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16. Abstract

The recently developed guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (M-E Design Guide) will change the way in which pavements are designed by replacing the traditional empirical design approach proposed in the AASHTO 1993 *Guide for the Design of Pavement Structures* with a mechanistic-empirical based approach. One of the most significant changes offered in the M-E Design Guide is the difference in the method used to account for highway traffic loading. Traffic volume and traffic loads, the two most important aspects required to characterize traffic for pavement design, are treated separately and independently. Traffic loading is accounted for by using the axle load spectrum of each axle type of each vehicle class. For the most accurate design cases, project specific weigh-in-motion (WIM) data should be used with appropriate growth factors, projected to the length of the analysis period. At present, the network of WIM systems in Texas consists of approximately twenty WIM stations, the majority of which are located on interstate facilities. Increased WIM density and sampling frequency are necessary to accommodate the current requirements of the M-E Design Guide. Currently, the Texas Department of Transportation (TxDOT) uses a statewide average to generate load data for most highways.

The goal of this research study was to assess and address the implications of the axle load spectra approach proposed by the M-E Design Guide. In addition, recommendations were developed regarding traffic data needs and availability to aid in deciding the installation locations of future WIM stations in Texas. A methodology for specifying the required accuracy of WIM equipment based on the effect that this accuracy has on pavement performance prediction was also developed. Regarding traffic volume forecasting, a methodology is presented that allows optimum use of available data by simultaneously estimating traffic growth and seasonal traffic variability. Through rigorous statistical analyses, it was determined that the use of continuous distribution functions offers numerous advantages. Associated with these analyses, the use of moment statistics was explored and were determined to be the best summary statistics to characterize axle load spectra from the viewpoint of load-associated pavement damage.

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Research performed in cooperation with the Texas Department of Transportation.

## Products

This report contains Products 0-4510-P1 and 0-4510-P3, in Chapters 3 and 5, respectively.

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# 1. Introduction

## 1.1 Background and Significance of the Study

During 1996, the National Cooperative Highway Research Program (NCHRP) undertook a substantial research effort, administered by the Transportation Research Board (TRB), to develop the guide for the *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* under NCHRP Project 1-37A, hereafter referred to as the M-E Design Guide. Although the project has been completed, work continues and several additional NCHRP projects have been commissioned to evaluate the products produced by 1-37A. The additional projects include NCHRP 1-40, NCHRP 1-40A, NCHRP 1-40B, NCHRP 1-40D(1), and NCHRP 1-40D(2). The goals of these projects range from facilitating the implementation of the M-E Design Guide, to critically reviewing the guide, to providing guidelines for local calibration and technical assistance for further software modifications and development.

This monumental effort will change the way in which pavements are designed by replacing the traditional empirical design approach proposed in the current American Association of State Highway and Transportation Officials (AASHTO) *Guide for the Design of Pavement Structures* (AASHTO, 1993) to a mechanistic-empirical based approach. In this approach, a mechanistic model is used to estimate stresses and strain within the pavement structure, which are, in turn, empirically correlated to expected performance by means of performance of transfer functions. In the case of flexible pavements, the mechanistic model incorporated in the new guide is a multi-layer linear elastic system. Although a finite element model was originally contemplated in the research for assessing the non-linear properties of granular materials, this has not been enabled in the latest release of the software (Version 0.800, November 4, 2005). In principle, the most important advantage of the mechanistic-empirical approach is the perceived ability to extrapolate results outside the original data range for which it was originally calibrated. This is an important limitation for empirical methods, which can be applied with confidence only within the original data range used for their development. Extensions of the predictions outside this range necessitate collection of experimental data and calibration to the new conditions.

From the perspective of this research study, the most important aspect of the M-E Design Guide is the difference in the method used to account for highway traffic loading. Traffic volume and traffic loads, the two most important aspects required to characterize traffic for pavement design, are treated separately and independently. The traditional empirical approach converts the entire traffic stream into its equivalent number of 18-kip single axle loads (ESALs) and predicts ESAL growth for the entire life of the project. In the M-E Design Guide, traffic loading is accounted for by using the axle load spectrum of each axle type of each vehicle class. For the most accurate design cases (Level 1), weigh-in-motion (WIM) data from the highway to be rehabilitated should be used with appropriate growth factors, projected to the length of the analysis period. Highways to be constructed on new rights-of-way will require traffic data estimates from highways in close proximity. For intermediate design levels (Level 2), regional axle load spectra data for facilities with similar truck volumes, and site-specific traffic classifications and counts will be used. Finally, for the less accurate design levels (Level 3), actual traffic counts or estimates will be used in conjunction with regional classification and WIM information.

At present, the network of WIM systems in Texas consists of approximately twenty WIM stations, the majority of which are located on interstate facilities. Increased WIM spatial (density) and temporal (sampling frequency) distributions are necessary to accommodate the current demands of the M-E Design Guide, especially for Level 1 and Level 2 designs. Currently, the Texas Department of Transportation (TxDOT) does not have adequate regional representation of weigh data and uses a statewide average to generate load data for most highways.

The goal of this research study was to assess and address the implications of the axle load spectra approach proposed by the M-E Design Guide. These implications are multi-dimensional. On one hand, the methods used to determine the data requirements of the M-E Design Guide were compared with the data available in the state. This was accompanied with the evaluation of traffic equipment and methodology for data collection and data management, with emphasis on the process required for delivering the data to the pavement designer. On the other hand, the implications of the axle load spectra approach on the structural design of pavement were considered and evaluated. In this process, guidelines for traffic data collection, processing, and usage were developed in conjunction with specifications for Level 1 (when available), Level 2, and Level 3 axle load spectra.

In addition, recommendations are provided regarding traffic data needs and availability to guide the spatial and temporal distribution of WIM stations to be installed in the near future in Texas. A methodology for specifying the required accuracy of WIM equipment, based on the effect that this accuracy has on pavement performance prediction, was also developed and is presented in this report. This methodology enables the joint quantification of random and systematic equipment errors on performance.

Regarding traffic volume forecasting, a methodology is proposed that allows optimum use of the limited data available by simultaneously estimating long-term (traffic growth) and short-term (seasonality) traffic volume variability. This methodology combines a time series model with the two most common traffic growth models (linear and compound growth) into a single model.

Through rigorous statistical analyses of WIM data, it was determined and demonstrated that the use of continuous distribution functions, instead of discrete distribution, offers numerous advantages. Associated with these analyses, the use of moment statistics was explored and determined to be the best summary statistics to characterize axle load spectra from the viewpoint of load-associated pavement damage.

## 1.2 Organization of the Report

This report (0-4510-4) is the third in a series of three project reports, which also includes 0-4510-1 and 0-4510-2. This report also contains Products 0-4510-P1 and 0-4510-P3, in Chapters 3 and 5, respectively.

Report 0-4510-1 presents a literature review and summary of data collection and processing procedures for characterizing traffic for pavement design, highlights practices and procedures used in Texas, and presents a detailed summary of the traffic data requirements of the M-E Design Guide. The report includes a sensitivity analysis of the M-E Design Guide in reference to design variables such as traffic volume, axle load, axle configuration, pavement type, and environmental conditions.

Report 0-4510-2 presents a literature review of currently used WIM equipment in the U.S., with particular emphasis on accuracy and calibration aspects. In addition, the report presents a summary of current trends and expected developments regarding vehicle weights and dimensions in the U.S. that may impact traffic in Texas in the future. The report includes a lengthy appendix containing the Level 1 axle load spectra data to be used in conjunction with the M-E Design Guide.

This report presents an overview of the project background and objectives in Chapter 1. Chapter 2 presents a detailed summary of all data input needs of the M-E Design Guide. These needs are presented by design level and include traffic, structural, and environmental inputs. Brief comments and recommendations on the hierarchical design approach are presented.

Chapter 3 entitled, *Data Management: Collection, Processing and Usage*, constitutes Product 1 of the research study. The chapter includes a description of the current availability of data in Texas, data processing recommendations, and a methodology for selecting WIM equipment based on the desired accuracy of pavement performance prediction. Chapter 3 also makes recommendations on the spatial and temporal distribution of WIM stations for supporting pavement design and rehabilitation in Texas.

Chapter 4 focuses on two main topics: 1) issues related to traffic forecasting, and 2) development of continuous axle load distribution functions. To address the first topic, a novel methodology is presented to simultaneously estimate traffic growth and seasonal variability by combining traditional growth models with time series analysis using trigonometric functions. For the second topic, it is demonstrated that multi-modal lognormal distribution can accurately capture actual axle load spectra. Furthermore, it is shown that the use of continuous distribution offers the advantage of facilitating the uncomplicated estimation of summary statistics that capture the load-associated pavement damage of a given axle load spectra.

Chapter 5, *Axle Load Specifications for Levels 2 and 3*, constitutes Product 3 of this research study. The chapter provides a justification for the practical advantages of using continuous functions rather than histograms to specify axle load distributions, as well as providing the reasons for integrating the axle spectra of all vehicle classes into four classes: single axles with single wheels, single axles with dual wheels, tandem axles, and tridem axles. The chapter concludes by providing Level 2 and Level 3 statistics and is complemented with Appendices F and G, which present the same information in a format compatible with that currently required to run software accompanying the M-E Design Guide.

Finally, conclusions and recommendations are presented in Chapter 6, followed by a list of references. The report is completed by a series of seven appendices (A through G) that complement the information contained in the various chapters.

# 2. Data Requirements for the M-E Pavement Design Guide

## 2.1 Introduction

Traditionally, the structural design of pavement makes use of empirical or empirical-mechanistic methods. The most widely used empirical design method is the current AASHTO Design Guide (AASHTO, 1993). In this guide, pavement life is accounted for in terms of accumulated number of equivalent single axle loads (ESALs). The basic design equation was obtained through regression analysis based on the results of the American Association of State Highway Officials (AASHO) Road Test during the late 1950s in Ottawa, Illinois (HRB, 1962). The limitations of the empirical approach have been identified in many research studies and in actual practice. On the other hand, the newly developed Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (the M-E Design Guide) aims at improving the facilitation of pavement design by focusing on highway pavement performance prediction (NCHRP, 2005). In the mechanistic-empirical (M-E) approach, pavement responses can be calculated through mechanistic analysis, such as finite element analysis or multi-layer linear-elastic theory. These responses (stresses and strains) are then correlated with pavement performance through performance or transfer functions, which are calibrated using field data such as the Long Term Pavement Performance (LTPP) database. One of the significant advantages of M-E design is its ability in location-oriented pavement performance prediction. In addition, hierarchical levels are considered to accommodate new and rehabilitated pavement design based on the significance of underlying project and resources availability. The three levels are designated as Level 1, Level 2, and Level 3, respectively.

This chapter consists of two main parts. The first part presents a brief review of the M-E Design Guide approach, aimed at identifying the detailed requirements of the major input components for pavement design: traffic, environment, and structure. The hierarchical input level approach is highlighted. The second part is concerned with recommendations on when and where to use the different input levels based on the findings of this research and other studies.

## 2.2 Inputs to the M-E Design Guide

The M-E Design Guide was developed under NCHRP Project 1-37 A (NCHRP, 2005). Figure 2.1 presents the main menu interface of the software developed under the project. It should be noted that the software and accompanying documentation are available online (NCHRP, 2005). The system is organized into four fundamental modules:

1. Project, which includes General information, Site/project information, and Analysis parameters,

2. Inputs, which includes Traffic, Climate, Structure, and Distresses potential,

3. Results, which includes Input summary and Output summary, and

4. Miscellaneous, which includes Analysis status, General project information, and Properties.

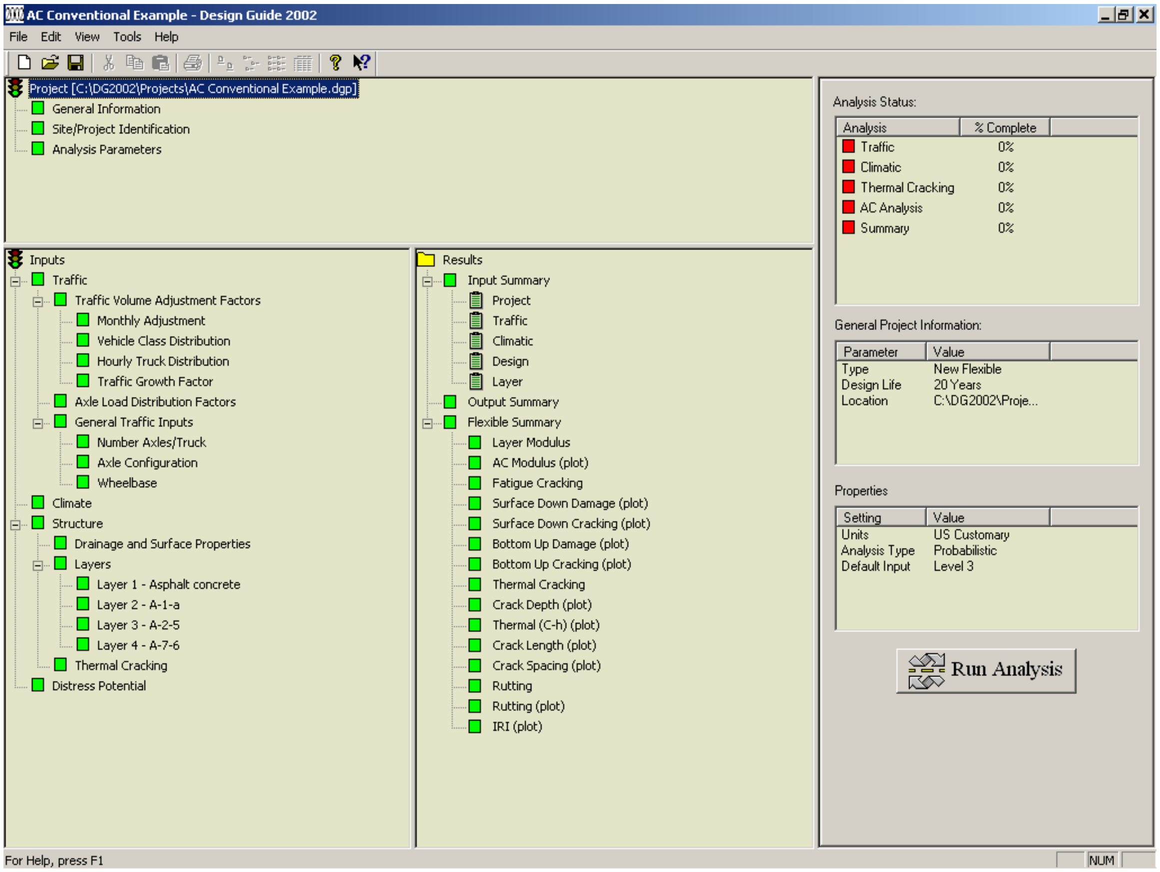


*Figure 2.1: Main Menu Interface of the M-E Design Guide*

The relevant information on inputs is now presented, along with a discussion of the three factors dominating pavement performance—traffic, climate/environment, and structure/material inputs. The organization follows the layout structure proposed by the M-E Design Guide software.

### 2.2.1 Traffic Input

1. Design Life (years)
2. Opening date
3. Initial two-way AADTT
    a. Two-way annual average daily traffic (AADT)
    b. Percentage of heavy vehicles (Class 4 or higher)
4. Number of lanes in design direction
5. Percentage of trucks in design direction (%)

6. Percentage of trucks in design lane (%)

7. Operational speed (mph)

8. Traffic volume adjustment

    a. Monthly adjustment factors (MAF) requires the factors (usually fluctuating around 1.0) from each month of a year for all the truck classes (Class 4 or higher). Input from Level 1 and Level 3 are available for this item. Level 1 is for site-specific MAF and Level 3 is for state default MAF.

    b. Vehicle class distribution requires AADTT distribution in percentage by vehicle class. The sum of the total truck class percentages should be 100 percent. It allows for input from Level 1 and Level 3. Level 1 is for site-specific distributions and Level 3 is for default distributions. With regard to default distributions, Truck Traffic Classifications (TTC) with seventeen categories are provided in the M-E Design Guide for users to select.

    c. Hourly distribution requires truck traffic distribution by period in each hour of a day.

    d. Traffic growth factors. Three growth functions are available for selection: none/zero growth, linear growth, and compound growth. For the latter two alternatives, percentage of growth rate is required for input. In addition, vehicle-class-based traffic growth models can be selected to accommodate a design with traffic growth estimates for each truck class.

9. Axle load distribution factor. Four types of axles are required with their respective axle load distributions (load spectra): single axle, tandem axle, tridem axle, and quad axle. An axle load spectrum of a given truck type is composed of normalized frequencies for all the load bins of that axle type on that type of truck. The number of load bins for single and tandem axles is 39, and the number for the tridem and quad axles is 31. In particular, the load spectrum of each axle type on each truck class in each month is required as input. Input of axle load spectra from Level 1 and Level 3 can be specified in the M-E Design Guide. Level 1 requires site-specific axle load spectra, while Level 3 uses the default information.

10. General traffic inputs, which include:

    a. Lateral traffic wander consisting of: mean wheel location (inches from the lane marking), traffic wander standard deviation (in inches), and design lane width (in feet).

    b. Number of axles per truck, which includes information on average number of axles for each of the four types for each truck class.

    c. Axle configuration covers axle and tire properties such as 1) average axle width (edge-to-edge outside dimensions, in feet); 2) dual tire spacing (in inches); 3) tire inflation pressure (in psi) for single and dual tires separately; and 4) axle spacing (in inches) for tandem, tridem and quad axle.

d. Wheelbase requires the input of average axle spacing (ft) and percent of trucks (%) for short, medium, and long wheelbase conditions.

### 2.2.2 Climate Input

Two alternatives can be used to specify the information for characterizing climate and environmental conditions: 1) import a previously generated climatic data file; 2) generate a new climatic data file from the relevant available weather stations. The weather database covers a wide range of stations across the U.S.

If the second alternative is chosen, two further options can be used to create the climatic file: 1) upload the weather data from a single specific weather station; or 2) create a virtual weather station through interpolating the information among up to six geographically close weather stations with available information. The additional information required for the interpolation is latitude (degrees, minutes), longitude (degrees, minutes), elevation (ft), and depth of water table (ft), which can use annual averages or (four) seasonal inputs.

