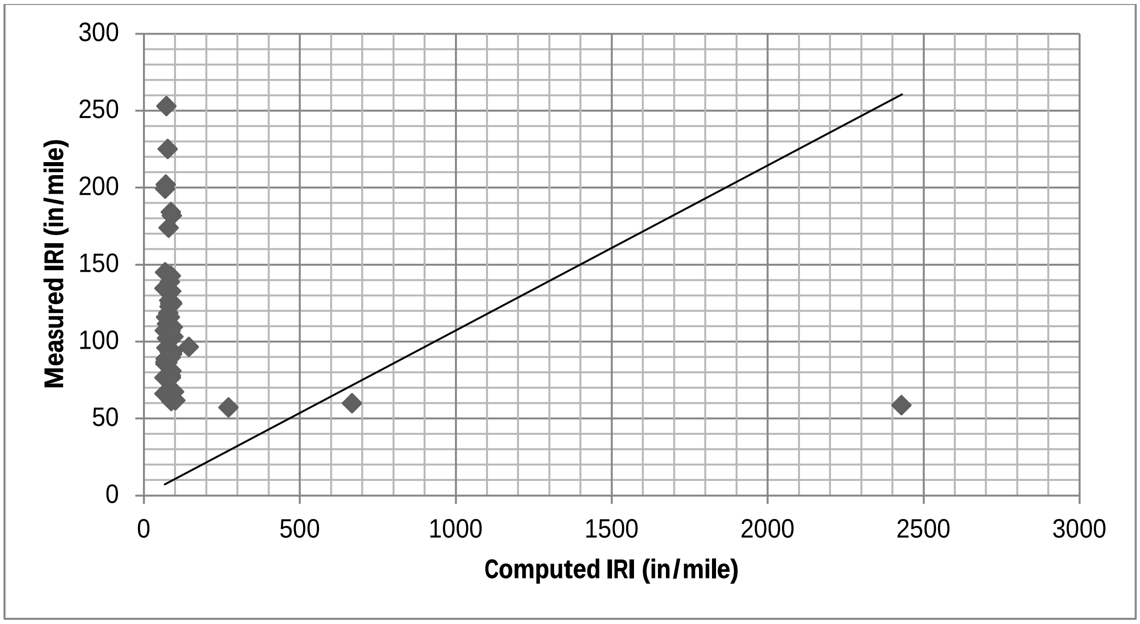


Figure 3.8: Measured IRI vs. Computed IRI (Globally Calibrated)

Table 3.13: Summary of Local Bias Assessment

Model	Regression Coefficients	Bias	SSE	R^2	S_e/S_y	*P* Value	Hypothesis; Ho:Σ(Meas.-Pred.) = 0
Alligator Cracking	C1=1 C2=1	-3.2	3,645	0.001	1	0.0006	Reject; P<0.05
Total Rutting	βr1=1 βr2=1 βr3=1	0.056	11.5	0.55	1.37	0.0013	Reject; P<0.05
Thermal Cracking	βt=1	129.1	234,373	0.31	1.116	0.0081	Reject; P<0.05
IRI	C1 =0.015 C2 = 0.4 C3 =0.008 C4 = 40	-24.7	754,583	0.09	7.82	0.02	Reject; P<0.05

3.3.5 Elimination of the Local Bias

To eliminate the bias, the Microsoft Excel Solver was used to optimize the calibration coefficients of the performance models. Data in Appendices D and E were used for this purpose. Table 3.14 lists the optimized calibration coefficients, the Bias, SSE, R^2, and the *P* value. The *P* value was used to judge the hypothesis. The following steps were performed to eliminate the local bias:

- The AASHTOWare Pavement ME 2.1 design problems were run with the global calibration coefficients to compute the distresses.
- The computed distresses were listed with the corresponding measured distresses for each segment (and at same time).
- The residual errors and the SSE were computed.
- The Microsoft Excel Solver was employed to adjust the regression coefficients, so that the minimum SSE is obtained.

3.3.5.1 Elimination of the Local Bias for the Alligator Cracking Model

First, the measured fatigue cracking data, the cumulative damages, and required parameters to compute the alligator cracking were extracted from Appendices D and E. The following steps explain the process in detail:

- A separate Excel spreadsheet file containing the extracted data, including the accumulated damage, was assembled.
- The regression coefficients (from global calibration) of the alligator cracking transfer function were listed in the same file.
- Then, the transfer function was defined. The alligator cracking values were computed from the accumulated damage values and were compared with the values computed by the AASHTOWare. Since they were found to be the same, the written equation was considered correct.
- The residuals errors were computed as the difference between the measured and the computed distresses. The Sum Squared of Errors (SSE) was computed from squaring the residuals, as shown in Table 3.14.

- Then, the Microsoft Solver was used to optimize the regression coefficients in the alligator cracking model to minimize the SSE. The optimized regression coefficients became the local calibration coefficients. Then, SSE and R^2 were computed and listed in Table 3.14. As shown in the table, the SSE reduced and the R^2 slightly improved.
- To identify the local bias, paired *t*-tests at 95% confidence level were conducted. At 95% confidence level, the null hypothesis was accepted as shown in Table 3.14. This indicates that the calibration improved the alligator cracking model.
- The measured versus computed alligator cracking is plotted in Figure 3.9. The plot shows an improvement in the location of the points relative to the equality line. However, it is clear that outliers still exist. Since it was not possible to validate the measured LTPP data, outlier analysis was not conducted.

Table 3.14: The Summary of Local Calibration and Elimination the Local Bias

Model Type	Regression Coefficients	Bias	SSE	R^2	S_e/S_y	*P* Value	Hypothesis; Ho:Σ(Meas.-Pred.) = 0
Alligator Cracking	C1=0.501711 C2=0.227186	0.21	2,766	0.07	0.96	0.85	Accepted; P>0.05
Rutting	βr1=0.59 βr2=0.821 βr3=0.74	-0.04	8.80	0.56	1.21	0.008	Reject; P<0.05
IRI	C1 = 168.709 C2 = -0.0238 C3 = 0.00017 C4 = 0.015	-6.0	115,777	0.87	1.053	0.33	Accepted; P>0.05

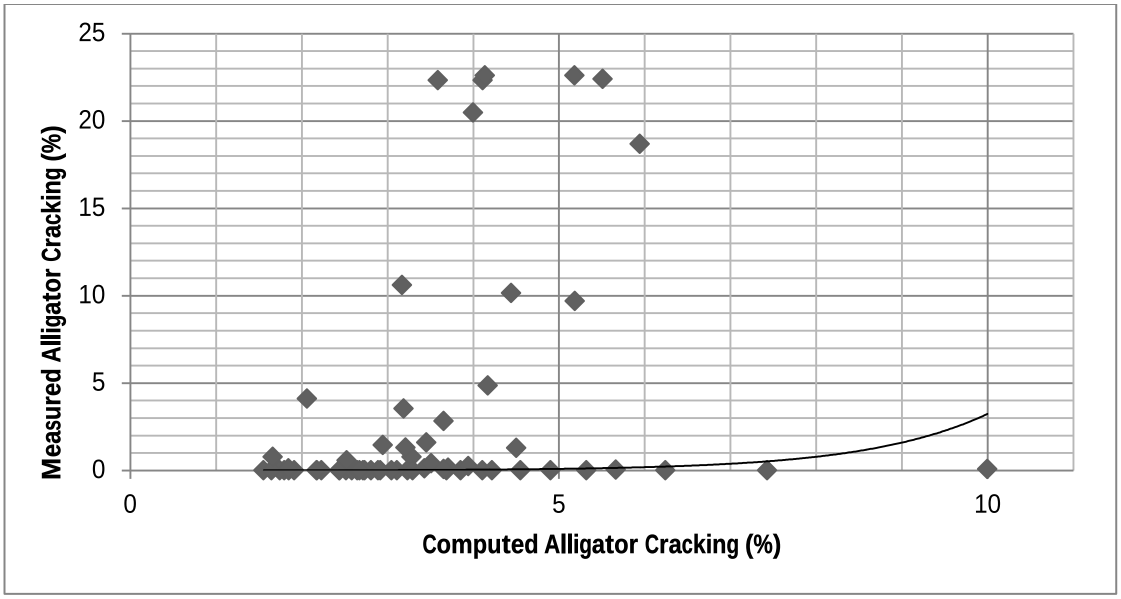


Figure 3.9: Measured vs. Computed Alligator Cracking (Locally Calibrated)

3.3.5.2 Elimination of Local Bias for the Rutting Models

The measured and computed total rutting data were extracted from Appendices D and E in a separate Excel file. The bias for total rutting was eliminated by performing the optimization approach mentioned earlier. The following steps explain the process:

- The total rutting was defined as the sum of the rutting in the subdivided layers (Asphalt Concrete [AC], Base, and Subgrade layers).
- The computed rut depth in each layer was multiplied with the correspondent global regression coefficients (e.g., $\beta r1 * AC_{Rutting}$).
- The residual errors for the full set of data were obtained as the difference between the computed total rutting and measured total rutting. From the residuals, SSE was obtained.
- The Microsoft Solver was employed to adjust the regression coefficients for the three pavement layers to compute the distresses that give the minimum SSE.
- The optimized regression coefficients were defined as the local calibration coefficients.
- Then the R^2 and the Bias were computed, as listed in Table 3.14. A slight improvement in the SSE and R^2 was noted.

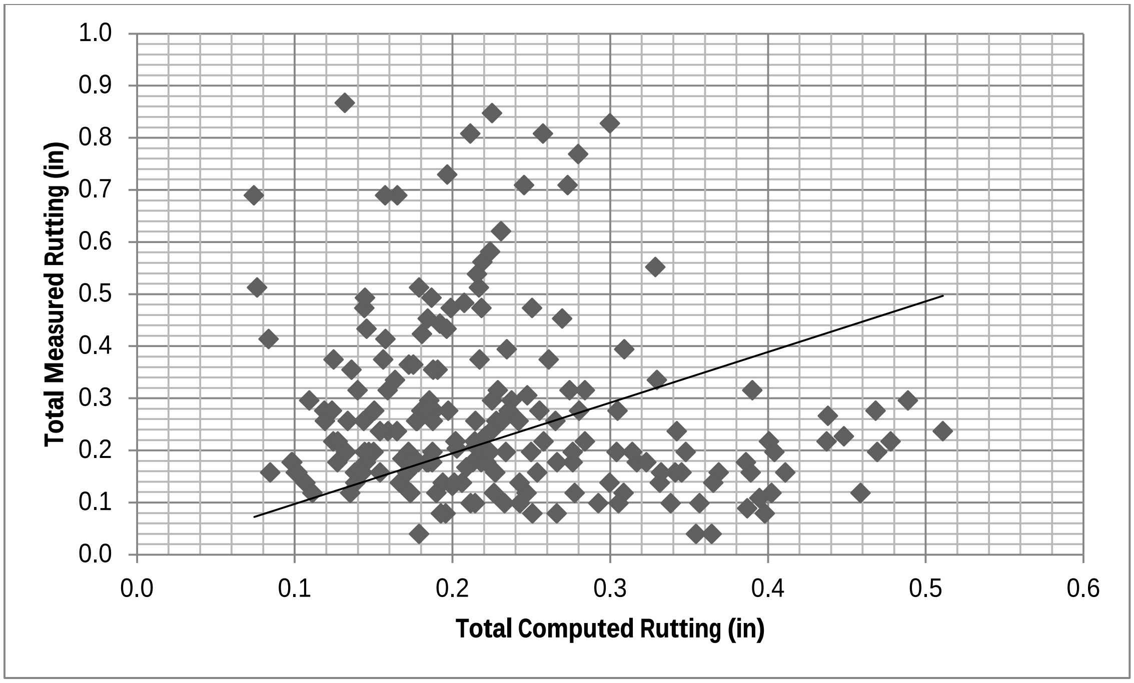


Figure 3.10: Measured vs. Computed Total Rutting (Local Calibration)

- To identify the local bias, a paired t-test at 95% confidence level was used. After the test was performed, the null hypothesis was rejected. Therefore, at 95% confidence level, there is a significant difference between the measured and the computed total rutting (Table 3.14).
- The outlier analysis was performed by running the SAS software. The existence of outliers in the measured distresses was determined by obtaining the absolute values of t-studentized $/t_i/$ for each segment. Then, the Bonferroni test was applied. The test proved there are no outliers in the measured dataset.
- The measured total rutting versus computed total rutting plot was performed, as illustrated in Figure 3.10. The plot shows an improvement in the location of the data relative to the equality line, so the optimization improved the model.

3.3.5.3 Elimination of Local Bias for the IRI Model

The bias of the IRI model was eliminated by performing the following steps:

- The IRI equation was defined as the summation of Site Factor (SF), Sum of alligator cracking and the thermal cracking (TC), and average total rut depth (RD).
- Each variable was multiplied with the corresponding global calibration coefficient.
- Then, the difference between the measured IRI and the initial IRI was found for each segment.
- Then residual errors were calculated.
- The Microsoft Solver was employed to optimize the global calibration coefficients to obtain the minimum SSE.
- To identify the local bias, a paired t-test at 95% confidence level was used. After the test was performed, the null hypothesis was accepted. Therefore, at 95% confidence level there is no significant difference between the measured and the locally computed distresses, as shown in Table 3.14.
- Thus, the optimized regression coefficients were defined as the local calibration coefficients. Then, R^2 and Bias were computed as shown in Table 3.14. A significant improvement in the SSE, (R^2), and $(\frac{S_e}{S_y})$ was noticed.
- The plot of measured versus computed IRI distresses is given in Figure 3.11. The plot shows the improvement of the location of the data points, indicating that the optimization improved the model.

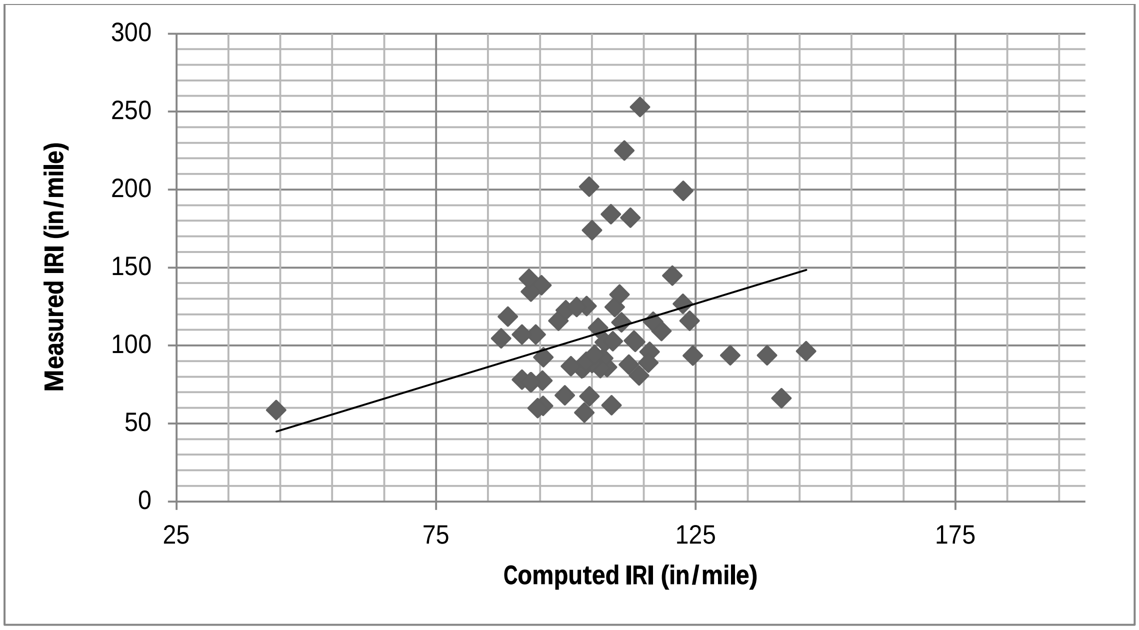


Figure 3.11: Measured IRI vs. Computed IRI (Locally Calibrated)

3.4 Validation of the Performance Models

The Jack-knife testing approach was performed only for the rutting and the IRI models since these models are linear. The validation of the alligator cracking model was not possible. The following steps were performed to validate the rutting model:

- The measured total rutting observations were extracted and listed in a separate Excel file.
- The extracted observations set was split into two groups: one for the calibration, and the other for prediction. These groups were randomly selected.
- Therefore, the prepared matrix consists of group X (as variables) and group Y (as predictor), with (*i=1,,,,,, n)* sets of observations.
- At the beginning, one set of (X_i, Y_i) was removed. By removing one set such as (x_1, y_1), the validation matrix contained *(n-1*) observations to perform the calibration.
- After the calibration was performed on the *n-1* matrix, the calibrated coefficients were used to predict (y) which are listed in a new group, called K_{th} group.

- Now the standard error ($new\ e_1$) was found as the difference between the measured and computed distress value of the removed dataset. For example ($y_{kth1} - y_1$).
- The removed dataset (x_1, y_1) was replaced with the second observations set in the *(n-1)* validation matrix which was (y_2, x_2).
- Same steps were repeated for all observations in the dataset.
- The F-Test at 95% confidence level was employed to identify if the new standard errors are significantly higher than the standard errors of the calibration. The test concluded that at 95% confidence level, the new standard errors are not significantly higher than the standard errors of the calibration. Thus, the calibrated model is valid.

The above mentioned steps were repeated to validate the IRI model. F-Test was used to test the validation at 95% confidence level. It was found that the calibrated IRI model is also valid.

Chapter 4: Development of the AASHTOWare Pavement ME 2.1 Design Cases

4.1 Overall Concept for Developing the Design Cases

The design cases were developed to design the new flexible pavement structures based on the AASHTO ME Pavement Design Guide (MEPDG). For this purpose, AASHTOWare Pavement ME 2.1 was used with the local calibration factors. The design cases were built based on a combination of truck traffic volume, climatic conditions, subgrade soil stiffness, pavement structure, and materials properties. The NYSDOT (2014a) CPDM was the main source providing the design inputs, along with NYSDOT standards and laboratory experimental data. The following conditions were considered when developing the design cases:

- The pavement structures for new flexible pavement classified as Principal Arterial – Interstate.
- A design life of 15 or 20 years was used.
- A design reliability of 90% was used.
- A water table depth of 10 feet was used.

The developed design cases simulated the current NYSDOT pavement configurations shown in Table 1.2. Thus, the following were considered during the development process for the design cases:

- Subgrade soil resilient modulus (M_r) of 4.0, 5.0, 6.0, 7.0, 8.0, and 9.0 ksi (28, 34, 41, 48, 55, and 62 Mpa).
- Annual Average Daily Truck Traffic (AADTT) in the design lane of 50, 100, 250, 500, 1,000, 2,000, 4,000, and 5,000.
- Pavement structures starting with the design cases included in the NYSDOT CPDM. The thicknesses of base and subbase layers were kept the same as those in the recommended design template included in CPDM. Nevertheless, the thicknesses of asphalt concrete and selected subgrade soil layers were varied to optimize the design solutions.

- The project location and the corresponding climatic data for all 23 climatic stations available in the AASHTOWare Pavement ME 2.1 for New York State.

A unique name format was used for each design case to distinguish the structure design components. The name format included the resilient modulus of subgrade soil layer (M_r), the total HMA thickness, the select granular subgrade layer thickness, and the Annual Average Daily Truck Traffic in the design lane. Accordingly, the name template of each design case was as below:

M_r (ksi) - HMA Thickness (in.) - Select Granular Subgrade Soil Thickness (in.) - AADTT (one lane)

4.2 General Information

Since AASHTOWare runs were performed for hypothetical design cases, AASHTOWare default dates for construction and opening-to-traffic dates were used, as illustrated in Figure 4.1. They would represent a typical road construction schedule, where the unbound granular layers are placed in the Spring and early Summer, while the asphalt layers are paved in late Summer or early Fall.

The opening to traffic typically takes place in the Fall. The construction month in AASHTOWare refers to the month and year that the unbound layers have been compacted and finished in the month that the Hot Mix Asphalt (HMA) has been placed, while the traffic opening data represent the date of opening the road to the public.

AASHTOWare accounts the monthly traffic loading and climatic inputs based on the selected construction and opening-to-traffic dates. Therefore, the monthly modulus values of each layer are affected by the selected dates.

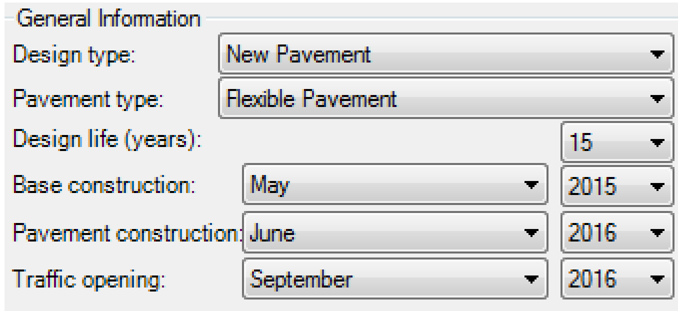


Figure 4.1: Selected Construction and Opening-To-Traffic Dates

4.3 Design Criteria and Reliability

In order to perform the design by AASHTOWare, the design criteria (trigger values) for flexible pavement distresses should be selected. The trigger values normally represent the distress values for which the asphalt pavement structure would be rehabilitated with an overlay. Thus, the NYSDOT Pavement Management Unit (PMU) was contacted and the typical distress values that trigger rehabilitation with an overlay were obtained. NYSDOT uses a trigger value range of 200–250 in/mile for the IRI only when deciding to rehabilitate flexible pavements. The values recommended by the Calibration Guide (AASHTO, 2008) were used to obtain the design criteria for other distresses.

For this research, the trigger value of the IRI was selected as 225 in/mile according to AASHTO recommendations. AASHTO suggests using the average of the agency design criteria; the initial IRI value was selected 60 in/mile because AASHTO recommends this value for full depth asphalt pavements (AASHTO, 2008). In addition to this, NYSDOT approved the use of 0.75 in. as the trigger value for total rutting in this research.

Therefore, the design solutions adequacy was determined based on the IRI and total rutting only. It should be noticed that NYSDOT uses 90% design reliability to design the new flexible pavement structures. For this reason, this value of design reliability was used. Table 4.1 gives the design criteria and reliability used for this study.

Table 4.1: Design Criteria and Reliability for this Study

Performance Criteria	Limit	Reliability
Initial IRI (in/mile)	60	-
Terminal IRI (in/mile)	225	90%
AC Longitudinal Cracking (ft/mile)	2000	90%
AC Fatigue Cracking (Percent)	10	90%
AC Thermal Cracking (ft/mile)	500	90%
Permanent Deformation-Total Rutting (in.)	0.75	90%
Permanent deformation-AC only (in.)	0.25	90%

4.4 Traffic Inputs

AASHTOWare offers the designer a hierarchical design input level. Level 1 input data should be used when the traffic data specific for the design project is known. Level 2 data should be used when limited traffic data for the specific project site is known, while Level 3 data uses default, regional, or statewide average traffic inputs. The work of Intaj (2012) was reviewed and evaluated to use the traffic data for this research. Intaj recommended the use of statewide average traffic data collected in 2010 for design of the new pavement structures. A detailed justification for this recommendation has been presented in Chapter 2 of this report.

The statewide average traffic data for 2010 were adopted to develop the design cases for this research. The statewide average traffic data for 2010 are summarized in:

- *Vehicle Class Distribution (VCD)*: Table 4.2
- *Monthly Adjustment Factors (MDF)*: Table 4.3
- *Number of Axles per Truck*: Table 4.4
- *Axle Load Spectra*: from Intaj (2012)

Table 4.2: Average Statewide VCD for 2010

Vehicle Class	4	5	6	7	8	9	10	11	12	13	Total
Distribution (%)	2.64	27.3	13.4	3.04	10.43	36	5.45	0.79	0.25	0.7	100

Table 4.3: Average Statewide MDF for 2010

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0.8	0.84	0.8	0.8	0.94	0.94	0.74	0.95	0.95	0.95
February	0.85	0.83	0.85	0.85	0.97	0.97	0.74	1.02	1.02	1.02
March	0.87	0.85	0.87	0.87	1.05	1.05	0.83	1.16	1.16	1.16
April	1.12	0.96	1.12	1.12	1.07	1.07	0.96	1.18	1.18	1.18
May	1.13	1.1	1.13	1.13	1.01	1.01	1.09	1.09	1.09	1.09
June	1	1.1	1	1	1.04	1.04	1.16	1.09	1.09	1.09
July	1.12	1.06	1.12	1.12	0.97	0.97	1.21	1.03	1.03	1.03
August	1.18	1.08	1.18	1.18	1.02	1.02	1.27	1.01	1.01	1.01
September	1.06	1.19	1.06	1.06	1.02	1.02	1.17	0.87	0.87	0.87
October	1.06	1.16	1.06	1.06	1.04	1.04	1.11	0.92	0.92	0.92
November	0.91	0.97	0.91	0.91	0.99	0.99	0.93	0.92	0.92	0.92
December	0.89	0.86	0.89	0.89	0.88	0.88	0.79	0.77	0.77	0.77

Table 4.4: Average Statewide Number of Axle per Truck for 2010

Vehicle Class	Single	Tandem	Tridem	Quad
Class 4	1.32	0.68	0	0
Class 5	2	0	0	0
Class 6	1	1	0	0
Class 7	1.32	0.28	0.64	0.05
Class 8	2.45	0.59	0.02	0
Class 9	1.23	1.89	0	0
Class 10	1.07	0.99	0.95	0.05
Class 11	3.7	0.27	0.25	0.01
Class 12	3.71	1.09	0.03	0
Class 13	2.11	0.76	0.28	0.32

Since AASHTOWare calls for additional traffic inputs, CPDM was reviewed and the appropriate values were selected:

- *Annual Average Daily Truck Traffic (AADTT)*: AADTT was selected as 100, 200, 500, 1000, 2,000, 4,000, 8,000, and 10,000.
- *Percentage of Trucks in Design Direction*: A 50% value has been used, as recommended by CPDM.
- *Number of Lanes in Design Direction*: It was assumed to be one lane in the design direction
- *Percentage of Trucks in Design Lane*: Since only one lane was selected for the design direction, 100% value was used.

- *Operational Speed*: The default value of 65 mph was used.
- *Truck Traffic Growth Rate*: The exponential traffic growth model, with a growth rate of 2% was used. This is the value recommended by NYSDOT when project-specific values are not available (CPDM).

AASHTOWare ME default values were used for traffic inputs that do not have values specified in the CPDM.

- *Axle Configuration*
- *Lateral Wander*
- *Wheel Base*
- *Identifiers*

4.5 Climatic Data

Climatic data has a significant impact on the distress prediction since the hourly basis records of temperature, precipitation, relative humidity, wind speed, and cloud cover are used by the Enhanced Integrated Climatic Model (EICM). The temperature and the moisture are computed by EICM for each sublayer of the pavement structure. The dynamic modulus and the resilient modulus are adjusted and modified over the design life by the EICM model. AASHTOWare contains climatic files for 851 climatic stations located throughout the United States.

In this research, all AASHTOWare climatic stations for New York State (NYS) were employed for the development of the design cases, as listed in Table 4.5. The weather conditions for the same region were represented in at least one climatic station per region. AASHTOWare does not have climatic data for a weather station in Region 9. Therefore, a virtual climatic station was created as a combination of the following climatic stations:

- Albany (14735)
- Elmira (14748)
- Montgomery (04789)
- Syracuse (14771)
- Utica (94794)

Since the CPDM and NYSDOT specifications do not provide recommendation on values to be used for the water table depth, in collaboration with NYSDOT it was decided to design for a water table depth value of 10 feet. It is important to mention that AASHTOWare uses the water table depth to calculate the moisture content in the unbounded layer which is used for the estimation of the resilient modulus of unbound materials during the design life. Previous work indicated that a water table depth higher than 10 feet has no effect on the predicted distresses (AASHTO, 2008). Figure 4.2 shows a screen capture of the AASHTOWare climatic data tab.

Figure 4.2: AASHTOWare Climate Tab

Table 4.5: AASHTOWare Climatic Stations Used for this Study

Climatic Stations					***Annual Water Table (ft)***
County	**Station ID**	**Longitude (Decimal Degrees)**	**Latitude (Decimal Degrees)**	**Region**	
Saratoga	Albany (14735)	-73.803	42.748	1	10
Warren	Glens Falls (14750)	-73.61	43.341	1	10
Oneida	Utica (94794)	-75.384	43.145	2	10
Onondaga	Syracuse (14771)	-76.103	43.109	3	10
Monroe	Rochester (14768)	-77.677	43.117	4	10
Erie	Buffalo (14733)	-78.736	42.941	5	10
Chautauqua	Dunkirk (14747)	-79.272	42.493	5	10
Niagara	Niagara Falls (04724)	-78.945	43.107	5	10
Steuben	Dansville (94704)	-77.713	42.571	6	10
Chemung	Elmira/Corning (14748)	-76.892	42.159	6	10
Allegany	Wellsville (54757)	-77.992	42.109	6	10
St. Lawrence	Massena (94725)	-74.846	44.936	7	10
Clinton	Plattsburgh (94733)	-73.523	44.687	7	10
Jefferson	Watertown (94790)	-76.022	43.992	7	10
Orange	Montgomery (04789)	-74.265	41.509	8	10
Dutchess	Poughkeepsie (14757)	-73.884	41.627	8	10
Westchester	White Plains (94745)	-73.708	41.067	8	10
Nassau	Farmingdale (54787)	-73.417	40.734	10	10
Suffolk	Islip (04781)	-73.102	40.794	10	10
Suffolk	Shirley (54790)	-72.869	40.822	10	10
New York	New York (94728)	73.967	40.783	11	10
Queens	New York (94789)	-73.796	40.655	11	10
Queens	New York (14732)	-73.881	40.779	11	10

4.6 Pavement Structure and Materials Data

A typical flexible pavement in New York State is a full-depth asphalt pavement built with Superpave asphalt mixes. The full depth asphalt flexible pavement structure is divided into three layers:

- Top course layer
- Binder course layer
- Base course layer

These asphalt concrete layers are placed on top of an Asphalt Treated Permeable Base (ATPB) layer. AASHTOWare has no material that simulates an ATPB. Therefore, a crushed stone layer with a resilient modulus of 45,000 psi was used to simulate the ATPB (AASHTO, 2008). The CPDM and NYSDOT specifications were used to assemble the required materials data (NYSDOT, 2008). The mix properties for several asphalt concrete mixes produced and compacted by the NYSDOT Asphalt Laboratory and tested at the University of Texas at Arlington were used to assemble the aggregate gradation data of the asphalt mixtures.

4.6.1 Pavement Structure Layers Thicknesses

Based on the CPDM typical section, the structural layer thicknesses were assembled. As previously mentioned, varied HMA and select granular subgrade thicknesses were used to develop the design cases. The minimum allowed thickness by AASHTOWare is 1.0 inch. Thus, the minimum Asphalt Concrete (AC) thickness was 3.0 inches. The thickness of the asphalt concrete base was gradually increased in 0.5-inch increments to obtain the design solutions. This increase was only applied to the base course layer; the surface and the binder layers have a fixed thickness. The assembling of the structure layers thicknesses for all the design cases was done as follows:

- *Asphalt Concrete (AC) surface layer:*
  - The thickness was selected as 1.0 in. when total AC thickness was less than 5.0 in.
  - The thickness was selected as 1.25 in. when total AC thickness was greater than or equal to 6.0 in.

- *Asphalt Concrete (AC) binder layer:*
  - The thickness was selected as 1.0 in. when total AC thickness was less than 5.0 in.
  - The thickness was selected as 2.0 in. when total AC thickness was greater than or equal to 6.0 in.
- *Asphalt Concrete (AC) base layer*: The HMA thickness was gradually increased or decreased in 0.5-inch increments to reach the satisfied pavement structure layer thickness. In the initial step of this study, the CPDM tables given in Table 2.2 were used as a reference to calculate the base course thickness. Then, the base course layer thickness was increased and decreased from these values.
- *Asphalt Treated Permeable Base (ATPB) layer*: The selected thickness was 4.0 inches (Figure 2.1).
- *Subbase course layer*: The selected thickness was 12.0 inches (Figure 2.1).
- *Select granular subgrade layer*: The thickness was varied from 0.0 to 12.0 inches in 6.0-inch increments to obtain the optimized structure thickness.
- *Subgrade soil layer:* The thickness was assumed to be semi-infinite.

4.6.2 Asphalt Concrete Volumetric Properties

The CPDM and NYSDOT specifications do not provide exact volumetric properties for the asphalt mixes since they vary from project to project. Because of this, the recommended volumetric properties inputs by AASHTO were used. The volumetric properties given in Figure 4.3 for the *Air Void Content, Asphalt Content, Unit Weight, and Poisson's Ratio* were used; these are the values recommended for Level 3 design input.

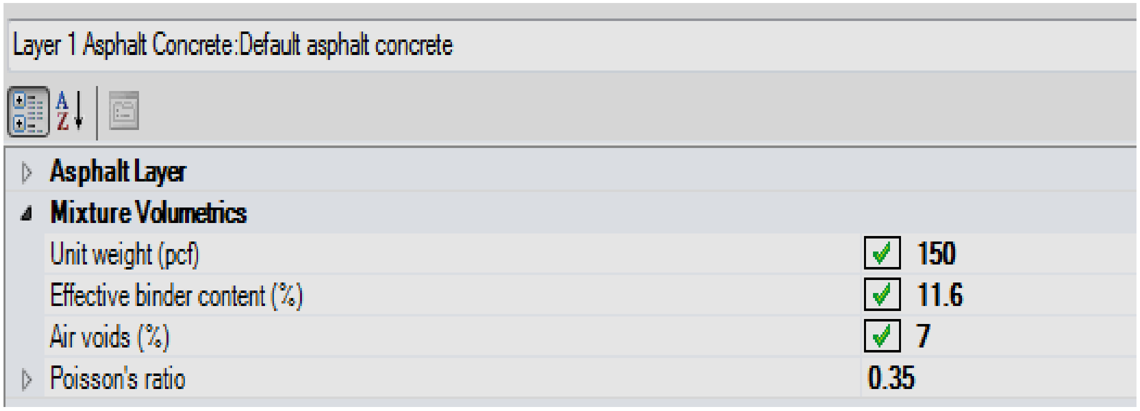


Figure 4.3: Asphalt Concrete Volumetric Properties

4.6.3 Asphalt Concrete Mechanical and Thermal Properties

Since NYSDOT does not have a database of dynamic modulus test results and could not provide Dynamic Shear Rheometer (DSR) test results for representative asphalt binders used in the state, specifically (AASHTO, 2008), Level 3 design inputs were used. They are:

- *Hot Mix Asphalt (HMA) Aggregate Gradation:* The HMA aggregate gradations were not mentioned in the CPDM so the NYSDOT mix design specifications and the aggregate gradation (summarized in Table 4.6) for the asphalt concrete samples tested at the University of Texas at Arlington were used to obtain the information:
    - § Asphalt concrete surface course: NMAS 12.5 mm or 9.5 mm is recommended.
    - § Asphalt concrete binder course: NMAS 19 mm or 25 mm is recommended.
    - § Asphalt concrete base course: NMAS 19 mm or 25 mm is recommended.
- *Select HMA E* Predictive Model:* Since the SuperPave asphalt mixture was used, the shear modulus of the asphalt binders (G*) were used for the equation to predict the dynamic modulus, as shown in Figure 4.4.
- *Reference Temperature:* Since the CPDM and NYSDOT specifications did not mention it, 70 °F, as recommended by AASHTO, was used. Figure

4.4 illustrates the reference temperature value in AASHTOWare. It defines the baseline temperature that is used in deriving the dynamic modulus master curve (AASHTO, 2008).

- *Asphalt Binder:* Once the Level 3 dynamic modulus was selected, AASHTOWare automatically defines same input level for the binder properties. Therefore, SuperPave performance grade (PG) was used as input; the values in Table 2.3 were used. The listed PG values in Table 2.3 were according to the AASHTO M 332 binder classification. However, AASHTOWare requires the PG in the form described in AASHTO M 320. Hence, NYSDOT suggested substituting the values in Table 2.3 into AASHTO M 320 classification (NYSDOT, 2014b). Table 4.7 compares the PG values according to AASHTO M 332 and 320.
- *Indirect Tensile Strength:* There is no recommended value by the CPDM and NYSDOT specifications. For this reason, Level 3 input was used, as shown in Figure 4.4.
- *Creep Compliance:* Level 3 inputs were used for the creep compliance at 4 °F, 14 °F, and 32 °F due to unavailability of measured creep compliance values, as given in Figure 4.5. AASHTOWare automatically calculates the creep compliance values based on the statistical relationship with other input values (AASHTO, 2008).
- *Thermal Properties:* The default values for the thermal conductivity and heat capacity of the asphalt materials were selected. In addition, the default coefficient of thermal contraction for HMA aggregates was selected. Figure 4.4 shows the inputs related to Thermal Properties.

Table 4.6: HMA Aggregate Gradation for Downstate and Upstate New York

Aggregate Gradation data for Upstate			
Sieve #	**% Passing**	**Layer**	**Nominal Maximum Aggregate Size**
3/4"	100	Top	9.5 mm
3/8"	100		
No. 4	82		
No. 200	4		
3/4"	92	Binder	19 mm
3/8"	67		
No. 4	49		
No. 200	2		
3/4"	86	Base	25 mm
3/8"	67		
No. 4	43		
No. 200	5		
Aggregate Gradation data for Downstate			
Sieve #	**% Passing**	**Layer**	**Nominal Maximum Aggregate Size**
3/4"	100	Top	12.5mm
3/8"	89		
No. 4	60		
No. 200	4		
3/4"	78	Binder	19 mm
3/8"	63		
No. 4	48		
No. 200	5		
3/4"	65	Base	37.5 mm
3/8"	56		
No. 4	34		
No. 200	4		

Table 4.7: NYSDOT Binder Substitution Guidance

AASHTO M 320 PG Binder Grade	AASHTO M 332 PG Binder Grade
PG 64-22	PG 64S-22
PG 70-22	PG 64H-22

Mechanical Properties	
Dynamic modulus	☑ **Input level:3**
Select HMA Estar predictive model	**Use G* based model (nationally uncalibrated).**
Reference temperature (deg F)	☑ **70**
Asphalt binder	☑ **SuperPave:64-22**
Indirect tensile strength at 14 deg F (psi)	☑ **361.14**
Creep compliance (1/psi)	☑ **Input level:3**
Thermal	
Thermal conductivity (BTU/hr-ft-deg F)	☑ **0.67**
Heat capacity (BTU/lb-deg F)	☑ **0.23**
Thermal contraction	**1.301E-05 (calculated)**
Is thermal contraction calculated?	**True**
Mix coefficient of thermal contraction (in./in./deg F)	☐
Aggregate coefficient of thermal contraction (in./in./deg F)	☑ **5E-06**
Voids in Mineral Aggregate (%)	☑ 18.6

Figure 4.4: AC Mechanical and Thermal Properties

Creep compliance level: 3

Loading Time(sec)	Low Temp (-4 deg F)	Mid Temp (14 deg F)	High Temp (32 deg F)
1	2.93692E-07	4.788056E-07	6.546713E-07
2	3.227263E-07	5.589023E-07	8.376009E-07
5	3.655588E-07	6.857071E-07	1.160117E-06
10	4.016979E-07	8.004151E-07	1.484279E-06
20	4.414097E-07	9.34312E-07	1.899019E-06
50	4.999939E-07	1.14629E-06	2.630232E-06
100	5.494233E-07	1.338047E-06	3.365177E-06

Figure 4.5: Input Level 3 Creep Compliance

4.6.4 Aggregate Gradation of Unbound Granular Layers

The granular materials type, AASHTO soil classification, and aggregate gradation were assembled from the NYSDOT specifications. The extracted data of each unbound layer are:

- *Asphalt Treated Permeable (ATPB) Base Layer:* NYSDOT uses the ATPB as base layer; NYSDOT recommended aggregate gradation data is given in Table 4.8. Two types of aggregate gradation are recommended by NYSDOT; the selection is based on the site characteristics. For this research, Type 1 was used since the design cases are hypothetical. It should be noticed in Table 4.8 that all the percentages are based on total weight of aggregate and the asphalt content is based on the total weight of mix. A-1-a AASHTO soil classification was used for this layer because it would resemble ATPB the best (Ahammed et al., 2013).

- *Subbase Course Layer:* It is defined by NYSDOT as any materials that do not consist of concrete, asphalt, glass, brick, stone, sand, gravel, or blast furnace slag. Four types of aggregate gradation are recommended by NYSDOT, as shown in Table 4.9. Type 2 was selected to be used in this research at the recommendation of NYSDOT. According to NYSDOT, Type 2 is defined as furnish materials consisting of approved Blast Furnace Slag or of Stone which is the product of crushing or blasting ledge rock, or a blend of Blast Furnace Slag and of Stone. A-1-a AASHTO soil classification was used for this layer.
- *Select granular subgrade layer:* NYSDOT recommended two options, either using well-graded rock with particles greatest dimension of 12 inches or any other materials except well-graded rock with no particles greater than 6 inches. NYSDOT recommended aggregate gradation is given in Table 4.10. A-1-a AASHTO soil classification was used for this layer.
- *Subgrade soil layer:* It is the natural ground. There is no available information for this layer in the CPDM or the NYSDOT specification. Thus, A-7-6 AASHTO soil classification was used for this layer.

Table 4.8: ATPB Aggregate Gradation

	Permeable Base				Shim	
	Type 1		Type 2		Type 5	
Screen Size	General Limits % Passing	Job Mix Tolerance %	General Limits % Passing	Job Mix Tolerance %	General Limits % Passing	Job Mix Tolerance %
2 in.	100	-	100	-	-	-
1 1/2 in.	95–100	-	75–100	±7	-	-
1 in.	80–95	±6	55–80	±8	-	-
1/2 in.	30–60	±6	23–42	±7	-	-
1/4 in.	10–25	±6	5–20	±6	100	-
1/8 in.	3–15	±6	2–15	±4	80–100	±6
No. 20	-	-	-	-	32–72	±7
No. 40	-	-	-	-	18–52	±7
No. 80	-	-	-	-	7–26	±4
No. 200	0–4	±2	-	-	2–12	±2
Asphalt Content %	2–4	NA	2.5–4.5	NA	7–9.5	NA
Mixing and Placing Temperature Range (F^0)	225–300		225–301		250–325	

Table 4.9: Subbase Course Layer Aggregate Gradation

Sieve Size Designation	Type			
	1	2	3	4
4 in.	-	-	100	-
3 in.	100	-	-	-
2 in.	90–100	100	-	100
1/4 in.	30–65	26–60	30–75	30–65
No. 40	5–40	5–40	5–40	5–40
No. 200	0–10	0–10	0–10	0–10

Table 4.10: Select Granular Subgrade Layer

Sieve Size	Percent Passing by Weight
1/4 in.	30 to 100
No. 40	0 to 50
No. 200	0 to 10

4.6.5 Granular Layers Materials Properties and Design Strategies

The CPDM and NYSDOT specifications were reviewed in order to assemble the input properties for the granular layers. Figure 4.6 shows an example of granular materials properties inputs for the select subgrade layer. Since no available information was found, Level 3 inputs were used for:

- Liquid Limit (LL) and Plasticity Index (PI)
- Maximum unit weight (pcf)
- Saturated hydraulic conductivity (ft/hr)
- Specific gravity of the soil
- Optimum gravimetric water content (%)
- User-Defined Soil Water Characteristic Curve (SWCC)
- Resilient Modulus (M_r)

ATPB was simulated as a crushed stone layer with high quality aggregate according to AASHTO. Resilient modulus (M_r) of ATPB layer was selected to be 45,000 psi. Figure 4.7 shows an example of design input properties for the ATPB layer.

The subbase layer was considered a crushed stone layer. The resilient modulus (M_r) value was estimated based on Figure 4.8 since no specific value is recommended by NYSDOT (AASHTO, 2008). The design inputs and properties of the subbase layer are given in Figure 4.9.

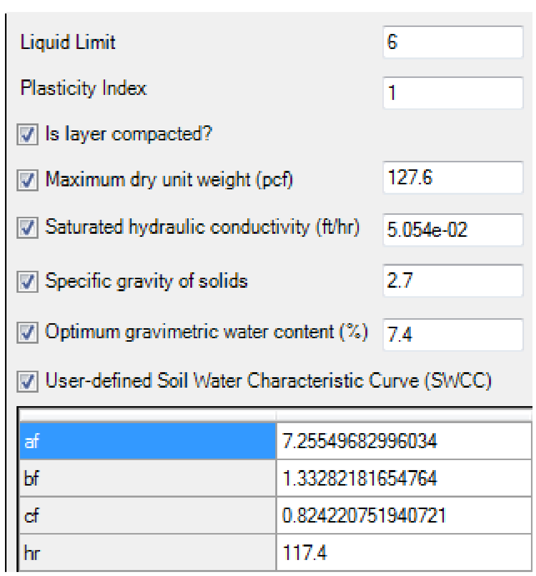


Figure 4.6: Select Granular Subgrade Materials Properties

Unbound	
Layer thickness (in.)	4
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	45000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	Crushed stone
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4.7: Inputs for the ATPB Layer in AASHTOWare

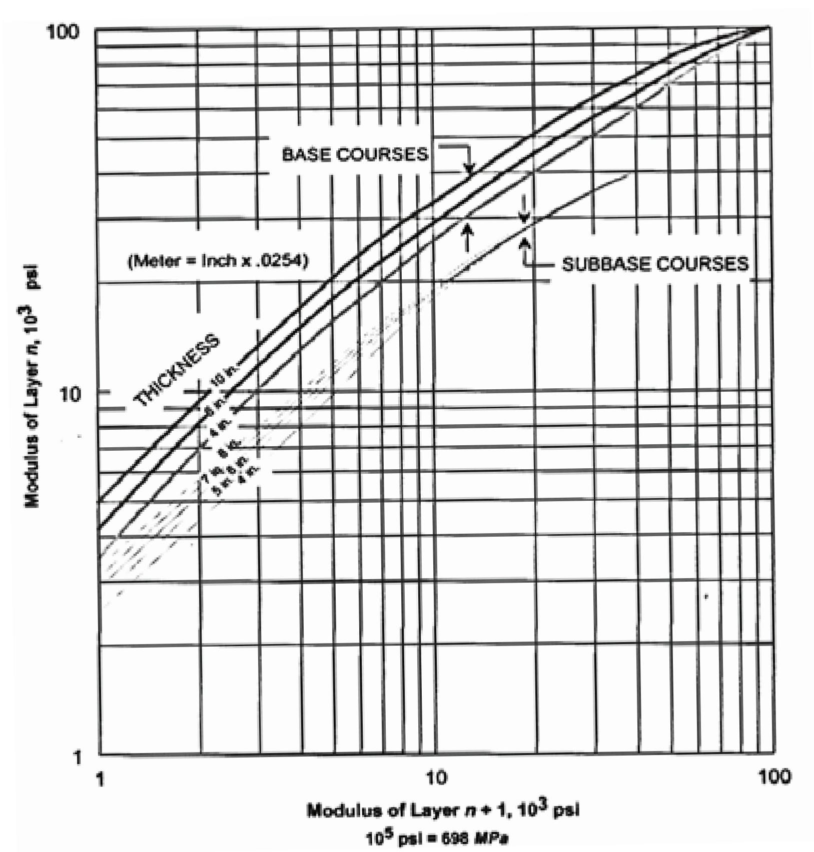


Figure 4.8: Modulus Criteria of Unbound Aggregate Base and Subbase Layers

Unbound	
Layer thickness (in.)	12
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	35000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	Crushed stone
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

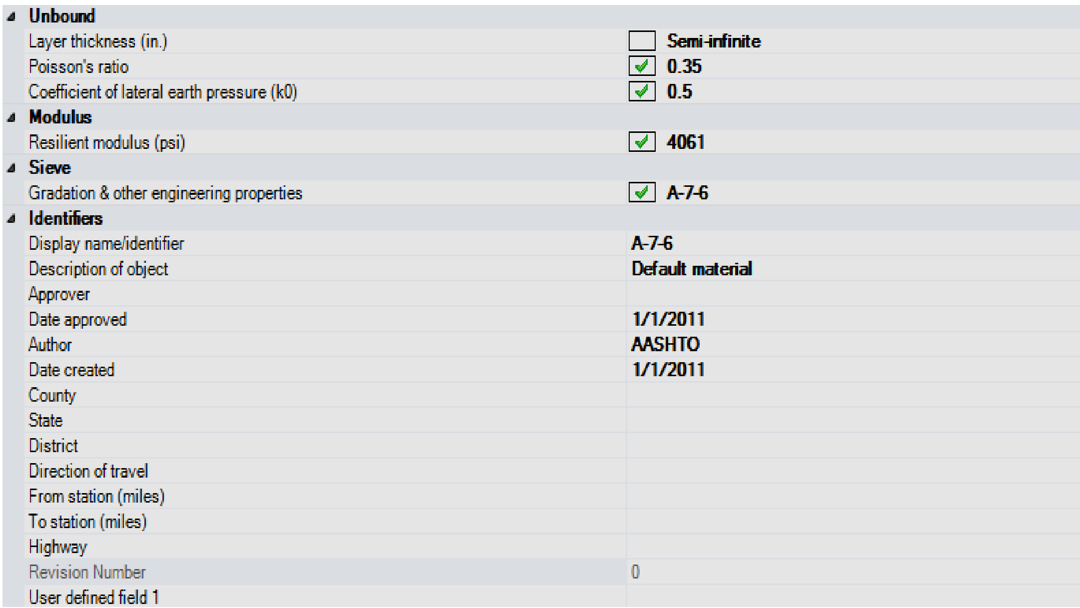
Figure 4.9: Simulated Subbase Course Layer in AASHTOWare

Unbound	
Layer thickness (in.)	6
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	25000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	A-1-a
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4.10: Simulated Select Granular Subgrade Soil Layer in AASHTOWare

The select granular subgrade layer material was considered an A-1-a soil. The design inputs and properties of the select subgrade layer are given in Figure 4.10. AASHTO recommended that the resilient modulus of the top granular layer should not exceed three times the resilient modulus of the bottom layer. Figure 4.10 illustrates the materials input and properties of select granular subgrade layer.

Subgrade soil layer was considered an A-7-6 soil with resilient modulus values varying from 28 to 62 MPa (4 to 9 ksi) to obtain design tables similar to those in the CPDM. It is important to mention that AASHTOWare does not consider the frost susceptibility of the subgrade soil. Figure 4.11 gives the materials inputs and properties for subgrade soil layer. Table 2.2 was used as a reference to input the resilient modulus for this layer.



Unbound	
Layer thickness (in.)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	4061
Sieve	
Gradation & other engineering properties	A-7-6
Identifiers	
Display name/identifier	A-7-6
Description of object	Default material
Approver	
Date approved	1/1/2011
Author	AASHTO
Date created	1/1/2011
County	
State	
District	
Direction of travel	
From station (miles)	
To station (miles)	
Highway	
Revision Number	0
User defined field 1	

Figure 4.11: Simulated Subgrade Soil Layer in AASHTOWare

4.7 Distress Models

In AASHTOWare, the JULEA linear-elastic multi-layer model computes the response of the pavement under traffic load throughout the design period. Then, the pavement response values are used to calculate the evolution of pavement distresses during the same design period. These distress models, also called performance models, were globally calibrated using a large set of the LTPP data for the national calibration. However, the distress models must be calibrated for local or regional conditions.

Momin (2011) successfully calibrated the distress models incorporated in the MEPDG 1.1 pavement design software for the Northeast (NE) region of the Unites States. The calibration factors he obtained are listed in Table 4.10. However, AASHTOWare Pavement ME 2.1 is the latest release of the Pavement ME Design computer software and it is the only version currently available to the public. Since this version of the software is greatly improved, the model calibration was repeated for the AASHTOWare software; the detailed description of the calibration of AASHTOWare distress models has been given in Chapter 3. The set of calibration coefficients developed in this study for the AASHTOWare software and the calibration coefficients developed by Momin for the MEPDG 1.1 software are given in Table 4.11; they are different. In this research, the AASHTOWare local calibration factors were used to develop the design solutions.

Table 4.11: Calibration Coefficients Used for the Flexible Pavement Performance Models

Distress	Layer	Coefficient	Momin's Study (MEPDG)	National	Obtained in this study (AASHTOWare)
Permanent Deformation	HMA	$ß_{r1}$	1.308	1	0.59
		$ß_{r2}$	1	1	1
		$ß_{r3}$	1	1	1
	Base	$ßr_{GB}$	2.0654	1	0.82
	Subgrade	$ßr_{SG}$	1.481	1	0.74
Alligator Cracking	HMA	C_1	-0.06883	1	0.501711
		C_2	1.27706	1	0.227186
Longitudinal Cracking	HMA	C1	-1	7	7
		C2	2	3.5	3.5
		C3	1856	1000	1000
IRI	HMA	C1	51.6469	40	168.709
		C2	0.000218	0.4	-0.0238
		C3	0.0081	0.008	0.00017
		C4	-0.9351	0.015	0.015

Chapter 5: Development of Design Tables for New Flexible Pavement Structures based on AASHTOWare Pavement 2.1

5.1 Overall Concept for Developing the Design Tables

The design solutions were developed by running several thousand AASHTOWare design cases. For each case run, the computed distresses were extracted using a macro in Microsoft Excel and were tabulated in Excel spreadsheet files. The selection of the successful design solutions was based on the following design criteria:

- IRI = 225 in/mile
- Total Rutting = 0.75 in.

The successful design solutions were determined, for each subgrade soil resilient modulus (M_r) and AADTT combination, as the run cases with the minimum select granular subgrade thickness and total asphalt layer thickness for which the IRI and total rutting were less than corresponding design criteria (225 in/mile for IRI and 0.75-inch total rut depth). It was found that the IRI design criteria (225 in/mile) was almost always reached before the design criteria for total rutting (0.75 inches) was reached. This is in agreement with NYSDOT practice of using the IRI as the trigger value for deciding when a distressed flexible pavement must be rehabilitated with an overlay.

Because of the very large number of cases run, the results of the runs are available only in electronic form. The electronic file can be obtained from the authors. These design solutions were then assembled in separate design tables for each region, in a format similar to that in Table 1.2. The new design tables are given in Appendix F. The first digit (before the "/" sign) is the total thickness of all asphalt concrete layers while the second number (after the "/" sign) is the design thickness for the select granular subgrade layer. US customary units were used for layer thickness at the request of NYSDOT.

The design tables given in Appendix F are for 15-year and 20-year design lives. However, only the design thicknesses for the 15-year design life are discussed further.

For some NYSDOT regions, more than one design table was obtained because more than one climatic station exists in that region, as indicated in Table 4.5. It was thus possible to compare the design tables for locations within the same NYSDOT region. A comparison was also made between the design tables obtained for the Upstate and the Downstate parts of New York State.

It was also important to compare the new design tables derived with the locally calibrated AASHTOWare 2.1 models with the CPDM design table reproduced in Table 5.16. To allow the comparison, the AADTT values were converted into equivalent ESALs values and were added to the design tables. This conversion was possible since the statewide average values of the traffic volume characteristics and axle load spectra for 2010 were used for the AASHTOWare runs.

5.2 Design Tables for Upstate New York

The design tables were developed for 14 locations in the Upstate part of the state, as shown in Table 5.1; all regions have at least one location.

Table 5.1: Climatic Stations in Upstate New York

County	Station ID	Region
Saratoga	Albany (14735)	1
Warren	Glens Falls (14750)	1
Oneida	Utica (94794)	2
Onondaga	Syracuse (14771)	3
Monroe	Rochester (14768)	4
Erie	Buffalo (14733)	5
Chautauqua	Dunkirk (14747)	5
Niagara	Niagara Falls (04724)	5
Steuben	Dansville (94704)	6
Chemung	Elmira/Corning (14748)	6
Allegany	Wellsville (54757)	6
St. Lawrence	Massena (94725)	7
Clinton	Plattsburgh (94733)	7
Jefferson	Watertown (94790)	7

5.2.1 Comparison of Design Tables for Region 1

In order to identify difference in the weather data among the studied locations in Region 1, the annual climate statistics are given in Table 5.2. To ease the comparison, the design tables for Region 1 are given in Table 5.3. The table suggests that, in general, for the same subgrade soil resilient modulus (M_r) and AADTT, the design solutions of Regions 1 are similar.