### 2.2.3 Structural Input

The structural inputs thoroughly cover the pavement-related information. Focus is placed on the material properties that are related to pavement performance. The environmental factors that affect material properties are also included, together with the relevant material inputs. Considering the increased complexity of characterizing asphalt concrete materials and the focus of this research study, the case example presented herein deals with the design inputs for a new flexible pavement.

#### *2.2.3.1 Drainage and surface properties*

1. Surface shortwave absorption

2. Drainage Parameters, which include infiltration, drainage path length (ft), and pavement cross slope (%).

#### *2.2.3.2 Layers*

For flexible pavement, asphalt materials and unbound materials are generally used. Asphalt materials serve primarily as the surface or base layer of a pavement structure. Unbound materials are mainly referred to as untreated materials used in the base, sub-base, and subgrade. If bedrock exists under an alignment, the properties for bedrock should be provided.

1. Asphalt materials. Three levels of input are allowed for characterizing the asphalt material inputs in the M-E Design Guide. The three hierarchical ranks are Levels 1, 2, and 3, as previously described, and involve three input aspects: Asphalt mix, Asphalt binder, and Asphalt general. However, the detailed input requirements may vary between different levels.

a. Asphalt mix

   (i) For Level 1, the dynamic modulus of asphalt mix (E*) is required for establishing the master curve and shift factors. The number of temperatures (ranging from 3 to 8) and number of frequencies (ranging from 3 to 6) at which measurements are made should be specified. The dynamic modulus corresponding to each pair of temperature and frequency should be determined through a laboratory test under a dynamic (repeated) triaxial setup.

   (ii) For Levels 2 and 3, the dynamic modulus prediction equation is used to generate the master curve based on asphalt binder and asphalt general information. In addition, aggregate gradation is a required input, which includes the following details: cumulative percentage retained on the 3/4 in. sieve, cumulative percentage retained on the 3/8 in. sieve, cumulative percentage retained on the #4 sieve, and percentage passing the #200 sieve.

b. Asphalt binder

   (i) For Levels 1 and 2, either of two options concerning short term aging under the Rolling Thin Film Oven (RTFO) test can be selected: 1) Superpave binder test data, which requires dynamic complex modulus (G*) and phase angle (δ) (under test specification by AASHTO) (AASHTO, 2006) under the conditions of different temperatures determined by designer; or 2) Conventional binder test data, which requires a series of conventional binder properties under AASHTO specifications: softening point, absolute viscosity, kinematic viscosity, specific gravity, penetration, and Brookfield (rotational) viscosity.

   (ii) For Level 3, three options are available in the M-E Design Guide: 1) Superpave binder grading, which is specified by selecting the high- and low-temperature performance grade in the software; 2) Conventional viscosity grade, ranging from AC 2.5 to AC 40; and 3) Conventional penetration grade, which covers penetration grade from PEN 40-50 to PEN 200-300.

c. Asphalt general

   (i) Although asphalt general information can be input in three hierarchical levels, the input parameters into the M-E Design Guide software are exactly the same. The only difference is the way these parameters are obtained. The input parameters are: 1) reference temperature; 2) effective binder content; 3) air voids; 4) total unit weight; 5) thermal conductivity; 6) heat capacity; and 7) Poisson's ratio. The M-E Design Guide emphasizes the determination of three levels of input for Poisson's ratio. The guidelines are: 1) for Level 1, Poisson's ratio can be estimated from laboratory testing; 2) for Level 2, the ratio is based on the material density characteristics, which are further divided into three sublevels: Level 2A uses user-defined parameters a and b; Level 2B suggests using a specific pair of a and b; and Level 2C provides a series of typical ranges of Poisson's ratios by the M-E Design Guide for the user to choose; 3) for Level 3, typical Poisson's ratios are provided.

2. Unbound materials. The parameters used in the M-E Design Guide for unbound materials are standard to AASHTO and Unified Soil Classification (USC) definitions. The input for unbound materials is centered on the parameters related to strength properties.

    a. Strength properties allow for three-level hierarchical inputs. For all of the levels, Poisson's ratio and the coefficient of lateral pressure are required. The difference among the three design levels comes from the varying input required for the resilient modulus.

        (i) For Level 1, two options are available: 1) Integrated Climate Model (ICM) calculated modulus, which requires the input of three parameters K1, K2, and K3 for determining the modulus in lieu of the generalized model used in the M-E Design Guide, and other input (to be discussed particularly in the following for ICM input); 2) User input modulus, which further has two alternatives: seasonal input, requiring K1, K2, and K3 input for each month of a year, or representative values for K1, K2, and K3.

        (ii) For Level 2, general correlations between soil index and strength properties and resilient modulus are used. The alternative parameters involving the use of correlations are CBR, R-value, AASHTO layer coefficient, penetration from Dynamic Cone Penetrometer (DCP), and (based on) plasticity index (PI) and gradation. Furthermore, if ICM-calculated modulus or User Input modulus representative value icons are selected, a representative value should be provided. Alternatively, if the User Input Modulus Seasonal Input icon is selected, the input value for each month is required.

        (iii) For Level 3, only the default value for resilient modulus is required. Typical resilient moduli for unbound granular and subgrade materials (at optimum moisture content) are available in the M-E Design Guide software.

    b. Integrated Climatic Model (ICM). If the ICM-calculated modulus is selected in the strength properties screen, detailed ICM input is required. The purpose of incorporating ICM is to make seasonal adjustments to the strength values for seasonal changes. The required input is composed of:

        (i) Gradation and plasticity index, which includes: 1) plasticity index (PI); 2) percentage passing #200 sieve; 3) percentage passing #4 sieve; and 4) D60 (mm).

(ii) Calculated or derived parameters, which include: 1) maximum dry unit weight (pcf); 2) specific gravity of solids (Gs); 3) saturated hydraulic conductivity (ft/hr); 4) optimum gravimetric (%), and calculated degree of saturation (%). In addition, soil water characteristic curve parameters can be selected.

(iii) Finally, either the item of compacted unbound material or uncompacted/natural unbound material should be selected to represent the compaction condition during the construction phase.

3. Bedrock. The presence of bedrock can lead to significant change of pavement mechanistic response. If bedrock exists with 10 ft of the finished grade, the input for bedrock properties includes: 1) material type, which has two alternative options: highly fractured and weathered bedrock, and massive and continuous bedrock; 2) unit weight (pcf); 3) Poisson's ratio; and 4) resilient modulus (psi).

*2.2.3.3 Thermal cracking*

For asphalt pavements, the parameters to estimate thermal cracking are required as input. The relevant properties used for thermal cracking prediction are tensile strength, creep compliance, coefficient of thermal contraction, surface shortwave absorption, thermal conductivity, and heat capacity. Three-level hierarchical input should be specified for these parameters.

1. Average tensile strength at 14ºF (psi). Level 1 uses the information from actual laboratory tests in accordance with AASHTO specifications (AASHTO, 2006). For Level 2, the tensile strength estimated from correlations with other properties of the asphalt concrete is used. For Level 3, typical values are recommended in the M-E Design Guide.

2. Creep compliance. First, the creep test duration is to be specified. Two alternatives are 100 seconds and 1,000 seconds. Second, concerning the creep compliance, different design levels have specific input requirements.

   a. For Level 1, the input is from actual laboratory tests. The creep compliance for each loading time and temperature condition (low, middle, and high) is required.

   b. Level 2 uses the estimation from correlation. Only the creep compliance under middle temperature conditions is required.

   c. The input parameters for Level 3 are the same as that of Level 1, with the exception that typical test values are recommended by the M-E Design Guide rather than measured.

3. Surface shortwave absorption. This information was already used in the input for Drainage and Surface Properties.

4. Mix coefficient of thermal contraction. Two options are available for selection:

    a. direct input of this parameter, or

    b. input of the mixture's voids in the mineral aggregate (VMA) and aggregate coefficient of thermal contraction and letting the embedded equation calculate the corresponding value.

## 2.3 Recommendations on Input for Hierarchical Design Levels

Since the development of the M-E Design Guide was first initiated, the primary effort in the hierarchical level input has been focused on the requirement of information and the determination of the corresponding input for each level. The foremost issue concerning when and where to use the individual design levels has not yet been objectively established because of the difficulty in determining the design level to practice; a balance should be achieved for all inputs, including material properties and performance and traffic characterization. Theoretically, the determination of one specific design level is dependent on the importance of the project. For instance, the design of a project with significant importance, such as an interstate highway, will be assigned Level 1 input, whereas the design of a local low volume road can be categorized as Level 3 design. On the other hand, as was described in the previous section, each design level requires specific input, especially for the higher levels; but the resources may not exist. For instance, a new highway design project with Level 1 input requirements usually does not have site-specific traffic information, because there is no WIM or AVC deployed at that site. In this situation, Level 2 may be adopted as an alternative, although its importance requires Level 1 data input.

In summary, the selection of a specific design level can be dependent on two major factors: 1) the importance of the to-be-designed highway; and 2) the availability and affordability of the necessary resources. The recommendations for selecting a design level are as follows:

1. Level 1 requires the highest accuracy level and represents the case in which project site-specific information has been clearly determined. It is recommended for most high volume highways, where early failures may cause important safety or economic consequences. The highway facilities using Level 1 design may include interstate highways, high volume U.S. highways, and state highways. It is more likely that rehabilitation projects on these types of highways will involve the use of this level because the required input information may be readily available. In addition, as is recommended by the Federal Highway Administration's (FHWA) Design Guide Implementation Team (DGIT) (FHWA, 2005), research and forensic studies may be included with this level.

2. Level 2 represents the intermediate level of accuracy and reliability and is reserved for cases where there is some knowledge of the ongoing project. This level can be used for most high-grade highway facilities, which may include interstate, U.S., and state highways. In addition, under certain conditions, such as the design of a new highway (Level 1) when not all site-specific information may be available, Level 2 may also be used in combination with Level 1. It is suggested that Level 2 design be consistent with the current AASHTO Design Guide (1993). In addition, it is clear that Level 2 may

become the most widely and practically used level for new and rehabilitated pavement designs.

3. Level 3 requires the lowest accuracy level and should be used when there is little knowledge of the ongoing project. This level can be used to design low volume highways such as Farm to Market (FM) and other local roadways where the potential implication of an early failure will not be associated with significant economic impacts. In addition, if there is insufficient data to support a highway design with Level 2 input, Level 3 should be used instead.

Finally, it should be pointed out that for pavement design practices, the input levels can be mixed in order to match a given situation. For example, Level 2 traffic, Level 3 material, and Level 1 climatic data can be used as inputs. The process for conducting the calculation using the M-E Design Guide software is the same regardless of the input levels used. The only difference is the reliability of the final design. It should be noted that the lower accuracy level will primarily control the design reliability.

# 3. Data Management: Collection, Processing and Usage (P1)

## 3.1 Data Collection and Processing

The database used in this study for establishing axle load spectra was obtained from the existing Weigh-in-Motion (WIM) systems in Texas. To date, twenty WIM stations are deployed on the highway systems in Texas. Table 3.1 indicates that among the sites installed with WIM stations, two are located on state highways (SH), six are on U.S. highways (US), and the remaining twelve are located on the interstate highway system (IH). With regard to the functional classes, all of the WIM stations in Texas are installed to monitor truck traffic on rural segments of the highway system. The temporal distribution of the individual traffic data collection differs among the WIM stations. Table 3.1 indicates that the sampling duration of individual stations ranges from 1 year (such as D77) to 8 years (such as D512 and D516). The entire WIM system is managed by TxDOT's Transportation Planning and Programming (TPP) Division. The axle load data used for pavement design and rehabilitation are provided by TPP, typically in terms of the number of ESALs.

Two data sources were provided by TPP for this study. One contains raw data files in binary format collected during the period from January 1998 to March 2002 at WIM stations D512 and D516. The other data source is the database that includes pre-processed traffic information (converted from binary code to ASCII code). To cover the entire process of data preparation, the following discussion focuses on the first data source, starting from the very first step: raw data. The raw data files were downloaded from WIM stations D512 and D516 using the CC200 remote data collection program.

Daily traffic records are stored in one file in a binary format. These files are transferred into ASCII format by data evaluation software called REPORTER, which was designed for use with the PAT DAW 100 WIM system used in this project. The ASCII codes are imported into the database for further use.

### 3.1.1 Details on Processing Raw Data

The raw data are filed in a specified format by the REPORTER program with the name "Dsssmmdd.yy," in which:

| | | |
|---|---|---|
| D | : | raw data file designator, |
| sss | : | site number (e.g., 512 from Table 3.1), |
| mm | : | month, |
| dd | : | day, and |
| yy | : | year. |

The first step in processing the raw data is to generate statistical and traffic record files from the original "D" files. Each "D" file is then split into a classification data file with prefix "C," and the weight data file with prefix "W." In order to obtain axle load information on each individual vehicle, the weight file is converted into an ASCII file with prefix "A," which can be imported to other data analysis packages for weight analysis. For example, the generated output file name for D512 can be in the form of A5120920.99. Table 3.2 shows the fields of the individual vehicle records in each of the "A" files.

**Table 3.1: WIM Station Distribution in Texas**

| Sta ID | 1994 | 1995 | 1996 | 1997 | 1998 | 1999 | 2000 | 2001 | 2002 | County | District | Location |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 502 | √ | √ | √ | √ | √ | √ | | √ | √ | Guadalupe | San Antonio | Southwest of Seguin on IH 10 |
| 504 | √ | √ | √ | √ | √ | √ | | | | Nolan | Abilene | Southwest of Sweetwater on IH 20 |
| 506 | √ | √ | | | | | | | √ | Wichita | Wichita Falls | Northwest Wichita Falls on US 287 |
| 507 | √ | √ | √ | √ | √ | | | √ | √ | Walker | Bryan | South of Huntsville on IH 45 |
| 509 | √ | √ | | | | | | | | Hunt | Paris | East of Greenville on IH 30 |
| 510 | | √ | √ | √ | √ | | | | | El Paso | El Paso | Northwest El Paso on IH 10 |
| 512 | √ | √ | √ | √ | √ | √ | √ | √ | √ | Live Oak | Corpus Christi | North of Three Rivers on IH 37 |
| 513 | √ | √ | | | | √ | √ | √ | √ | Bell | Waco | South of Salado on IH 35 |
| 514 | √ | √ | | √ | √ | | | | √ | Kaufman | Dallas | Northeast of Kaufman on IH 20 |
| 515 | √ | √ | | | | | | | | Hidalgo | Pharr | South of Falfurrios on US 281 |
| 516 | √ | √ | √ | √ | √ | √ | √ | √ | √ | Bexar | San Antonio | Southeast of San Antonio on IH 35 |
| 517 | √ | √ | √ | | | | | | | Hidalgo | Pharr | Northeast Pharr on US 83 |
| 518 | | | | | √ | | | √ | √ | Kerr | San Antonio | East of Kerrville on IH 10 |
| 519 | | | | | | | | √ | √ | Mitchell | Abilene | East of Westwood on IH 20 (west of Colorado City) |
| 520 | | √ | √ | √ | √ | | | | | Randall | Amarillo | East of Canyon on IH 27 |
| 522 | | | | | √ | √ | √ | √ | √ | Hidalgo | Pharr | North of Site 515 on US 281 |
| 181 | | | | | | | | √ | √ | Cameron | Pharr | Northeast side of Brownsville on SH 48 |
| 71 | | | | | | | | √ | √ | McMullen | San Antonio | South of Tilden on SH 16 |
| 74 | | | | | | | | √ | √ | Kenedy | Pharr | East side of Sarita on US 77 |
| 77 | | | | | | | | | √ | Cameron | Pharr | Southeast of San Benito on US 77/83 |

**Table 3.2: Truck Record Data Fields in ASCII Format File**

| Field | Length | Range |
|---|---|---|
| Lane | 1 | 1..6 |
| Month | 2 | 1..12 |
| Day | 2 | 1..31 |
| Year | 2 | 0..99 |
| Hour | 2 | 0..23 |
| Minute | 2 | 0..59 |
| Second | 2 | 0..59 |
| Vehicle Number | 5 | 0..99999 |
| Vehicle Class | 1 | 1..15 |
| Gross Weight | 6:1 | |
| Vehicle Length | 6:1 | |
| Vehicle Speed | 5:1 | |
| Violations Code | 3 | 0..127 |
| Axle 1 RT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle 1 LT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle 2 RT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle 2 LT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle DIST. AX1-AX2 | 4:1 | 0..99.9 or Space |
| Axle 3 RT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle 3 LT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle DIST. AX2-AX3 | 4:1 | 0..99.9 or Space |
| … | | |
| Axle 13 RT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle 13 LT. Wheel WT. | 4:1 | 0..99.9 or Space |
| Axle DIST. AX12-AX13 | 4:1 | 0..99.9 or Space |
| Lane Direction | 1 | 0..9 |
| Number of Axles | 2 | 2..13 |

After obtaining the output file with traffic records in ASCII format, data cleansing of the erroneous records is required. In general, erroneous records are caused by: 1) inaccurate scales; 2) several vehicles combined in one record; 3) unreasonable axle spacings; 4) errors in axle spacing compared with TxDOT vehicle classifications; 5) ghost records; and 6) combined errors (Qu and Lee, 1997). However, it was found by Qu and Lee that erroneous records account for less than 1 percent of the sampled vehicle numbers; therefore, all the truck records are included in this study.

Because loads are recorded as individual wheel loads on each axle, wheel loads are converted to axle loads to obtain the axle load spectrum. In the next step, the axle type should be identified according to the spacing between adjacent axles. Four types of axles are defined by the Traffic Monitoring Guide (TMG) (FHWA, 2001): single axles, tandem axles, tridem axles, and quad axles. Notice that no distinction is made between single axles with single wheels and single axles with dual wheels (Prozzi and Hong, 2005; Prozzi et al., 2006).

## 3.2 Recommendations on Accuracy and Calibration Regimen of WIM Devices

Popularity of state-of-the-art WIM technology has increased, due in large part to its ability to effectively collect continuous traffic data. Theoretically speaking, a WIM scale, once installed, is able to continuously collect and record vehicle information. The WIM scale's main advantage is that it is capable of collecting population data samples, instead of just short-duration samples. Nevertheless, WIM system instability due to sensor technology, environmental effects, pavement conditions, and other factors gives rise to concerns about its measurement accuracy. The reliability of the WIM system for collecting accurate data relies heavily on its accuracy and calibration, which leads to an in-depth investigation of WIM measurement error and, more importantly, its effect on pavement performance estimation.

### 3.2.1 WIM Measurement Error

Generally speaking, measurement error can be caused by: 1) the measurement system or inspector; 2) the inspected objects; or 3) the processing of collected data. The focus in this study is specifically placed on the first source. Three major factors should be taken into consideration regarding the accuracy of a WIM scale: 1) roadway factors, among which pavement smoothness and longitudinal and transverse profiles play a central role; 2) vehicular factors, including speed, acceleration, tire condition, load, and body type; and 3) environmental factors, including wind, water, and temperature (Lee, 1998). In other words, WIM measurement error results from the combined effect of these relevant factors.

Mathematically, the measurement error of a WIM scale can be expressed in terms of percentage difference (relative error) as (Davies and Sommerville, 1987; Bergan et al., 1998):

$$\varepsilon(\%) = \frac{WIM\ Weight - Static\ Weight}{Static\ Weight} \times 100 \tag{3.1}$$

where WIM Weight = the weight recorded by WIM on one pass of a given axle load; Static Weight = the axle load weighed by a static scale.

The measurement error, $\varepsilon$, is comprised of two independent components according to the nature of the error, per se—random error and systematic error, respectively. The random error is described as the statistical fluctuations of measurement (in either direction) from the truth, and it is intrinsic to the measurement due to the inability of the device to precisely determine the truth. On the other hand, the systematic error persistently generates the inaccuracies along one direction, which could be due to issues such as faulty design or inadequate calibration.

Provided that a WIM system is properly installed in a sound road structure, and calibrated and subject to normal traffic and environmental conditions, only the random error occurs. The random errors of WIM observations exhibit a normal distribution with zero mean (Davies and Sommerville, 1987; Bergan et al., 1998). The standard deviation of the underlying normal distribution (sigma, $\sigma_\varepsilon$) is a measure that indicates the WIM accuracy or reliability (Bergan et al., 1998). The term $\sigma_\varepsilon$ herein is defined as the WIM accuracy indicator. Figure 3.1 illustrates the distributions of random error for weighing Gross Vehicle Weight (GVW) by three typical WIM systems [Single Load Cell ($\sigma_\varepsilon$ = 1.5%), Bending Strain ($\sigma_\varepsilon$ = 5%), and Piezo ($\sigma_\varepsilon$ = 10%)].

Systematic error in WIM observations is caused by inadequate calibration or calibration bias. The calibration bias may be due to initial improper calibration or the WIM system being out of calibration after being in service for a long time. An illustration of WIM systematic error is presented in Figure 3.2, which shows that the shift of the mean of the random error distribution (in this case $\sigma_{\varepsilon} = 5\%$ is fixed) leads to the systematic error. In the example of -10 percent bias, the WIM system is under calibrated, whereas +10 percent bias is an example of over calibration. In the case of ideal calibration, only random error occurs. In addition, it is assumed that when calibration bias occurs, both random and systematic errors exist.

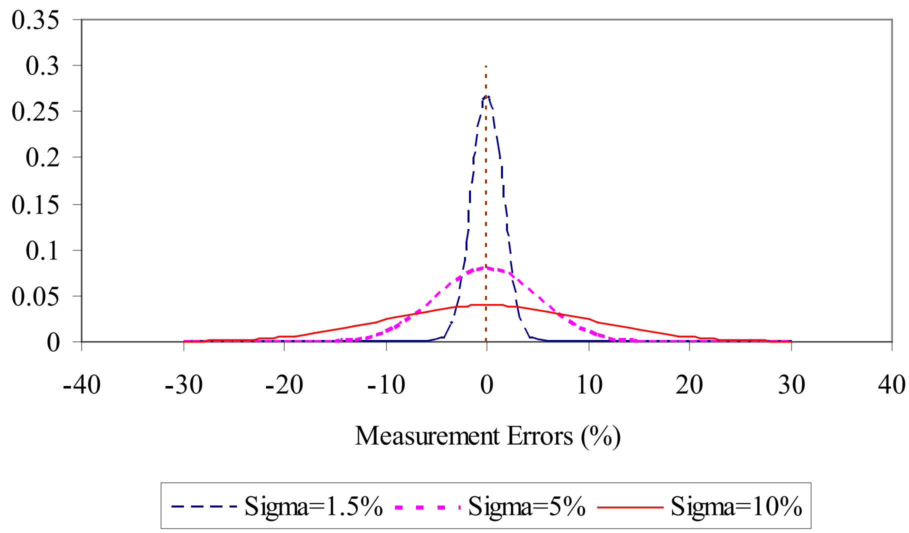


*Figure 3.1: Example of Random Errors of WIM System*

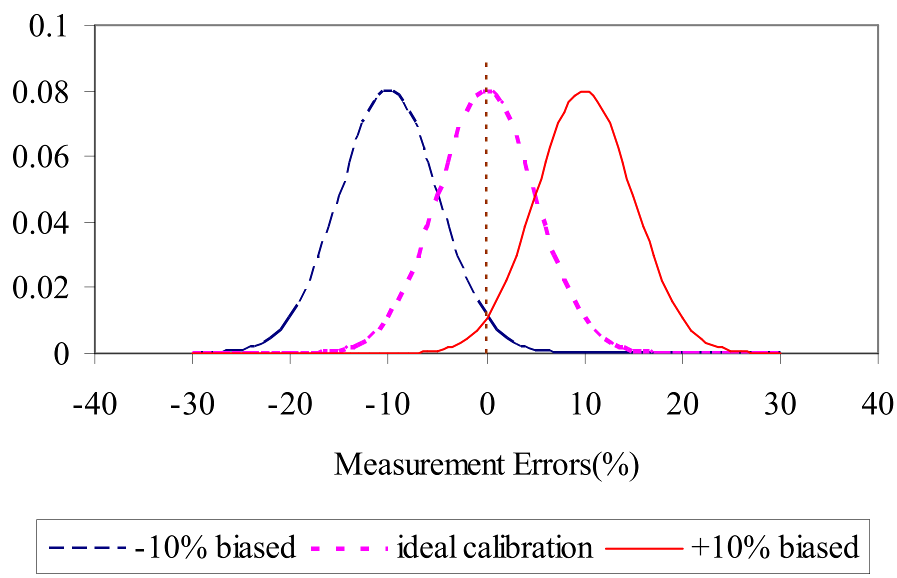


*Figure 3.2: Example of Systematic Errors of WIM System*

### 3.2.2 Effect of WIM Measurement Error on Pavement Performance Estimation

Two scenarios are investigated concerning the effect of WIM measurement error on pavement performance estimation. First, the load-related pavement damage estimation is derived under the condition of ideal calibration of a WIM scale (i.e., with zero calibration bias and involving random errors only). A comparison is carried out between the estimated load-related pavement damage with random measurement error present (normal distribution, $\sigma_\varepsilon$ not equal to zero) and that from the reference (true value). The second scenario investigates the load-related pavement damage estimation under the presence of biased WIM calibration. In such cases, not only systematic error but also random error is involved because the latter is inevitable. The results are presented in Figures 3.3, 3.4, and 3.5. Appendix A provides the details on how these results were derived.

With respect to WIM measurement under only random error, a range of WIM accuracy indicator $\sigma_\varepsilon$'s from 0 to 20 percent are adopted to address how these random errors affect the estimation of pavement performance. Figure 3.3 shows the relationship between varying WIM accuracy indicator $\sigma_\varepsilon$ (representing random error) and relative errors of load-related pavement damage estimation. It is shown that the random error leads to overestimation of load-related pavement damage. The overestimated load-related pavement damage could be as large as 25 percent.

When calibration bias occurs, both random and systematic errors should be addressed. Load-related pavement damage estimation error shows a significant variation. Figure 3.4 illustrates the effect of a series of combinations of WIM accuracy indicator $\sigma_\varepsilon$ and calibration bias $\alpha$ on load-related pavement damage estimation. It is shown that both overestimation and underestimation may occur. Also indicated is that load-related pavement damage estimation error is more sensitive to calibration bias than to WIM accuracy indicator $\sigma_\varepsilon$. In addition, the sensitivity of load-related pavement damage estimation to calibration bias is examined. To this purpose, a typical WIM scale with accuracy indicator $\sigma_\varepsilon = 5\%$ is employed, as shown in Figure 3.5. It was found that 10 percent over calibration results in up to 51 percent over estimation of load-related pavement damage, which is more significant than had been reported in previous work (FHWA, 1998). However, 10 percent under calibration produces results similar to those previously reported (approximately 31 percent underestimation of load-related pavement damage).

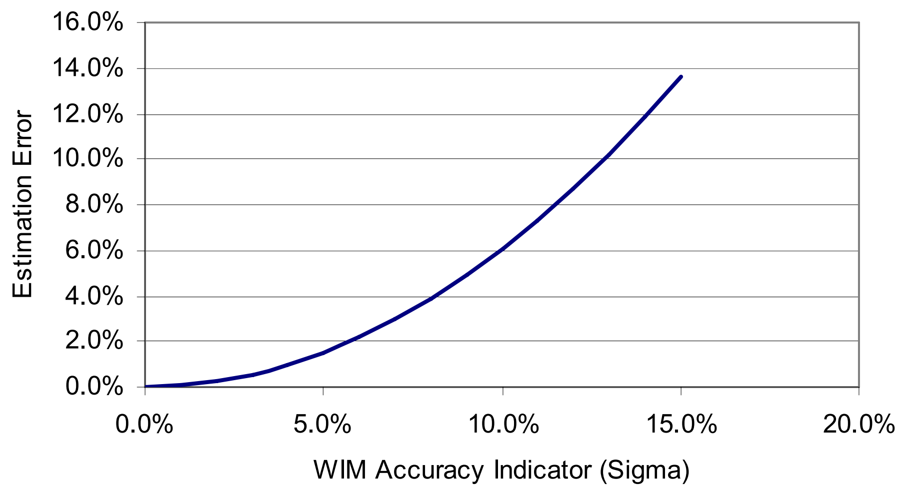


*Figure 3.3: Load-related Pavement Damage Estimation Error vs. Random Error*

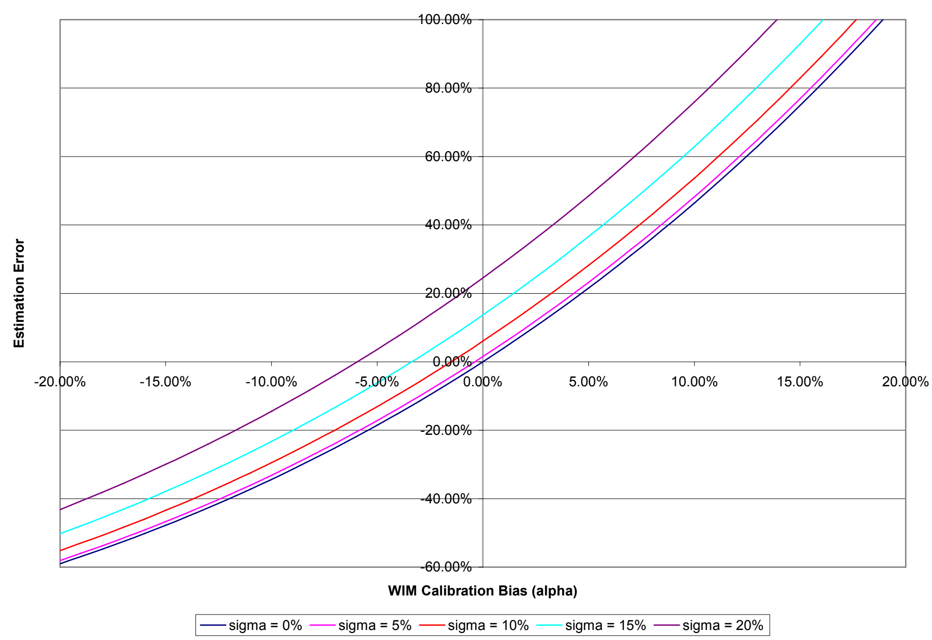


*Figure 3.4: Load-pavement Estimation Error vs. Random and Systematic Errors*

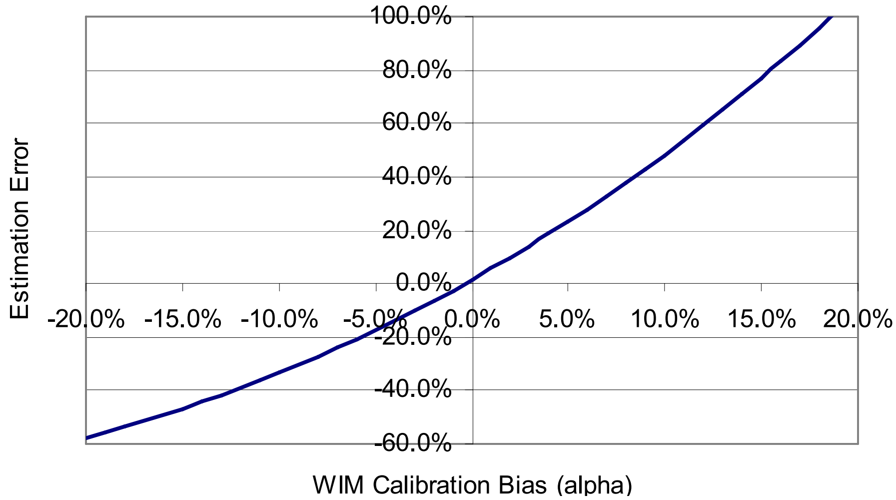


*Figure 3.5: Sensitivity of Performance Estimation Error on Calibration Bias*

## 3.3 Recommendations on Sampling Density and Frequency

As previously discussed, theoretically speaking, a WIM scale is able to continuously collect and record vehicle information. However, in practice, because of the large amount of space required for storing all the collected data, short-duration data collection is usually adopted. This leads to the need to evaluate the accuracy of estimating axle load spectra obtained from a limited sample data set.

The objective in this report is to quantify how much axle load spectra estimated by varying sample sizes deviate from the spectra determined by using the entire population. Consequently, the result can be applied to determine the sample size needed to accommodate pavement design and rehabilitation with different levels of accuracy or reliability.

### 3.3.1 Sampling Scheme

A series of samples of varying sizes (survey duration) are randomly drawn from the population. The population is from WIM station D512 (Table 3.1), with the data collection duration covering the period from 1998 to 2002. Three scenarios are utilized to draw the various samples. The first scenario focuses on a 1-day basis sample from different duration units: 1 day/month, 1 day/quarter, and 1 day/year. This scenario is considered because these 1-day samples are used to represent the minimum cost data collection strategy. The second scenario is based on 2-continuous-day basis samples: 2 days/month, 2 days/quarter, and 2 days/year. Considering current practice, whereby 2 continuous days' data per quarter are reported by the state to FHWA, this scenario is proposed to cater to a similar requirement. The third scenario is based on the 1-week basis sample (i.e., 7 consecutive days): 1 week/month, 1 week/quarter, and 1 week/ year. Typically, traffic volume varies among days, particularly between weekdays and weekends. A sample collected seven days in a row overcomes this potential source of data variability. Hence, a week-based sample is aimed at eliminating the possible within-week variation, which has no relevance to the structural design of pavements. The sampling scheme and sizes of the individual samples are summarized in Table 3.3. To facilitate the discussion in the following paragraphs, the analysis for two representative truck classes (Class 10 [Class 9 in

TMG, 2001, i.e., 18-wheeler] and Class 5) are highlighted because these two types of trucks account for the vast majority of truck traffic on Texas highways as well as in most states.

**Table 3.3: Sampling Scheme and Sample Sizes**

| Sampling Scheme | Scenario 1 | | | Scenario 2 | | | Scenario 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1d/m | 1d/q | 1d/y | 2d/m | 2d/q | 2d/y | 1w/m | 1w/q | 1w/y |
| No. of Days/Year | 12 | 4 | 1 | 24 | 8 | 2 | 84 | 28 | 7 |

### 3.3.2 Sensitivity Analysis

Intuitively, with increasing sample sizes (number of days per year in Table 3.3), the difference in axle load distribution between individual samples and the population should decrease. This hypothesis is supported by the examples given in Figures 3.6 to 3.8, which compare tandem load spectra for an 18-wheeler truck from three sample sizes: 1 day/year, 1 day/quarter, and 1 day/month with that from the population, respectively. With the growing sample sizes, the sample load spectrum curve moves closer to the population load spectrum curve. To specifically quantify the difference between the load spectra obtained from varying sample sizes and the populations, two alternative criteria were utilized. The first criterion is based on the Sum of Absolute Error (SAE) and the second criterion is based on the associated load-related pavement damage.