Table 5.2: Region 1 Annual Statistics Climate Records

Region 1 Climatic Station	Albany	Glens Falls
Mean annual air temperature (°F)	48.88	44.8
Mean annual precipitation (in.)	35.53	37.27
Freezing Index (°F-days)	1436.7	2667.9
Average annual number of freeze/thaw cycles	68.35	88.9

Table 5.3: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 1

AADTT	Albany	Glens Falls	Albany	Glens Falls	Albany	Glens Falls
	Mr = 4 ksi		Mr = 5 ksi		Mr = 6 ksi	
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	4 / 0	4.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	6 / 0	6.5 / 0	5 / 0	5 / 0	4 / 0	4.5 / 0
500	8.5 / 0	8.5 / 6	7 / 0	7 / 0	6 / 0	6 / 0
1,000	10.5 / 6	10.5 / 6	9.5 / 6	9.5 / 6	8 / 0	8 / 0
2,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6	11 / 6	11 / 6
4,000	14 / 6	14.5 / 6	13.5 / 6	13.5 / 6	13 / 6	13 / 6
5,000	15 / 6	15 / 6	14 / 6	14 / 6	13.5 / 6	13.5 / 6
	Mr = 7 ksi		Mr = 8ksi		Mr = 9 ksi	
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	5.5 / 0	5.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0
1,000	7 / 0	7.5 / 0	6.5 / 0	6.5 / 0	6 / 0	6 / 0
2,000	10.5 / 6	10 / 0	9 / 0	9 / 0	9 / 0	8.5 / 0
4,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6	12 / 0	12 / 0
5,000	13 / 6	13 / 6	13 / 6	13 / 6	12.5 / 6	12.5 / 6

5.2.2 Comparison of Design Tables for Region 5

In order to identify difference in the weather data among the studied locations in Region 5, the annual climate statistics are given in Table 5.4. To ease the comparison, the design tables for Region 5 are given in Table 5.5. The table suggests that, in general, the design solutions of Region 5 for the same subgrade soil resilient modulus (M_r) and AADTT are different. Few design solutions are found identical at low AADTT and stiffer soil.

Table 5.4: Annual Climate Statistics for Three Locations in Region 5

Region 5 Climatic Station	Buffalo	Dunkirk	Niagara Falls
Mean annual air temperature (°F)	48.71	49.65	47.43
Mean annual precipitation (in.)	37.62	34.59	31.1
Freezing Index (°F-days)	1279.9	1099.5	1723.1
Average annual number of freeze/thaw cycles	47.36	55.98	52.94

Table 5.5: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 5

AADTT	Buffalo	Dunkirk	Niagara Falls	Buffalo	Dunkirk	Niagara Falls	Buffalo	Dunkirk	Niagara Falls
	Mr = 4 ksi			Mr = 5 ksi			Mr = 6 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0	3.5 / 0
500	7 / 0	7.5 / 0	7 / 0	6 / 0	6.5 / 0	6.5 / 0	5 / 0	5 / 0	4.5 / 0
1,000	8.5 / 6	9.5 / 6	9 / 6	8 / 0	8.5 / 0	8 / 0	7 / 0	7.5 / 0	6.5 / 0
2,000	11 / 6	12 / 6	11 / 6	10 / 0	11 / 6	10 / 6	9 / 0	10 / 0	9 / 0
4,000	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	13 / 6	12 / 6	11.5 / 6	12 / 6	11 / 6
5,000	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	13 / 6	11.5 / 6
	Mr = 7 ksi			Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4 / 0	4.5 / 0	5 / 0	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	6.5 / 0	6 / 0	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
2,000	8 / 0	9 / 0	8 / 0	7.5 / 0	8.5 / 0	7 / 0	6.5 / 0	8 / 0	6.5 / 0
4,000	11 / 6	11.5 / 6	11 / 6	10 / 6	11 / 6	9.5 / 6	10 / 0	11 / 0	9.5 / 0
5,000	11.5 / 6	12.5 / 6	11.5 / 6	11 / 6	12 / 6	10.5 / 6	10 / 6	11.5 / 6	10 / 6

5.2.3 Comparison of Design Tables for Region 6

The difference in weather data among Region 6 locations was identified by listing the annual climate statistics as shown in Table 5.6. To fulfill the comparison, the design tables of Region 6 locations are listed in Table 5.7. Overall, the design solutions are varied though a few design solution are identical for the same M_r and AADTT values.

Table 5.6: Annual Climate Statistics for Three Locations in Region 6

Region 6 Climatic Station	Dansville	Elmira	Wellsville
Mean annual air temperature (°F)	49.14	47.33	45.13
Mean annual precipitation (in.)	30.24	31.54	35.87
Freezing Index (°F-days)	1309.3	1611.9	2014.5
Average annual of freeze/thaw cycles	67.97	87.81	55.98

Table 5.7: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 6

AADTT	Dansville	Elmira	Wellsville	Dansville	Elmira	Wellsville
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	4 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5.5 / 0	6 / 0	4.5 / 0	4.5 / 0	4.5 / 0
500	8 / 0	7.5 / 0	7.5 / 0	6.5 / 0	6.5 / 0	6 / 0
1,000	9.5 / 6	9.5 / 6	9 / 6	9 / 0	8 / 6	8 / 6
2,000	12 / 6	11.5 / 6	11.5 / 6	11 / 6	11 / 6	10.5 / 6
4,000	13.5 / 6	13.5 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6
5,000	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	4 / 0	3 / 0	3.5 / 0	3.5 / 0
500	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0	6 / 0
2,000	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9 / 0	8.5 / 0
4,000	12.5 / 6	12 / 6	11.5 / 6	12 / 6	11.5 / 6	11 / 6
5,000	13 / 6	12.5 / 6	12 / 6	12.5 / 6	12 / 6	11.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	5 / 0
2,000	9 / 0	8 / 0	8 / 0	8 / 0	7.5 / 0	7 / 0
4,000	11.5 / 6	11 / 6	10.5 / 6	11 / 0	10.5 / 0	10 / 0
5,000	12 / 6	11.5 / 6	11 / 6	12 / 6	11.5 / 6	11 / 6

5.2.4 Comparison of Design Tables for Region 7

For the three locations in Region 7, the weather data is given in Table 5.8 while the design tables are assembled in Table 5.9. In general, the design solutions are very close; the difference in the total thickness of the asphalt layers is less than 1.0 inch.

Table 5.8: Annual Climate Statistics for Three Locations in Region 7

Region 7 Climatic Station	Massena	Plattsburgh	Watertown
Mean annual air temperature (°F)	44.06	44.92	46.03
Mean annual precipitation (in.)	32.8	29.27	33.36
Freezing Index (°F-days)	2866.4	2471.7	2208
Average annual of freeze/thaw cycles	71.95	74.78	71.7

Table 5.9: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 7

AADTT	Massena	Plattsburgh	Watertown	Massena	Plattsburgh	Watertown
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	4 / 0	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
250	6 / 0	6 / 0	6 / 0	5 / 0	5 / 0	5 / 0
500	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	6.5 / 0
1,000	10 / 6	10 / 6	9.5 / 6	9 / 0	9 / 0	9 / 0
2,000	12 / 6	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6
4,000	14 / 6	14 / 6	13.5 / 6	13 / 6	13 / 6	13 / 6
5,000	14 / 6	14 / 6	14 / 6	13.5 / 6	13.5 / 6	13 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4.5 / 0	4.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
500	6 / 0	6 / 0	5.5 / 0	5 / 0	5 / 0	4.5 / 0
1,000	8 / 0	8 / 0	8 / 0	7 / 0	7 / 0	7 / 0
2,000	10 / 6	10 / 6	10 / 0	9.5 / 0	9.5 / 0	9.5 / 0
4,000	12.5 / 6	12.5 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	4.5 / 0	4 / 0	4 / 0	3.5 / 0
1,000	6 / 0	6 / 0	6 / 0	6 / 0	6 / 0	5 / 0
2,000	9 / 0	9 / 0	8.5 / 0	8 / 0	8 / 0	8 / 0
4,000	11.5 / 6	11.5 / 6	11 / 6	11.5 / 0	11.5 / 0	11 / 0
5,000	12 / 6	12 / 6	12 / 6	12 / 6	12 / 6	11.5 / 6

5.3 Design Tables for Downstate New York

The design tables were developed for the listed Downstate New York regions and climatic stations as shown in Table 5.10. It is important to mention that a virtual climatic station was created for Region 9 since no data were available from a weather station in that region (see Table 4.5).

Table 5.10: Climatic Stations in Downstate New York

County	Station ID	Region
Orange	Montgomery (04789)	8
Dutchess	Poughkeepsie (14757)	8
Westchester	White Plains (94745)	8
Virtual Climatic Station	Combination of Climatic Stations	9
Nassau	Farmingdale (54787)	10
Suffolk	Islip (04781)	10
Suffolk	Shirley (54790)	10
New York	New York City (94728)	11
Queens	New York City (94789)	11
Queens	New York City (14732)	11

5.3.1 Comparison of Design Tables for Region 8

The weather data of Region 8 locations are listed in Table 5.11 to show the climate variation. The comparison was conducted by listing the design tables as shown in Table 5.12. It can be observed that the change in locations affected the design thicknesses for high traffic volumes regardless of the stiffness of the subgrade soil M_r. At low AADTT, the design solutions were identical in general.

Table 5.11: Annual Climate Statistics for Three Locations in Region 8

Region 8 Climatic Station	Montgomery	Poughkeepsie	White Plains
Mean annual air temperature (°F)	49.43	50.42	51.26
Mean annual precipitation (in.)	38.2	40.96	94.17
Freezing Index (°F-days)	1274.8	1191.4	852.4
Average annual of freeze/thaw cycles	89.81	86.94	55.96

Table 5.12: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 8

AADTT	Montgomery	Poughkeepsie	White Plains	Montgomery	Poughkeepsie	White Plains
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	4 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	6 / 0	6 / 0	5.5 / 0	4.5 / 0	5 / 0	4.5 / 0
500	7.5 / 0	8 / 0	7 / 0	6.5 / 0	7 / 0	6 / 0
1,000	9.5 / 6	10 / 6	9 / 6	9 / 0	9 / 6	8 / 6
2,000	12 / 6	12 / 6	11 / 6	11 / 6	11 / 6	10 / 6
4,000	13.5 / 6	14 / 6	13 / 6	13 / 6	12.5 / 6	12 / 6
5,000	14 / 6	14.5 / 6	13.5 / 6	13.5 / 6	13 / 6	12.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	4 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0
500	5 / 0	5.5 / 0	5 / 0	4.5 / 0	5 / 0	4.5 / 0
1,000	7.5 / 0	8 / 0	7 / 0	7 / 0	7 / 0	6 / 0
2,000	10.5 / 6	10 / 6	9 / 6	9.5 / 0	9.5 / 6	8 / 6
4,000	12.5 / 6	12.5 / 6	11.5 / 6	12 / 6	12 / 6	11.5 / 6
5,000	13 / 6	13 / 6	12 / 6	12.5 / 6	12.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	5 / 0	4.5 / 0	4 / 0	5 / 0	4 / 0	4 / 0
1,000	6 / 0	6 / 0	5 / 0	5.5 / 0	6 / 0	5 / 0
2,000	8.5 / 0	9 / 0	7.5 / 0	9.5 / 0	8.5 / 0	7 / 0
4,000	11.5 / 6	11.5 / 6	10 / 6	11 / 0	11 / 0	9.5 / 0
5,000	12 / 6	12 / 6	11 / 6	11.5 / 6	12 / 6	10.5 / 6

5.3.2 Comparison of Design Tables for Region 10

For the three studied locations in Region 10, the annual average climatic statistical indicators are listed in Table 5.13, while the design tables are shown in Table 5.14. It is clear there are variations due to location change, but the difference in total HMA thickness is less than 1.0 inch.

Table 5.13: Annual Climate Statistics for Three Locations in Region 10

Region 10 Climatic Station	Farmingdale	Islip	Shirley
Mean annual air temperature (°F)	52.72	52.2	51.97
Mean annual precipitation (in.)	39.22	39.18	42.09
Freezing Index (°F-days)	637.686	672.3	702.414
Average annual of freeze/thaw cycles	52.18	64.17	73.17

Table 5.14: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 10

AADTT	Farmingdale	Islip	Shirley	Farmingdale	Islip	Shirley
	Mr = 4 ksi			Mr = 5 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3.5 / 0	3.5 / 0	3 / 0	3 / 0	3 / 0
250	5 / 0	5 / 0	5 / 0	4 / 0	4 / 0	4.5 / 0
500	7 / 0	7 / 0	7.5 / 0	5.5 / 0	5.5 / 0	6 / 0
1,000	8.5 / 6	8.5 / 6	9.5 / 6	8.5 / 0	8 / 0	8 / 0
2,000	11 / 6	11 / 6	11.5 / 6	10 / 6	10 / 6	10.5 / 6
4,000	13 / 6	13 / 6	13.5 / 6	12.5 / 6	12.5 / 6	12.5 / 6
5,000	13.5 / 6	13.5 / 6	14 / 6	13 / 6	13 / 6	13.5 / 6
	Mr = 6 ksi			Mr = 7 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3.5 / 0	3.5 / 0	4.5 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4.5 / 0	6 / 0	4.5 / 0	4.5 / 0	5 / 0
1,000	7 / 0	7 / 0	8 / 0	6 / 0	6 / 0	6 / 0
2,000	9.5 / 6	9 / 6	10.5 / 6	8 / 0	8 / 0	9 / 6
4,000	11.5 / 6	11.5 / 6	12.5 / 6	11 / 6	11 / 6	11.5 / 6
5,000	12 / 6	12 / 6	13.5 / 6	11.5 / 6	11.5 / 6	12 / 6
	Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	3.5 / 0	3.5 / 0	4 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	5 / 0	5 / 0	5 / 0	4.5 / 0	4.5 / 0	4.5 / 0
2,000	7.5 / 0	7.5 / 0	8 / 0	6.5 / 0	6.5 / 0	7 / 0
4,000	10.5 / 6	10.5 / 6	11 / 6	9.5 / 0	9.5 / 0	10 / 0
5,000	11 / 6	11 / 6	11.5 / 6	10.5 / 6	10.5 / 6	11.5 / 6

5.3.3 Comparison of Design Tables for Region 11

The differences in weather data of Region 11 locations are indicated by the annual statistics data shown in Table 5.15. To facilitate the comparison, the design tables of Region 11 locations are listed in Table 5.16. The listed values show the majority of the design solutions are dissimilar due to the climate variations.

Table 5.15: Annual Climate Statistics for Three Locations in Region 11

Region 11 Climatic Station	NYC 94728	NYC 94789	NYC 14723
Mean annual air temperature (°F)	55.01	54.14	55.61
Mean annual precipitation (in.)	44.39	39.58	42.39
Freezing Index (°F-days)	429.48	429.444	384.084
Average annual of freeze/thaw cycles	31.86	41.74	29.24

Table 5.16: Design Thickness (in.) of HMA and Select Granular Subgrade Layers – Region 11

AADTT	NYC 94728	NYC 94789	NYC 14732	NYC 94728	NYC 94789	NYC 14732	NYC 94728	NYC 94789	NYC 14732
	Mr = 4 ksi			Mr = 5 ksi			Mr = 6 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3.5 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	5.5 / 0	5 / 0	5 / 0	4.5 / 0	4 / 0	4 / 0	3.5 / 0	3 / 0	3.5 / 0
500	8 / 0	6.5 / 0	7 / 0	6.5 / 0	5 / 0	5.5 / 0	5 / 0	4.5 / 0	4.5 / 0
1,000	9.5 / 6	8.5 / 6	8.5 / 6	9 / 0	7.5 / 0	8 / 0	8 / 0	6.5 / 0	6.5 / 0
2,000	12.5 / 6	11 / 6	11 / 6	11.5 / 6	9.5 / 6	10 / 6	10.5 / 6	8.5 / 6	9 / 6
4,000	14 / 6	12.5 / 6	13 / 6	13.5 / 6	12 / 6	12 / 6	13 / 6	11 / 6	11.5 / 6
5,000	14 / 12	13.5 / 6	13.5 / 6	14 / 6	12.5 / 6	13 / 6	13.5 / 6	11.5 / 6	12 / 6
	Mr = 7 ksi			Mr = 8 ksi			Mr = 9 ksi		
50	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
100	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
250	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0	3 / 0
500	4.5 / 0	4 / 0	4 / 0	4.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0	3.5 / 0
1,000	7 / 0	5.5 / 0	5.5 / 0	6 / 0	4.5 / 0	5 / 0	5.5 / 0	4.5 / 0	4.5 / 0
2,000	10 / 6	7.5 / 6	8 / 6	9 / 0	7 / 0	7.5 / 0	8.5 / 0	6 / 0	6.5 / 0
4,000	12.5 / 6	10 / 6	10.5 / 6	12 / 6	9.5 / 6	10 / 6	11.5 / 0	9 / 0	9.5 / 0
5,000	13 / 6	11 / 6	11.5 / 6	12.5 / 6	10 / 6	11 / 6	12 / 6	10 / 6	10.5 / 6

5.4 Comparison between the Design Tables for Upstate and Downstate New York

Because of differences in the aggregate gradation and PG binder grades used for HMA mixes and the climatic conditions in the two parts of the state, it was expected that some differences in design solutions may exist. Tables 5.3, 5.5, 5.8, and Tables 5.11, 5.13, and 5.15

indicate that, at low AADTT, the corresponding design solutions are the same for the Upstate and Downstate regardless of the subgrade soil. However, at high AADTT and for soft subgrade soil, the design solutions for the same traffic and subgrade soil are thicker for the Upstate than for the Downstate.

5.5 Comparison of ME and CPDM Design Tables

To facilitate the comparison, the design solutions of Table 2.2 were converted into US customary units as shown in Table 5.17. The AADTT values were converted into equivalent ESALs values and were added to the design tables in Appendix F. Figures 5.1, 5.2, 5.3, 5.4, 5.5, and 5.6 show the design thickness for the asphalt layer in the CPDM (Solid Line) and the newly developed tables (X-Y Scatters) for different traffic volumes and for a subgrade layer resilient modulus.

The comparison reveals that, for low traffic volumes, the design asphalt layer thickness in the CPDM table is greater than the corresponding thickness in the newly developed tables, mainly because the CPDM thicknesses for low traffic volumes were not calculated but recommended minimum. The difference in the design solutions from the two methods are due to the fact that the two methods use different design criteria and design inputs and they rely on different principles and assumptions. However, it is recommended that a minimum of 6.0 inches be required for the total thickness of asphalt concrete layers because the minimum total thickness used by NYSDOT for the past 25 years has been 6.5 inches (Table 5.17). This minimum required thickness is also needed to reduce the potential for pavement failure caused by occasional overloaded vehicles that may use the road.

At medium traffic volume (500 and 1,000 AADTT), the design thickness for the asphalt layers are about the same in both tables. For high traffic volumes, the design total asphalt layer thickness in the newly developed tables is greater than the corresponding total thickness in the CPDM table. However, the thickness of the select granular subgrade layer is much less in the newly developed design tables. It was decided to use thinner granular select subgrade layer, because it is more economical to increase the thickness of asphalt concrete base and reduce the thickness of the select granular subgrade layer. The equivalency in terms of contribution to pavement performance is about 1.0 inch HMA to 4.0 inches of select granular subgrade but, according to the NYSDOT PMS unit, 1.0 inch of HMA costs about the same as 3.0 inches of select granular subgrade material.

Table 5.17: Design Layer Thicknesses in CPDM in US Customary Units

CPDM for Mr=4 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 2	6.6	0
2 < ESALs <= 4	7	0
4 < ESALs <= 8	8	0
8 < ESALs <= 13	9	0
13 < ESALs <= 23	10	0
23 < ESALs <= 45	10	6
45 < ESALs <= 80	10	12
80 < ESALs <= 140	10	18
140 < ESALs <= 300	10	18

CPDM for Mr=5 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 4	6.6	0
4 < ESALs <= 7	7	0
7 < ESALs <= 13	8	0
13 < ESALs <= 23	9	0
23 < ESALs <= 40	10	0
40 < ESALs <= 70	10	6
70 < ESALs <= 130	10	12
130 < ESALs <= 235	10	18
235 < ESALs <= 300	10	18

CPDM for Mr=6 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 6	6.6	0
6 < ESALs <= 11	7	0
11 < ESALs <= 20	8	0
20 < ESALs <= 35	9	0
35 < ESALs <= 60	10	0
60 < ESALs <= 110	10	6
110 < ESALs <= 200	10	12
200 < ESALs <= 300	10	18

CPDM for Mr=7 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 8	6.6	0
8 < ESALs <= 16	7	0
16 < ESALs <= 30	8	0
30 < ESALs <= 50	9	0
50 < ESALs <= 85	10	0
85 < ESALs <= 160	10	6
160 < ESALs <= 300	10	12

CPDM for Mr=8 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 12	6.6	0
12 < ESALs <= 20	7	0
20 < ESALs <= 40	8	0
40 < ESALs <= 65	9	0
65 < ESALs <= 115	10	0
115 < ESALs <= 215	10	6
215 < ESALs <= 300	10	12

CPDM for Mr=9 ksi		
ESALs (million)	HMA Thickness	Select Granular Subgrade Thickness
ESALs <= 15	6.6	0
15 < ESALs <= 30	7	0
30 < ESALs <= 50	8	0
50 < ESALs <= 90	9	0
90 < ESALs <= 150	10	0
150 < ESALs <= 300	10	6