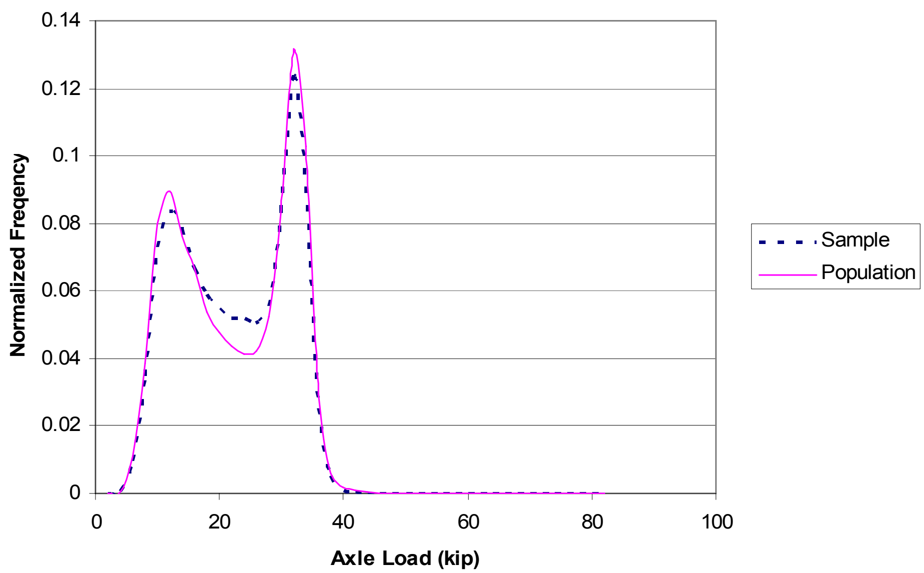


*Figure 3.6: Load Spectra Comparison Between 1 Day/year Sample and Population*

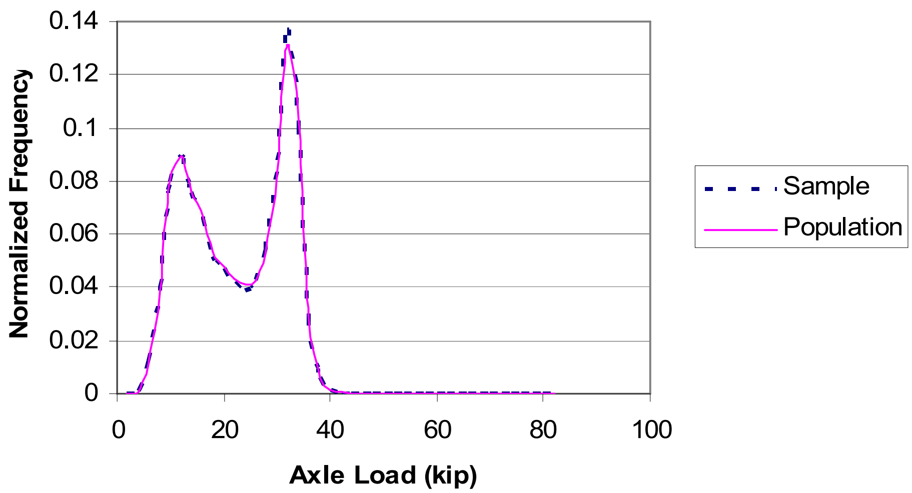


*Figure 3.7: Load Spectra Comparison Between 1 Day/quarter Sample and Population*

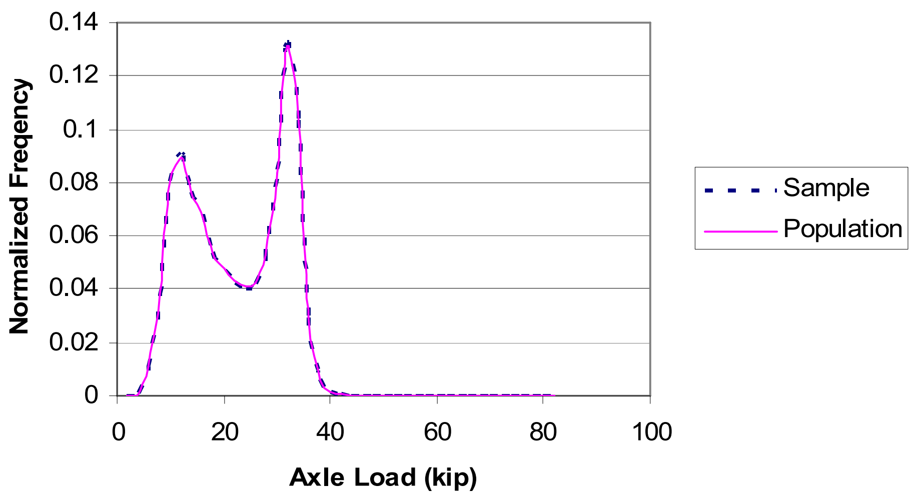


*Figure 3.8: Load Spectra Comparison Between 1 Day/month Sample and Population*

*3.3.2.1 Criterion 1: Sensitivity analysis in terms of SAE*

As shown in figures 3.6 through 3.8, the approximation of load spectrum from sampled data to that from population data can be employed as a criterion to measure the accuracy of the axle load spectrum under varying sample sizes. The SAE is proposed to quantify this difference. SAE is defined as follows:

$$SAE = \sum_i \left| f_i^s - f_i^p \right| \tag{3.2}$$

where,

$f_i^s$ : normalized frequency of the i^th bin of the sample spectrum, and

$f_i^p$ : normalized frequency on the i^th bin from the population spectrum.

Consequently, the SAE for each of the sample load spectrum are obtained. A number of random samples are drawn repeatedly for each of the sample schemes mentioned previously. Figure 3.9 shows the distribution of the SAE for the different sample sizes for Class 5 single axle loads. The x-axis represents the total number of surveying days or sample size. For more detailed information regarding the number of days of each sample, see Table 3.3. The line in the figure connects the means of corresponding to each given sample size. In general, as the sample size increases, the SAE decreases, i.e., the precision of load spectrum is improved. Roughly, the average SAE decreases from 7.4 percent for the smallest sample size (1 day/year) to 1.1 percent for the largest sample size (7 days/month or 84 days /year). Increased precision (reduced SAE) is not only the result of increasing the sample size (survey days) but also of the distribution of the surveying periods during the year. For instance, Figure 3.9 shows that the estimated error in the 1 day/quarter case (sample size = 4 days) has approximately the same error as the case of 7 day/year (sample size = 7 days). This is because during the latter, although data are collected during more days, the survey is more affected by underlying seasonal effects.

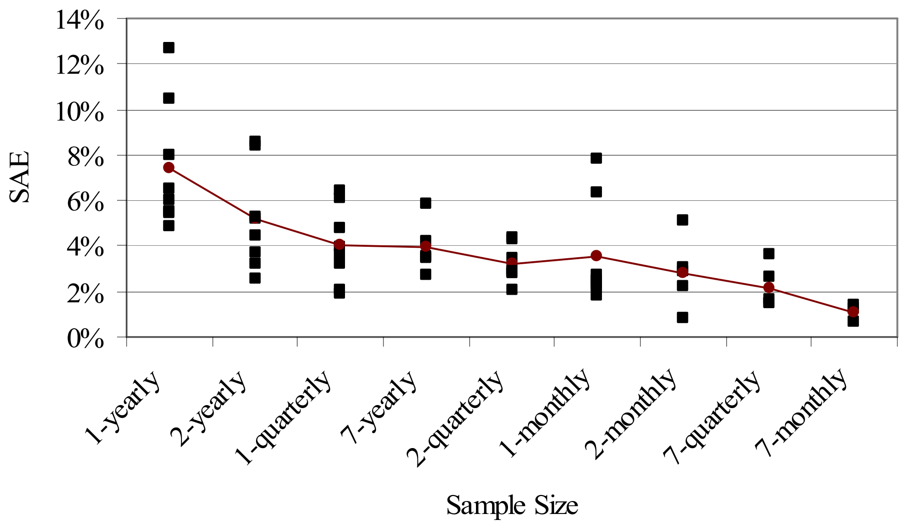


*Figure 3.9: Sensitivity of Class 5 Truck Single Axle Load Spectra*

Figures 3.10 and 3.11 present the load spectra estimation error for the single and tandem axles of truck Class 10 (*"18-wheeler"*). Notice that the single axle incorporates both the steering axle and single axle with dual wheels.

Similar to the load spectrum error estimated for Class 5, both the mean SAE for the single and tandem axle load spectra decrease as the sample sizes increase. The SAE decreases from around 8.7 percent to 1.4 percent as the sample varies from the smallest to the largest size for single axles; the corresponding values for the tandem axles vary from 7.7 percent to 1.3 percent. The high value of SAE corresponding to the sample of 1 week/year is of particular interest (Figure 3.10). The reason for this peak might be due to the load spectra of truck Class 10 being sensitive to seasonal fluctuations, and the 7 consecutive days' sample per year fails to capture this seasonality. Furthermore, there is no significant difference in terms of the SAE among the sampling alternatives on the monthly basis, whatever the length of sample time within the monthly duration unit. In this sense, it is concluded that spreading the surveys throughout the years (more surveys) is more important than increasing the survey length (more consecutive days), especially in areas where traffic is significantly affected by seasonal effects. This implication is meaningful and conducive to a more efficient WIM data collection scheme for the sake of improving axle load spectra precision.

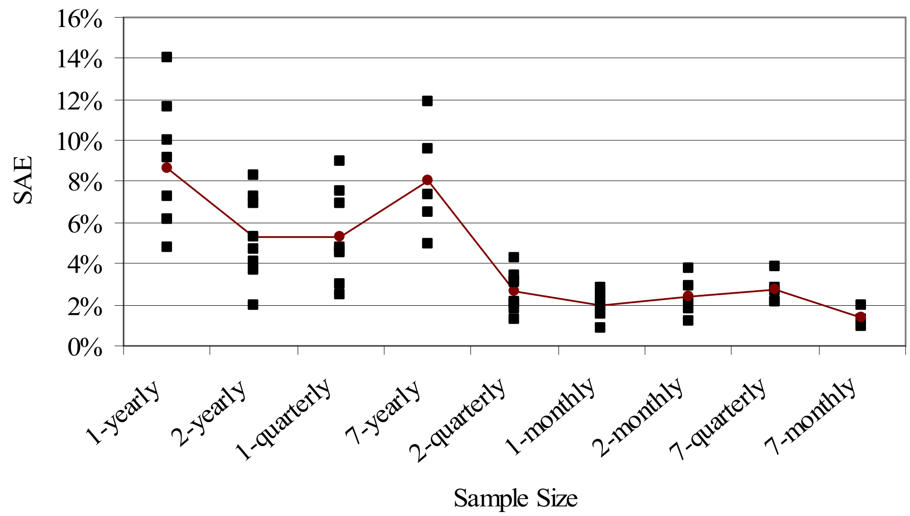


*Figure 3.10: Sensitivity of Class 10 Truck Single Axle Load Spectra*

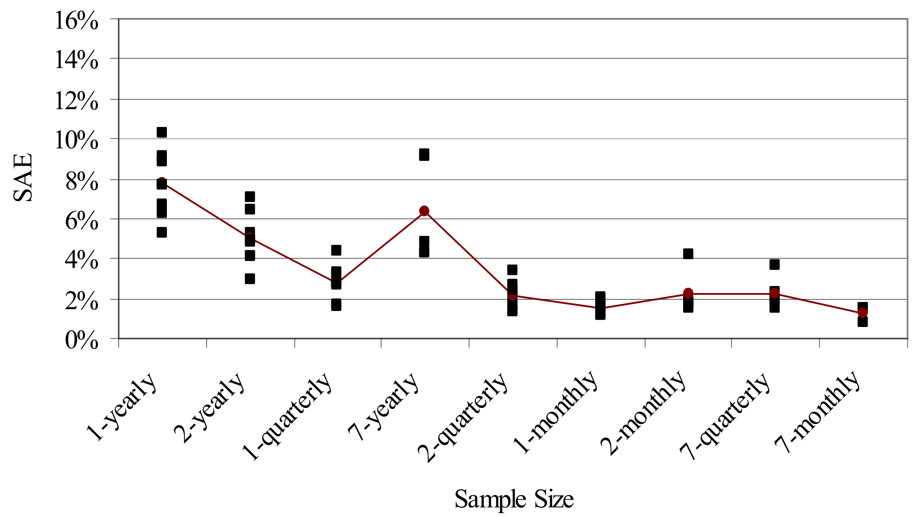


*Figure 3.11: Sensitivity of Class 10 Truck Tandem Axle Load Spectra*

*3.3.2.2 Criterion 2: Sensitivity analysis in terms of load-related pavement damage*

Criterion 1 is concerned with the mathematical fit of axle load distribution, per se. In the context of pavement design and rehabilitation, it is not the error in the actual data that is important but the error in the relevant statistics that relate to load-related pavement damage. As will be discussed in Chapter 4, load-related pavement damage based on axle load spectra can be captured by means of a variety of moment-related statistics. In this section, only the summary statistic is presented, denoted as load spectra factor (LSF):

$$LSF = \sum_{i=1}^{I} \left( \frac{x_i}{L_s} \right)^m f_i \tag{3.3}$$

where

$i$ : $i^{th}$ bin of axle load distribution,

$I$ : total number of bins,

$x_i$ : load weight falling in the $i^{th}$ bin,

$L_s$ : standard axle load for a given axle configuration,

$m$ : moment order, and

$f_i$ : normalized frequency of axle load falling in $i^{th}$ bin.

In addition, Chapter 4 will show that the order of the moments that correlate with load associated pavement damage usually ranges from 1 to 4. The lower moment orders (i.e., 1 to 2) tend to more adequately capture damage caused by rutting, and higher values (around 4) tend to more adequately represent damage caused by fatigue cracking and loss of serviceability.

Four representative moment order conditions are selected for analysis: 1, 2, 3, and 4. The analysis of the load-related pavement damage sensitivity on sampling frequency focuses on the largest truck class: Class 10. Both single and tandem axles are evaluated. For each sample frequency, two samples are randomly obtained, denoted as S1 and S2. The results are presented in Tables 3.4 and 3.5, respectively. The error is defined as the relative difference between the moments obtained from sample and population.

**Table 3.4: Load-pavement Estimation Sensitivity (Error in %) for Single Axle of 3S2**

| Moment Order | 1 day /year | | 2 days /year | | 1 day /quarter | | 1 week /year | | 2 days /quarter | | 1 day /month | | 2 days /month | | 1 week /quarter | | 1 week /month | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 |
| 1 | -0.4 | 1.6 | -0.2 | 0.5 | -0.4 | 0.3 | 0.8 | 0.7 | 0.3 | -0.6 | 0.0 | 0.1 | 0.0 | 0.5 | -0.3 | -0.2 | 0.1 | 0.2 |
| 2 | -0.5 | 3.2 | -0.6 | 1.0 | -1.0 | 0.5 | 1.6 | 1.5 | 0.6 | -1.1 | -0.1 | 0.1 | 0.1 | 0.9 | -0.6 | -0.5 | 0.1 | 0.3 |
| 3 | -0.4 | 4.8 | -1.1 | 1.6 | -1.7 | 0.5 | 2.3 | 2.5 | 1.0 | -1.6 | -0.2 | 0.1 | 0.2 | 1.3 | -1.1 | -0.9 | 0.1 | 0.5 |
| 4 | -0.2 | 6.4 | -2.0 | 2.0 | -2.3 | 0.2 | 2.9 | 3.4 | 1.6 | -2.2 | -0.4 | 0.1 | 0.2 | 1.8 | -1.7 | -1.4 | 0.1 | 0.6 |

**Table 3.5: Load-pavement Estimation Sensitivity (Error in %) for Tandem Axle of 3S2**

| Moment Order | 1 day /year | | 2 days /year | | 1 day /quarter | | 1 week /year | | 2 days /quarter | | 1 day /month | | 2 days /month | | 1 week /quarter | | 1 week /month | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 | S1 | S2 |
| 1 | -0.4 | 2.2 | -0.6 | 1.3 | 0.0 | 0.2 | 0.8 | 1.8 | 0.9 | -0.1 | -0.1 | 0.1 | 0.0 | 0.7 | -0.4 | 0.2 | 0.2 | 0.3 |
| 2 | -1.9 | 3.6 | -1.5 | 2.7 | -0.1 | 0.4 | 0.8 | 2.7 | 1.8 | -0.2 | -0.2 | 0.1 | -0.1 | 1.4 | -0.6 | 0.3 | 0.5 | 0.4 |
| 3 | -3.6 | 4.3 | -2.5 | 4.2 | -0.2 | 0.5 | 0.4 | 3.0 | 2.7 | -0.6 | -0.4 | 0.2 | -0.1 | 2.1 | -0.8 | 0.2 | 0.7 | 0.4 |
| 4 | -5.4 | 4.6 | -3.5 | 5.6 | -0.7 | 0.5 | -0.3 | 3.1 | 3.4 | -1.0 | -0.8 | 0.1 | -0.1 | 2.8 | -1.1 | -0.1 | 0.8 | 0.4 |

The results are consistent with those obtained by applying Criterion 1. That is, as the sample size increases, the error decreases. Furthermore, it is interesting to note that the errors from different samples are relatively small. It is implied that axle load spectra are not significantly sensitive to the sample sizes when they are accounted for in terms of load-associated pavement damage. In this sense, smaller sample sizes or frequencies (such as 2 days/year and 1 day/ quarter) are adequate to accommodate pavement design with relatively high accuracy. In addition, the current practice of sampling WIM data in Texas (2 days/quarter) provides fairly accurate traffic load data for pavement design.

## 3.4 Recommendations on Location and Number of WIM Stations

One of the most distinctive features between the M-E Design Guide and the current design guide (AASHTO, 1993) is the adoption of hierarchical design levels. It is proposed that a pavement can be designed to one of three design levels based on the importance of the underlying project or availability of information and resources. With regard to traffic input, different traffic load information is required for each of the three levels:

1. Level 1 is the most accurate design level and will require site-specific axle load spectra for each truck class developed from WIM systems.

2. Level 2 is the intermediate design level and will require regional axle load spectra for each truck class.

3. Level 3 is the least accurate design level, which will make use of default or statewide axle load spectra based on the available traffic data.

In practice, site-specific axle load spectra required by Level 1 can be obtained from the available WIM data in a straightforward manner, but only from those locations where the WIM system has been installed. For the design level requiring the lowest accuracy (Level 3), the default axle load spectra for each truck class can be obtained by pooling the data from all the available WIM sites. For the intermediate design level (Level 2), a series of aspects increases the difficulty in establishing the regional axle load spectra. It is known that traffic volume is positively correlated with the highway functional class: the *higher* the facility class, the higher the volume. However, unlike traffic volume, axle load spectrum reflects the axle load distribution along a period of time in terms of normalized frequencies. Hence, the possible situations are: 1) there is no significant difference among load spectra in terms of load-related pavement damage among sites, although traffic volumes differ; and 2) the load spectra difference does exist regardless of whether or not the volumes differ. In the latter case, care should be taken when applying the load spectra in pavement design. For instance, two highways with similar traffic volumes may have different load distributions, which in turn may result in a pronounced difference in their service lives. In addition, the division into regions remains a challenge. The regional load spectrum should be representative of its region's load characteristics. This is usually related to the geographic boundary, commodity flow, or industrial, agricultural, and commercial operations prevailing in the given region. Therefore, characterizing and providing regional axle load spectra for the implementation of the upcoming M-E Design Guide is a critical and complex issue. To begin, data have to be available for each region and condition (facility type, environment). Therefore, the minimum number and locations of WIM stations should be established. To this effect, the TMG recommends categorizing the roads into Truck Weight Road Groups (TWRG) (FHWA, 2001), based on research conducted in Washington. Six WIM sites are recommended for each TWRG.

Some qualitative guidelines are provided by the TMG for establishing TWRG. It is suggested that TWRG be created in such a way that each group of roads experiences loads with similar characteristics. The two most basic grouping criteria could be through-truck percentages and geographic regions with specific economic traits (FHWA, 2001). However, it was pointed out in a pioneering research study using traffic data in Washington that forming homogenous groups is not always possible (Hallenbeck and Kim, 1993).