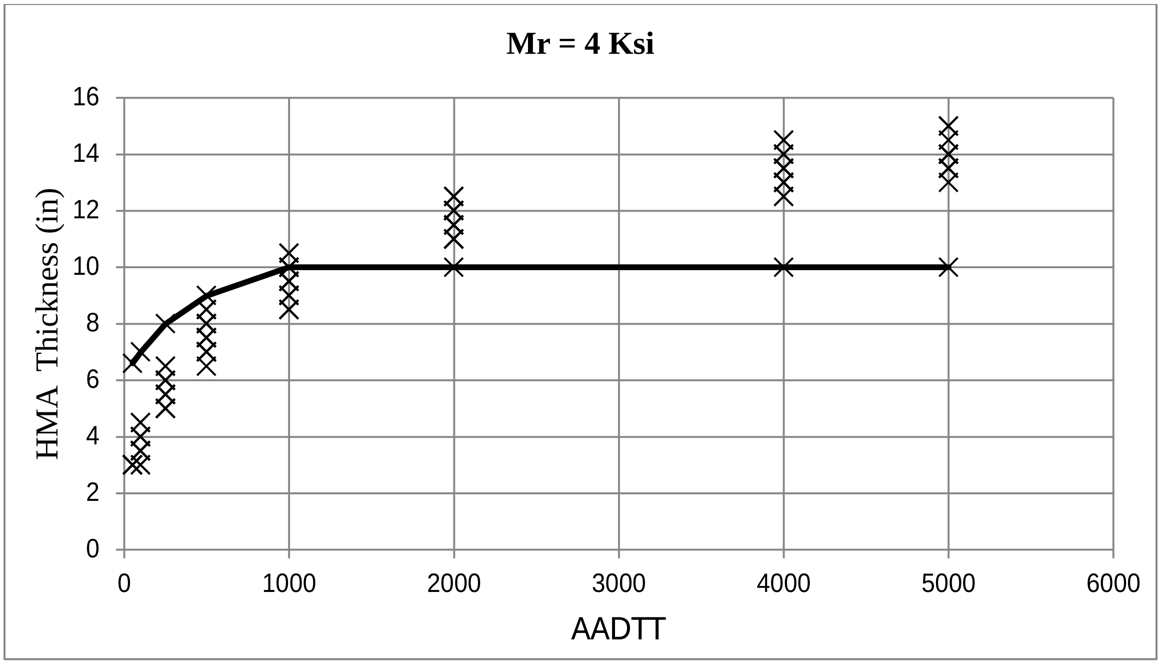


Figure 5.1: AADTT versus HMA Thickness (in.) for M_r =4 ksi

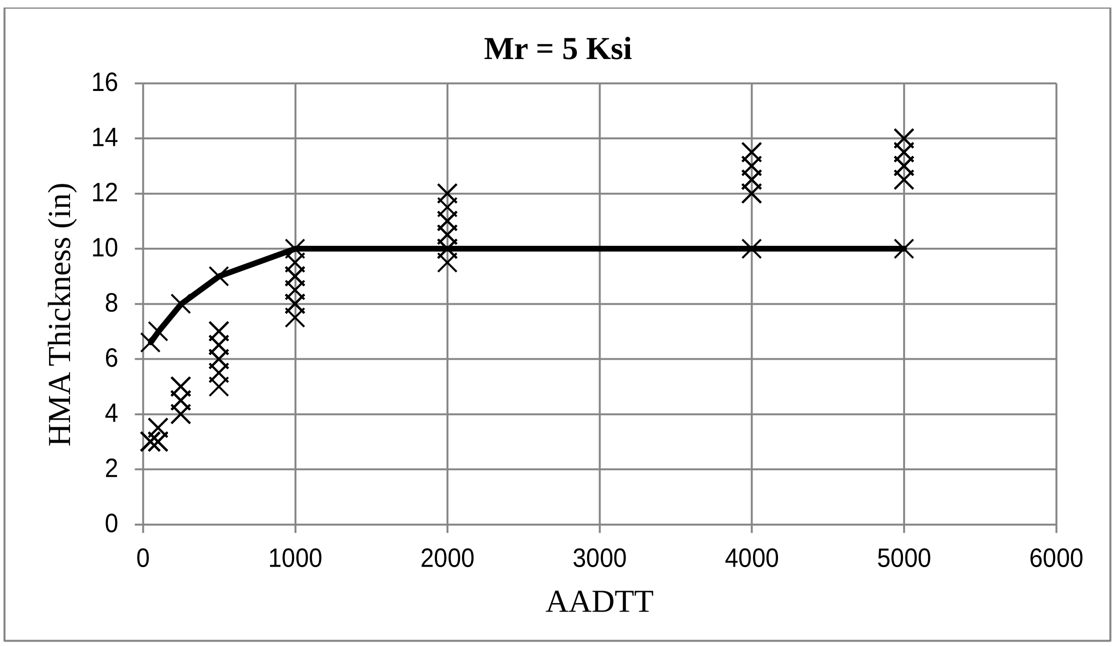


Figure 5.2: AADTT versus HMA Thickness (in.) for M_r =5 ksi

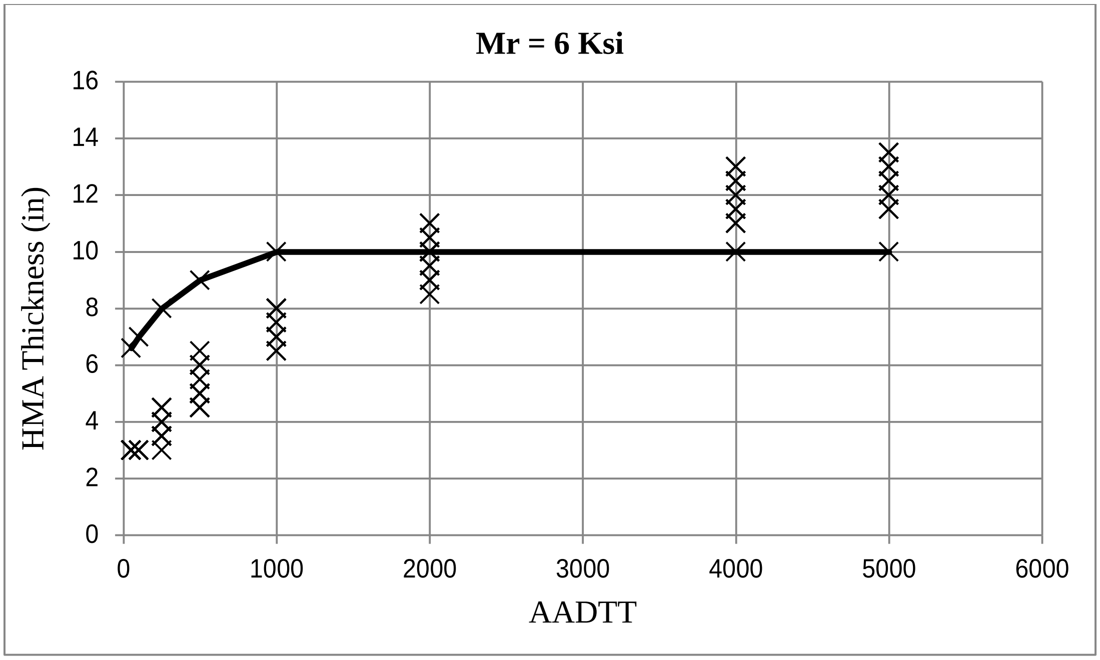


Figure 5.3: AADTT versus HMA Thickness (in.) for M_r =6 ksi

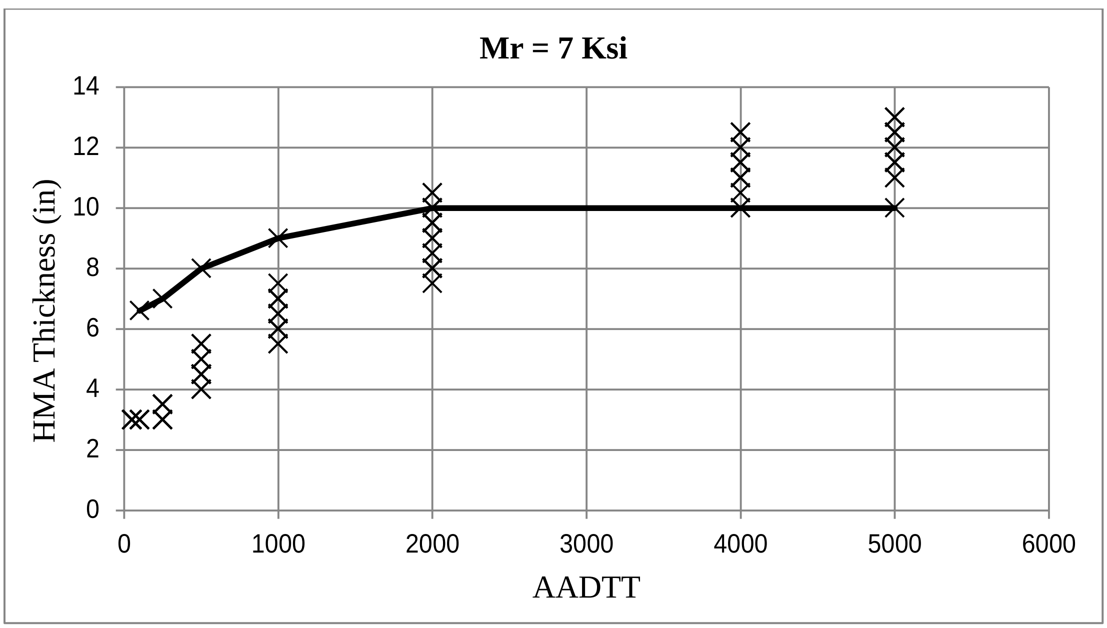


Figure 5.4: AADTT versus HMA Thickness (in.) for M_r =7 ksi

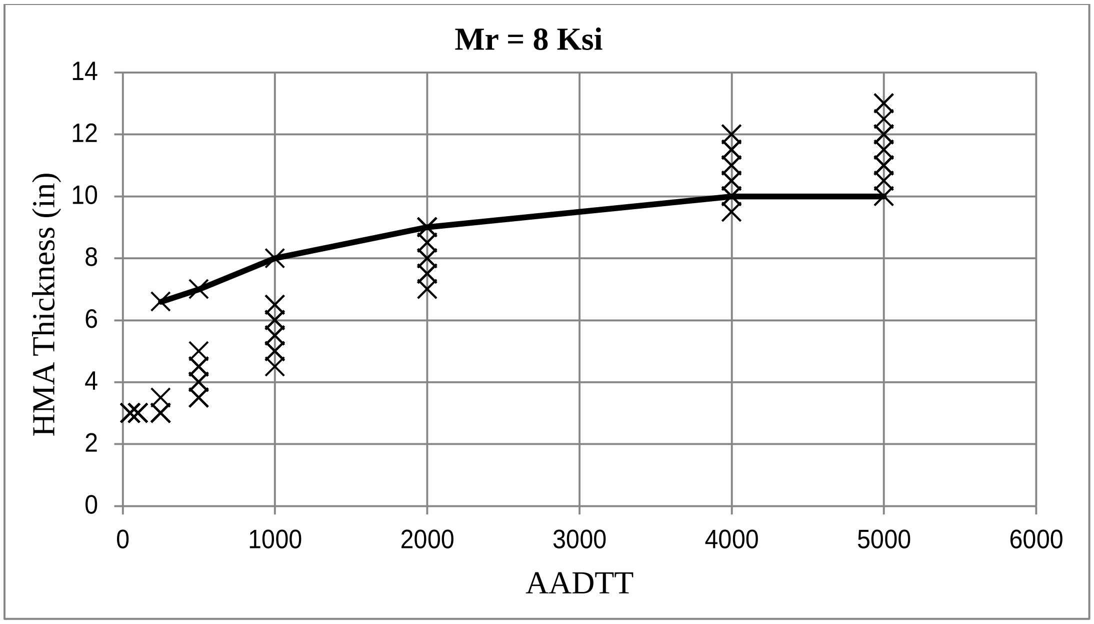


Figure 5.5: ESALs (million) versus HMA Thickness (in.) for M_r =8 ksi

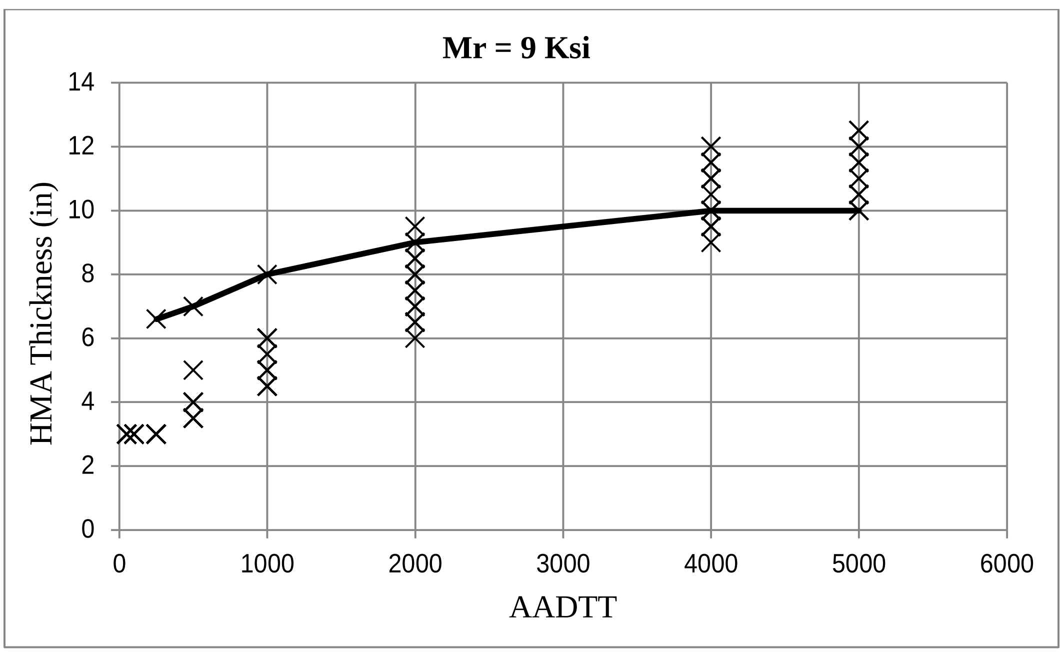


Figure 5.6: ESALs (million) versus HMA Thickness (in.) for M_r =9 ksi

Chapter 6: Conclusions and Recommendations

The New York State Department of Transportation has decided to use the Mechanistic-Empirical Pavement Design Guide (MEPDG) for the design of new flexible pavement structures in the future. The process of implementing the MEPDG has commenced with the development of a database containing material and traffic inputs, as well as the calibration of the distress models to local conditions. Since the design of new and overlaid pavement structures is almost exclusively done in NYSDOT regional offices, which likely do not have designers with expertise in running the AASHTOWare Pavement 2.1 software, a simpler design method based on AASHTOWare is needed. This simplified design method could utilize design tables; the designer would need to select directly from these tables the design pavement structure based on a limited number of inputs. Currently, NYSDOT is using only two tables for the design of new flexible pavements; the NYSDOT design engineers are very familiar with their use. However, these tables were developed based on the AASHTO 1993 Design Guide.

The first objective of this research work was to calibrate the AASHTOWare distress models for the local conditions in New York State. For this purpose, construction, traffic, and performance data on 18 LTPP sites in the Northeast region of the United States were used. The alligator cracking, rutting, and IRI models in AASHTOWare were successfully calibrated. The longitudinal cracking and low-temperature cracking models could not be calibrated because the field measured data were erroneous. The calibrated AASHTOWare model can be used for the design of new flexible pavement structures in New York State.

The second objective of this research work was to develop design tables based on AASHTOWare to be used by NYSDOT for the design of new flexible pavement structures. The development of the design tables was done by running the calibrated AASHTOWare software for combinations of climatic conditions, traffic load level, subgrade soil stiffness, and pavement structures. The runs were done for the following conditions:

- Design pavement structure for a new flexible pavement classified as Principal Arterial – Interstate.
- Design reliability level of 90%.

- Design life of 15 years. This was selected since, according to the PMS unit of NYSDOT, new flexible pavement structures in New York State are resurfaced with an overlay for the first time 12 to 15 years after their construction.
- Water table depth of 10 feet.
- At least one location for each of the 11 regions of NYSDOT. For Region 9, a virtual weather station was created. For all other regions, the AASHTOWare software contains climatic files for at least one location.
- Statewide average values for traffic volume parameters and for axle load spectra.

Design cases were established as a combination of the following design situations:

- Subgrade soil resilient modulus of 4, 5, 6, 7, 8, and 9 ksi (28, 34, 41, 48, 55, and 62 MPa).
- AADTT in the design lane of 50, 100, 250, 500, 1,000, 2,000, 4,000, and 5,000 trucks.
- Pavement structures starting with the design cases included in the CPDM. The granular subbase materials and thicknesses recommended by the CPDM were used but only the asphalt concrete layer thickness and the select granular subgrade layer thickness were varied to include several values higher and lower than those recommended by the CPDM. The thickness of the asphalt binder and surface layers were kept constant.

For each design case, the predicted distresses were compared with the corresponding performance criteria, 225 in/mile for IRI and 0.75 in. for total rutting. The design case with the thinnest asphalt base layer for which the predicted distresses were lower than the design criteria was selected as the design solution. The design solutions were then assembled in design tables for each of the 24 locations.

The following conclusions were derived from this research:

- The calibration of the rutting, alligator cracking, and IRI models was successful

- The development of simple design tables was successful. The designer needs only AADTT and M_r to design the pavement structure.
- The climate variation has an impact on the design thicknesses; different design tables were obtained for different locations within the New York state.
- For high truck traffic volumes and soft subgrade soils, the design solutions vary from location to location, even within the same region. For low traffic volumes, the design solutions are the same throughout the state.
- The design solutions for the Upstate part of New York requires thicker asphalt concrete layers than the corresponding design solutions for the Downstate part of the state. This may be explained by differences in the binder grade and aggregate gradation for the asphalt mixes used in the two parts of the state and the difference in the climatic conditions between the two parts of the state.
- At lower AADTT, the new design tables recommend thinner asphalt concrete layers than those recommended in the CPDM table, while at higher AADTT the design asphalt layer thickness is greater in the new design tables than in the CPDM table.

The following recommendations are resulting from this study:

- NYSDOT should develop a new flexible pavement performance database. It is recommended to monitor in-service or accelerated pavement structures in order to obtain a larger database of performance and construction data and thus improve the calibration of the distress models.
- The flexible pavement performance models should be recalibrated if the new pavement performance database will be available or any of the distress models change.
- Additional design tables should be developed for water table depths of less than 10 feet.

- For high AADTT values, a life-cycle cost analysis (LCCA) should be conducted to determine the cost effectiveness of full-depth asphalt pavement designs included in the tables when compared to that of rigid pavement designs.

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Appendix A: Vehicle Classification Data Provided by NYSDOT

Table A.1: Functional Classification of Roads

FC Code	Description
1	Rural Principal Arterial-Interstate
2	Rural Principal Arterial-Other
6	Rural Minor Arterial
7	Rural Major Collector
8	Minor Collector
9	Rural Local
11	Urban Principal Arterial-Interstate
12	Urban Principal Arterial-Other Freeways and Expressways
14	Urban Principal Arterial-Other
16	Urban Minor Arterial
17	Urban Major Collector
19	Urban Local Road

Table A.2: Direction of Traffic

Direction	Description
0	E/W or SE/NW Combined
1	North
2	Northeast
3	East
4	Southeast
5	South
6	Southwest
7	West
8	Northwest
9	N/S or NE/SW Combined

Table A.3: Vehicle Classification Sites

Site_ID	Direction	N_Lanes	Region	FC Code	AADTT (2007)	AADTT (2008)	AADTT (2009)	AADTT (2010)	AADTT (2011)
00180	1	3	11	11	3965	3756	3373		
00180	5	3	11	11	3905	3713	3596		
00199	1	3	11	11		9042			
00199	5	3	11	11		9495			
00280	1	2	11	14	952	876	818		
00280	5	2	11	14	1067	1003	917		
00299	1	3	11	11		5955	5830		
00299	5	3	11	11		4569	3731		
00580	3	3	11	11		4495	4123		
00580	7	3	11	11		4169	3753		
00581	1	3	11	11				5137	
00698	1	4	11	12					2827
00698	5	3	11	12					2152
00710	1	1	10	16			159	165	151
00710	5	1	10	16			154	182	170
00797	3	1	10	14		189	172		176
00797	7	1	10	14		181	170		169
01280	3	1	1	16	108	100	101	94	
01280	7	1	1	16	97	93	87	86	
01281	1	1	1	6	81	75	58	41	
01281	5	1	1	6	85	77	61	44	
01511	1	3	1	11	2480	2472			
01511	5	3	1	11	2665	2566			
01512	1	3	1	11	2133	2068	1932	1970	1971
01512	5	3	1	11	2327	2273	2074	2123	2126
01800	1	1	1	2	566	526	500	524	510
01800	5	1	1	2	550	451	491	513	507
02180	1	1	2	7		27	27	28	30
02180	5	1	2	7		27	27	28	31
02181	1	1	2	14	74	63	52	52	55
02181	5	1	2	14	53	48	41	44	43
02280	1	1	2	6		46	35	41	40
02280	5	1	2	6		60	46	55	55
02680	1	2	2	12		161	168		155
02680	5	2	2	12		318	318		315
03311	1	2	3	1		2105	1978		2033
03311	5	2	3	1		2066	1946		1995
03381	1	1	3	7		53	41	41	44
03381	5	1	3	7		53	45	48	52
03382	1	2	3	12	589	595			
03382	5	2	3	12	585	534			
03482	3	1	3	2	129	97	109	111	
03482	7	1	3	2	143	127	127	134	
03411	1	2	3	1	1338	1273	1155	1214	1195
03411	5	2	3	1	1409	1337	1114	1274	1239

Table A.3: Vehicle Classification Sites (Continued)

Site_ID	Direction	N_Lanes	Region	FC Code	AADTT (2007)	AADTT (2008)	AADTT (2009)	AADTT (2010)	AADTT (2011)
03480	1	1	3	7		100	96		131
03480	5	1	3	7		102	95		147
03481	3	1	3	7	48	39			43
03481	7	1	3	7	40	36			43
03482	3	1	3	2					123
03482	7	1	3	2					144
03580	1	1	3	16	286	266	231	229	
03580	5	1	3	16	247	217	182	181	
03681	1	2	3	12		584	558	574	553
03681	5	2	3	12		455	429	453	435
03680	1	1	3	7		42	46	49	52
03680	5	1	3	7		45	54	55	54
04181	1	2	4	14	476	483	470		
04181	5	2	4	14	453	462	433		
04280	1	1	4	16	57	58			
04280	5	1	4	16	66	59			
04342	1	3	4	12	802	795			753
04342	5	3	4	12	902	880			786
04380	3	2	4	12	45	20	44		
04380	7	2	4	12	62	54	58		
04481	1	1	4	6	43	46	43	46	44
04481	5	1	4	6	42	41	44	45	41
04482	1	1	4	7	37	39	33	32	33
04482	5	1	4	7	38	39	34	34	36
04483	3	2	4	14	259		186		
04483	7	2	4	14	301		195		
04780	1	1	4	7		34	30	27	27
04780	5	1	4	7		32	29	26	28
05183	3	2	5	1			665		
05183	7	2	5	1			662		
05280	3	2	5	1	698	649	598		
05280	7	2	5	1	724	720	702		
05282	3	2	5	2	221	265	267	268	
05282	7	2	5	2	244	273	277	278	
05281	1	1	5	2	205	196	176	162	183
05281	5	1	5	2	193	189	170	159	173
05380	3	2	5	12			571		
05380	7	2	5	12			624		
05381	1	2	5	16		139	138		
05381	5	2	5	16		137	136		
05384	1	2	5	2	479	350	341	362	
05384	5	2	5	2	435	351	341	356	
05383	3	3	5	14		278	255	266	276
05383	7	3	5	14		361	327	316	316

Table A.3: Vehicle Classification Sites (Continued)

05480	1	1	5	16	129	110	60	111	105
05480	5	1	5	16	143	125	61	124	118
05484	3	1	5	2	83	77		80	85
05484	7	1	5	2	87	79		82	82
06100	3	2	6	1		756		791	795
06100	7	2	6	1		732		776	770
06280	1	1	6	6	100	108	115		117
06280	5	1	6	6	97	112	109		121
06282	3	2	6	12	121	107	100	119	124
06282	7	2	6	12	115	97	102	120	116
06281	3	1	6	16		46	37	33	
06281	7	1	6	16		52	44	39	
06340	1	1	6	2					160
06340	5	1	6	2					206
06480	1	1	6	7	62	58	56	48	42
06480	5	1	6	7	73	67	70	62	61
06500	3	2	9	2	1496	1493	1082		
06500	7	2	9	2	1569	1515	1096		
06680	3	1	6	16	42	44	40	42	44
06680	7	1	6	16	41	42	37	42	43
07183	1	1	7	2	190	176	159	169	175
07183	5	1	7	2	214	199	182	187	192
07182	1	1	7	14	41	44	41	41	38
07182	5	1	7	14	36	41	38	35	35
07181	3	1	7	2		212	232		231
07181	7	1	7	2	269	251	243		243
07111	1	2	7	1			831	834	
07111	5	2	7	1			848	861	
07100	1	2	7	1	904	864	746	753	
07100	5	2	7	1	1002	936	794	804	
07231	3	1	7	6	112		71	72	66
07231	7	1	7	6	164		75	75	69
07280	1	2	7	14			182	294	
07280	5	2	7	14	645		189	311	
07381	1	2	7	1		564	505	523	
07381	5	2	7	1		567	517	534	
07380	1	2	7	14		381	367	363	
07380	5	2	7	14		399	376	368	
07480	3	1	7	2	58	61		56	
07480	7	1	7	2	62	65		59	
07481	3	1	7	7	77	91	83	82	79
07481	7	1	7	7	106	95	94	93	88
07580	3	1	7	6	208	193	155	177	
07580	7	1	7	6	278	247	211	210	
07581	3	1	7	6	279	281			
07581	7	1	7	6	342	247			

Table A.3: Vehicle Classification Sites (Continued)

08180	3	2	8	2		110	106	111	
08180	7	2	8	2		116	110	118	
08280	3	2	8	11	3450	3208	2947	3096	
08280	7	2	8	11	3358	2951	2740	2903	
08380	3	2	8	2					820
08380	7	2	8	2					840
08381	1	1	8	6	65	56	51	50	52
08381	5	1	8	6	68	64	57	57	55
08382	3	2	8	11		1287			
08382	7	2	8	11		1148			
08383	1	2	8	16	249	238	239	228	228
08383	5	2	8	16	252	242	245	240	245
08411	3	2	8	11			3420		
08411	7	2	8	11			3342		
08481	1	1	8	14	444	413	364		387
08481	5	2	8	14	374	347	302		314
08580	1	1	8	16	32	52			
08580	5	1	8	16	26	326			
09121	1	2	9	1		1972	1974		
09121	5	2	9	1		2134	1903		
09181	3	2	9	14		291	279	280	
09181	7	2	9	14		199	192	195	
09180	1	1	9	6	113	107	98	121	164
09180	5	1	9	6	113	105	86	107	141
09122	1	2	9	12	555	564	580	609	
09122	5	2	9	12	480	568	547	587	
09280	1	1	9	14		203	164	171	189
09280	5	1	9	14		198	182	198	203
09381	1	1	9	7	29	32	26	31	29
09381	5	1	9	7	25	27	23	27	26
09380	1	1	9	2	172	170	146	140	
09380	5	1	9	2	162	182	158	156	
09480	1	1	9	14	140	117	118		
09480	5	1	9	14	147	136	127		
09580	3	3	9	1	957	906	884		921
09580	7	2	9	1	957	880	857		905
09581	1	1	9	7	53	45	45	48	47
09581	5	1	9	7	56	45	46	50	52
09582	3	1	9	16	73	65	69	69	
09582	7	1	9	16	68	64	65	67	
09631	3	2	9	2	528	516	493		
09631	7	2	9	2	569	564	531		

Appendix B: Extracted Long-Term Pavement Performance (LTPP) Traffic Design Inputs

Table B.1: Annual Average Daily Truck Traffic, Momin (2011)

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
091803	1992	100	341003	2003	790
091803	1993	110	341003	2004	870
091803	1994	170	341011	1993	1100
091803	1995	190	341011	1994	950
091803	2004	170	341011	1995	1000
091803	2005	170	341011	1996	1050
091803	2006	160	341011	1997	1220
231001	2001	660	341011	1999	1330
231001	2002	640	341011	2000	1340
231001	2003	630	341011	2001	1460
231009	2000	290	341011	2002	1510
231009	2002	290	341011	2003	1590
231009	2003	280	341011	2004	1600
231009	2006	300	341011	2005	1420
231028	2000	250	341011	2007	1230
231028	2001	270	341030	1993	360
231028	2002	310	341030	1994	360
231028	2003	290	341030	1995	350
231028	2004	320	341030	1996	320
231028	2005	300	341030	1997	330
231028	2006	360	341030	1999	390
231028	2007	400	341030	2001	360
251003	1992	100	341030	2006	390
251003	1993	90	341030	2007	330
251003	1994	120	341031	1994	1050
251003	1995	170	341031	1995	1120
251003	1996	230	341031	1996	1040
251003	1997	200	341031	1998	1310
251003	1994	120	341031	1994	1050
251003	1995	170	341031	1995	1120
251003	1994	120	341031	1994	1050

Table B.1: Annual Average Daily Truck Traffic, Momin (2011) (Continued)

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
251003	1995	170	341031	1995	1120
251003	1996	230	341031	1996	1040
251003	1997	200	341031	1998	1310
251003	1998	200	341031	1999	1340
341003	1994	670	341033	1994	260
341003	1995	750	341033	1995	270
341003	1996	940	341033	2000	320
341003	1997	1520	341033	2002	300
341003	1998	1020	341033	2003	250
341003	1999	640	341033	2004	290
341003	2000	820	341034	1994	1190
341003	2001	830	341034	1995	1180
341003	2002	750	341034	1996	1230
341034	2004	1640	341034	1997	1290
341034	2007	1330	341034	1998	1340
341638	1994	1150	341034	1999	1310
341638	1995	1170	341034	2000	1370
341638	1996	1190	341034	2001	1450
341638	1997	1250	341034	2002	1560
341638	1998	1270	341034	2003	1570
341638	1999	1180	501002	2005	310
341638	2002	1610	501002	2006	490
341638	2003	1910	501002	2007	380
341638	2004	1960	501004	1992	170
341638	2005	1700	501004	1993	160
341638	2007	1350	501004	1994	170
361643	1995	770	501004	1996	210
421597	1998	90	501004	1997	210
421597	1999	90	501004	1998	210
421597	2004	150	501004	1999	180
421597	2005	130	501004	2000	200

Table B.1: Annual Average Daily Truck Traffic, Momin (2011) (Continued)

SHRP ID	YEAR	AADTT	SHRP ID	YEAR	AADTT
421597	2006	160	501004	2001	200
421597	2007	130	501004	2002	190
421599	1998	450	501004	2003	180
421599	1999	470	501004	2004	190
421599	2000	510	501004	2005	200
421599	2001	490	501681	1992	400
421599	2003	490	501681	1993	390
421599	2004	490	501681	1994	400
421599	2005	490	501681	1995	400
421599	2006	500	501681	1996	410
421599	2007	530	501681	1997	440
501002	1992	240	501681	1998	490
501002	1993	220	501681	1999	530
501002	1994	220	501683	1992	390
501002	1995	220	501683	1993	380
501002	1996	250	501683	1994	400
501002	1997	260	501683	1995	400
501002	1998	260	501681	2006	710
501002	1999	380	501683	1996	410
501002	2000	370	501683	1997	430
501002	2001	320	501683	1998	470
501002	2002	290	501683	1999	510
501002	2003	300	501683	2000	520
501002	2004	280	501683	2001	550
501681	2000	540	501683	2002	630
501681	2001	560	501683	2003	490
501681	2002	520	501683	2004	510
501681	2003	570	501683	2005	570
501681	2004	660	501683	2006	480
501681	2005	710			

Table B.2: Vehicle Class Distribution

SHRP ID	Year	Vehicle Class										Total
91803	1992	0.87	50.95	17.49	12.75	2.75	14.52	0.51	0.14	0.01	0.01	100
231001	2002	3.45	20.82	2.32	0.05	3.47	57.85	11.03	0.87	0.13	0.01	100
231009	2000	9.49	35.83	10.71	2.05	8.17	23.71	9.89	0.15	0	0	100
231028	2000	6.71	18.51	7.85	2.29	2.5	24.48	37.61	0	0	0.05	100
251003	1993	1.75	56.96	24.43	0.31	7.37	8.88	0.26	0.04	0	0	100
341003	1994	1.05	61.56	9.98	0.24	4.95	21.62	0.5	0.1	0	0	100
341011	1993	1.6	31.16	17.69	1.64	8.9	36.53	1.13	1.19	0.06	0.1	100
341030	1999	1.82	62.91	12.14	4.9	3.95	13.82	0.46	0	0	0	100
341031	1998	1.74	28.45	5.25	9.68	7.25	44.94	1.96	0.7	0.02	0.01	100
341033	2002	2.54	48.96	14.17	1.23	6.12	25.95	0.7	0.26	0.05	0.02	100
341034	1997	2.23	41.07	9.47	3.58	7.68	34.19	1.19	0.55	0.02	0.02	100
341638	1996	1.59	37.31	6.4	3.38	9.68	39.95	1.05	0.61	0.02	0.01	100
421597	2004	4.69	42.94	14.61	3.43	8.21	23.62	0.35	2.11	0.01	0.03	100
421599	2001	1.02	15.98	9.49	9.13	4.55	58.67	0.45	0.54	0.03	0.14	100
501002	1992	3.45	32.84	18.81	1.26	8.21	33.28	0.77	0.74	0.63	0.01	100
501004	1994	1.91	53.98	10.32	0.19	10.21	22.59	0.51	0.1	0.19	0	100
501681	1992	2.52	26.82	8.2	0.39	8.81	50.24	2.24	0.76	0.02	0	100
501683	1992	2.52	26.56	8.62	0.52	9.7	49.86	1.72	0.45	0.04	0.01	100

Table B.3: Monthly Adjustment Factors

Site: 231001-2002

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.2	0.84	1.56	0	1.08	1.2	1.08	1.32	1.344	0
February	1.2	0.96	1.44	0	1.2	1.272	1.08	1.08	1.332	0
March	1.2	0.84	1.92	0	0.96	1.296	1.08	1.32	1.332	0
April	1.32	1.08	1.92	0	1.08	1.296	1.2	1.32	1.332	0
May	1.44	1.32	1.68	6	1.32	1.296	1.32	1.32	1.332	0
June	1.56	1.68	1.44	0	1.5	1.08	0.96	1.44	1.332	0
July	1.32	1.68	2.04	0	1.5	1.08	1.08	1.44	1.332	0
August	0	0	0	0	0	0	0	0	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.08	1.44	0	6	1.2	1.2	1.44	1.08	1.332	0
November	0.84	1.2	0	0	1.08	1.2	1.44	0.84	1.332	0
December	0.84	0.96	0	0	1.08	1.08	1.32	0.84	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 231009-2000

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0.924	0.792	0.792	1.188	1.056	1.056	0.66	0	0	0
March	1.056	0.792	0.792	1.056	1.32	1.056	0.924	0	0	0
April	1.056	0.66	1.056	1.548	1.188	0.924	1.188	0	0	0
May	1.32	0.924	1.188	1.056	1.32	1.188	1.32	2.64	0	0
June	1.452	1.32	1.452	1.452	1.32	1.452	1.32	2.64	0	0
July	1.452	1.452	1.584	1.188	1.056	1.188	1.452	2.64	0	0
August	1.452	1.848	1.452	1.188	1.32	1.452	1.512	0	0	0
September	1.056	2.112	1.188	0.792	1.056	1.188	0.924	0	0	0
October	1.188	1.452	1.452	1.536	1.32	1.452	1.524	2.64	0	0
November	1.188	1.056	1.452	1.536	1.188	1.188	1.452	2.64	0	0
December	1.056	0.792	0.792	0.66	1.056	1.056	0.924	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 231028-2000

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.98	1.056	0.528	1.056	1.188	1.056	1.188	0	0	0
February	2.244	1.056	0.66	1.188	1.188	1.056	1.32	0	0	0
March	1.32	0.924	0.66	1.188	1.188	1.188	1.32	0	0	0
April	1.188	0.792	0.792	0.792	1.188	1.056	1.056	0	0	0
May	0.924	0.924	1.32	0.792	1.188	1.32	1.056	0	0	0
June	0.924	1.188	1.716	1.716	1.32	1.32	1.32	0	0	0
July	0.792	1.452	1.452	0.792	1.188	1.188	1.188	0	0	0
August	0.792	1.716	1.716	2.112	1.452	1.32	1.32	0	0	0
September	1.056	1.716	1.452	1.056	1.188	1.188	1.056	0	0	0
October	1.056	1.584	1.584	1.848	1.188	1.32	1.32	0	0	0
November	0.924	0.792	1.32	0.66	0.924	1.188	1.056	0	0	0
December	0	0	0	0	0	0	0	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 251003-1993

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.308	1.188	1.836	0	2.592	1.404	0	0	0	0
February	1.356	1.296	2.16	0	1.728	1.728	0	0	0	0
March	1.356	1.404	1.836	0	2.052	1.404	0	0	0	0
April	1.356	0.972	2.376	0	1.728	2.916	0	0	0	0
May	1.356	1.188	0.864	0	1.08	1.404	0	0	0	0
June	0	1.296	0.432	0	0.324	0.54	0	0	0	0
July	1.356	1.08	0.54	0	0.324	0.54	0	0	0	0
August	1.356	1.08	0.108	0	0.324	0.324	0	0	0	0
September	1.356	1.296	0.648	0	0.648	0.54	0	0	0	0
October	0	0	0	0	0	0	0	0	0	0
November	0	0	0	0	0	0	0	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341003-1994

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	4.2	1.2	0.6	0	0.84	1.44	0.72	0	0	0
February	4.56	1.32	0.6	0	1.08	1.2	0.72	0	0	0
March	0.72	1.2	0.24	0	0.36	0.24	0	0	0	0
April	1.08	1.2	0.6	0	0.6	0.6	0	0	0	0
May	1.44	0.96	0.6	0	0.72	0.6	0.72	0	0	0
June	0	1.2	1.92	1.68	1.8	1.8	1.56	6	0	0
July	0	1.2	2.04	3.48	1.8	1.68	2.28	6	0	0
August	0	1.56	2.28	3.48	2.16	2.04	3	0	0	0
September	0	1.2	2.04	1.68	1.8	1.68	2.28	0	0	0
October	0	0.96	1.08	1.68	0.84	0.72	0.72	0	0	0
November	0	0	0	0	0	0	0	0	0	0
December	0	0	0	0	0	0	0	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341011-1993

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	4.032	1.44	1.44	2.4	1.248	1.296	1.464	1.344	0	1.2
June	0	1.248	1.92	1.248	1.248	1.296	1.476	1.152	1.62	1.2
July	0	1.152	1.44	0.96	1.248	1.152	1.476	1.248	0	1.2
August	1.44	1.248	1.44	1.152	1.248	1.248	1.344	1.248	1.596	1.2
September	1.344	1.152	1.056	1.056	1.248	1.152	1.056	1.152	1.596	1.2
October	1.152	1.152	0.768	1.152	1.152	1.152	0.96	1.248	1.596	1.2
November	0.96	1.152	0.864	0.864	1.152	1.152	0.96	1.248	1.596	1.2
December	0.672	1.056	0.672	0.768	1.056	1.152	0.864	0.96	1.596	1.2

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341030-1999

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.452	1.716	0.792	0.264	1.056	0.792	0	0	0	0
February	1.32	0.924	0.924	0.396	0.792	1.056	0.792	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	1.32	1.32	1.452	1.056	1.32	1.32	2.112	0	0	0
May	0.66	1.056	0.924	1.056	0.924	1.188	1.452	0	0	0
June	1.452	1.188	1.452	1.584	1.584	1.584	1.452	0	0	0
July	1.776	1.188	1.512	1.716	1.584	1.32	1.452	0	0	0
August	1.788	1.32	1.32	2.376	1.98	1.32	1.452	0	0	0
September	1.32	1.188	1.056	0.66	1.452	1.188	0.792	0	0	0
October	1.056	1.056	1.056	0.792	1.056	1.188	1.452	0	0	0
November	0.528	1.056	1.188	1.452	0.66	1.056	0.792	0	0	0
December	0.528	1.188	1.524	1.848	0.792	1.188	1.452	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341031-1998

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	1.32	1.32	1.56	1.32	1.68	1.44	1.56	1.2	12	0
April	0.84	1.08	1.08	0.96	1.44	1.2	1.2	0.72	0	0
May	0.96	1.2	1.08	0.96	1.08	1.08	1.08	0.84	0	0
June	1.56	1.2	1.2	1.32	1.2	1.2	1.32	1.2	0	0
July	1.32	1.32	1.2	1.2	1.2	1.2	0.96	1.08	0	0
August	1.2	1.2	1.08	1.32	1.08	1.2	1.2	1.2	0	0
September	1.32	1.2	1.2	1.32	1.08	1.2	1.2	1.32	0	0
October	1.2	1.2	1.2	1.32	1.08	1.2	1.32	1.32	0	0
November	1.2	1.2	1.2	1.2	1.08	1.2	1.2	1.2	0	0
December	1.08	1.08	1.2	1.08	1.08	1.08	0.96	1.92	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341033-2002

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.2	0.84	0.96	0.96	1.08	1.2	1.632	1.5	0	0
February	1.2	0.72	0.6	0.36	0.96	0.96	1.08	1.5	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	1.56	1.8	1.32	0.96	1.2	1.32	1.08	1.5	0	0
June	1.2	2.16	1.488	1.32	1.2	1.296	1.08	1.5	0	0
July	1.08	1.56	1.476	1.56	1.38	1.296	1.08	1.5	0	0
August	1.2	1.2	1.2	1.32	1.38	1.296	1.644	1.5	0	0
September	1.2	0.96	1.32	1.68	1.38	1.296	1.08	1.5	0	0
October	1.2	0.96	1.476	1.32	1.38	1.296	1.644	1.5	0	0
November	1.08	0.96	1.2	1.56	1.08	1.2	1.08	0	0	0
December	1.08	0.84	0.96	0.96	0.96	0.84	0.6	0	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341034-1997

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.08	1.08	1.08	1.08	1.188	1.296	0.864	1.188	0	0
February	1.188	1.188	1.08	0.864	1.188	1.188	0.864	1.296	0	0
March	1.296	1.08	1.08	0.972	1.08	1.08	1.08	1.296	0	0
April	1.296	1.296	1.296	1.188	1.404	1.404	1.728	1.56	10.8	0
May	1.296	1.296	1.296	1.08	1.404	1.404	1.728	1.572	0	0
June	1.296	1.296	1.296	1.512	1.296	1.296	1.62	1.296	0	0
July	0	0	0	0	0	0	0	0	0	0
August	0	0	0	0	0	0	0	0	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.296	1.296	1.62	1.944	1.404	1.296	1.188	1.296	0	0
November	1.08	1.08	1.188	1.188	0.972	0.972	0.972	0.756	0	0
December	0.972	1.188	0.864	0.972	0.864	0.864	0.756	0.54	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 341638-1996

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	1.056	1.188	0.528	1.056	1.188	0.792	1.188	0	0
February	0	1.188	1.188	0.66	1.188	1.344	1.056	1.188	0	0
March	0.264	1.188	1.32	1.452	1.188	1.356	1.32	1.188	0	0
April	1.584	1.248	1.38	2.376	1.356	1.356	1.716	1.32	0	0
May	1.908	1.272	1.392	1.98	1.368	1.356	1.32	1.32	0	0
June	1.92	1.272	1.188	1.584	1.188	1.188	1.32	1.188	0	0
July	1.716	1.272	1.188	1.452	1.188	1.188	1.584	1.188	0	0
August	1.716	1.272	1.32	1.188	1.368	1.188	1.584	1.452	0	0
September	0	0	0	0	0	0	0	0	0	0
October	1.452	1.188	0.924	0.66	1.188	1.056	0.66	1.188	0	0
November	1.452	1.188	1.056	0.792	1.056	1.056	1.056	1.188	0	0
December	1.188	1.056	1.056	0.528	1.056	0.924	0.792	0.792	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 421597-2004

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.188	0.956	0.528	0.792	0.824	0.774	0.3	0.792	0	0
February	1.188	1.088	0.528	1.056	1.088	0.906	0.3	1.02	0	0
March	1.584	1.088	0.528	1.056	1.088	0.906	0.3	1.02	0	0
April	1.584	0.952	0.924	1.056	1.12	1.038	0.3	1.084	0	0
May	1.32	0.956	1.32	0.792	0.956	1.038	2.3	1.02	0	0
June	0.792	0.924	1.452	1.284	1.12	1.302	0.3	1.12	0	0
July	0.528	1.056	1.188	1.056	1.088	1.038	0.3	1.12	0	0
August	0.792	1.088	1.64	1.112	1.088	1.002	2.3	1.07	0	0
September	0.848	1.088	1.056	1.12	0.92	1.17	0.3	0.792	0	0
October	0.98	1.084	1.052	1.084	0.92	1.17	2.3	1.32	0	0
November	0.396	0.92	1.084	0.792	0.888	0.906	2.3	0.792	0	0
December	0.8	0.8	0.7	0.8	0.9	0.75	0.7	0.85	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 421599-2001

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0.72	1.2	0.84	0.96	1.2	1.08	0.6	0.96	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	1.08	1.2	1.2	1.08	1.2	1.2	1.2	1.38	0	1.5
May	1.32	1.272	1.56	1.512	1.2	1.2	1.2	1.356	0	1.5
June	1.32	1.2	1.44	1.524	1.32	1.2	1.8	1.356	0	1.5
July	0.72	1.08	1.32	1.32	1.2	1.2	1.8	1.356	0	0
August	1.08	1.284	1.2	1.524	1.2	1.32	1.2	1.356	0	1.5
September	1.32	1.2	1.2	1.2	1.08	1.2	1.2	1.356	0	1.5
October	1.56	1.284	1.32	1.2	1.32	1.32	1.8	0.96	0	1.5
November	1.32	1.2	0.96	0.84	1.2	1.2	0.6	0.96	0	1.5
December	1.56	1.08	0.96	0.84	1.08	1.08	0.6	0.96	0	1.5

Table B.3: Monthly Adjustment Factors (Continued)

Site: 501002-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.152	0.72	0.864	1.44	1.008	1.296	1.584	0.864	1.776	0
February	1.44	0.576	0.864	1.44	1.008	1.152	0.72	1.536	1.008	0
March	0.864	0.432	1.008	0.864	1.008	1.152	0.72	0.864	1.008	0
April	1.008	1.152	0.864	1.44	1.296	1.296	1.584	1.536	1.008	0
May	1.152	1.296	1.008	0.864	1.152	1.296	0.72	1.536	1.008	0
June	1.44	1.584	1.296	1.44	1.728	1.368	1.584	0.864	1.008	0
July	1.152	1.584	2.16	1.44	1.728	1.296	2.16	1.536	1.008	0
August	1.44	1.44	2.016	0.864	1.44	1.296	1.584	1.536	1.776	0
September	1.44	1.584	1.872	2.88	1.152	1.368	1.584	1.536	1.776	0
October	1.44	1.44	1.008	0.432	1.152	1.008	0.72	0.864	1.008	0
November	0.864	1.296	0.72	0.432	0.864	0.864	0.72	0.864	1.008	0
December	1.008	1.296	0.72	0.864	0.864	1.008	0.72	0.864	1.008	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 501683-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.296	0.576	0.864	1.152	1.008	1.008	1.152	0.72	0	0
February	1.296	0.576	0.864	1.152	1.008	1.008	1.152	0.72	0	0
March	1.296	0.576	0.864	1.152	1.008	1.152	1.152	0.72	0	0
April	1.296	0.576	0.864	1.152	1.152	1.152	1.152	0.72	0	0
May	0.864	1.296	1.152	1.152	1.152	1.152	1.152	0.72	0	0
June	1.152	1.44	1.296	1.152	1.152	1.152	1.44	1.44	0	0
July	1.296	1.44	1.44	2.016	1.344	1.44	1.872	1.44	0	0
August	1.152	1.512	1.44	1.152	1.356	1.296	1.152	1.44	0	0
September	1.008	1.512	1.296	1.152	1.356	1.296	1.008	1.44	0	0
October	1.584	1.44	1.152	2.016	1.356	1.296	1.152	1.44	0	0
November	1.44	1.44	1.728	1.152	1.356	1.152	1.008	1.44	0	0
December	1.008	1.296	1.296	0.576	1.008	1.152	1.008	1.44	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 501681-1992

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.152	0.576	1.008	0.864	1.008	1.152	1.152	0.864	0	0
February	2.016	0.576	0.864	0.864	1.152	1.152	1.152	0.864	0	0
March	1.008	0.576	1.008	1.536	1.152	1.152	1.152	0.864	0	0
April	0.864	1.152	1.152	1.536	1.296	1.152	1.008	1.296	0	0
May	0.864	1.44	1.296	1.536	1.152	1.152	1.008	0.864	0	0
June	1.152	1.632	1.44	0.864	1.296	1.296	2.016	1.296	0	0
July	1.296	1.632	1.296	1.536	1.296	1.296	1.296	1.296	0	0
August	1.152	1.632	1.44	1.536	1.44	1.296	1.008	1.872	0	0
September	1.44	1.44	1.296	0.864	1.296	1.296	1.296	1.296	0	0
October	1.44	1.44	1.44	1.536	1.152	1.152	1.296	1.296	0	0
November	1.008	1.152	1.152	0.864	1.008	1.152	1.008	1.296	0	0
December	1.008	1.152	1.008	0.864	1.152	1.152	1.008	1.296	0	0

Table B.3: Monthly Adjustment Factors (Continued)

Site: 501004-1994

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0	0	0	0	0	0	0	0	0	0
February	0	0	0	0	0	0	0	0	0	0
March	0	0	0	0	0	0	0	0	0	0
April	0	0	0	0	0	0	0	0	0	0
May	1.44	1.056	0.96	0	1.056	1.152	1.92	0	4.8	0
June	1.44	1.152	1.248	0	1.44	1.32	1.92	0	4.8	0
July	1.152	1.272	1.248	0	1.248	1.152	0	0	0	0
August	0.768	1.272	1.248	0	1.248	1.308	1.92	0	0	0
September	1.44	1.272	1.248	0	1.248	1.308	0	0	0	0
October	1.44	1.272	1.248	0	1.248	1.248	1.92	0	0	0
November	1.152	1.152	1.152	0	1.056	1.152	1.92	0	0	0
December	0.768	1.152	1.248	9.6	1.056	0.96	0	0	0	0

Table B.4: Number of Axles Per Vehicle

Site	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
091803	Single	1.84	2	1	1	2.36	1.05	1.01	2	4	1
	Tandem	0.67	0	1	0.11	0.72	1.96	0.99	0	1	0
	Tridem	0	0	0	1	0.82	0.08	0.99	1	0	2
	Quad	0	0	0	0	0	0	0.26	0	0	0
231001	Single	1.83	2.14	1	1.01	2.34	1.47	1.01	5	4	1.71
	Tandem	0.17	0.04	1	0.02	0.66	1.76	1.09	0	1	1.82
	Tridem	0	0	0	0.85	0	0	0.91	0	0	0.65
	Quad	0	0	0	0.13	0	0	0	0	0	0
231009	Single	1.76	2.11	1	1	2.19	1.21	1.03	5	4	0
	Tandem	0.24	0.03	1	0	0.81	1.89	1.22	0	1	0
	Tridem	0	0	0	1	0	0	0.78	0	0	0
	Quad	0	0	0	0	0	0	0	0	0	0
231028	Single	1.57	2.17	1	1	2.35	1.45	1.01	5	4	1.16
	Tandem	0.43	0.03	1	0	0.64	1.77	1.11	0	1	0.32
	Tridem	0	0	0	0.99	0	0	0.89	0	0	1.81
	Quad	0	0	0	0.01	0	0	0	0	0	0
251003	Single	1.87	2	1	1	2.18	1.04	1	2.75	0	0
	Tandem	0.64	0.04	1	0	0.83	1.96	1.47	1	0	0
	Tridem	0	0	0	1	0.11	0.17	0.97	0.25	0	0
	Quad	0	0	0	0	0	0	0	0	0	0
341003	Single	1.37	2	1	0.91	2.34	1.07	1.02	2.04	2.5	1
	Tandem	0.66	0.01	1	1.13	0.66	1.95	1.01	0.55	1	0.4
	Tridem	0	0	0	0.91	0	0.02	0.99	1	0.5	2
	Quad	0	0	0	0	0	0	0.31	0	0	0.4
341011	Single	1.33	2	1	0.99	2.11	1.08	1.04	4.12	3.86	1.02
	Tandem	0.67	0	1	0.14	0.89	1.95	1	0.11	1.05	0.9
	Tridem	0	0	0	0.99	0	0.01	0.96	0.37	0.51	1.35
	Quad	0	0	0	0	0	0	0.15	0	0	0.86
341030	Single	1.53	2	1	0.98	2.41	1.1	1.02	0	0	0
	Tandem	0.47	0	1	0.04	0.59	1.95	1.11	0	0	0
	Tridem	0	0	0	0.97	0	0	0.86	0	0	0
	Quad	0	0	0	0.01	0	0	0	0	0	0
341031	Single	1.4	1.99	1	1	2.17	1.09	1.01	4.86	2.53	1.24
	Tandem	0.6	0.01	1	0.02	0.83	1.95	1	0.09	1.16	1.12
	Tridem	0	0	0.01	1	0	0	0.99	0.12	1.09	1.9
	Quad	0	0	0	0	0	0	0.03	0	0	0.84

Table B.4: Number of Axles Per Vehicle (Continued)

Site	Axles	Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
41033	Single	1.61	2.04	1	0.91	2.27	1.16	1.01	4.34	1.33	1
	Tandem	0.39	0.01	1	0.45	0.67	1.91	1.47	0.27	1.08	0.79
	Tridem	0	0	0	0.63	0.02	0	0.52	0	0.72	1
	Quad	0	0	0	0	0	0	0	0	0	0.29
341034	Single	1.48	2	1	0.99	2.19	1.09	1.01	4.64	3.21	1.13
	Tandem	0.52	0	1	0.06	0.81	1.95	1	0.11	0.95	1.15
	Tridem	0	0	0	0.99	0	0.01	0.99	0.23	1.02	1.06
	Quad	0	0	0	0	0	0	0.07	0	0	1.04
341638	Single	1.51	2	1	1	2.19	1.08	1.01	4.69	3.18	1.24
	Tandem	0.49	0	1	0.05	0.81	1.95	1	0.1	1.31	1.86
	Tridem	0	0	0	1	0.01	0.01	0.99	0.19	0.72	1.81
	Quad	0	0	0	0	0	0	0.08	0	0	0.92
421597	Single	1.91	2	1	1	2.26	1.26	1.06	5	4	1.13
	Tandem	0.09	0	1	0	0.74	1.87	1.12	0	1	0.5
	Tridem	0	0	0	1	0	0	0.86	0	0	0.88
	Quad	0	0	0	0	0	0	0	0	0	0.63
421599	Single	1.94	2	1	1	2.33	1.23	1.02	5	4	2.65
	Tandem	0.06	0	1	0	0.67	1.89	1.16	0	1	1.65
	Tridem	0	0	0	1	0	0	0.83	0	0	0.38
	Quad	0	0	0	0	0	0	0	0	0	0.09
501002	Single	1.24	2	1	0.96	2.14	1.02	1.06	2.99	2	1.25
	Tandem	0.76	0.01	1	0.66	0.86	1.98	1.03	1.01	2	2.5
	Tridem	0	0	0	0.96	0	0.02	0.97	0	0.14	1
	Quad	0	0	0	0	0	0	0.39	0	0	0
501004	Single	1.71	2	1	0.89	2.24	1.12	1.07	2	1.54	1
	Tandem	0.42	0	1	1.67	0.77	1.93	1.07	1.08	1.49	1
	Tridem	0	0	0	0.89	0	0.03	0.94	1	0.72	0
	Quad	0	0	0	0	0	0	0	0	0	1
501681	Single	1.3	1.99	1	0.93	2.16	1.03	1.03	3.02	2.28	1
	Tandem	0.7	0.01	1	1.03	0.84	1.97	1.02	0.99	1.86	2.5
	Tridem	0	0	0	0.93	0.01	0.01	0.98	0	0	1.17
	Quad	0	0	0	0	0	0	0.12	0	0	0
501683	Single	1.36	2	1	0.97	2.14	1.02	1.1	3	2.02	2.08
	Tandem	0.64	0.01	1	0.78	0.85	1.98	1.08	1	1.89	1.33
	Tridem	0	0	0	0.97	0	0.01	0.94	0	0.82	1.08
	Quad	0	0	0	0	0	0	0.13	0	0	0

Appendix C: Extracted Long-Term Pavement Performance (LTPP) Structural and Materials Properties Design Inputs

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Table C.1: General Information on the Selected LTPP Sections

STATE CODE	SHRP ID	CONSTRUCTION DATE		Number of LTPP LANES	TOTAL LANES	Functional Class	Direction
		1	2				
9	1803	1-Jul-88	17-Jan-95	1	2	Rural Major Collector	N
23	1001	1-Jul-88	6-Jun-95	2	4	Rural Principal Arterial - Interstate	N
23	1009	1-Jul-88	22-Aug-93	1	2	Rural Principal Arterial - Other	N
23	1028	1-Jul-88	12-May-92	1	2	Rural Principal Arterial - Other	E
25	1003	1-Jun-88	7-Jun-88	1	2	Urban Other Principal Arterial	N
34	1003	1-Aug-88	8-Apr-94	2	4	Rural Minor Arterial	N
34	1011	1-Jul-88	28-Apr-98	2	4	Rural Principal Arterial - Interstate	E
34	1030	1-Dec-88	24-Feb-91	2	4	Rural Principal Arterial - Other	S
34	1031	1-Jul-88	4-Apr-96	2	4	Urban Principal Arterial - Other Freeways	N
34	1033	1-Jul-88	11-Sep-97	2	4	Rural Principal Arterial - Other	S
34	1034	1-Dec-88	-	2	4	Urban Principal Arterial - Other Freeways	S
34	1638	1-Dec-88	-	2	4	Urban Principal Arterial - Other Freeways	N
36	1008	1-May-89	25-Aug-89	2	4	Urban Other Principal Arterial	E
36	1011	1-Jun-88	14-Sep-93	2	4	Urban Principal Arterial - Interstate	S
36	1643	1-May-89	12-Oct-89	1	2	Rural Principal Arterial - Other	N
36	1644	1-May-89	19-Jun-96	1	2	Rural Minor Arterial	W
42	1597	1-Aug-88	12-Jun-90	1	2	Rural Minor Arterial	E
42	1599	1-Aug-88	1-Jun-99	1	2	Urban Other Principal Arterial	W
50	1002	1-Aug-88	-	1	2	Rural Principal Arterial - Other	N
50	1004	1-Aug-88	6-Oct-98	1	2	Rural Principal Arterial - Other	E
50	1681	1-Jun-89	8-Sep-91	1	2	Rural Principal Arterial - Other	N
50	1683	1-Jun-89	23-Sep-91	1	2	Rural Principal Arterial - Other	S

Table C.2: Gradation Data of HMA Aggregates

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
23	1001	1												63				45
23	1001	2							63					17				3
23	1001	3					51							8				3
23	1001	4						87	74	64					10			2
23	1001	5	88		62		49	44	36	31		27			11		3	2
23	1001	6	100		99		70		39	33		27			13		7	3
23	1001	7	100	100	100	100	100	98	41	18			9					4
50	1002	2	75				51		24									4
50	1002	3	75		60		52		31	23								1
50	1002	4	100		99		81	71	52	38		29	20		10			2
50	1002	5	100	100	100		99	82	64	48		34	23		12			3
25	1003	1												70				20.3
25	1003	2	83		77		71	66	56		47	31		16		6		3
25	1003	3	100	100	93		65		35		25			12				2
25	1003	4	100	100	100	100	100	88	60		39	26		18		10		4
34	1003	2			86				56						9			5
34	1003	3	98				70		50	40					16			
34	1003	4	100	100	100	100	100	98	69	50					19			7
50	1004	1												77				19.5
50	1004	2	69				46		30									5
50	1004	3	79		60		48		28	23								2
50	1004	4	100	100	100		83	72	55	40		29	20		13			3
50	1004	5	100	100	100	100	100	84	61	47		35	25		16			3

Table C.2: Gradation Data of HMA Aggregates (Continued)

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
23	1009	2												16				2
23	1009	3					61							12				3
23	1009	4			64			47	42	34		25			10		8	3
23	1009	5	100	100	100	100	100	99	71	51		38	25		15		8	4
34	1011	2			91				73						18			4
34	1011	3			87				49	37					15			6
34	1011	4	100	100	100	100	100	98	72	46					18			6
23	1028	1												13				1.2
23	1028	2												16				5
23	1028	3					61							16				3
23	1028	4	100		96		77	59	40	32		26	18		12		6	2
23	1028	5	94		73		55	44	35	29		23	16		11		8	3
34	1030	2	100	100	100	100	100		95									6
34	1030	3			67				52					25				6
34	1030	4	7			3												
34	1030	5			83				48	42					17			6
34	1030	6	100	100	100	100	100	97	62	51					19			6
34	1031	2			94				69						12			6
34	1031	3	99				69		36	30					13			3
34	1031	4	100	100	100	100	100	93	60	48					18			6
34	1033	2			81				47						11			4
34	1033	3	100				77		49	40					15			7
34	1033	4	100				75		45	32					12			5
34	1033	5	100	100	100	100	100	98	70	51					18			7

Table C.2: Gradation Data of HMA Aggregates (Continued)