As one of the largest states in the U.S., the characteristics of axle load distribution in Texas are diverse. Moreover, bordering with Mexico, Texas experiences traffic impacts due to the North American Free Trade Agreement (NAFTA). It is expected that traffic load patterns on the highways affected by NAFTA-related traffic will be different from those of other highways.

To meet the requirement of establishing the minimum number of WIM stations, three significant factors were considered in detail in this research study:

- Regions: Establishing regions to characterize traffic loading by accounting for factors such as geographic condition, industrial, agricultural, and commercial activity. Because of the size of the state of Texas and its economic diversity, traffic loading patterns vary across regions. Integrating the considerations of district boundaries and freight distribution, the following eight regions have been established (TTI, 2003). A map of regions by districts is illustrated in Figure 3.12:

    a) Panhandle,

    b) West,

    c) North Interstate Highway (IH) 35 corridor,

    d) Central Texas,

    e) South IH 35 Corridor (adjacent to Mexico border),

    f) Piney Woods,

    g) South Coastal, and

    h) North Coastal.

- Location (rural, urban, or suburban): Currently, all of the WIM stations in Texas are located in rural areas. Freight modes may differ between rural and urban areas, in addition to the levels and types of economic activity. For instance, the major trucks running on urban area highways focus on short-distance transportation. Relatively light trucks and partially loaded trucks may account for the majority of the truck traffic. On the other hand, rural highways (especially interstate highways) are utilized by a significant proportion of long-distance and fully loaded heavy trucks (such as the "18-wheeler" truck). Consequently, load distribution patterns vary among the highways in rural and urban areas. In addition, a highway located in a suburban area may also demonstrate different load patterns from a highway in an urban or rural area, perhaps because of the area's unique economic development. It is important to note that urban traffic may already be saturated, while a suburban area may be experiencing rapid traffic growth as part of the area's urbanization process.
- Class: As recommended by TMG (FWHA, 2001), different functional classes of highways may experience distinct axle load spectra characteristics. In this study, highways are categorized into four groups based on their facility classes: 1) Interstate highways; 2) U.S. highways; 3) state highways; and 4) other lower-class highways (mostly consisting of FM or Ranch-to-Market [RM] highways). Currently, the twenty WIM stations in Texas are all located on the first three classes of highway facilities (see Table 3.1).

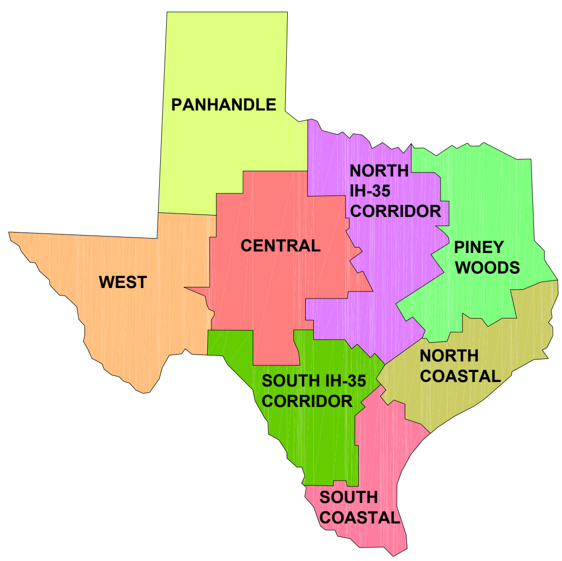


*Figure 3.12: Texas Regions for WIM System Deployment Analysis*

Therefore, the numbers and locations of WIM stations in Texas are to be determined based on the three factors just discussed. In order for TxDOT to better implement the deployment of WIM systems, two schemes are suggested.

Scheme 1 fully considers the three factors' combinations, which include eight regions, three locations (urban [U], suburban [S], and rural [R] areas), and four highway functional classes. In addition, to address the variation of the traffic data, three repetitions of WIM scales are suggested for each possible combination. As a result, the total number of WIM stations suggested in Scheme 1 is:

| |
|---|
| 8 (regions) ×3 (location) × 4 (highway classes) × 3 (replicates) = 288 |

Alternatively, in Scheme 2, a less ambitious plan is suggested that contemplates a smaller number of WIM stations as compared to Scheme 1. Two modifications are suggested: 1) seven regions are proposed instead of eight regions (two coastal areas are combined); and 2) only urban and rural areas are differentiated. The rest remains the same. As a result, the total number of WIM stations suggested in Scheme 2 is:

| |
|---|
| 7 (regions) ×2 (U/R) × 4 (highway classes) × 3 (replicates) = 168 |

Considering current budget constraints and the high cost of deploying the entire proposed WIM system, it is suggested that the implementation plan adopt a phase-by-phase approach. For instance, the three replicates for each combination of region, location, and class could be installed in consecutive phases. Thus, in Scheme 1, the first ninety-six WIM stations could be installed, followed by the remainder of the stations in stages, as funds become available. The phased approach would also enable monitoring of the plan's effectiveness as further developments occur in the area of pavement design.

The exact locations of the individual WIM stations are not presented in this report because of the need for numerous practical considerations, the most important being the analysis of current highway construction plans in Texas. The installation of a WIM scale needs to involve a series of technical criteria. The major technical considerations suggested by TMG (FHWA, 2001) are:

1. Flat pavement with adequate riding quality,
2. Pavement that is in good structural condition and that has enough strength to adequately support axle sensors,
3. Vehicles traveling at constant speeds over the sensors, and
4. Access to power and communication systems.

# 4. Recommendations for Project-Specific Traffic Data Validation and Usage

## 4.1 Long-term Traffic Volume Analysis: Traffic Growth

Traffic characterization for highway pavement design comprises two aspects: traffic loads and traffic volumes. Traffic loads, expressed in terms of axle load spectra, are developed based on Weigh-in-Motion (WIM) data and have been discussed previously in this report. In this chapter, traffic volume is addressed from the point of view of its statistical characteristics in conjunction with forecasting. In order to reflect traffic growth characteristics among different highway facilities in Texas, fifteen representative facilities have been selected in this study to cover four highway functional classes: interstate highways (IH), U.S. highways (US), state highways (SH), and farm to market roads (FM). In addition, considering that most of the damage to a pavement structure is caused by commercial trucks, only truck volumes are investigated. With regard to growth characteristics, both linear and compound growth trends are evaluated based on historical data.

The data set used in this part of the study is from the database developed and maintained by Transportation and Logistics (TLOG). The data records cover the period from 1979 to 2002, with truck traffic data covering the period from 1986 to 2002. For preparing traffic data for pavement design, the main truck traffic characteristics are highlighted. The TLOG database provides detailed traffic information for the major fields, including district (Dist), county (Co), beginning mark point (Beg Mpt), ending mark point (End Mpt), highway number (Highway #), year (Yr of AADT), current AADT (Cur AADT), percentage of truck in AADT (% Trk in AADT), and number of trucks in AADT (# of trucks). Specific traffic volumes at a series of segments along each highway are reported in terms of AADT during the period. The length of each segment varies and is measured by the distance between its beginning mark point and its ending mark point. In addition, district and county numbers are available for capturing the location of each segment of highway. TxDOT divides the state into twenty-five districts, and there are a total of 254 counties in the state. The geographical distribution of the TxDOT districts is shown in Figure 4.1, with the name abbreviations shown in Table 4.1. Detailed geographical information in terms of a county's boundaries is presented in Figure 4.2, and the numbers and county names are shown in Table 4.2.


*Figure 4.1: TxDOT District Boundaries*

**Table 4.1: TxDOT District Abbreviations**

| District | | District | | District | | District | | District | |
|---|---|---|---|---|---|---|---|---|---|
| Name | Abbreviation | Name | Abbreviation | Name | Abbreviation | Name | Abbreviation | Name | Abbreviation |
| Abilene | ABL | Brownwood | BWD | El Paso | ELP | Lufkin | LFK | San Antonio | SAT |
| Amarillo | AMA | Bryan | BRY | Forth Worth | FTW | Odessa | ODA | Tyler | TYL |
| Atlanta | ATL | Childress | CHS | Houston | HOU | Paris | PAR | Waco | WAC |
| Austin | AUS | Corpus Christi | CRP | Laredo | LRD | Pharr | PHR | Wichita Falls | WFS |
| Beaumont | BMT | Dallas | DAL | Lubbock | LBB | San Angelo | SJT | Yoakum | YKM |

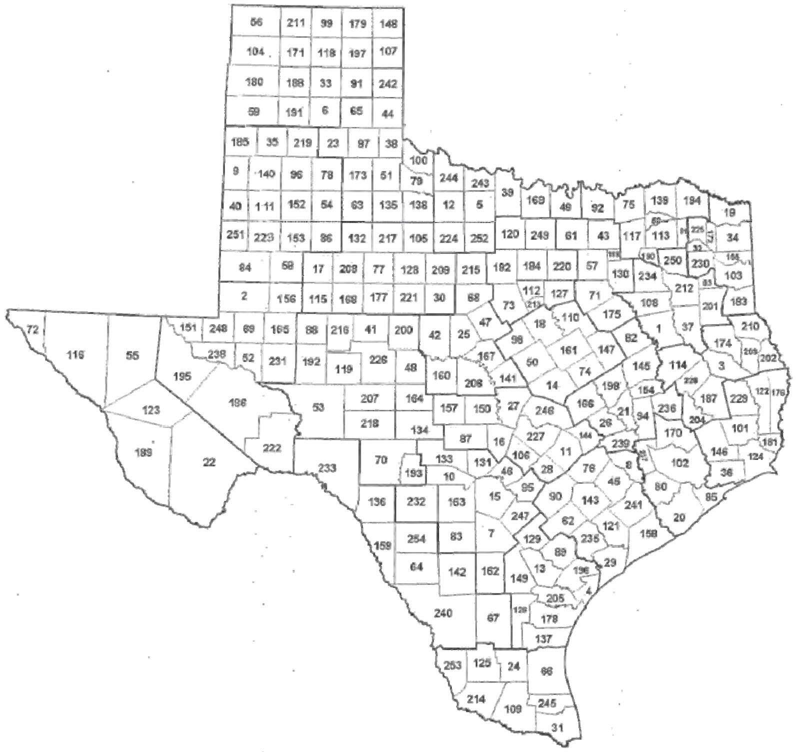

*Figure 4.2: Texas County Boundaries*

**Table 4.2: Texas County Numbers and Corresponding Districts**

| County | | | County | | | County | | | County | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| # | Name | District | # | Name | District | # | Name | District | # | Name | District |
| 1 | Anderson | TYL | 65 | Donley | CHS | 129 | Karnes | CRP | 193 | Real | SJT |
| 2 | Andrews | ODA | 66 | Kenedy | PHR | 130 | Kaufman | DAL | 194 | Red River | PAR |
| 3 | Angelina | LFK | 67 | Duval | LRD | 131 | Kendall | SAT | 195 | Reeves | ODA |
| 4 | Aransas | CRP | 68 | Eastland | BWD | 132 | Kent | ABL | 196 | Refugio | CRP |
| 5 | Archer | WFS | 69 | Ector | ODA | 133 | Kerr | SAT | 197 | Roberts | AMA |
| 6 | Armstrong | AMA | 70 | Edwards | SJT | 134 | Kimble | SJT | 198 | Robertson | BRY |
| 7 | Atascosa | SAT | 71 | Ellis | DAL | 135 | King | CHS | 199 | Rockwall | DAL |
| 8 | Austin | YKM | 72 | El Paso | ELP | 136 | Kinney | LRD | 200 | Runnels | SJT |
| 9 | Bailey | LBB | 73 | Erath | FTW | 137 | Lleberg | CRP | 201 | Rusk | TYL |
| 10 | Bandera | SAT | 74 | Falls | WAC | 138 | Knox | CHS | 202 | Sabine | LFK |
| 11 | Bastrop | AUS | 75 | Fannin | PAR | 139 | Lamar | PAR | 203 | San Augustine | LFK |
| 12 | Baylor | WFS | 76 | Fayette | YKM | 140 | Lamb | LBB | 204 | San Jacinto | LFK |
| 13 | Bee | CRP | 77 | Fisher | ABL | 141 | Lampasas | BWD | 205 | San Patricio | CRP |
| 14 | Bell | WAC | 78 | Floyd | LBB | 142 | Lasalle | LRD | 206 | San Saba | BWD |
| 15 | Bexar | SAT | 79 | Foard | CHS | 143 | Lavaca | YKM | 207 | Schleicher | SJT |
| 16 | Blanco | AUS | 80 | Fort Bend | HOU | 144 | Lee | AUS | 208 | Scurry | ABL |
| 17 | Borden | ABL | 81 | Franklin | PAR | 145 | Leon | BRY | 209 | Shackelford | ABL |
| 18 | Bosque | WAC | 82 | Freestone | BRY | 146 | Liberty | BMT | 210 | Shelby | LFK |

| County | | | County | | | County | | | County | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 19 | Bowie | ATL | 83 | Frio | SAT | 147 | Limestone | WAC | 211 | Sherman | AMA |
| 20 | Brazoria | HOU | 84 | Gaines | LBB | 148 | Lipscomb | AMA | 212 | Smith | TYL |
| 21 | Brazos | BRY | 85 | Galveston | HOU | 149 | Live Oak | CRP | 213 | Somervell | FTW |
| 22 | Brewster | ELP | 86 | Garza | LBB | 150 | Llano | AUS | 214 | Starr | PHR |
| 23 | Briscoe | CHS | 87 | Gillespie | AUS | 151 | Loving | ODA | 215 | Stephens | BWD |
| 24 | Brooks | PHR | 88 | Glasscock | SJT | 152 | Lubbock | LBB | 216 | Sterling | SJT |
| 25 | Brown | BWD | 89 | Goliad | CRP | 153 | Lynn | LBB | 217 | Stonewall | ABL |
| 26 | Burleson | BRY | 90 | Gonzales | YKM | 154 | Madison | BRY | 218 | Sutton | SJT |
| 27 | Burnet | AUS | 91 | Gray | AMA | 155 | Marion | ATL | 219 | Swisher | LBB |
| 28 | Caldwell | AUS | 92 | Grayson | PAR | 156 | Martin | ODA | 220 | Tarrant | FTW |
| 29 | Calhoun | YKM | 93 | Gregg | TYL | 157 | Mason | AUS | 221 | Taylor | ABL |
| 30 | Callahan | ABL | 94 | Grimes | BRY | 158 | Matagorda | YKM | 222 | Terrell | ODA |
| 31 | Cameron | PHR | 95 | Guadalupe | SAT | 159 | Maverick | LRD | 223 | Terry | LBB |
| 32 | Camp | ATL | 96 | Hale | LBB | 160 | McCulloch | BWD | 224 | Throckmorton | WFS |
| 33 | Carson | AMA | 97 | Hall | CHS | 161 | McLennan | WAC | 225 | Titus | ATL |
| 34 | Cass | ATL | 98 | Hamilton | WAC | 162 | McMullen | SAT | 226 | Tom Green | SJT |
| 35 | Castro | LBB | 99 | Hansford | AMA | 163 | Medina | SAT | 227 | Travis | AUS |
| 36 | Chambers | BMT | 100 | Hardeman | CHS | 164 | Menard | SJT | 228 | Trinity | LFK |
| 37 | Cherokee | TYL | 101 | Hardin | BMT | 165 | Midland | ODA | 229 | Tyler | BMT |
| 38 | Childress | CHS | 102 | Harris | HOU | 166 | Milam | BRY | 230 | Upshur | ATL |
| 39 | Clay | WFS | 103 | Harrison | ATL | 167 | Mills | BWD | 231 | Upton | ODA |
| 40 | Cochran | LBB | 104 | Hartley | AMA | 168 | Mitchell | ABL | 232 | Uvalde | SAT |
| 41 | Coke | SJT | 105 | Haskell | ABL | 169 | Montague | WFS | 233 | Val Verde | LRD |
| 42 | Coleman | BWD | 106 | Hays | AUS | 170 | Montgomery | HOU | 234 | Van Zandt | TYL |
| 43 | Collin | DAL | 107 | Hemphill | AMA | 171 | Moore | AMA | 235 | Victoria | YKM |
| 44 | Collingsworth | CHS | 108 | Henderson | TYL | 172 | Morris | ATL | 236 | Walker | BRY |
| 45 | Colorado | YKM | 109 | Hidalgo | PHR | 173 | Motley | CHS | 237 | Walter | HOU |
| 46 | Corral | SAT | 110 | Hill | WAC | 174 | Nacogdoches | LFK | 238 | Ward | ODA |
| 47 | Comanche | BWD | 111 | Hockley | LBB | 175 | Navarro | DAL | 239 | Washington | BRY |
| 48 | Concho | SJT | 112 | Hood | FTW | 176 | Newton | BN1T | 240 | Webb | LRD |
| 49 | Cooke | WFS | 113 | Hopkins | PAR | 177 | Nolan | AB L | 241 | Wharton | YKM |
| 50 | Coryell | WAC | 114 | Houston | LFK | 178 | Nueces | CRP | 242 | Wheeler | CHS |
| 51 | Cottle | CHS | 115 | Howard | ABL | 179 | Ochiltree | AMA | 243 | Wichita | WFS |
| 52 | Crane | ODA | 116 | Hudspeth | ELP | 180 | Oldham | AMA | 244 | Wilbarger | WFS |
| 53 | Crockett | SJT | 117 | Hunt | PAR | 181 | Orange | BMT | 245 | Willacy | PHR |
| 54 | Crosby | LBB | 118 | Hutchinson | AMA | 182 | Palo Pinto | FTW | 246 | Williamson | AUS |
| 55 | Culberson | ELP | 119 | Irion | SJT | 183 | Panola | ATL | 247 | Wilson | SAT |
| 56 | Dallam | AMA | 120 | Jack | FTW | 184 | Parker | FTW | 248 | Winkler | ODA |
| 57 | Dallas | DAL | 121 | Jackson | YKM | 185 | Parmer | LBB | 249 | Wise | FPN |
| 58 | Damson | LBB | 122 | Jasper | BMT | 186 | Pecos | ODA | 250 | Wood | TYL |
| 59 | Deaf Smith | AMA | 123 | Jeff Davis | ELP | 187 | Polk | LFK | 251 | Yoakum | LBB |
| 60 | Delta | PAR | 124 | Jefferson | BMT | 188 | Potter | AMA | 252 | Young | WFS |
| 61 | Denton | DAL | 125 | Jim Hogg | PHR | 189 | Presidio | ELP | 253 | Zapata | PHR |
| 62 | De Witt | YKM | 126 | Jim Wells | CRP | 190 | Rains | PAR | 254 | Zavala | LRD |
| 63 | Dickens | CHS | 127 | Johnson | FTW | 191 | Randall | AMA | | | |
| 64 | Dimmit | LRD | 128 | Jones | ABL | 192 | Reagan | SJT | | | |

### 4.1.1 Truck Growth Analysis

The objective of this part of the study is to characterize truck growth by performing statistical analyses. The growth analyses are first carried out for each individual highway segment for all available years, i.e., the traffic volume in question corresponds to the same segment along the years. Additionally, in order to reflect traffic growth in the long term while minimizing the non-representative data points, only those segments with truck traffic records available in the earliest year (1986) and latest year (2002) were selected for the sample. Finally, the statistics for traffic growth from all of the segments along each individual highway are integrated to derive highway-specific traffic growth characteristics.

#### *4.1.1.1 Models for estimating traffic growth*

Traditionally, in pavement design, two basic approaches are used for traffic forecasting: the linear growth model and the compound (or exponential) growth model. The formula for the linear growth model is:

$$Y_i = a + bx = Y_0 \times (1 + r \times i) \tag{4.1}$$

where:

| | | |
|---|---|---|
| *a, b* | : | parameters of the linear model, |
| $x$ | : | dependent variable (usually time t), |
| $Y_i$ | : | the traffic volume in year i, |
| $Y_0$ | : | the traffic volume in the base year (usually denoted as i = 0), |
| $r$ | : | annual growth factor as a percentage of base year's traffic volume, and |
| $i$ | : | number of years from the base year. |

The formula for the compound growth model is:

$$Y_i = Y_0 \times (1 + r)^i \tag{4.2}$$

$$\log(Y_i) = \log(Y_0) + i \times \log(1 + r) \tag{4.3}$$

Both models can be estimated through the ordinary least square (OLS) technique. For Equation 4.1, the OLS is applied directly. For the compound model (Equation 4.2), after transformation, the OLS can also be applied to the transformed linear model, Equation 4.3. It is implied in Equation 4.1 that the annual growth factor varies with the different base years adopted. It is suggested that within the linear growth trend model, the change of traffic volume from year to year (i.e., growth rate) remains the same, represented by the parameter *b*, which, however, results in the change of growth factor *r* provided the base year shifts. The higher the traffic volume in the year selected as the base year, the lower the growth factor. Examples are given to demonstrate how the change of base year alters the value of the growth factor given by Equation 4.1. Unlike the linear growth model, the growth rate obtained from the compound model (Equation 4.2) is time-independent. Regardless of the base year, the annual growth factor remains constant for a given data set, while the growth rate varies.

In addition, comparing the two models, the projected traffic by the compound model will be larger in the long run than that projected by the linear model. In practice, for pavement designers, the adoption of either growth trend model depends on the properties of existing historical data and the level of understanding of traffic forecasting in the area.

*4.1.1.2 Growth estimates*

Traffic growth can be considered a dependent variable mainly influenced by the combined effect of factors such as industrial, agricultural, commercial, and regional economic development. For example, the rapidly growing industrial demand on raw materials in a region will boost truck volumes in the vicinity. Another example is that the Texas highway system has experienced increased truck traffic after the implementation of NAFTA. Moreover, the highways within the NAFTA corridor will probably experience higher traffic growth in the near future, as legislation regarding the movement of trucks at the border between Mexico and the U.S. is finalized. In this research, in order to identify traffic growth characteristics on highways across Texas, samples from a variety of typical highway facilities were evaluated, which included interstate, U.S. and state highways, and FM roads. Within each specific highway functional class of interest, data available through the TLOG database concerning traffic growth on the individual segments along Texas highways was thoroughly examined. The analysis process is presented in detail for IH 10. The investigation on the remaining highways follows the same procedure. The accompanying tables and figures illustrating truck traffic growth are presented in Appendix B and Appendix C.

1. Interstate Highways. As representatives of interstate highways in Texas, IH 10, IH 20, and IH 35 are now described:

    a. IH 10. The longest interstate highway in Texas, IH 10 is 879 miles in length within the state borders. It runs eastward from the Texas-New Mexico border northwest of El Paso via El Paso to the junction with IH 20, southwest of Pecos, to a junction with IH 35 in San Antonio, and from another junction with IH 35 in San Antonio via Houston to the Texas-Louisiana border near Orange. The Texas counties that IH 10 crosses include: El Paso, Hudspeth, Jeff Davis, Reeves, Pecos, Crockett, Sutton, Kimble, Kerr, Gillespie, Kendall, Bexar, Guadalupe, Seguin, Caldwell, Gonzales, Fayette, Colorado, Austin, Waller, Harris, Chambers, Jefferson, and Orange. Traffic growth statistics for IH 10 are shown in Table B1, Appendix B. The "Section #" column gives the order of sections along the highway eastbound. The second column presents the county number where the section is located. In order to show the difference in the linear growth factor due to different base years, the growth factors obtained by adopting the following base years are presented: Year 1 (1987), Year 8 (1994), and the most recent year, Y16 (2002). Growth factors obtained by the compound model are also listed in the table. The empty cells indicate a lack of statistics for those specific sections. Additionally, growth factor statistics are shown in Figure C1 (Appendix C). For the linear growth trend, it is shown that the growth factor varies with the shift of the base year. Basically, as the base year moves toward more recent years, the traffic increases. Thus, the growth factor decreases with the numerator (growth rate) fixed and denominator (AADTT in selected base year) increased. Therefore, it is more rational to provide the growth rate for pavement design when the linear model is applied to historical data in growth trend analysis. For the compound model, as previously discussed, the growth factor is time-independent, i.e., invariant with the base year. In the following process, the analysis focuses on the growth rate for the linear model and the growth factor for the compound model. Figures C2 and C3 (Appendix C) show the cumulative distribution functions (cdf) of the growth rate and growth factor, respectively. To better understand the growth trend of truck traffic as well as to accommodate the selection of traffic growth level for a particular highway, the statistic "percentile" is presented. A percentile is a value that indicates the percent of the sample that is equal to or below the given value. For example, if the 95th percentile of traffic growth factor is 5 percent, it means that 95 percent of the growth factors in the sample are smaller than 5 percent. The typical n-th percentiles as well as mean for growth rate (GR) and growth factor (GF) are shown in Table B2 of Appendix B.

b. IH 20. The length of IH 20 within Texas is 636 miles. It runs eastbound from its junction with IH 10 southwest of Pecos via Pecos, Fort Worth, and Dallas to the Texas-Louisiana border east of Marshall. The Texas counties that IH 20 runs through include: Reeves, Ward, Crane, Ector, Midland, Martin, Howard, Mitchell, Nolan, Taylor, Callahan, Eastland, Erath, Palo Pinto, Parker, Tarrant, Dallas, Kaufman, Van Zandt, Smith, Gregg and Harrison. The growth rate in the linear model and the growth factor of each section in the compound model are shown in Figures C4 and C5 of Appendix C. The direction along the highway is from west to east. The cumulative distributions for the growth rate and factor are presented in Figures C6 and C7 (Appendix C), respectively. In addition, Table B3 (Appendix B) displays the percentiles and means for the two parameters. Like IH 10, the mean of growth factor is close to the 50th percentile value, while the mean of the growth rate exceeds the 60th percentile value.

c. IH 35. IH 35 covers a distance of 407 miles within Texas. It runs northbound from the International Border at Laredo through San Antonio and Austin to the junction of IH 35W and IH 35E near Hillsboro, and from the junction of IH 35W and IH 35E near Denton to the Texas/Oklahoma border north of Gainesville. The Texas counties that IH 35 goes through include Webb, LaSalle, Frio, Medina, Atascosa, Bexar, Guadalupe, Comal, Hays, Travis, Williamson, Bell, Falls, McLennan, Hill, Denton, and Cooke. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C8 and C9 of Appendix C, respectively. In both of the figures, the direction along the highway is northbound. In addition, the cumulative distributions for growth rate and growth factor are shown in Figures C10 and C11 (Appendix C) with the respective percentiles and means displayed in Table B4 of Appendix B.

2. US Highways. As representative examples, truck traffic growth characteristics for US highways 59, 82, 281, and 290 are now presented:

   a. US 59. The length of US 59 within Texas is 612 miles. It begins at the Texas-Mexico border at Laredo, leading northeast and then north to the Texas-Arkansas state line. The Texas counties US 59 passes through are: Webb, Duval, McMullen, Live Oak, Bee, Goliad, Victoria, Jackson, Wharton, Ft. Bend, Harris, Montgomery, Liberty, San Jacinto, Polk, Angelina, Nacogdoches, Shelby, Panola, Harrison, Marion, Cass, and Bowie. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C12 and C13 (Appendix C), respectively. In both figures, the direction along US 59 is from south to the north/northeast. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C14 and C15 (Appendix C), respectively. The percentiles and means are displayed in Table B5 of Appendix B.

b. US 82. The length of US 82 within Texas is 505 miles. It runs from FM 769 at the New Mexico border northeastward to Plains, continuing eastward and concurrent with US 380 to Brownfield; then northeast and concurrent with US 62 through Lubbock to Ralls; thence eastward via Dickens, Guthrie, Benjamin to Seymour; thence northeast and concurrent with US 277 to Wichita Falls; thence eastward and concurrent with US 287 to Henrietta; thence eastward via Gainesville to FM 1417, northeast of Sherman; thence southward along FM 1417 to SH 56; continuing eastward and concurrent with SH 56 to Bonham; then continues eastward via Paris, Clarksville, De Kalb, and New Boston to US 67 in Texarkana at the Arkansas border. The Texas counties that US 82 passes through are: Yoakum, Terry, Hockley, Lubbock, Crosby, Dickens, King, Knox, Baylor, Archer, Wichita, Clay, Montague, Cooke, Grayson, Fannin, Lamar, Red River, and Bowie. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C16 and C17 (Appendix C), respectively. In both figures, the direction along US 82 is from south to the north/northeast. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C18 and C19 (Appendix C), respectively. The percentiles and means are displayed in Table B6 of Appendix B.

c. US 281. The length of US 281 in Texas is 582 miles. It begins at the Texas-Mexico border, running northward to the Texas-Oklahoma state line. The Texas counties that US 281 runs through are: Cameron, Hidalgo, Brooks, Jim Wells, Live Oak, Atascosa, Bexar, Comal, Blanco, Burnet, Lampasas, Coryell, Hamilton, Erath, Palo Pinto, Jack, Archer, and Wichita. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C20 and C21 (Appendix C), respectively. In both figures, the direction along US 281 is from south to north. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C22 and C23 (Appendix C), respectively. The percentiles and means are displayed in Table B7 of Appendix B.

d. US 290. The length of US 290 in Texas is 261 miles. It starts from IH 10 southeast of Junction, running eastward to Houston. The Texas counties US 290 passes through are: Kimble, Gillespie, Blanco, Hays, Travis, Bastrop, Lee, Fayette, Washington, Waller, and Harris. The growth rate of the linear model and growth factor of the compound model of each section are shown in Figures C24 and C25 (Appendix C), respectively. In both of the figures, the direction along US 290 is from west to east. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C26 and C27 of Appendix C. The percentiles and means are displayed in Table B8 of Appendix B.

3. State Highways. Two representative highways SH 16 and SH 71, are selected for the truck traffic growth analysis:

    a. SH 16. The length of SH 16 is 542 miles. It begins at Zapata at the Texas-Mexico border, running through San Antonio, then northward, terminating at US 281 south of Windthorst. The Texas counties SH 16 passes through are Zapata, Jim Hogg, Duval, McMullen, Atascosa, Bexar, Medina, Bandera, Kerr, Gillespie, Llano, San Saba, Mills, Comanche, Eastland, Palo Pinto, and Young. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C28 and C29 (Appendix C), respectively. In both figures, the direction along SH 16 is from south to north. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C30 and C31 (Appendix C), respectively. The percentiles and means are displayed in Table B9 of Appendix B.

    b. SH 71. SH 71 is 253 miles in length. It starts from approximately 1.5 miles east of Blessing, running northwestward, terminating at US 87 approximately 2 miles south of Brady. The Texas counties SH 71 passes through are Matagorda, Wharton, Colorado, Fayette, Travis, Blanco, Burnet, Llano, Mason, San Saba, and McCulloch. The growth rate for the linear model and growth factor for the compound model of each section are shown in Figures C32 and C33 (Appendix C), respectively. In both figures, the direction along SH 71 is west to east. In addition, the cumulative distributions for the growth rate and growth factor are shown in Figures C34 and C35 (Appendix C), respectively. The percentiles and means are displayed in Table B10 (Appendix B).

4. FM Road. Six FM roads were selected for the truck traffic growth analysis: FM 1329, FM 1450, FM 2088, FM 2111, FM 2222, and FM 2917. The designations for the six roads are shown in Table 4.3.

**Table 4.3: Designations of Six Selected FM Roads**

| FM Number | Length (miles) | Through Counties |
|---|---|---|
| 1329 | 33.808 | Bell and Williamson |
| 1450 | 44.227 | Reeves and Pecos |
| 2088 | 30.990 | Wood and Upshur |
| 2111 | 18.482 | Runnels |
| 2222 | 10.844 | Travis |
| 2917 | 9.397 | Brazoria |

### 4.1.2 Findings

Based on the traffic growth study of each individual highway described in the previous section, the findings are presented in this section for each highway group, as well as a comparison among them.

### *4.1.2.1 Interstate Highways*

Truck traffic growth rates and growth factors vary among sections of each individual highway. The growth rates and factors vary among highways, as well. Tables B1 and B4 (in Appendix B) show that the mean growth rate on IH 10 and IH 20 are the same (223 vehicles), but are less than half the growth rate on IH 35 (516 vehicles). Concerning the growth factor, IH 35 shows the largest value with 6.7 percent, followed by IIH 10 with 4.4 percent, and IH 20 with 3.7 percent, representing the lowest among the three highways. It is suggested that since the mid-1980s, IH 35 has experienced the fastest traffic growth. In addition, it is implied that as one of the NAFTA corridor arteries, IH 35 will carry heavier traffic volumes compared to IH 10 and IH 20.

By comparing the individual mean and 50th percentile (median) of the growth rate, Tables B1 to B4 of Appendix B show that the former is larger than the latter, whereas for the growth factor those two parameters are fairly close.

The lengths of the three interstate highways range from around 407 miles to 879 miles, and each passes through both rural and urban areas. By observing each highway, it can be concluded that when a highway passes through a metropolitan area (e.g., Houston, San Antonio, Austin, Dallas, and Fort Worth) the traffic growth factor is the lowest, followed by the traffic growth factor when a highway passes through a rural area. Where a highway bypasses a metropolitan area (thus running through a suburban area), the traffic growth factor is the largest (as can be seen where IH 20 bypasses Dallas). To illustrate the difference in traffic growth factors between rural and urban areas, a hypothesis test is conducted to compare IH 10 and IH 35. It is assumed that for each individual highway there is no significant difference between the mean growth factors on the rural and urban sections. The test process is described here:

Hypothesis test: $\quad H_0: \mu_R - \mu_U = 0 \qquad (4.4)$

$H_A: \mu_R - \mu_U > 0$

Statistic:

$$Z = \frac{\mu_R - \mu_U}{\sqrt{\frac{S_U^2}{n_U} + \frac{S_R^2}{n_R}}}$$

where

$H_0$ : null hypothesis;

$H_A$: alternative;

$Z$: statistic;

$\mu_R$: mean growth factor at rural areas;

$\mu_U$: mean growth factor at urban area;

$S_R$: standard error of sample growth factors at rural area;

$S_U$: standard error of sample growth factors at urban area;

$n_R$: sample size of growth factors at urban area;

$n_U$: sample size of growth factors at rural area.

In the case of IH 10, the Z statistic is 6.99. The null hypothesis is rejected because the one tail critical value with 95 percent confidence level is 1.645. It means that the mean growth factor where the highway passes through a rural area is significantly larger, with 95 percent confidence, than that of an urban area.

Similarly, in the case of IH 35, the Z statistic is 4.65. Hence, the null hypothesis is rejected in favor of the alternative, implying that the mean growth factor in a rural area is also significantly larger, with 95 percent confidence, than that of an urban area.

Therefore, for pavement design and rehabilitation of interstate highways, each highway should be treated individually in terms of traffic forecasting. In addition, different segments should also be considered accordingly. At a minimum, urban and rural areas should be considered separately.

#### *4.1.2.2 US Highways*

Tables B5 to B8 (Appendix B) show that the growth rate varies from 21 (US 82) to 153 (US 59) and the growth factor varies from 2.3 percent (US 82) to 6.0 percent (US 290). It is suggested that when planning highway design and rehabilitation, US highways should be treated individually in terms of traffic forecasting. In addition, the mean of the growth rate is larger than its median, while the mean of the growth factor is around its median.

#### *4.1.2.3 State Highways*

Tables B9 and B10 (Appendix B) show that the mean growth rates for both state highways are very close (21 and 22, respectively), while their growth factors differ. The mean growth factor for SH 16 is 5.8 percent, while that for SH 71 is less than half, 2.8 percent. Both mean growth rates are larger than their medians. The mean growth factor of SH 16 is the same as its median, while the growth factor of SH 71 is larger than its median.

#### *4.1.2.4 FM Roads*

Figures C36 through C47 (Appendix C) show that the growth rates vary significantly among the representative FM roadways. The growth rate on FM 2222 ranges from 50 to 80 vehicles, while the rates for the others are less than 10 vehicles. One possible reason for the growth rate on FM 2222 is that most of the roadway is within the urban/suburban area of Austin, which boosts traffic numbers. However, for those FM roads located in rural areas, the traffic growth rates are relatively small. The growth factor varies from around -5 percent to 10 percent.

By comparing the growth rate among the groups of highways, it can be seen that the interstate highways have the largest increase, followed by US highways, and then state highways. The FM roads have the lowest growth rate except for those, like FM 2222, that are located at or connecting with urban/suburban areas. The growth factor varies among these groups.

In summary, when performing traffic forecasting, because of the varying characteristics among the highway functional classes and along each individual highway, attention should be paid to their particular growth rates and growth factors. The tables and figures provided in Appendices B and C can be used by pavement design engineers to select the appropriate growth rate or factor depending on the roadway and the desired percentile.