STATE CODE	SHRP ID	LAYER NO	1	7/8	3/4	5/8	1/2	3/8	#4	#8	#10	#16	#30	#40	#50	#80	#100	#200
			Percent Passing															
34	1034	2	100				74		45	38					16			6
34	1034	3	100		98		82	71	46	40					16			5
42	1597	2	100		76			53	37	27		20						5
42	1597	3																
42	1597	4	100	100	100	100	100	90	63	45		33	23		15		9	7
42	1599	2			76			51	24			6						3
42	1599	3			90		69	57	36	25		16	11		8		6	4.5
42	1599	4	98				69	57	36	25		16	11		8		6	4.5
42	1599	5	100	100	100	100	100	95	60	42		26	17		11		8	5.5
34	1638	3	100				74		45	38					16			6
34	1638	4	100		98		82	71	46	40					16			5
50	1681	1												17.6				10.2
50	1681	3	69		66		61	57	47	36		26	18		10		5	3
50	1681	5	100	100	100		93	76	53	37		29	24		20			5
50	1683	1												51.7				41.5
50	1683	3	86		83		78	73	60	51		40	31		20		10	6
50	1683	5	100	100	100		92	79	54			29	23		19			6
9	1803	2						47			34			17			5	2
9	1803	3	100		72				35	30					14			4
9	1803	4	100	100	100		99	78	52	42					17			5

Table C.3: Binder Content

STATE CODE	SHRP ID	LAYER NO	MAX SP. GRAVITY	BULK SP.GRAVITY MEAN	ASPHALT CONTENT MEAN	PERCENT AIR VOIDS MEAN	VOIDS MINERAL AGGREGATE	EFFECTIVE ASPHALT CONTENT
23	1001	4		2.24	4			
23	1001	5	2.49	2.38	5.1	4.3	15	
23	1001	6	2.47	2.33	5.4	5.7	14.7	
23	1001	7	2.512	2.455	6.2	10.8	22.3	
33	1001	5	2.521	2.41	4.5	6.7	15.3	
33	1001	6	2.457	2.34	6.3	4.9	17.7	
25	1002	4	2.67	2.53	4.4	4.8		
25	1002	5	2.58	2.33	6.3	8.8		
50	1002	4	2.488	2.382	5.5	4.2	15.6	4.9
25	1003	3	2.45	2.27	5	6.5		
25	1003	4	2.39	2.26	6.4	5.3		
25	1004	4	2.63	2.54	4.5	3.6		
25	1004	5	2.63	2.54	4.5	3.6		
50	1004	3	2.502	2.389	5	4.5	14	4.1
50	1004	4	2.471	2.38	5.5	3.7	14.2	4.5
50	1004	5	2.439	2.359	6.2	3.1	15.4	5.3
23	1009	4	2.49	2.41	5.1		15.5	
23	1009	5	2.415	2.405	7.1	7.2	16.8	
23	1012	4	2.448	2.405	5.2	1.7	13.5	5
23	1012	5	2.397	2.39	6.5	0.2	15.3	6.4
23	1026	4	2.545	2.48	5	2.7	14.9	
23	1026	5	2.515	2.455	5	5	16.6	

Table C.3: Binder Content (Continued)

STATE CODE	SHRP ID	LAYER NO	MAX SP.GRAVITY	BULK SP.GRAVITY MEAN	ASPHALT CONTENT MEAN	PERCENT AIR VOIDS MEAN	VOIDS MINERAL AGGREGATE	EFFECTIVE ASPHALT CONTENT
23	1028	4	2.52	2.36	5.1	6.5	18	
23	1028	5	2.5	2.34	5.1	6.5	17.7	
42	1599	3	2.637		3.4			
42	1599	4	2.571	2.486	4.6	3.3	14	4.3
42	1599	5	2.522	2.425	6	3.9	16.3	5.3
9	1803	3	2.546		4.3	7.6		
9	1803	4	2.526	2.449	5.2	3.1	15.7	
34	1003	3			4.4			
34	1003	4			5.8			
34	1011	3			5			
34	1011	4			5.8			
34	1030	5			4.2			
34	1030	6			5.4			
34	1031	3			4.6			
34	1031	4			5.6			
34	1033	3			4.7		16.4	
34	1033	4			4.7		16.6	
34	1033	5			5.9		19.5	
34	1034	2			4.9			
34	1034	3			4.4		13.9	
34	1638	3			4.4			
34	1638	4			4.9			

Table C.4: Binder Gradation

STATE CODE	SHRP ID	Layer No	AC Grade	AC SG	AC viscosity 140°F	AC viscosity 275°F	AC Penetration 77°F	Lab viscosity 140°F	Lab viscosity 275°F	Lab Duct. 77°F	Lab penetration 77°F
23	1001	4	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	5	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	6	AC-10	1.031	1058	350	114	1120	323.8	150	56
23	1001	7	AC-20	1.04	1810	418.33	83	1800	425	150	48
50	1002	3	85-100 pen	1.022	1144	308	92				
50	1002	4	85-100 pen	1.022	1144	308	92				
50	1002	5	85-100 pen	1.022	1144	308	92				
25	1003	3	AC-20	1.026	2064	401	73	4042			
25	1003	4	AC-20	1.026	1772	377	82	3976			
34	1003	3	AC-20	1.025	2021		72				
34	1003	4	AC-20	1.025	2021		72				
50	1004	3	85-100 pen	1.022	1159	311	90				58
50	1004	4	85-100 pen	1.023	1159	311	90				60
50	1004	5	85-100 pen	1.023	1159	311	59				60
23	1009	4	85-100 pen	1.023	1778	400	89			150	58
23	1009	5	85-100 pen	1.023	1765	390.5	90			150	60
34	1011	3	85-100 pen	1.025			91				
34	1011	4	85-100 pen	1.029			91				
36	1011	4	AC-20	1.024							
23	1028	4	AC-10	1.014	1125	311	120	2420		150	74
23	1028	5	AC-10	1.014	1125	311	120	2420		150	74

Table C.4: Binder Gradation (Continued)

STATE CODE	SHRP ID	Layer No	AC Grade	AC SG	AC viscosity 140°F	AC viscosity 275°F	AC Penetration 77°F	Lab viscosity 140°F	Lab viscosity 275°F	Lab Duct. 77°F	Lab penetration 77°F
34	1030	5	AC-20	1.025							
34	1030	6	AC-20	1.025							
34	1031	3	AC-20	1.025	1793	465	74				
34	1031	4	AC-20	1.025	1968	412	70				
34	1033	3	AC-20	1.025	2124	446	67				
34	1033	4	AC-20	1.025	2124	446	67				
34	1033	5	AC-20	1.025	2124	446	67				
34	1034	2	AC-20	1.02	2108	406	77				
34	1034	3	AC-20	1.02	2108	406	77				
42	1597	3	AC-20		2000						
42	1597	4	AC-20	1.01	2000						
1599	42	3	AC-20	1.024	2037	452	79				
1599	42	4	AC-20	1.024	2037	452	79				
1599	42	5	AC-20	1.024	2037	452	79				
34	1638	3	AC-20	1.02	2108	406	77				
34	1638	4	AC-20	1.02	2108	406	77				
50	1681	5	85-100 pen	1.01							
50	1683	5	85-100 pen	1.01							
9	1803	3	AC-20	1.01	2052		69	4462			54
9	1803	4	AC-20	1.01	2052		69	4462			54

Table C.5: Subgrade Soil Data

State Code	SHRP Id	Construction No	Layer No	AASHTO Soil Class	CBR	Plasticity Index	Liquid Limit	Maximum Lab Dry Density	Optimum Lab Moisture Content	In Situ Dry Density Mean	In Situ Moisture Optimum Mean
23	1001	1	1	A-4				135	6.7		
50	1002	1	1	A-7-6							
25	1003	1	1	A-2-4	10			114	12	106	
50	1004	1	1	A-6		0	0	112	12.6	102	82.1
23	1009	1	1	A-4							
23	1028	1	1	A-1a		0	0	128	8.5		
50	1681	1	1	A-1a		3	18				
50	1683	1	1	A-1a		11	26				
9	1803	1	1					122	12.4		118.2
34	1003	1	1	A-7-6							
34	1011	1	1	A-7-6							
34	1030	1	1	A-4							
34	1031	1	1	A-7-6							
34	1033	1	1	A-2-4							
34	1034	1	1	A-1-a							
34	1638	1	1	A-1-b							
42	1597	1	1	A-7-5							
42	1599	1	1	A-7-5							

Table C.6: Base Layer Data

State Code	SHRP Id	Construction No	Layer No	AASHTO Soil Class	Plasticity Index	Max Lab Dry Density	Optimum Lab Moisture	In Situ Dry Density Mean	In Situ Moisture Mean
23	1001	1	2	A-1-b	1	131	6.5	129	7
23	1001	1	3	A-1-a		139	6.1		
25	1003	1	2	A-1-a		125	8.4		
23	1009	1	2	A-1-b	1	133	10	126	3
23	1009	1	3	A-1-a		139	7.9	139	3
23	1028	1	2	A-1-a		142	6.2	141	4
23	1028	1	3	A-1-a		143	7.4	137	3
34	1031	1	2	A-1-a					7
34	1033	1	2	A-1-a					5
9	1803	1	2	A-1-a		137	7.6	138	5

Table C.7: Layer Thickness

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
23	1001	1	Subgrade	Poorly Graded Sand	
23	1001	2	Subbase Layer	Sand	42
23	1001	3	Base Layer	Crushed Stone, Gravel or Slag	4
23	1001	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1001	5	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1001	6	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2.2
23	1001	7	Friction Course	Hot Mixed, Hot Laid Asphalt Concrete, Open Graded (Porous Friction Course)	0.8
50	1002	1	Subgrade	Gravel	
50	1002	2	Base Layer	Crushed Stone, Gravel or Slag	24
50	1002	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
50	1002	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.8
50	1002	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.3
25	1003	1	Subgrade	Poorly Graded Sand	
25	1003	2	Base Layer	Gravel (Uncrushed)	12
25	1003	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4.7
25	1003	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.2
34	1003	1	Subgrade	Sandy Silt	
34	1003	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	24

Table C.7: Layer Thickness (Continued)

State Code	SHRP ID	Layer No	Description	Material Type	Mean Thickness
50	1004	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
50	1004	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.8
50	1004	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.3
23	1009	1	Subgrade	Poorly Graded Sand	
23	1009	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	20
23	1009	3	Base Layer	Crushed Stone, Gravel or Slag	4
23	1009	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
23	1009	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
34	1011	1	Subgrade	Silty Sand	
34	1011	2	Subbase Layer	Soil-Aggregate Mixture (Predominantly Coarse-Grained Soil)	10
34	1011	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	7.5
34	1011	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5

Table C.7: Layer Thickness (Continued)

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
34	1031	1	Subgrade	Silty Sand	
34	1031	2	Base Layer	Crushed Stone, Gravel or Slag	16
34	1031	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	6.5
34	1031	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
34	1033	1	Subgrade	Clayey Gravel	
34	1033	2	Subbase Layer	Crushed Stone, Gravel or Slag	14
34	1033	3	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4
34	1033	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	1.5
34	1033	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
34	1034	1	Subgrade	Poorly Graded Sand	
34	1034	2	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	10
34	1034	3	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2
42	1597	1	Subgrade	Silty Clay	
42	1597	2	Base Layer	Gravel (Uncrushed)	17
42	1597	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
42	1597	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	1.5
42	1599	1	Subgrade	Silty Clay	
42	1599	2	Base Layer	Gravel (Uncrushed)	12
42	1599	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	5
42	1599	4	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4

Table C.7: Layer Thickness (Continued)

State Code	SHRP ID	Layer #	Description	Material Type	Mean Thickness
34	1031	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	2
50	1681	1	Subgrade	Gravel	
50	1681	2	Subbase Layer	Sand	12
50	1681	3	Subbase Layer	Gravel (Uncrushed)	20
50	1681	4	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
50	1681	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
50	1683	1	Subgrade	Silty Sand	
50	1683	2	Subbase Layer	Sand	12
50	1683	3	Subbase Layer	Gravel (Uncrushed)	20
50	1683	4	Base Layer	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	3
50	1683	5	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3
9	1803	1	Subgrade	Silty Sand	
9	1803	2	Base Layer	Gravel (Uncrushed)	10
9	1803	3	AC Layer Below Surface (Binder Course)	Asphalt Bound, Dense Graded, Hot Laid, Central Plant Mix	4
9	1803	4	Original Surface Layer	Hot Mixed, Hot Laid Asphalt Concrete, Dense Graded	3

Appendix D: Extracted Long-Term Pavement Performance (LTPP) Performance Data

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Table D.1: Measured Rutting Data from the LTPP Database

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
231001	1989	August	0.204	0.147	0.164	0.515
231001	1990	August	0.199	0.129	0.145	0.473
231001	1991	August	0.182	0.118	0.134	0.434
231001	1992	April	-	-	-	-
231001	1993	April	0.235	0.129	0.149	0.513
231001	1994	August	-	-	-	-
231009	1989	August	0.037	0.092	0.127	0.257
231009	1990	August	0.044	0.095	0.137	0.276
231009	1991	August	0.045	0.093	0.138	0.276
231009	1992	April	-	-	-	-
231028	1989	August	0.104	0.156	0.152	0.413
231028	1990	August	0.137	0.159	0.158	0.453
231028	1991	August	0.144	0.164	0.164	0.471
251003	1989	August	0.022	0.045	0.090	0.157
341003	1989	July	0.155	0.284	0.290	0.728
341003	1990	September	0.230	0.278	0.298	0.861
341003	1991	August	0.208	0.237	0.264	0.799
341003	1992	September	0.263	0.264	0.300	0.827
341003	1993	June	-	-	-	-
341011	1989	October	0.100	0.042	0.154	0.295
341011	1990	September	0.140	0.049	0.184	0.374
341011	1991	September	-	-	-	-
341011	1992	April	0.113	0.037	0.145	0.295
341011	1993	February	0.153	0.045	0.177	0.375
341011	1994	June	-	-	-	-
341011	1995	November	0.176	0.043	0.175	0.394
341011	1997	July	0.154	0.035	0.146	0.335
341030	1989	July	0.098	0.215	0.377	0.692
341030	1990	September	0.121	0.244	0.443	0.886
341031	1989	October	0.146	0.101	0.246	0.493
341031	1990	September	0.157	0.090	0.226	0.472
341031	1991	September	-	-	-	-
341031	1992	April	0.169	0.084	0.220	0.473
341031	1993	February	0.169	0.078	0.206	0.453
341031	1994	June	-	-	-	-
341031	1995	November	0.239	0.085	0.229	0.552
341033	1989	October	0.064	0.075	0.135	0.274

Table D.1: Measured Rutting Data from the LTPP Database (Continued)

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
341033	1990	September	0.097	0.093	0.166	0.356
	1991	September	-	-	-	-
	1992	April	0.082	0.068	0.126	0.276
	1993	February	0.110	0.079	0.146	0.336
	1994	June	-	-	-	-
	1995	November	0.130	0.078	0.145	0.354
341034	1989	October	0.046	0.000	0.092	0.138
	1990	September	0.103	0.000	0.173	0.276
	1991	September	-	-	-	-
	1992	April	0.070	0.000	0.107	0.178
	1993	February	0.097	0.000	0.139	0.237
	1994	June	-	-	-	-
	1995	November	0.112	0.000	0.144	0.256
	1997	July	0.080	0.000	0.097	0.178
	1998	August	-	-	-	-
	1999	September	-	-	-	-
	2000	July	0.105	0.000	0.112	0.217
	2001	December	-	-	-	-
	2002	June	0.104	0.000	0.103	0.275
	2004	May	-	-	-	-
	2005	November	0.135	0.000	0.121	0.256
	2007	June	0.146	0.000	0.131	0.276
341638	1989	October	0.071	0.050	0.076	0.197
	1990	September	0.124	0.075	0.116	0.315
	1991	August	0.079	0.045	0.073	0.197
	1992	April	-	-	-	-
	1993	February	0.067	0.035	0.056	0.158
	1994	June	-	-	-	-
	1995	November	0.081	0.037	0.059	0.177
	1997	July	0.092	0.040	0.064	0.197
	1998	August	-	-	-	-
	1999	September	-	-	-	-
	2000	July	0.115	0.043	0.069	0.227
	2001	December	-	-	-	-
	2002	June	0.102	0.036	0.059	0.197
	2003	May	0.113	0.040	0.064	0.217
	2004	May	-	-	-	-
	2005	November	0.129	0.042	0.066	0.236

Table D.1: Measured Rutting Data from the LTPP Database (Continued)

Site	Year	Month	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
421597	1989	August	0.026	0.055	0.078	0.158
421599	1989	August	0.035	0.037	0.105	0.177
	1990	September	0.054	0.040	0.122	0.216
	1991	August	0.052	0.035	0.109	0.197
	1992	October	-	-	-	-
	1993	March	0.089	0.053	0.173	0.315
	1995	June	0.080	0.044	0.151	0.275
	1996	July	0.084	0.043	0.148	0.275
	1997	November	-	-	-	-
	1998	March	0.087	0.042	0.147	0.275
501002	1989	August	0.088	0.095	0.113	0.295
	1990	August	0.110	0.115	0.148	0.373
	1991	September	0.095	0.094	0.125	0.314
	1993	April	0.124	0.106	0.144	0.374
	1994	August	0.130	0.100	0.135	0.365
	1995	October	0.155	0.113	0.156	0.424
	1996	October	0.133	0.093	0.129	0.355
	1997	October	0.168	0.115	0.162	0.445
	1998	June	0.167	0.111	0.156	0.434
	1999	November	0.194	0.120	0.170	0.483
	2000	June	0.225	0.130	0.185	0.540
	2001	September	0.235	0.135	0.193	0.563
	2002	May	0.243	0.138	0.199	0.590
	2003	November	0.267	0.144	0.209	0.620
	2004	April	-	-	-	-
501004	1989	August	0.024	0.047	0.088	0.158
	1990	August	0.042	0.073	0.140	0.255
	1991	September	0.036	0.054	0.106	0.196
	1993	April	0.053	0.068	0.135	0.256
	1995	October	0.057	0.059	0.120	0.237
	1997	November	0.067	0.062	0.127	0.256
501681	1989	August	0.078	0.202	0.134	0.413
	1990	August	0.115	0.228	0.149	0.492
501683	1989	August	0.133	0.374	0.184	0.692
	1990	August	0.210	0.442	0.214	0.866

Table D.2: Measured Cracking and IRI data from the LTPP database

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
231001	1989	August	4814.515	0.000	779.328	118.407
	1990	August	5008.481	0.000	2286.029	138.492
	1991	August	5060.436	0.771	1233.070	115.695
	1992	April	-	-	-	109.651
	1993	April	668.490	0.000	4010.941	-
	1994	August	-	-	-	125.275
231009	1989	August	4727.923	1.597	824.356	61.231
	1990	August	5753.172	0.000	973.294	67.238
	1991	August	5625.016	1.292	1780.332	61.485
	1992	April	-	-	-	62.258
231028	1989	August	8628.027	0.000	803.574	85.523
	1990	August	9812.605	0.000	1423.572	86.056
	1991	August	5271.721	0.000	1001.004	91.707
251003	1989	August	17245.663	0.000	3082.675	122.564
341003	1989	July	1246.925	22.335	3262.787	124.471
	1990	September	1818.432	22.604	3245.468	-
	1991	August	1378.545	22.407	2279.101	102.998
	1992	September	5649.262	18.675	2549.268	95.750
	1993	June	-	-	-	103.442
341011	1989	October	5971.384	0.000	1364.690	101.972
	1990	September	6033.731	0.000	2036.644	102.529
	1991	September	-	-	-	108.548
	1992	April	5472.614	0.000	1728.376	102.136
	1993	February	5933.284	0.000	1804.577	109.220
	1994	June	-	-	-	115.645
	1995	November	10474.168	0.036	6383.562	115.746
	1997	July	-	-	-	117.951
341030	1989	July	2590.833	10.602	744.691	225.004
	1990	September	4208.371	20.469	2895.636	252.857
341031	1989	October	9750.259	3.534	3532.954	111.247
	1990	September	7155.963	2.834	1977.761	114.720
	1991	September	-	-	-	121.791
	1992	April	6082.222	4.862	1887.706	115.100
	1993	February	6549.819	10.154	6179.205	126.593
	1994	June	-	-	-	155.409
	1995	November	10692.380	9.688	5898.647	144.702
341033	1989	October	1271.171	0.000	1967.370	201.726

Table D.2: Measured Cracking and IRI data from the LTPP database (Continued)

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
341033	1990	September	1319.662	0.000	2448.822	173.796
	1991	September	-	-	-	176.610
	1992	April	1167.260	1.310	2102.454	184.010
	1993	February	710.054	0.108	2279.101	181.716
	1994	June	-	-	-	183.845
	1995	November	1420.109	0.251	2930.273	199.115
341034	1989	October	2002.007	0.000	0.000	85.245
	1990	September	2871.391	0.000	0.000	85.447
	1991	September	-	-	-	88.159
	1992	April	3484.462	0.000	0.000	87.678
	1993	February	3990.159	0.000	0.000	88.843
	1994	June	-	-	-	90.820
	1995	November	5410.268	0.000	0.000	93.279
	1997	July	-	-	-	94.153
	1998	August	-	-	-	94.964
	1999	September	-	-	-	93.545
	2000	July	13234.721	0.000	1728.376	-
	2001	December	-	-	-	98.525
	2002	June	13865.111	0.161	2885.245	96.320
	2004	May	-	-	-	96.206
	2005	November	-	-	-	97.612
	2007	June	-	-	-	101.655
341638	1989	October	516.088	0.000	0.000	56.923
	1990	September	904.020	0.000	0.000	59.685
	1991	August	-	-	-	60.762
	1992	April	910.948	0.000	0.000	56.973
	1993	February	3338.988	0.000	0.000	58.469
	1994	June	-	-	-	60.864
	1995	November	4966.917	0.000	0.000	-
	1997	July	-	-	-	65.261
	1998	August	-	-	-	63.297
	1999	September	-	-	-	65.121
	2000	July	5524.570	0.000	148.938	-
	2001	December	-	-	-	67.364
	2002	June	6601.774	0.072	443.351	65.989
	2003	May	-	-	-	64.627
	2004	May	-	-	-	65.311
	2005	November	-	-	-	66.059

Table D.2: Measured Cracking and IRI data from the LTPP database (Continued)

Site	Year	Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
421597	1989	August	547.261	0.000	762.010	107.015
421599	1989	August	0.000	0.000	0.000	86.651
	1990	September	0.000	0.000	0.000	88.590
	1991	August	72.737	0.000	405.251	89.414
	1992	October	-	-	-	92.151
	1993	March	0.000	0.000	0.000	93.836
	1995	June	-	-	-	100.552
	1996	July	422.569	0.000	155.866	-
	1997	November	-	-	-	102.491
	1998	March	-	-	-	103.011
501002	1989	August	0.000	0.000	0.000	77.958
	1990	August	27.709	0.000	0.000	77.439
	1991	September	786.255	0.000	0.000	68.023
	1992	July	-	-	-	70.697
	1993	April	2445.358	0.000	976.758	-
	1994	August	-	-	-	80.090
	1995	October	1666.030	0.000	980.221	80.727
	1996	October	-	-	-	78.136
	1997	October	-	-	-	82.502
	1998	June	-	-	-	82.143
	1999	November	-	-	-	86.170
	2000	June	6463.227	0.413	4748.705	93.494
	2001	September	-	-	-	91.986
	2002	May	9192.607	0.072	4533.957	93.514
	2003	November	-	-	-	93.332
	2004	April	-	-	-	95.116
501004	1989	August	3480.998	0.000	45.028	104.544
	1990	August	3813.512	0.108	138.547	106.825
	1991	September	4527.030	4.108	308.268	92.379
	1992	July	-	-	-	93.329
	1993	April	5330.604	0.771	1918.879	131.459
	1994	July	-	-	-	131.789
	1995	October	5230.157	0.574	2985.692	132.600
	1997	November	-	-	-	129.495
501681	1989	August	2085.135	0.000	27.709	76.361
	1990	August	308.268	0.000	131.620	76.311
501683	1989	August	7914.509	0.771	1291.953	134.450
	1990	August	2251.392	1.453	1517.092	142.560

Appendix E: Computed Distresses for the LTPP Sites Using AASHTOWare Pavement ME 2.1

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Table E.1: Computed Rutting - Global Calibration Factors

Site #	Month	Year	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
091803	1989	0.9	0.03	0.03	0.08	0.14
091803	1990	2.1	0.04	0.04	0.10	0.18
091803	1991	3.0	0.05	0.04	0.10	0.19
091803	1992	4.1	0.06	0.04	0.11	0.21
091803	1994	6.1	0.07	0.05	0.12	0.23
091803	1995	7.2	0.08	0.05	0.12	0.24
091803	1996	8.2	0.08	0.05	0.12	0.25
091803	1997	9.2	0.09	0.05	0.13	0.26
091803	1998	9.8	0.09	0.05	0.13	0.27
091803	2000	11.9	0.10	0.05	0.13	0.28
091803	2002	13.8	0.11	0.05	0.13	0.29
091803	2003	14.8	0.11	0.05	0.14	0.30
091803	2004	15.7	0.12	0.05	0.14	0.31
091803	2007	18.8	0.13	0.05	0.14	0.32
231001	1989	1.0	0.01	0.04	0.05	0.10
231001	1990	2.0	0.06	0.06	0.08	0.20
231001	1991	3.0	0.06	0.06	0.08	0.20
231001	1993	4.7	0.08	0.07	0.10	0.25
231001	1995	7.2	0.10	0.08	0.11	0.29
231001	1999	11.1	0.13	0.08	0.11	0.32
231001	2000	11.8	0.13	0.08	0.11	0.33
231001	2002	13.8	0.14	0.08	0.11	0.33
231009	1989	1.0	0.01	0.06	0.10	0.18
231009	1990	2.0	0.06	0.11	0.17	0.34
231009	1991	3.0	0.07	0.12	0.19	0.38
231009	1993	4.8	0.08	0.13	0.21	0.41
231009	1995	7.2	0.10	0.14	0.22	0.46
231009	1997	9.0	0.12	0.14	0.23	0.49
231009	1998	10.1	0.12	0.14	0.24	0.50
231009	1999	10.8	0.12	0.14	0.24	0.51
231009	2001	13.0	0.14	0.15	0.25	0.53
231009	2003	14.8	0.14	0.15	0.26	0.55
231009	2004	15.7	0.15	0.15	0.26	0.56
231028	1989	1.0	0.06	0.06	0.10	0.22
231028	1990	2.0	0.08	0.06	0.12	0.26
231028	1991	3.0	0.09	0.06	0.13	0.28
231028	1993	4.7	0.10	0.07	0.13	0.31

Table E.1: Computed Rutting - Global Calibration Factors (Continued)

231028	1995	7.2	0.14	0.07	0.14	0.35
231028	1998	9.8	0.15	0.08	0.15	0.38
231028	1999	10.8	0.17	0.08	0.15	0.40
231028	2001	13.0	0.20	0.08	0.16	0.43
231028	2003	14.8	0.21	0.08	0.16	0.45
231028	2004	15.8	0.22	0.08	0.16	0.46
251003	1989	1.1	0.03	0.05	0.11	0.19
251003	1990	2.2	0.04	0.05	0.13	0.22
251003	1991	3.1	0.05	0.05	0.13	0.24
251003	1992	4.2	0.06	0.06	0.14	0.26
251003	1995	7.3	0.07	0.07	0.15	0.30
251003	1996	8.3	0.08	0.07	0.16	0.30
251003	1998	9.9	0.08	0.07	0.16	0.32
341003	1989	0.8	0.06	0.06	0.15	0.27
341003	1990	2.0	0.11	0.06	0.19	0.36
341003	1991	2.9	0.12	0.07	0.20	0.38
341003	1992	4.0	0.14	0.07	0.22	0.42
341003	1994	6.2	0.17	0.07	0.23	0.47
341003	1995	7.1	0.18	0.07	0.24	0.49
341003	1999	10.5	0.22	0.08	0.26	0.56
341003	2000	11.8	0.25	0.08	0.27	0.59
341003	2002	14.0	0.27	0.08	0.28	0.63
341003	2005	17.2	0.31	0.08	0.29	0.68
341011	1989	1.2	0.08	0.04	0.14	0.26
341011	1990	2.1	0.11	0.04	0.16	0.31
341011	1992	3.7	0.12	0.05	0.17	0.34
341011	1993	4.5	0.14	0.05	0.18	0.38
341011	1995	7.3	0.19	0.05	0.20	0.45
341011	1997	8.9	0.21	0.05	0.21	0.48
341011	1999	11.2	0.24	0.06	0.22	0.52
341011	2000	11.9	0.25	0.06	0.23	0.54
341011	2002	14.1	0.29	0.06	0.24	0.58
341011	2007	19.2	0.36	0.06	0.26	0.68
341030	1989	1.5	0.03	0.08	0.10	0.22
341030	1990	2.7	0.06	0.10	0.13	0.29
341030	1991	3.6	0.06	0.10	0.14	0.30
341030	1992	4.7	0.07	0.11	0.15	0.33
341030	1995	7.8	0.09	0.12	0.17	0.38
341030	1997	9.5	0.10	0.13	0.18	0.40
341030	1999	11.3	0.11	0.13	0.18	0.42

Table E.1: Computed Rutting - Global Calibration Factors (Continued)

341030	2000	12.5	0.12	0.13	0.18	0.43
341030	2001	13.7	0.12	0.13	0.19	0.45
341030	2005	17.8	0.14	0.14	0.20	0.47
341030	2007	19.4	0.14	0.14	0.20	0.48
341031	1989	1.2	0.08	0.04	0.15	0.26
341031	1990	2.1	0.10	0.04	0.17	0.31
341031	1992	3.7	0.13	0.05	0.18	0.36
341031	1993	4.5	0.15	0.05	0.19	0.39
341031	1995	7.3	0.21	0.05	0.22	0.48
341031	1996	8.0	0.22	0.05	0.22	0.50
341031	1999	11.1	0.27	0.06	0.24	0.57
341031	2000	11.9	0.29	0.06	0.25	0.59
341031	2002	14.1	0.33	0.06	0.26	0.65
341031	2005	17.3	0.39	0.06	0.28	0.73
341033	1989	1.2	0.04	0.04	0.08	0.16
341033	1990	2.1	0.06	0.05	0.09	0.19
341033	1992	3.7	0.07	0.05	0.09	0.21
341033	1993	4.5	0.08	0.05	0.10	0.23
341033	1995	7.3	0.11	0.06	0.11	0.27
341033	1997	9.2	0.13	0.06	0.11	0.30
341033	2000	12.2	0.16	0.06	0.12	0.34
341033	2002	13.8	0.17	0.06	0.12	0.35
341033	2003	14.9	0.18	0.07	0.12	0.37
341033	2004	15.7	0.18	0.07	0.12	0.37
341033	2007	18.8	0.21	0.07	0.13	0.41
341034	1989	1.8	0.04	0.03	0.08	0.15
341034	1990	2.7	0.06	0.03	0.09	0.17
341034	1992	4.3	0.07	0.04	0.10	0.21
341034	1993	5.1	0.08	0.04	0.10	0.22
341034	1995	7.8	0.11	0.04	0.11	0.26
341034	1997	9.5	0.11	0.04	0.11	0.27
341034	2000	12.8	0.14	0.04	0.12	0.29
341034	2002	14.4	0.15	0.05	0.12	0.31
341034	2005	17.8	0.17	0.05	0.12	0.34
341034	2007	19.4	0.18	0.05	0.12	0.35
341638	1989	1.8	0.10	0.10	0.12	0.32
341638	1990	2.7	0.14	0.11	0.14	0.39
341638	1991	3.6	0.17	0.11	0.15	0.43
341638	1993	5.1	0.19	0.12	0.16	0.47
341638	1995	7.8	0.25	0.13	0.17	0.56
341638	1997	9.5	0.27	0.14	0.18	0.58

Table E.1: Computed Rutting - Global Calibration Factors (Continued)

341638	2000	12.8	0.33	0.14	0.18	0.65
341638	2002	14.4	0.36	0.14	0.19	0.69
341638	2003	15.3	0.37	0.14	0.19	0.70
341638	2005	17.8	0.41	0.15	0.19	0.75
421597	1989	0.9	0.02	0.04	0.05	0.11
421597	1990	1.8	0.02	0.04	0.06	0.13
421597	1991	2.9	0.03	0.05	0.07	0.15
421597	1993	4.5	0.04	0.05	0.08	0.17
421597	1994	5.8	0.04	0.05	0.09	0.19
421597	1995	7.0	0.05	0.05	0.09	0.20
421597	1996	7.8	0.05	0.06	0.09	0.20
421597	1997	9.0	0.06	0.06	0.10	0.21
421597	2000	12.2	0.07	0.06	0.10	0.23
421597	2002	13.7	0.07	0.06	0.11	0.24
421597	2003	14.8	0.08	0.06	0.11	0.25
421597	2007	18.9	0.09	0.06	0.12	0.28
421599	1989	0.8	0.03	0.03	0.08	0.14
421599	1990	1.9	0.05	0.03	0.10	0.18
421599	1991	3.0	0.06	0.03	0.11	0.20
421599	1993	4.5	0.07	0.04	0.12	0.22
421599	1995	6.8	0.08	0.04	0.14	0.25
421599	1996	7.8	0.09	0.04	0.14	0.27
421599	1998	9.5	0.10	0.04	0.15	0.28
421599	2000	11.9	0.11	0.04	0.15	0.31
421599	2001	13.0	0.12	0.04	0.16	0.32
421599	2002	13.8	0.12	0.04	0.16	0.32
421599	2003	14.6	0.12	0.04	0.16	0.33
421599	2005	16.9	0.14	0.04	0.17	0.35
501002	1989	0.9	0.04	0.05	0.06	0.15
501002	1990	1.9	0.05	0.05	0.07	0.17
501002	1991	3.0	0.06	0.06	0.08	0.19
501002	1993	4.6	0.07	0.06	0.08	0.22
501002	1994	5.9	0.09	0.07	0.09	0.24
501002	1995	7.1	0.10	0.07	0.09	0.26
501002	1996	8.1	0.10	0.07	0.09	0.27
501002	1997	9.1	0.11	0.07	0.10	0.27
501002	1998	9.8	0.11	0.07	0.10	0.28
501002	1999	11.2	0.12	0.07	0.10	0.30
501002	2000	12.0	0.13	0.07	0.10	0.31

Table E.1: Computed Rutting - Global Calibration Factors (Continued)

501002	2001	13.0	0.14	0.07	0.10	0.31
501002	2002	14.1	0.14	0.07	0.11	0.32
501002	2003	15.2	0.15	0.08	0.11	0.33
501004	1989	0.9	0.02	0.04	0.08	0.14
501004	1990	1.9	0.03	0.05	0.09	0.16
501004	1991	3.0	0.04	0.05	0.10	0.18
501004	1993	4.6	0.04	0.05	0.10	0.20
501004	1995	7.1	0.06	0.06	0.11	0.23
501004	1997	9.2	0.07	0.06	0.12	0.25
501004	1999	10.8	0.08	0.06	0.12	0.26
501004	2000	11.8	0.08	0.06	0.12	0.26
501004	2001	12.9	0.08	0.06	0.13	0.27
501004	2002	13.7	0.08	0.06	0.13	0.27
501004	2004	15.7	0.09	0.06	0.13	0.28
501004	2007	18.9	0.10	0.07	0.14	0.30
501681	1989	1.1	0.02	0.06	0.04	0.11
501681	1990	2.1	0.04	0.09	0.06	0.19
501681	1991	3.2	0.06	0.11	0.07	0.24
501681	1993	4.8	0.07	0.11	0.07	0.26
501681	1995	7.3	0.10	0.13	0.08	0.30
501681	1998	9.9	0.13	0.13	0.08	0.34
501681	1999	10.9	0.13	0.13	0.08	0.35
501681	2001	13.2	0.15	0.14	0.09	0.37
501681	2003	14.9	0.16	0.14	0.09	0.39
501681	2004	16.1	0.17	0.14	0.09	0.40
501683	1989	1.1	0.02	0.05	0.03	0.10
501683	1990	2.1	0.04	0.09	0.04	0.18
501683	1991	3.2	0.06	0.10	0.05	0.21
501683	1993	4.8	0.07	0.11	0.05	0.24
501683	1995	7.3	0.10	0.12	0.06	0.28
501683	1998	9.9	0.12	0.13	0.06	0.31
501683	1999	10.9	0.13	0.13	0.06	0.32
501683	2001	13.2	0.15	0.13	0.06	0.35
501683	2003	14.9	0.16	0.14	0.07	0.36
501683	2004	16.1	0.17	0.14	0.07	0.38
501683	2007	19.1	0.19	0.15	0.07	0.40

Table E.2: Computed Cracking and IRI - Global Calibration Factors

Site #	Month	Year	Alligator Cracking (%)	Transverse Cracking (ft/mi)	IRI (in/mi)
231001	1989	1	0.0065	1,556.30	113.9
231001	1990	2	0.0123	1,590.53	116.4
231001	1991	3	0.0131	1,596.72	117.7
231001	1993	4.67	0.0224	2,112.00	128.5
231009	1989	1	0.0210	2,112.00	126.3
231009	1990	2	0.0462	2,112.00	130.2
231009	1991	3	0.0709	2,112.00	133.3
231028	1989	1	0.0169	0.02	97.9
231028	1990	2	0.0347	1.14	101
231028	1991	3	0.0526	1,535.28	120.9
251003	1989	1.08	0.0072	1,594.43	114.9
341003	1989	0.83	0.0415	1,461.87	117.9
341003	1990	2	0.1200	1,498.01	124.3
341003	1991	2.92	0.1610	1,553.44	127.4
341003	1992	4	0.2280	1,767.17	133.6
341011	1989	1.17	0.0197	0.02	100.7
341011	1990	2.08	0.0378	6.91	104.4
341011	1992	3.67	0.0587	826.01	117.6
341011	1993	4.5	0.0829	1,518.83	128.8
341011	1995	7.25	0.1610	1,552.86	138.4
341030	1989	1.5	0.0143	0.02	98.1
341030	1990	2.67	0.0411	0.54	103
341031	1989	1.17	0.0121	0.00	100.7
341031	1990	2.08	0.0225	0.01	104.3
341031	1992	3.67	0.0412	24.97	109.5
341031	1993	4.5	0.0551	24.99	112.5
341031	1995	7.25	0.1130	38.17	123.0
341033	1989	1.17	0.0045	12.88	95.2
341033	1990	2.08	0.0083	27.60	97.7
341033	1992	3.67	0.0134	1,075.56	113.0
341033	1993	4.5	0.0182	1,812.88	123.9
341033	1995	7.25	0.0342	1,908.68	131.8
341034	1989	1.75	0.0011	0.00	93.9
341034	1990	2.67	0.0021	0.00	96.0
341034	1992	4.25	0.0041	153.56	101.2
341034	1993	5.08	0.0054	153.58	103.0
341034	1995	7.83	0.0096	212.00	109.9

Table E.2: Computed Cracking and IRI - Global Calibration Factors (Continued)

341034	2002	14.42	0.0214	965.12	134.7
341638	1989	1.75	0.1360	0.00	104.0
341638	1990	2.67	0.2870	0.00	108.4
341638	1993	5.08	0.6510	325.74	120.6
341638	2002	14.42	2.6600	1,350.81	165.3
421597	1989	0.92	0.0010	0.02	92.1
421599	1989	0.83	0.0007	0.11	93.3
421599	1990	1.92	0.0020	5.44	96.8
421599	1991	3	0.0033	86.22	100.6
421599	1993	4.5	0.0048	1,890.96	125.0
501002	1989	0.92	0.0010	2,112.00	119.1
501002	1990	1.92	0.0021	2,112.00	121.2
501002	1991	3	0.0036	2,112.00	123.8
501002	1993	4.58	0.0054	2,112.00	127.5
501002	1995	7.08	0.0098	2,112.00	134.3
501002	2000	12	0.0186	2,112.00	149.1
501002	2002	14.08	0.0225	2,112.00	155.6
501004	1989	0.92	0.0005	2,112.00	118.4
501004	1990	1.92	0.0010	2,112.00	120.5
501004	1991	3	0.0017	2,112.00	122.8
501004	1995	7.08	0.0042	2,112.00	132.0
501681	1989	1.08	0.0007	0.00	91.4
501681	1990	2.08	0.0098	2,112.00	121.3
501683	1989	1.08	0.0008	0.00	90.7
501683	1990	2.08	0.0102	2,112.00	120.5

Table E.3: Computed Rutting - Local Calibration Factors

Site #	Month	Year	AC Rutting (in)	Base Rutting (in)	Subgrade Rutting (in)	Total Rutting (in)
91803	1989	0.92	0.02	0.03	0.06	0.10
91803	1990	2.08	0.02	0.03	0.07	0.13
91803	1991	3	0.03	0.03	0.08	0.14
91803	1992	4.08	0.03	0.03	0.08	0.15
91803	1994	6.08	0.04	0.04	0.09	0.17
91803	1995	7.17	0.05	0.04	0.09	0.17
91803	1996	8.17	0.05	0.04	0.09	0.18
91803	1997	9.17	0.05	0.04	0.09	0.18
91803	1998	9.83	0.05	0.04	0.09	0.19
91803	2000	11.92	0.06	0.04	0.10	0.20
91803	2002	13.75	0.06	0.04	0.10	0.21
91803	2003	14.83	0.07	0.04	0.10	0.21
91803	2004	15.67	0.07	0.04	0.10	0.21
91803	2007	18.83	0.08	0.04	0.11	0.23
231001	1989	1	0.01	0.03	0.04	0.08
231001	1990	2	0.04	0.05	0.06	0.14
231001	1991	3	0.04	0.05	0.06	0.15
231001	1993	4.67	0.04	0.06	0.08	0.18
231001	1995	7.17	0.06	0.06	0.08	0.20
231001	1999	11.08	0.08	0.07	0.08	0.23
231001	2000	11.83	0.08	0.07	0.08	0.23
231001	2002	13.83	0.08	0.07	0.08	0.23
231009	1989	1	0.01	0.05	0.07	0.13
231009	1990	2	0.03	0.09	0.13	0.26
231009	1991	3	0.04	0.10	0.14	0.28
231009	1993	4.75	0.05	0.10	0.15	0.30
231009	1995	7.17	0.06	0.11	0.17	0.34
231009	1997	9	0.07	0.11	0.17	0.35
231009	1998	10.08	0.07	0.12	0.18	0.36
231009	1999	10.83	0.07	0.12	0.18	0.37
231009	2001	13	0.08	0.12	0.18	0.39
231009	2003	14.83	0.09	0.12	0.19	0.40
231009	2004	15.67	0.09	0.12	0.19	0.40
231028	1989	1	0.03	0.05	0.08	0.16
231028	1990	2	0.05	0.05	0.09	0.18
231028	1991	3	0.05	0.05	0.09	0.20
231028	1993	4.67	0.06	0.06	0.10	0.22

Table E.3: Computed Rutting - Local Calibration Factors (Continued)

231028	1995	7.17	0.08	0.06	0.11	0.25
231028	1998	9.83	0.09	0.06	0.11	0.27
231028	1999	10.83	0.10	0.06	0.11	0.28
231028	2001	13	0.12	0.07	0.12	0.30
231028	2003	14.83	0.12	0.07	0.12	0.31
231028	2004	15.75	0.13	0.07	0.12	0.32
251003	1989	1.08	0.02	0.04	0.08	0.14
251003	1990	2.17	0.02	0.04	0.09	0.16
251003	1991	3.08	0.03	0.04	0.10	0.17
251003	1992	4.17	0.03	0.05	0.10	0.19
251003	1995	7.25	0.04	0.05	0.11	0.21
251003	1996	8.25	0.05	0.05	0.12	0.22
251003	1998	9.92	0.05	0.06	0.12	0.23
341003	1989	0.83	0.04	0.05	0.11	0.20
341003	1990	2	0.06	0.05	0.14	0.26
341003	1991	2.92	0.07	0.05	0.15	0.27
341003	1992	4	0.08	0.06	0.16	0.30
341003	1994	6.17	0.10	0.06	0.17	0.33
341003	1995	7.08	0.11	0.06	0.18	0.35
341003	1999	10.5	0.13	0.06	0.19	0.39
341003	2000	11.83	0.15	0.07	0.20	0.41
341003	2002	14	0.16	0.07	0.21	0.44
341003	2005	17.17	0.18	0.07	0.22	0.47
341011	1989	1.17	0.05	0.03	0.10	0.19
341011	1990	2.08	0.06	0.04	0.12	0.22
341011	1992	3.67	0.07	0.04	0.13	0.24
341011	1993	4.5	0.09	0.04	0.14	0.26
341011	1995	7.25	0.11	0.04	0.15	0.31
341011	1997	8.92	0.13	0.04	0.16	0.33
341011	1999	11.17	0.14	0.05	0.17	0.36
341011	2000	11.92	0.15	0.05	0.17	0.37
341011	2002	14.08	0.17	0.05	0.18	0.39
341011	2007	19.17	0.22	0.05	0.19	0.46
341030	1989	1.5	0.02	0.07	0.08	0.17
341030	1990	2.67	0.03	0.08	0.10	0.21
341030	1991	3.58	0.04	0.09	0.10	0.23
341030	1992	4.67	0.04	0.09	0.11	0.25
341030	1995	7.75	0.05	0.10	0.12	0.28
341030	1997	9.5	0.06	0.10	0.13	0.29
341030	1999	11.33	0.06	0.11	0.13	0.31

Table E.3: Computed Rutting - Local Calibration Factors (Continued)

341030	2000	12.5	0.07	0.11	0.14	0.31
341030	2001	13.67	0.07	0.11	0.14	0.32
341030	2005	17.83	0.08	0.11	0.15	0.34
341030	2007	19.42	0.08	0.12	0.15	0.35
341031	1989	1.17	0.05	0.03	0.11	0.19
341031	1990	2.08	0.06	0.03	0.12	0.22
341031	1992	3.67	0.08	0.04	0.14	0.25
341031	1993	4.5	0.09	0.04	0.14	0.27
341031	1995	7.25	0.12	0.04	0.16	0.33
341031	1996	8	0.13	0.04	0.17	0.34
341031	1999	11.08	0.16	0.05	0.18	0.39
341031	2000	11.92	0.17	0.05	0.18	0.40
341031	2002	14.08	0.20	0.05	0.19	0.44
341031	2005	17.25	0.23	0.05	0.21	0.49
341033	1989	1.17	0.02	0.04	0.06	0.12
341033	1990	2.08	0.03	0.04	0.06	0.14
341033	1992	3.67	0.04	0.04	0.07	0.15
341033	1993	4.5	0.05	0.04	0.07	0.16
341033	1995	7.25	0.06	0.05	0.08	0.19
341033	1997	9.17	0.08	0.05	0.08	0.21
341033	2000	12.17	0.10	0.05	0.09	0.23
341033	2002	13.83	0.10	0.05	0.09	0.24
341033	2003	14.92	0.11	0.05	0.09	0.25
341033	2004	15.67	0.11	0.05	0.09	0.25
341033	2007	18.83	0.13	0.06	0.09	0.28
341034	1989	1.75	0.02	0.03	0.06	0.11
341034	1990	2.67	0.03	0.03	0.06	0.12
341034	1992	4.25	0.04	0.03	0.07	0.15
341034	1993	5.08	0.05	0.03	0.07	0.15
341034	1995	7.83	0.06	0.03	0.08	0.18
341034	1997	9.5	0.07	0.03	0.08	0.18
341034	2000	12.75	0.08	0.04	0.09	0.20
341034	2002	14.42	0.09	0.04	0.09	0.21
341034	2005	17.83	0.10	0.04	0.09	0.23
341034	2007	19.42	0.11	0.04	0.09	0.24
341638	1989	1.75	0.06	0.08	0.09	0.23
341638	1990	2.67	0.08	0.09	0.10	0.27
341638	1991	3.58	0.10	0.09	0.11	0.30
341638	1993	5.08	0.11	0.10	0.12	0.33
341638	1995	7.83	0.15	0.11	0.13	0.39

Table E.3: Computed Rutting - Local Calibration Factors (Continued)

341638	1997	9.5	0.16	0.11	0.13	0.40
341638	2000	12.75	0.20	0.12	0.14	0.45
341638	2002	14.42	0.21	0.12	0.14	0.47
341638	2003	15.33	0.22	0.12	0.14	0.48
341638	2005	17.83	0.25	0.12	0.14	0.51
421597	1989	0.92	0.01	0.03	0.04	0.08
421597	1990	1.83	0.01	0.04	0.05	0.10
421597	1991	2.92	0.02	0.04	0.05	0.11
421597	1993	4.5	0.02	0.04	0.06	0.12
421597	1994	5.75	0.03	0.04	0.06	0.14
421597	1995	7	0.03	0.04	0.07	0.14
421597	1996	7.83	0.03	0.05	0.07	0.15
421597	1997	9	0.04	0.05	0.07	0.15
421597	2000	12.17	0.04	0.05	0.08	0.17
421597	2002	13.67	0.04	0.05	0.08	0.17
421597	2003	14.83	0.05	0.05	0.08	0.18
421597	2007	18.92	0.06	0.05	0.09	0.20
421599	1989	0.83	0.02	0.02	0.06	0.10
421599	1990	1.92	0.03	0.03	0.07	0.13
421599	1991	3	0.03	0.03	0.08	0.14
421599	1993	4.5	0.04	0.03	0.09	0.16
421599	1995	6.75	0.05	0.03	0.10	0.18
421599	1996	7.83	0.05	0.03	0.10	0.19
421599	1998	9.5	0.06	0.03	0.11	0.20
421599	2000	11.92	0.07	0.03	0.11	0.21
421599	2001	13	0.07	0.03	0.12	0.22
421599	2002	13.75	0.07	0.04	0.12	0.22
421599	2003	14.58	0.07	0.04	0.12	0.23
421599	2005	16.92	0.08	0.04	0.12	0.24
501002	1989	0.92	0.03	0.04	0.04	0.11
501002	1990	1.92	0.03	0.04	0.05	0.12
501002	1991	3	0.04	0.05	0.06	0.14
501002	1993	4.58	0.04	0.05	0.06	0.16
501002	1994	5.92	0.05	0.05	0.07	0.17
501002	1995	7.08	0.06	0.05	0.07	0.18
501002	1996	8.08	0.06	0.06	0.07	0.19
501002	1997	9.08	0.06	0.06	0.07	0.19
501002	1998	9.75	0.07	0.06	0.07	0.20
501002	1999	11.17	0.07	0.06	0.07	0.21
501002	2000	12	0.08	0.06	0.08	0.22
501002	2001	13	0.08	0.06	0.08	0.22

Table E.3: Computed Rutting - Local Calibration Factors (Continued)

501002	2002	14.08	0.08	0.06	0.08	0.22
501002	2003	15.17	0.09	0.06	0.08	0.23
501004	1989	0.92	0.01	0.03	0.06	0.10
501004	1990	1.92	0.02	0.04	0.06	0.12
501004	1991	3	0.02	0.04	0.07	0.13
501004	1993	4.58	0.03	0.04	0.08	0.14
501004	1995	7.08	0.04	0.05	0.08	0.16
501004	1997	9.17	0.04	0.05	0.09	0.18
501004	1999	10.83	0.05	0.05	0.09	0.18
501004	2000	11.75	0.05	0.05	0.09	0.19
501004	2001	12.92	0.05	0.05	0.09	0.19
501004	2002	13.67	0.05	0.05	0.09	0.19
501004	2004	15.67	0.05	0.05	0.10	0.20
501004	2007	18.92	0.06	0.05	0.10	0.21
501681	1989	1.08	0.01	0.05	0.03	0.08
501681	1990	2.08	0.02	0.08	0.04	0.14
501681	1991	3.17	0.04	0.09	0.05	0.18
501681	1993	4.75	0.04	0.09	0.05	0.19
501681	1995	7.25	0.06	0.10	0.06	0.22
501681	1998	9.92	0.08	0.11	0.06	0.24
501681	1999	10.92	0.08	0.11	0.06	0.25
501681	2001	13.17	0.09	0.11	0.06	0.27
501681	2003	14.92	0.10	0.11	0.06	0.28
501681	2004	16.08	0.10	0.12	0.07	0.28
501683	1989	1.08	0.01	0.04	0.02	0.07
501683	1990	2.08	0.02	0.08	0.03	0.13
501683	1991	3.17	0.04	0.08	0.04	0.16
501683	1993	4.75	0.04	0.09	0.04	0.17
501683	1995	7.25	0.06	0.10	0.04	0.20
501683	1998	9.92	0.07	0.11	0.05	0.22
501683	1999	10.92	0.08	0.11	0.05	0.23
501683	2001	13.17	0.09	0.11	0.05	0.25
501683	2003	14.92	0.10	0.11	0.05	0.26
501683	2004	16.08	0.10	0.11	0.05	0.27
501683	2007	19.08	0.11	0.12	0.05	0.28

Table E.4: Computed Cracking and IRI Distresses - Local Calibration Factors

Site #	Month	Year	Alligator Cracking (%)	IRI (in/mi)
231001	1989	1	2.8091	80.3
231001	1990	2	3.2349	96.9
231001	1991	3	3.2818	97.2
231001	1993	4.67	3.6901	105.2
231009	1989	1	3.4533	93.1
231009	1990	2	4.1074	121.1
231009	1991	3	4.5052	126.8
231028	1989	1	3.2940	99.6507
231028	1990	2	3.8541	106.285
231028	1991	3	4.2207	109.947
251003	1989	1.08	2.7144	94.8
341003	1989	0.83	4.1116	109.1
341003	1990	2	5.1821	128.1
341003	1991	2.92	5.5131	134.9
341003	1992	4	5.9447	107.2
341011	1989	1.17	3.5878	115.22
341011	1990	2.08	4.1362	120.485
341011	1992	3.67	4.5519	126.572
341011	1993	4.5	4.9039	138.952
341011	1995	7.25	5.6651	100.172
341030	1989	1.5	3.1728	111.082
341030	1990	2.67	3.9995	107.411
341031	1989	1.17	3.1884	115.4
341031	1990	2.08	3.6552	123.4
341031	1992	3.67	4.1710	128.3
341031	1993	4.5	4.4432	143.7
341031	1995	7.25	5.1880	90.6
341033	1989	1.17	2.5165	94.9
341033	1990	2.08	2.8909	98.8
341033	1992	3.67	3.2131	102.3
341033	1993	4.5	3.4345	109.2
341033	1995	7.25	3.9438	88.2
341034	1989	1.75	1.9147	92.4
341034	1990	2.67	2.2323	97.9
341034	1992	4.25	2.5843	100.1
341034	1993	5.08	2.7353	106.3
341034	1995	7.83	3.1123	116.1

Table E.4: Computed Cracking and IRI Distresses - Local Calibration Factors (Continued)

341034	2002	14.42	3.7101	113.7
341638	1989	1.75	5.3196	116.5
341638	1990	2.67	6.2426	108.1
341638	1993	5.08	7.4308	40.7
341638	2002	14.42	9.9965	82.2
421597	1989	0.92	1.7980	85.9
421599	1989	0.83	1.7465	93.0
421599	1990	1.92	2.1805	97.2
421599	1991	3	2.4428	101.1
421599	1993	4.5	2.6487	88.8
501002	1989	0.92	1.8494	92.5
501002	1990	1.92	2.1761	96.2
501002	1991	3	2.4391	100.3
501002	1993	4.58	2.6758	-
501002	1995	7.08	3.0472	106.5
501002	2000	12	3.5096	115.5
501002	2002	14.08	3.6576	117.6
501004	1989	0.92	1.5572	86.6
501004	1990	1.92	1.8475	90.7
501004	1991	3	2.0623	93.9
501004	1995	7.08	2.5268	101.9
501681	1989	1.08	1.6488	81.6
501681	1990	2.08	2.9187	95.9
501683	1989	1.08	1.6635	79.5
501683	1990	2.08	2.9471	92.9849

Appendix F: Pavement Design Tables for New York State Developed with the AASHTOWare Pavement ME 2.1