## 4.2 Short-Term Traffic Volume Analysis: Distribution and Variation

In addition to the required traffic growth prediction models for pavement design, the M-E Design Guide necessitates traffic class distribution and seasonal variation, both of which play a vital role in the new design approach. In this section, these two issues are addressed, based on the full database (data continuously collected from 1998 to 2002) from a representative and reliable WIM station, D512 on IH 37 in Corpus Christi.

### 4.2.1 Traffic Class Distribution

As discussed in Chapter 2, the 15-vehicle category classification scheme is adopted by the M-E Design Guide, which includes truck Class 4 to Class 15. The percentage of each truck class within a particular truck flow in terms of volume from the traffic records collected during 1998 to 2002 at WIM station D512 is presented in Table 4.4 as well as Figure 4.3. It is shown that within a truck flow, Class 10, denoted as 3S2 or 18-wheel semi-trailer, accounts for the largest portion with about 56 percent of the truck volume, followed by Class 5, single unit truck, with about 25 percent. In addition, truck Class 4, Class 6, Class 9, Class 12, and Class15 account for a volume of about 3 percent to 4 percent each. The remaining truck classes account for only around 1 percent or less.

**Table 4.4: Truck Volume Percentage of Each Class**

| Truck Class | Percentage |
|---|---|
| 4 | 2.59% |
| 5 | 24.83% |
| 6 | 3.33% |
| 7 | 0.01% |
| 8 | 1.44% |
| 9 | 4.26% |
| 10 | 55.71% |
| 11 | 0.44% |
| 12 | 2.75% |
| 13 | 0.87% |
| 14 | 0.00% |
| 15 | 3.76% |

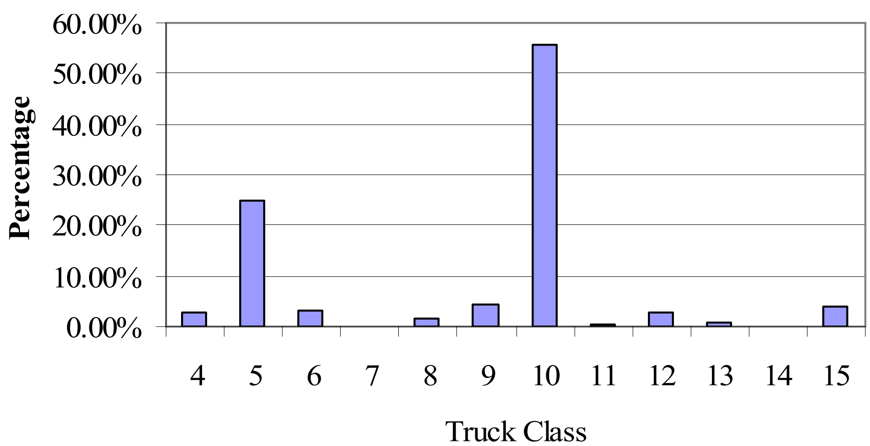


*Figure 4.3: Truck Volume Percentage of Each Class (1998-2002)*

Truck traffic percentages in each individual month by years are shown in detail in Figures D1 to D12 (Appendix D). In general, the percentages for individual truck classes in each month period have a similar pattern to that shown in Figure 4.3. However, the portion of each truck class varies within the same month over the years. A slight variation is observed for each truck percentage each month in a given year. Generally, no significant change of truck constitution is observed. It is implied that percentages for truck classes can be assumed as fixed in this case.

### 4.2.2 Average Number of Axles

The average number of axles for each axle set on each truck class is also a required input for the M-E Design Guide. By examining the data set from WIM station D512, the average number of axles for single axle (including steering axle and single axle with dual wheels), tandem axle, and tridem axle for each truck class is presented in Table 4.5. The case for quads is not shown here because of the relatively few records available. For a further breakdown of single axles, steering axles and single axles with dual wheels can be considered separately. By assuming that the average number of steering axles per truck is one, then the average number of single axles with dual wheels can be obtained by subtracting one from the average number of all single axles. From a structural point of view, the error introduced by this assumption is far less serious that the error that is made by assuming that single axles with single or dual wheels of the same load produce the same damage.

Although Table 4.5 is based on the data from WIM station D512, the results can be generalized to the other WIM stations because the number of axles for each particular truck class will not vary significantly from site to site.

**Table 4.5: Average Number of Axles**

| Class | Number of Axles | | |
|---|---|---|---|
| | Single | Tandem | Tridem |
| 4 | 1.41 | 0.59 | 0 |
| 5 | 2.06 | 0.07 | 0 |
| 6 | 1 | 1 | 0 |
| 7 | 1 | 0 | 1 |
| 8 | 3 | 0 | 0 |
| 9 | 2 | 1 | 0 |
| 10 | 1.11 | 1.94 | 0 |
| 11 | 1 | 1 | 0.99 |
| 12 | 5 | 0 | 0 |
| 13 | 3.85 | 1.04 | 0 |
| 14 | 3 | 2 | 0 |
| 15 | 2.75 | 0.59 | 0.06 |

### 4.2.3 Seasonal Variability

Seasonal traffic volume variation is significant for structural pavement design and rehabilitation. Pavement deterioration is the result of the joint impact of traffic and environment on pavement structures and materials. The modulus of asphalt materials changes an order of magnitude from very stiff in winter to very soft in summer. Traffic volume may exhibit a peak during the warmer months, due in part to the seasonal harvest. Consequently, rutting of the asphalt surface becomes a critical issue.

The monthly average daily truck traffic (MADTT) from 1999 to 2002 is presented in Figure 4.4. It can be seen that, except in year 2002, ADTT is between 3,000 and 4,000 with moderate fluctuation along each year, whereas the collected volume in 2002 ranged from 3,000 to 5,000 with more evident seasonal variation. By comparing the data from various years, Figure 4.4 also shows graphically the traffic growth at that specific station.

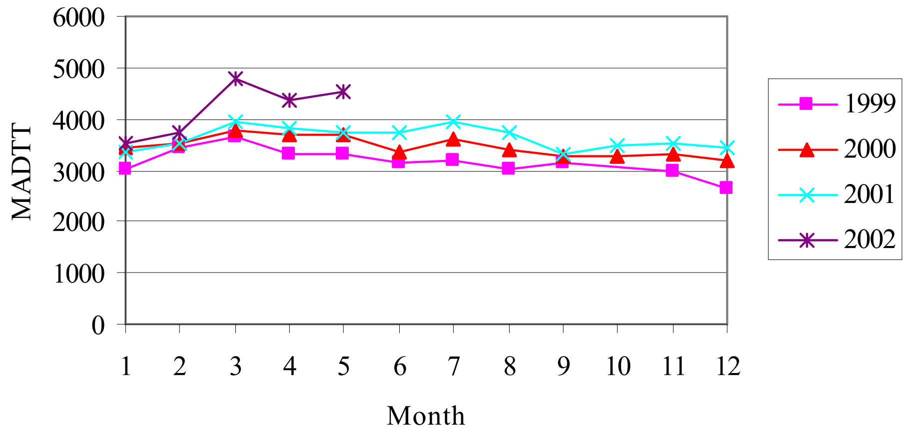


*Figure 4.4: ADTT Seasonal Fluctuation*

Table 4.6 and Figure 4.5 provide seasonal characteristics in terms of monthly fluctuation factors (MFF) from 1999 to 2001, which is defined as,

$$MFF = \frac{MADTT}{AADTT} \tag{4.5}$$

where

MADTT : monthly average daily truck traffic, and

AADTT : annual average daily truck traffic.

Generally, the monthly fluctuation factors range between 0.9 and 1.1, with the MFF greater than 1 during the period of February to July, and the remaining periods below 1. In addition, there are two seasonal peaks during each year, the highest peak occurring in March and a minor peak in July, with their MFFs well above the yearly average level, denoted as a unity. Moreover, traffic in March shows the highest MFF. In addition, the lowest MFF is found around December and January.

**Table 4.6: Monthly Fluctuation Factor of Truck Traffic**

| Month | 1999 | 2000 | 2001 |
|---|---|---|---|
| 1 | 0.956 | 0.990 | 0.928 |
| 2 | 1.088 | 1.019 | 0.975 |
| 3 | 1.151 | 1.085 | 1.087 |
| 4 | 1.045 | 1.066 | 1.052 |
| 5 | 1.041 | 1.067 | 1.031 |
| 6 | 0.998 | 0.973 | 1.032 |
| 7 | 1.003 | 1.046 | 1.083 |
| 8 | 0.959 | 0.982 | 1.033 |
| 9 | 0.989 | 0.943 | 0.913 |
| 10 | n.a. | 0.950 | 0.954 |
| 11 | 0.937 | 0.962 | 0.966 |
| 12 | 0.833 | 0.918 | 0.943 |

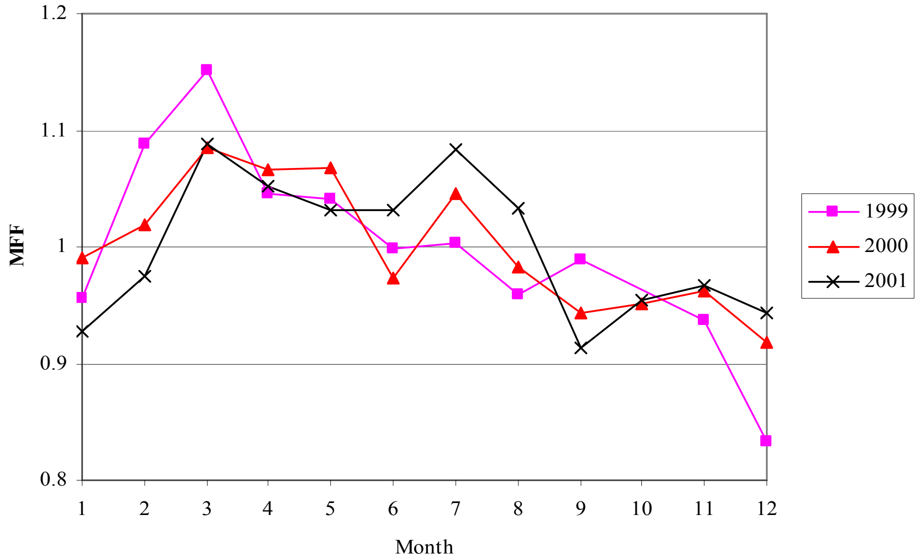


*Figure 4.5: Seasonal Fluctuation of Truck Traffic*

More details on seasonal traffic variations are studied based on each truck class. Figures E1 to E11 (Appendix E) present seasonal traffic fluctuations in terms of MFFs for each truck class. Each individual truck class exhibits significant seasonal variability. However, the variation patterns differ from each other among the various classes and also from the entire seasonal truck traffic pattern. For Class 4, the highest volume is in May, while the lowest is in January. For Class 5, two pronounced peaks are in April and July (which is the highest). Class 6 shows the largest variation among individual years. For Class 8 trucks, more frequent activity occurs at the beginning of each year, while inactivity is during the hot season, around August. Class 9 has a similar seasonal variation pattern to all of the truck classes. Class 10 trucks experience their highest volume in April, while July shows low activity. Class 11 trucks show the highest volume in March and April, except for the year 1999. Class 12 does not show significant seasonal fluctuation. Class 13 has different seasonal variation situations along different years. Class 15 shows the highest frequency in March and the lowest in September. Therefore, it can be concluded that addressing seasonal truck fluctuation characteristics based on each class is more rational than combining them all together. Alternatively, due to the relatively low frequencies of some classes, grouping should be considered.

In this section, a time series model is used to mathematically address the growth trend (long-term) and seasonal variation (short-term) of traffic volume. An additive decomposition time series model is adopted. Details can be found in Prozzi and Hong (2006). The model structure is denoted as follows:

$$z_t = T_t + S_t + \varepsilon_t \tag{4.6}$$

where

$z_t$ : time dependent variable, i.e., traffic volumes along months herein;

$T_t$ : trend component;

$S_t$ : seasonal component;

$\varepsilon_t$ : irregular (or error component);

t : time unit, such as in month.

As an example, by applying a trigonometric function to capture the seasonality, a fitted model for truck traffic volume forecasting is obtained as,

$$z_t = 3014.103 + 22.455t + 260.380\sin(2\pi t/12) \\ -223.407\cos(2\pi t/12) - 131.640\cos(4\pi t/12) + \varepsilon_t \tag{4.7}$$

The predictions obtained by applying the above model are presented in Figure 4.6, which shows that the model fits the observed data: both the growth trend and seasonal variation are precisely captured. With this model, traffic forecast can be obtained, as is represented by the dotted curve in Figure 4.6.

The modeling approach proposed herein can provide an effective and efficient way to incorporate both long-term growth trends and short-term seasonal variations into a single mathematical expression. This can be used to support the M-E Design Guide by facilitating traffic input and more efficient programming.

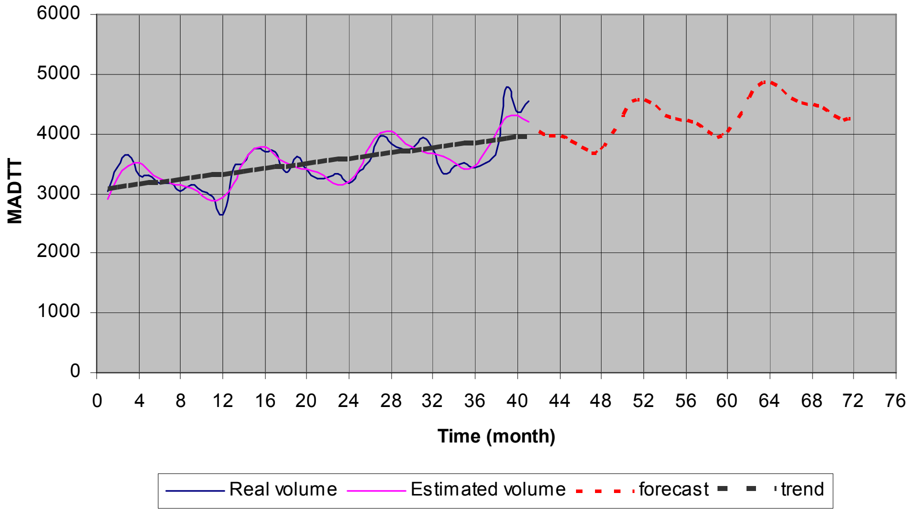


*Figure 4.6: Time Series Model to Address Growth Trend and Seasonal Variability*

### 4.2.4 Hourly Distribution

Truck traffic hourly distribution is one of the required inputs of the M-E Design Guide. Hourly distribution is defined as traffic percentage in each hourly interval within 1 day. With 1-day samples from each month during the period of 1998 to 2002, the truck hourly distribution is obtained, as shown in Table 4.7. Figure 4.7 shows truck hourly distribution as well as the variation (one standard deviation) in each time interval. It is indicated that 1:00 p.m. to 2:00 .p.m. is the truck traffic peak hour, whereas 2:00 a.m. to 3:00 a.m. is the interval with the lowest truck traffic volume for this specific site. The higher the hourly volume percentage is, the larger its variation.

**Table 4.7: Truck Traffic Hourly Distribution**

| Hour | (%) | Hour | (%) |
|---|---|---|---|
| 0-1 | 3.15 | 12-13 | 5.68 |
| 1-2 | 2.89 | 13-14 | 5.70 |
| 2-3 | 2.55 | 14-15 | 5.68 |
| 3-4 | 2.65 | 15-16 | 5.54 |
| 4-5 | 2.88 | 16-17 | 5.32 |
| 5-6 | 3.13 | 17-18 | 5.00 |
| 6-7 | 3.52 | 18-19 | 4.62 |
| 7-8 | 3.76 | 19-20 | 4.31 |
| 8-9 | 3.94 | 20-21 | 3.93 |
| 9-10 | 4.64 | 21-22 | 3.70 |
| 10-11 | 5.14 | 22-23 | 3.38 |
| 11-12 | 5.52 | 23-24 | 3.37 |

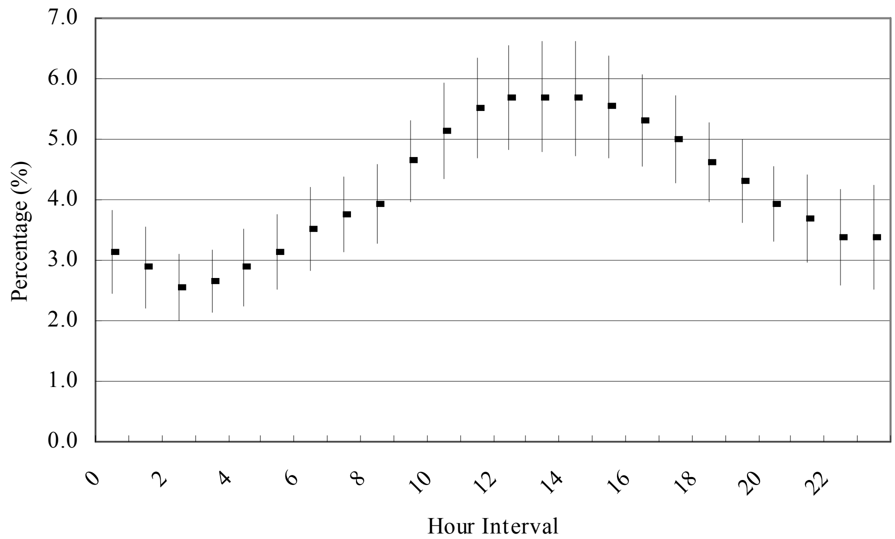


*Figure 4.7: Truck Traffic Hourly Distribution and Variation*

## 4.3 Continuous Characterization of Axle Load Distributions

There are two fundamental reasons for establishing well-accepted statistics for traffic axle load distributions. The first reason is to simplify and minimize traffic data processing and input into the M-E Design Guide without significant loss of accuracy and reliability. It has been shown that through the M-E Design Guide software, as many as thirty-nine parameters are needed for each individual single and tandem axle load distribution, respectively, and thirty-one parameters are needed for tridems and quads, respectively. This results in 140 combinations. Axle load distributions vary across the truck classes (e.g., ten classes of trucks by TMG 2001), which multiplies the number of parameters to be input into the Design Guide. Furthermore, each of the above combinations should be determined for each month of the year, resulting in approximately 12,000 parameters or more, depending on the traffic characteristics of the specific project. In this sense, a set of well-accepted statistics representing axle load distributions in a succinct way will simplify the implementation of the Design Guide. Hence, the objective of this section is to present the family of distributions that better capture actual axle load spectra.