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Table F.1: Design Thicknesses for Region 1 – Albany

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3	0	3	0	3	0	3	0	3	0
250	6	0	5	0	4	0	3.5	0	3	0	3	0
500	8.5	6	7	0	6	0	5.5	0	4.5	0	4	0
1,000	10.5	6	9.5	6	8	0	7	0	6.5	0	6	0
2,000	12.5	6	12	6	11	6	10.5	6	9	0	9	0
4,000	14	6	13.5	6	13	6	12.5	6	12	6	12	0
5,000	15	6	14	6	13.5	6	13	6	13	6	12.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	4	0	3	0	3	0	3	0	3	0
100	6	0	4.8	0	4	0	3.5	0	3	0	3	0
250	9	0	8	0	7	0	6	0	5	0	4.5	0
500	11	6	10	6	9	6	8	6	7	6	6.5	0
1,000	12.5	12	12	12	11	12	10.5	12	10	6	9.5	0
2,000	14.5	12	14	12	13	12	12.5	12	11	12	12	6
4,000	19.5	12	18	12	17.5	12	17	12	16	12	13.5	12
5,000	23	12	22.5	12	21.5	12	20	12	19	12	15	12

Table F.2: Design Thicknesses for Region 1 – Glens Falls

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4.5	0	3	0	3	0	3	0	3	0	3	0
250	6.5	0	5	0	4.5	0	3.5	0	3	0	3	0
500	8.5	6	7	0	6	0	5.5	0	4.5	0	4	0
1,000	10.5	6	9.5	6	8	0	7.5	0	6.5	0	6	0
2,000	12.5	6	12	6	11	6	10	0	9	0	8.5	0
4,000	14.5	6	13.5	6	13	6	12.5	6	12	6	12	0
5,000	15	6	14	6	13.5	6	13	6	13	6	12.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	5.5	0	4	0	3.5	0	3	0	3	0	3	0
100	6.5	0	5.5	0	4.5	0	4	0	3.5	0	3	0
250	9.5	0	8.5	0	7	0	4.5	0	5	0	4.5	0
500	11	6	10	6	9.5	0	8	0	7.5	0	7	0
1,000	13	12	11.5	12	11.5	6	10.5	6	10	0	9.5	0
2,000	15.5	12	14	12	13	12	12.5	12	12.5	6	12	0
4,000	19	12	18	12	17	12	15	12	15	12	14	6
5,000	22	12	20	12	19	12	17	12	16	12	14.5	12

Table F.3: Design Thicknesses for Region 2 – Utica

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4.5	0	3.5	0	3	0	3	0	3	0	3	0
250	6.5	0	5	0	4.5	0	3.5	0	3.5	0	3	0
500	8.5	6	7	0	6.5	0	5.5	0	4.5	0	4	0
1,000	10	6	9	0	8	0	7	0	6.5	0	6	0
2,000	12.5	6	12	6	11	6	10.5	6	9	0	8.5	0
4,000	14.5	6	13.5	6	13	6	12.5	6	11.5	6	11	0
5,000	15	6	14	6	13.5	6	13	6	13	6	12.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	5	0	4	0	3.5	0	3	0	3	0	3	0
100	7	0	5.5	0	4.5	0	4	0	3.5	0	3	0
250	9.5	0	8	0	7	0	6	0	5	0	4.5	0
500	11.5	6	10.5	6	9	0	8	0	7.5	0	7	0
1,000	13	12	13	12	11.5	6	10.5	0	10	0	9.5	0
2,000	15.5	12	14.5	12	14	12	12.5	6	12	6	12	0
4,000	19	12	17	12	16	12	15	12	14	0	13.5	0
5,000	21.5	12	19	12	18	12	16	12	16.5	12	15.5	6

Table F.4: Design Thicknesses for Region 3 – Syracuse

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	3.5	0	3	0	3	0	3	0
500	7.5	0	6	0	5	0	4.5	0	4	0	4	0
1,000	9.5	6	8	0	7	0	6	0	5.5	0	4.5	0
2,000	11.5	6	10.5	6	9.5	0	8.5	0	8	0	7.5	0
4,000	13.5	6	12.5	6	11	6	11.5	6	10.5	6	10	0
5,000	13.5	6	13.5	6	11.5	6	11	6	11	6	10.5	6
DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	4.5	0	4	0	3.5	0	3	0	3	0
250	8.5	0	7	0	6	0	5	0	4.5	0	4	0
500	10	6	9	6	8	6	7	0	6	0	5.5	0
1,000	12	12	10.5	12	9.5	12	9.5	0	8.5	0	8	0
2,000	13.5	12	13	12	12	12	12	6	11	0	10.5	0
4,000	16.5	12	15.5	12	15	12	13	12	13	6	12.5	0
5,000	17	12	16	12	15.5	12	14	12	13.5	12	13	6

Table F.5: Design Thicknesses for Region 4 – Rochester

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4.5	0	3.5	0	3	0	3	0	3	0
500	7	0	6	0	4.5	0	4.5	0	4	0	3.5	0
1,000	9	0	8	0	7	0	6	0	5.5	0	4.5	0
2,000	11.5	6	10.5	6	9	0	8	0	7.5	0	7	0
4,000	13.5	6	12.5	6	11	6	11	6	10	6	10	0
5,000	14	6	13	6	11.5	6	11.5	6	10.5	6	10	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3.5	0	3	0	3	0	3	0	3	0
100	5.5	0	4.5	0	3.5	0	3.5	0	3	0	3	0
250	8	0	7	0	5.5	0	4.5	0	4.5	0	3.5	0
500	9.5	6	9	6	7.5	0	6.5	0	6	0	5	0
1,000	11.5	12	10.5	12	9.5	6	9	12	9	0	7.5	0
2,000	13.5	12	12.5	12	11.5	12	11	6	10.5	0	10	0
4,000	15	12	14	12	13.5	12	13	12	12.5	6	12	0
5,000	16.5	12	14.5	12	14.5	12	13.5	12	13.5	12	13	6

Table F.6: Design Thicknesses for Region 5 – Buffalo

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	4	0	3	0	3	0	3	0
500	7	0	6	0	5	0	4	0	3.5	0	3.5	0
1,000	8.5	6	8	0	7	0	6	0	5	0	4.5	0
2,000	11	6	10	0	9	0	8	0	7.5	0	6.5	0
4,000	13	6	12.5	6	11.5	6	11	6	10	6	10	0
5,000	13.5	6	13	6	12.5	6	11.5	6	11	6	10	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	4.5	0	3.5	0	3.5	0	3	0	3	0
250	8	0	7	0	6	0	5	0	4.5	0	3.5	0
500	9.5	6	8.5	6	7.5	6	6.5	0	6	0	5	0
1,000	11.5	12	10.5	12	9.5	12	9	6	8	0	7.5	0
2,000	13.5	12	12.5	12	11.5	12	11	12	10.5	0	10	0
4,000	16.5	12	15.5	12	14.5	12	13.5	12	12.5	6	12	0
5,000	17.5	12	16.5	12	15.5	12	14.5	12	13.5	12	13	6

Table F.7: Design Thicknesses for Region 5 – Dunkirk

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	4	0	3	0	3	0	3	0
500	7.5	0	6.5	0	5	0	4.5	0	4	0	3.5	0
1,000	9.5	6	8.5	0	7.5	0	6.5	0	5.5	0	5	0
2,000	12	6	11	6	10	0	9	0	8.5	0	8	0
4,000	13.5	6	13	6	12	6	11.5	6	11	6	11	0
5,000	14	6	13.5	6	13	6	12.5	6	12	6	11.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	5.5	0	4	0	3.5	0	3	0	3	0
250	8.5	0	8	0	6.5	0	5	0	4.5	0	4.5	0
500	10.5	6	10	6	8.5	0	8.5	0	7	0	6	0
1,000	12	12	11	12	10.5	6	10	0	9.5	0	9	0
2,000	14	12	14.5	12	12.5	12	12	6	11.5	0	11	0
4,000	17.5	12	16.5	12	15	12	14	12	14	6	13	0
5,000	19.5	12	18	12	16.5	12	15	12	14	12	13.5	6

Table F.8: Design Thicknesses for Region 5 – Niagara Falls

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4.5	0	3.5	0	3	0	3	0	3	0
500	7	0	6.5	0	4.5	0	5	0	3.5	0	3.5	0
1,000	9	6	8	0	6.5	0	6	0	5	0	4.5	0
2,000	11	6	10	6	9	0	8	0	7	0	6.5	0
4,000	12.5	6	12	6	11	6	11	6	9.5	6	9.5	0
5,000	13	6	12.5	6	11.5	6	11.5	6	10.5	6	10	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3.5	0	3	0	3	0	3	0	3	0
100	8	0	4.5	0	3.5	0	3.5	0	3	0	3	0
250	8	0	7.5	0	5.5	0	4.5	0	4.5	0	3.5	0
500	11	6	8.5	6	7.5	0	7	0	6	0	5	0
1,000	12.5	6	10.5	6	9.5	6	9	6	8	0	7	0
2,000	13.5	6	12	12	12	12	11	12	10.5	0	10	0
4,000	15.5	12	14	12	13	12	12.5	12	12.5	6	12	0
5,000	15.5	12	14.5	12	13.5	12	13	12	13	12	13	6

Table F.9: Design Thicknesses for Region 6 – Dansville

DESIGN LIFE = 15 YEARS

AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	4	0	3	0	3	0	3	0
500	8	0	6.5	0	5	0	4.5	0	4	0	3.5	0
1,000	9.5	6	9	0	8	0	7	0	6	0	5	0
2,000	12	6	11	6	10	6	9.5	0	9	0	8	0
4,000	13.5	6	12	6	12.5	6	12	6	11.5	6	11	0
5,000	14	6	13.5	6	13	6	12.5	6	12	6	12	6

DESIGN LIFE = 20 YEARS

AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	4	0	3	0	3	0	3	0	3	0
100	6	0	5	0	4	0	3.5	0	3	0	3	0
250	9.5	0	7	0	6.5	0	5.5	0	5	0	4	0
500	10.6	6	9.5	6	8.5	0	8	6	7.5	0	7	0
1,000	12.5	6	11.5	6	11	6	10.5	6	10	6	9.5	0
2,000	15	6	14.5	6	13	6	12.5	6	12	0	11.5	0
4,000	18	12	15.5	12	14.5	12	14	12	13.5	6	13	0
5,000	19	12	18	12	16	12	15	12	14.5	12	14	6

Table F.10: Design Thicknesses for Region 6 – Elmira

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	4	0	3.5	0	3	0	3	0
500	7.5	0	6.5	0	5	0	4.5	0	4	0	3.5	0
1,000	9.5	6	8	6	8	0	6.5	0	6	0	5	0
2,000	11.5	6	11	6	10	0	9	0	8	0	7.5	0
4,000	13.5	6	12.5	6	12	6	11.5	6	11	6	10.5	0
5,000	14	6	13	6	12.5	6	12	6	11.5	6	11.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	5	0	4	0	3.5	0	3	0	3	0
250	9	0	7.5	0	6.5	0	5	0	5	0	4.5	0
500	11	6	10	0	8.5	0	8	0	7.5	0	6.5	0
1,000	12.5	12	11.5	6	10.5	0	10.5	0	10	0	9	0
2,000	14	12	13	12	13.5	6	12	6	11.5	0	11	0
4,000	17.5	12	15.5	12	15	12	13.5	12	13.5	6	13	0
5,000	19	12	18.5	12	16	12	15	12	14	12	13.5	6

Table F.11: Design Thicknesses for Region 6 – Wellsville

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3	0	3	0	3	0	3	0	3	0
250	6	0	4.5	0	4	0	3.5	0	3	0	3	0
500	7.5	0	6	0	5	0	4.5	0	4	0	3.5	0
1,000	9	6	8	6	7	0	6	0	5.5	0	5	0
2,000	11.5	6	10.5	6	9.5	0	8.5	0	8	0	7	0
4,000	13	6	12.5	6	11.5	6	11	6	10.5	6	10	0
5,000	13.5	6	13	6	12	6	11.5	6	11	6	11	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6.5	0	5.5	0	4	0	3.5	0	3	0	3	0
250	8.5	0	7	0	6	0	5.5	0	4.5	0	4.5	0
500	10	6	9	6	8	0	7.5	0	7	0	6	0
1,000	12	12	11	12	10.5	6	10	0	9	0	8	0
2,000	14	12	13.5	12	12	6	11.5	6	11.5	0	10.5	0
4,000	15.5	12	15	12	14	12	13.5	6	13	6	12.5	0
5,000	17	12	15	12	14.5	12	14	12	13.5	12	13	6

Table F.12: Design Thicknesses for Region 7 – Massena

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3.5	0	3	0	3	0	3	0	3	0
250	6	0	5	0	4.5	0	3.5	0	3	0	3	0
500	8	0	7	0	6	0	5	0	4.5	0	4	0
1,000	10	6	9	0	8	0	7	0	6	0	6	0
2,000	12	6	11	6	10	6	9.5	0	9	0	8	0
4,000	14	6	13	6	12.5	6	12	6	11.5	6	11.5	0
5,000	14	6	13.5	6	13	6	12.5	6	12	6	12	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	5	0	4	0	3.5	0	3	0	3	0	3	0
100	6.5	0	5.5	0	4.5	0	4	0	3.5	0	3	0
250	9	0	8	0	7	0	6	0	5	0	4.5	0
500	11	6	9.5	6	9	0	8	0	7	0	6.5	0
1,000	12	12	11.5	12	11	6	10.5	0	9.5	0	9	0
2,000	14	12	13.5	12	13	12	12.5	6	12	0	11.5	0
4,000	17.5	12	15.5	12	14	12	14	12	13.5	6	12.5	0
5,000	19.5	12	17	12	15.5	12	14	12	14	12	14	6

Table F.13: Design Thicknesses for Region 7 – Plattsburgh

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3.5	0	3	0	3	0	3	0	3	0
250	6	0	5	0	4.5	0	3.5	0	3	0	3	0
500	8	0	7	0	6	0	5	0	4.5	0	4	0
1,000	10	6	9	0	8	0	7	0	6	0	6	0
2,000	12	6	11	6	10	6	9.5	0	9	0	8	0
4,000	14	6	13	6	12.5	6	12	6	11.5	6	11.5	0
5,000	14	6	13.5	6	13	6	12.5	6	12	6	12	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6.5	0	5.5	0	4.5	0	4	0	3.5	0	3	0
250	9	0	7.5	0	6.5	0	6	0	5	0	4.5	0
500	11	6	9.5	6	9	0	8	0	7	0	6.5	0
1,000	12	12	11.5	12	11	6	10.5	0	9.5	0	9	0
2,000	14	12	13.5	12	13	12	12.5	6	12	0	11.5	0
4,000	17.5	12	15.5	12	14	12	14	12	13.5	6	12.5	0
5,000	19.5	12	17	12	15.5	12	14	12	14	12	14	6

Table F.14: Design Thicknesses for Region 7 – Watertown

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3.5	0	3	0	3	0	3	0	3	0
250	6	0	5	0	4	0	3.5	0	3	0	3	0
500	8	0	6.5	0	5.5	0	4.5	0	4.5	0	3.5	0
1,000	9.5	6	9	0	8	0	7	0	6	0	5	0
2,000	12	6	11	6	10	0	9.5	0	8.5	0	8	0
4,000	13.5	6	13	6	12	6	11.5	6	11	6	11	0
5,000	14	6	13	6	12.5	6	12.5	6	12	6	11.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6.5	0	5.5	0	4.5	0	4	0	3.5	0	3	0
250	9	0	7.5	0	6.5	0	5.5	0	5	0	4.5	0
500	10.5	6	9.5	6	8.5	0	7.5	0	7	0	6	0
1,000	12	12	11.5	12	11	6	10	0	9.5	0	9	0
2,000	14	12	13.5	12	13	12	12.5	6	12	0	11	0
4,000	17	12	15	12	14	12	14	12	13.5	6	13	0
5,000	19	12	16.5	12	14.5	12	14.5	12	14	12	13.5	6

Table F.15: Design Thicknesses for Region 8 – Montgomery

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	6	0	4.5	0	4	0	3.5	0	3	0	3	0
500	7.5	0	6.5	0	5	0	4.5	0	5	0	5	0
1,000	9.5	6	9	0	7.5	0	7	0	6	0	5.5	0
2,000	12	6	11	6	10.5	6	9.5	0	8.5	0	9.5	0
4,000	13.5	6	13	6	12.5	6	12	6	11.5	6	11	0
5,000	14	6	13.5	6	13	6	12.5	6	12	6	11.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	4	0	3	0	3	0	3	0	3	0
100	6	0	5	0	4	0	3.5	0	3	0	3	0
250	8.5	0	7.5	0	6	0	5	0	5	0	4.5	0
500	10.5	6	9	12	8.5	0	7.5	0	7	0	6	0
1,000	12.5	6	13.5	6	11	6	10	0	9.5	0	9	0
2,000	14.5	12	13.5	6	13	6	12.5	6	12.5	0	11.5	0
4,000	17	12	15.5	12	15	12	14	12	13.5	6	13	0
5,000	20	12	17	12	15.5	12	14.5	12	14	12	13.5	6

Table F.16: Design Thicknesses for Region 8 – Poughkeepsie

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	4	0	3	0	3	0	3	0	3	0	3	0
250	6	0	5	0	4	0	3.5	0	3	0	3	0
500	8	0	7	0	5.5	0	5	0	4.5	0	4	0
1,000	10	6	9	6	8	0	7	0	6	0	6	0
2,000	12	6	11	6	10	6	9.5	6	9	0	8.5	0
4,000	14	6	12.5	6	12.5	6	12	6	11.5	6	11	0
5,000	14.5	6	13	6	13	6	12.5	6	12	6	12	6
DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	5	0	4.5	0	3.5	0	3	0	3	0
250	9	0	7.5	0	6.5	0	5.5	0	5	0	4.5	0
500	11	6	10	0	9	0	8	0	7	0	6.5	0
1,000	13	6	12.5	6	11	6	10	6	9.5	0	9	0
2,000	15	12	14	12	13.5	12	12.5	6	12	12	11.5	0
4,000	18.5	12	16	12	14.5	12	14	12	13.5	6	13.5	0
5,000	20.5	12	17.5	12	15.5	12	14.5	12	14	12	14	6

Table F.17: Design Thicknesses for Region 8 – White Plains

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	3.5	0	3	0	3	0	3	0
500	7	0	6	0	5	0	4.5	0	4	0	4	0
1,000	9	6	8	6	7	0	6	0	5	0	5	0
2,000	11	6	10	6	9	6	8	6	7.5	0	7	0
4,000	13	6	12	6	11.5	6	11.5	6	10	6	9.5	0
5,000	13.5	6	12.5	6	12	6	12	6	11	6	10.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.5	0	4	0	3	0	3	0	3	0	3	0
100	6	0	5	0	4	0	3.5	0	3	0	3	0
250	8	0	7	0	6	0	5	0	4.5	0	4	0
500	10	6	8.5	12	8	0	7	0	6	0	5.5	0
1,000	12	6	11	0	10	0	9	0	8.5	0	7.5	0
2,000	14	12	13	6	12	6	11.5	6	10.5	0	10	0
4,000	15.5	12	14	12	13.5	12	13	6	12.5	6	12.5	0
5,000	16.5	12	15	12	14	12	13.5	12	13	12	13	6

Table F.18: Design Thicknesses for Region 9 – Virtual Climatic Station

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	3.5	0	3.5	0	3	0	3	0
500	7.5	0	6	0	5	0	4.5	0	4	0	3.5	0
1,000	9	6	8	6	7.5	0	6.5	0	5.5	0	5	0
2,000	11.5	6	10.5	6	9.5	6	9	0	8	0	7.5	0
4,000	13.5	6	12.5	6	12	6	11.5	6	11	6	11	0
5,000	14	6	13	6	12.5	6	12	6	11.5	6	11	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	5	0	3.5	0	3	0	3	0	3	0	3	0
100	6	0	5.5	0	4	0	3.5	0	3	0	3	0
250	8.5	0	7	0	6	0	5.5	0	5	0	4.5	0
500	10	6	9	6	8	0	7.5	0	6.5	0	6	0
1,000	12	12	11.5	12	10.5	6	10	6	9	0	9.5	0
2,000	13.5	12	13.5	12	13	12	12.5	12	12	0	12	0
4,000	16.5	12	15.5	12	15	12	14.5	12	14	6	14	0
5,000	18	12	16.5	12	16	12	15.5	12	15	12	14.5	6

Table F.19: Design Thicknesses for Region 10 – Farmingdale

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4	0	3.5	0	3	0	3	0	3	0
500	7	0	5.5	0	4.5	0	4.5	0	3.5	0	3.5	0
1,000	8.5	6	8.5	0	7	0	6	0	5	0	4.5	0
2,000	11	6	10	6	9.5	6	8	0	7.5	0	6.5	0
4,000	13	6	12.5	6	11.5	6	11	6	10.5	6	9.5	0
5,000	13.5	6	13	6	12	6	11.5	6	11	6	10.5	6
DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3	0	3	0	3	0	3	0	3	0
100	5	0	5	0	4	0	3	0	3	0	3	0
250	8.5	0	7	0	6	0	5	0	4	0	4	0
500	10	6	9	6	8	0	7	0	6	0	5	0
1,000	12	6	11	6	10	12	9	12	8	0	8	0
2,000	14.5	12	13.5	12	12	0	12.5	6	11	0	11.5	0
4,000	16	12	15.5	12	14	0	13	12	13	6	12.5	0
5,000	17	12	16	12	14	0	14	12	14	12	13.5	6

Table F.20: Design Thicknesses for Region 10 – Islip

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4	0	3.5	0	3	0	3	0	3	0
500	7	0	5.5	0	4.5	0	4.5	0	3.5	0	3.5	0
1,000	8.5	6	8	0	7	0	6	0	5	0	4.5	0
2,000	11	6	10	6	9	6	8	0	7.5	0	6.5	0
4,000	13	6	12.5	6	11.5	6	11	6	10.5	6	9.5	0
5,000	13.5	6	13	6	12	6	11.5	6	11	6	10.5	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	3	0	3	0	3	0	3	0	3	0
100	5	0	4.5	0	3.5	0	3	0	3	0	3	0
250	8.5	0	6.5	0	5.5	0	4.5	0	4	0	3.5	0
500	9.5	6	8.5	6	7.5	0	6.5	0	6	0	5.5	0
1,000	12	6	10.5	12	10	6	8.5	12	8	0	7.5	0
2,000	14.5	12	12.5	12	12	6	12.5	6	10.5	0	10	0
4,000	16	12	15.5	12	13.5	12	13	12	14	6	12	0
5,000	16.5	12	16	12	14	12	13.5	12	14	12	13	6

Table F.21: Design Thicknesses for Region 10 – Shirley

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4.5	0	4.5	0	3	0	3	0	3	0
500	7.5	0	6	0	6	0	5	0	4	0	3.5	0
1,000	9.5	6	8	0	8	0	6	0	5	0	4.5	0
2,000	11.5	6	10.5	6	10.5	6	9	6	8	0	7	0
4,000	13.5	6	12.5	6	12.5	6	11.5	6	11	6	10	0
5,000	14	6	13.5	6	13.5	6	12	6	11.5	6	11.5	6
DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4	0	4	0	3.5	0	3	0	3	0	3	0
100	5.5	0	4.5	0	4.5	0	3	0	3	0	3	0
250	8	0	7	0	7	0	5	0	4.5	0	4	0
500	10	6	8.5	6	9	0	7	0	6.5	0	5.5	0
1,000	12.5	12	11	12	11	6	9.5	0	8.5	0	8	0
2,000	13.5	12	13	12	12.5	12	12	6	11	0	10.5	0
4,000	16	12	14.5	12	14	12	13.5	12	13	6	13	6
5,000	18	12	15.5	12	15	12	14	12	13.5	12	13.5	6

Table F.22: Design Thicknesses for Region 11 – New York City 94728

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3.5	0	3	0	3	0	3	0	3	0	3	0
250	5.5	0	4.5	0	3.5	0	3	0	3	0	3	0
500	8	0	6.5	0	5	0	4.5	0	4.5	0	3.5	0
1,000	9.5	6	9	0	8	0	7	0	6	0	5.5	0
2,000	12.5	6	11.5	6	10.5	6	10	6	9	0	8.5	0
4,000	14	6	13.5	6	13	6	12.5	6	12	6	11.5	0
5,000	14	12	14	6	13.5	6	13	6	12.5	6	12	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	4.0	0	3.5	0	3.0	0.0	3.0	0.0	3.0	0.0	3.0	0.0
100	6.0	0.0	4.5	0	5.5	0.0	3.0	0.0	3.0	0.0	3.0	0.0
250	8.5	0	7.0	0	6.0	0	5.0	0	4.5	0	4.5	0
500	10.5	6	9.5	6	8.5	0	7.5	0	7.0	0	6.0	0
1,000	12.5	12	11.5	12	11.0	6	10.5	0	9.5	0	9.5	0
2,000	14.0	12	13.5	12	13.0	12	12.5	6	12.0	0	11.5	0
4,000	18.5	12	16	12	15.5	12	14.5	12	14.0	6	13.5	0
5,000	21	12	18.0	12	16	12	15.0	12	14.0	6	14.0	6

Table F.23: Design Thicknesses for Region 11 – New York City 94789

DESIGN LIFE = 15 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4	0	3	0	3	0	3	0	3	0
500	6.5	0	5	0	4.5	0	4	0	3.5	0	3.5	0
1,000	8.5	6	7.5	0	6.5	0	5.5	0	4.5	0	4.5	0
2,000	11	6	9.5	6	8.5	6	7.5	6	7	0	6	0
4,000	12.5	6	12	6	11	6	10	6	9.5	6	9	0
5,000	13.5	6	12.5	6	11.5	6	11	6	10	6	10	6

DESIGN LIFE = 20 YEARS												
AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrad e (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3.5	0	3	0	3	0	3	0	3	0	3	0
100	5.5	0	4	0	3.5	0	3	0	3	0	3	0
250	5.5	0	6.5	0	5	0	4.5	0	3.5	0	3.5	0
500	9	6	8	6	7	0	6	0	5	0	5	0
1,000	11	12	10	12	9	6	8	0	7.5	0	7	0
2,000	13	12	12	12	11.5	12	11	6	10	0	9.5	0
4,000	13.5	12	13.5	6	12.5	12	12	6	11.5	12	12.5	0
5,000	15.5	12	14.5	12	13	12	13.5	12	13	12	12.5	6

Table F.24: Design Thicknesses for Region 11 – New York City 14732

DESIGN LIFE = 15 YEARS

AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3	0	3	0	3	0	3	0	3	0	3	0
100	3	0	3	0	3	0	3	0	3	0	3	0
250	5	0	4	0	3.5	0	3	0	3	0	3	0
500	7	0	5.5	0	4.5	0	4	0	3.5	0	3.5	0
1,000	8.5	6	8	0	6.5	0	5.5	0	5	0	4.5	0
2,000	11	6	10	6	9	6	8	6	7.5	0	6.5	0
4,000	13	6	12	6	11.5	6	10.5	6	10	6	9.5	0
5,000	13.5	6	13	6	12	6	11.5	6	11	6	10.5	6

DESIGN LIFE = 20 YEARS

AADTT in the Design Lane	Mr = 4 Ksi		Mr = 5 Ksi		Mr = 6 Ksi		Mr = 7 Ksi		Mr = 8 Ksi		Mr = 9 Ksi	
	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)	Total HMA (in)	Select Granular Subgrade (in)
50	3.5	0	3	0	3	0	3	0	3	0	3	0
100	5.5	0	4	0	3.5	0	3	0	3	0	3	0
250	7.5	0	6.5	0	5	0	4.5	0	4	0	3.5	0
500	9.5	6	8	6	7	0	6	0	6	0	5	0
1,000	11	12	10	12	11.5	12	8.5	0	8	0	7	0
2,000	13	12	12.5	12	11.5	12	11	6	10.5	0	13	0
4,000	14.5	12	14	12	13.5	12	13	12	12.5	6	12.5	0
5,000	16	12	14.5	12	14	12	13.5	12	13	12	12.5	6

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