The second reason for establishing sound statistics stems from the need to evaluate and compare different axle load distributions. In this sense, summary statistics (such as ESALs) will always be required. Discrete axle load distributions can be generated from collected WIM data. The varying loads in the distribution lead to different rates of damage to a pavement structure. In addition to the distribution, per se (i.e., data fit), evaluation of a given axle load distribution in terms of its damage potential to the pavement is a key issue for pavement design. Comparison among different axle load distributions is also critical because many projects may not have site-specific WIM and may need to "borrow" axle load distributions from adjacent sites. In such conditions, only those load distributions producing close or equal pavement damage should be adopted. A quantitative evaluation of load-related pavement damage based on sound statistics plays a meaningful role and is presented in the next section.

### 4.3.1 Moment-based Statistics

It was established through the AASHO Road Test (HRB, 1962) that the impact of each individual axle load on flexible pavement can be estimated according to the so-called fourth power law (AASHTO, 1993; Huang, 2003). The fourth power law implies that pavement damage by passing vehicles increases exponentially with the increase of their axle loads. This relationship is denoted by the Load Equivalence Factor (LEF):

$$LEF = \left( \frac{x_r}{L_s} \right)^m \tag{4.8}$$

where $x_r$ = axle load in the rth bin, (lbs), (assume that axle loads within each bin are identical); $L_s$ = load on a standard axle with the same number of axles as $x_r$; and m = power denoting the relative damage to the pavement of a given load $x_r$.

When the *LEF* is multiplied for the number of axles of that given load, the numbers of ESALs is obtained. Total ESAL is then obtained by determining the sum of the ESALs corresponding to each axle load of each axle type. As a result, the load-related pavement damage based on a given axle load spectrum can be obtained by summing the contributions from all the loads $x_r$'s in the distribution, denoted as load spectra factor (*LSF*), which is,

$$LSF = \sum_{r=1}^{R} \left( \frac{x_r}{L_s} \right)^m q_r \tag{4.9}$$

where R is the total number of load bins and $q_r$ is the normalized frequency of load in the rth bin.

In this sense, *LSF* represents the numbers of ESALs of a representative axle within the specific traffic stream.

It can be demonstrated that LSF is exactly the m-th sample moment statistic divided by a constant $L_s^m$. Thus, load-related pavement damage based on any axle load spectrum can be represented quantitatively by a moment-based statistic.

The study of load-related pavement damage is associated with varying values of power "m." It should be noted that the power m equal to 4 in the fourth power law was obtained in the AASHO Road Test based on pavement damage due to loss of serviceability. After that, a series of further studies, which involved the determination of the value of m, suggested that m vary over a wide range under different pavement types and distress conditions. Of particular interest for this study are the two major distresses in flexible pavements: fatigue cracking and surface rutting. Concerning fatigue cracking, Salam and Monismith (1972) implied the power could be 3.8 through their analysis of fatigue test results. The Asphalt Institute recommended that 3.291 be used (Huang, 2003). It was found by Pell and Cooper (1975) that the exponential power in their fatigue performance test could range from 2.5 to 6.3. With regard to rutting, however, it is believed that lower power values are more appropriate. For instance, in their pavement rutting model, Archilla and Madanat (2000) found that the power for the single axle is 2.98, while it is 3.89 for the tandem axle. Additionally, in the research utilizing the Canterbury Accelerate Pavement Testing Indoor Facility (CAPTIF), it was suggested in its compaction-wear model that the exponent value of the power law exhibited a variation between around 1.0 and 3.4 (Pont et al., 2002). Therefore, for comprehensive consideration of load-related pavement damage (i.e., LSF), all the alternatives associated with those powers should be considered.

### 4.3.2 Sensitivity Analysis to Moment Order

Because it is well established that varying power values can be applied to estimate load-related pavement damage, the sensitivity of LSF estimation to the variation of the power values is of interest in the context of pavement design. Particularly, the relationship between estimated damage based on different power values and the most used value 4 is examined in the following paragraphs. A series of hypothetical scenarios (referred to as load spectra level) under the conditions of increased axle load (represented by shifting the axle load distribution rightward) are incorporated to explore the sensitivity of load-related pavement damage to power values.

As an example, Figure 4.8 shows a typical single axle load spectrum and a new (hypothetical) load spectrum obtained by shifting all the loads with 1 kip rightward. Three other scenarios are adopted with shifting loads varying from 1 kip to 3 kips. As a result, the sensitivity of LSF to power values under each of the scenarios is illustrated in Figure 4.9. For all of the scenarios, LSF first decreases and then increases with the increase of power values. For a clearer understanding of the phenomenon, see the results presented in Figure 4.10 for tandem axle load spectra. Figure 4.11 summarizes the results for all three types of axles, with focus on the moment order at which minimum LSF occurs. For each type of axle, with the shifted load increasing, the moment order with minimum LSF decreases. For instance, for 0-kip shift level, i.e., load spectra from real world collected data, it is implied that adoption of the fourth power does not lead to conservative designs. Most importantly, contrary to general belief, increasing the value of the power does not necessarily result in the estimation of a higher number of ESALs. Thus, a sensitivity analysis to the power value is strongly recommended when designing pavement structures.

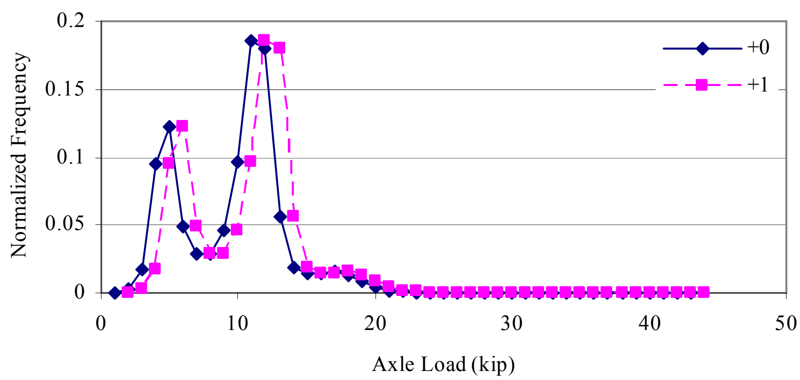


*Figure 4.8: Single Axle Load Spectrum and a New Spectrum Shifted 1-kip Rightward*

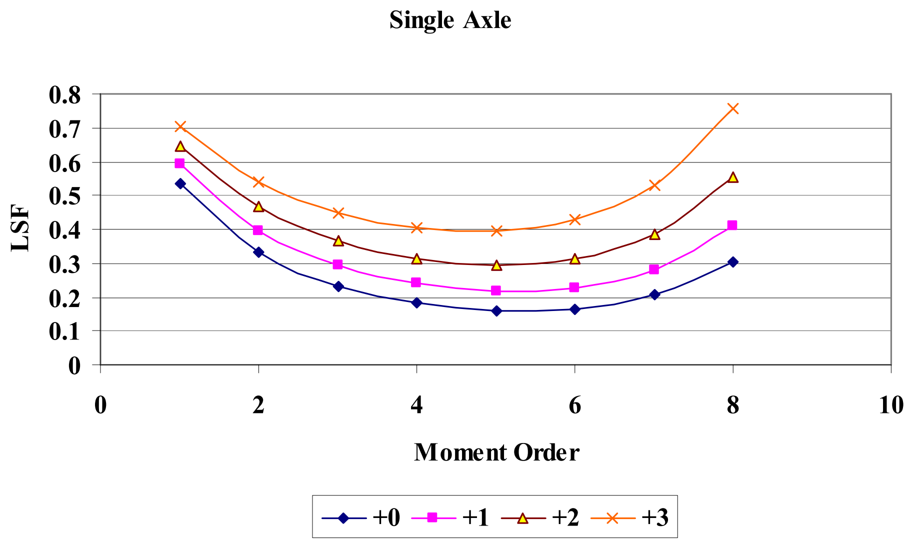


*Figure 4.9: Sensitivity of LSF to Power Value under Different Scenarios for Single Axle*

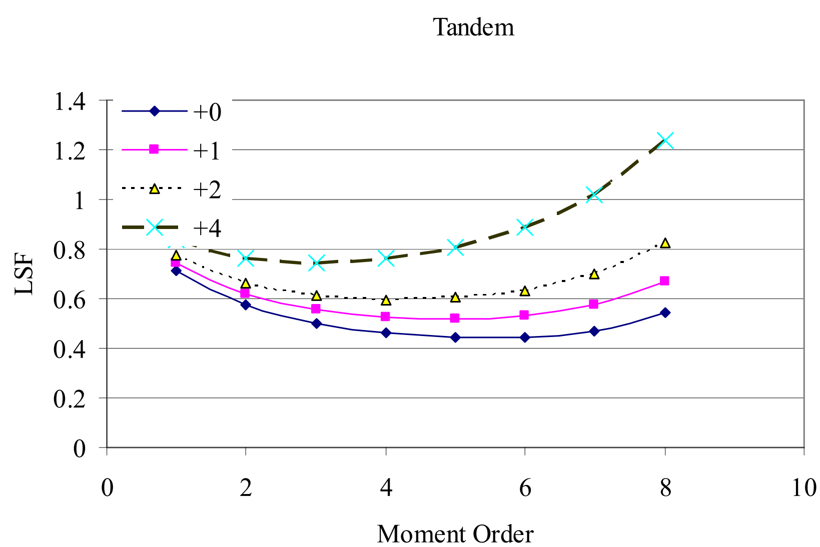


*Figure 4.10: Sensitivity of LSF to Power Value under Different Scenarios for Tandem Axle*

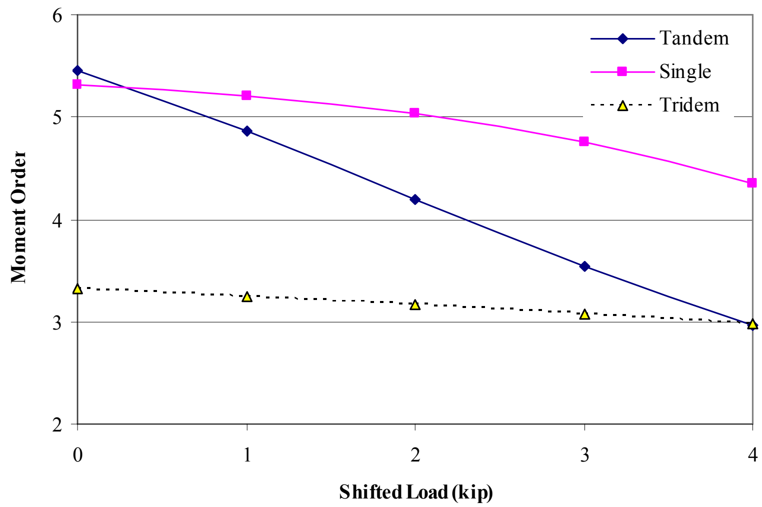


*Figure 4.11: Sensitivity of LSF to Moment Order of Single, Tandem, and Tridem Axles*

### 4.3.3 Axle Load Spectrum Fit Function and Relevant Statistics

After examining all of the axle load distributions for all the truck classes, it was found that mixed lognormal distributions are appropriate for fitting axle load spectra because the positive axle load can be represented by the lognormal explanatory variable, and the load frequency peak(s) can be captured by the various peaks of the mixed lognormal distribution (Prozzi and Hong, 2005; Prozzi et al., 2006).

As has been previously reported (Hallenbeck and Kim, 1993), axle load spectra with two pronounced peaks are commonly observed in the U.S. However, in the fit function, a third mode is introduced in this study as a means of improving the data fit. The spectra represented in Figure 4.12 exhibits three lognormal distributions and their combination to form the mixed lognormal fit function. In addition, the distribution functions can be described by Equation 4.10 as lognormal distribution, and Equation 4.11 as mixed lognormal distribution (DeGroot and Schervish, 2002). Strictly speaking, the axle load function is truncated on the heavy load tail side because axle loads extend within a finite weight range. Nevertheless, the difference between the truncated distribution and the non-truncated distribution, in terms of the heavy load "tail" side of the cumulative probability function, is proven to be very small and negligible for pavement design purposes (Prozzi et al., 2006). Hence, non-truncated distribution function is adopted directly in this study.

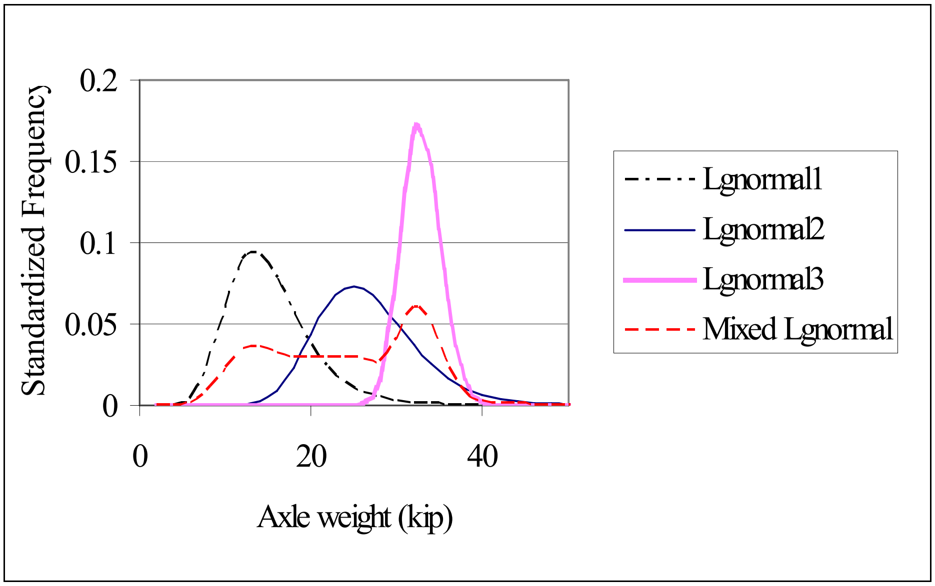


*Figure 4.12: Load Spectrum Function Illustration*

For each axle type on a given truck class (i.e., steering axle, single axle with dual wheels, and tandem axle), the parameters of the fit functions were determined, excluding Class 14 due to the nonexistence of traffic records within the available data, and Class 7 and Class 15 because of their irregularity of load distribution due in part to small sample size and measurement errors, respectively. The load distribution functions for tridem and quad axles are also not included here because of the very small sample size for the two axle groups.

Once the axle load distribution fit functions are obtained, what follows is the task of examining the goodness-of-fit of those functions. As examples, the original discrete load distributions and the corresponding fit functions for steering axle, single axle with dual wheels, and tandem axle on Class 10 are presented in Figures 4.13 to 4.15. It is shown that the empirical curves (obtained from fit functions) fit the real data very well. Alternatively, the coefficient of determination, $R^2$, can also be adopted as a criterion to measure the goodness-of-fit. Table 4.8 shows that all of the fit functions have very high $R^2$-values, almost equal to 1, which implies that the empirical functions fit the actual data extremely well. Therefore, the continuous functions with fitted parameters can be equally valid to represent axle load spectra and have the added important advantage that, at most, eight parameters are required, as opposed to the thirty-nine parameters needed for the discrete spectra proposed in the M-E Design Guide. The approach proposed in this study is neither better nor worse; it is just simpler and has the potential to significantly reduce data storage needs and computer running time.

$$X = \ln(\lambda, \zeta) \qquad X > 0$$

$$f(x; \lambda, \zeta) = \frac{1}{\sqrt{2\pi}x\zeta} e^{-\frac{1}{2}(\frac{\ln(x)-\lambda}{\zeta})^2} \tag{4.10}$$

$$f(x; \lambda_1, \zeta_1, \lambda_2, \zeta_2, \lambda_3, \zeta_3, W_1, W_2) = \frac{W_1}{\sqrt{2\pi}x\zeta_1} e^{-\frac{1}{2}(\frac{\ln(x)-\lambda_1}{\zeta_1})^2} + \frac{W_2}{\sqrt{2\pi}x\zeta_2} e^{-\frac{1}{2}(\frac{\ln(x)-\lambda_2}{\zeta_2})^2}$$
$$+ \frac{(1 - W_1 - W_2)}{\sqrt{2\pi}x\zeta_3} e^{-\frac{1}{2}(\frac{\ln(x)-\lambda_3}{\zeta_3})^2} \tag{4.11}$$

where

$x$ : axle load weight (kip),

$\lambda_i$, $\xi_i$ : parameters for lognormal distribution, and

$W_i$ : weight for the i$^{th}$ mode.

**Table 4.8: Parameters for Load Distribution Functions of Generalized Load Spectra**

| Axle set | Parameters | Class4 | Class5 | Class6 | Class8 | Class9 | Class10 | Class11 | Class12 | Class13 |
|---|---|---|---|---|---|---|---|---|---|---|
| Steering | W1 | 0.598 | 0.650 | 0.391 | 0.412 | 0.431 | 0.564 | 0.817 | 0.910 | 0.928 |
| | λ1 | 2.303 | 1.476 | 2.219 | 1.506 | 1.512 | 2.417 | 2.371 | 2.316 | 2.304 |
| | ξ1 | 0.243 | 0.120 | 0.406 | 0.165 | 0.153 | 0.089 | 0.124 | 0.126 | 0.123 |
| | λ2 | 2.504 | 1.939 | 2.356 | 2.137 | 2.236 | 2.310 | 2.331 | 1.621 | 2.085 |
| | ξ2 | 0.120 | 0.313 | 0.129 | 0.190 | 0.163 | 0.139 | 0.325 | 0.234 | 0.200 |
| | R2 | 0.999 | 1.000 | 0.998 | 0.992 | 0.998 | 0.999 | 0.999 | 0.998 | 0.999 |
| Single (Dual Wheels) | W1 | 0.036 | 0.505 | | 0.504 | 0.132 | 0.285 | | 0.381 | 0.305 |
| | λ1 | 1.592 | 1.349 | | 1.397 | 1.702 | 1.543 | | 2.098 | 1.868 |
| | ξ1 | 0.235 | 0.193 | | 0.581 | 0.131 | 0.273 | | 0.344 | 0.240 |
| | λ2 | 2.487 | 2.077 | | 2.417 | 2.217 | 2.533 | | 2.649 | 2.497 |
| | ξ2 | 0.185 | 0.520 | | 0.351 | 0.616 | 0.348 | | 0.222 | 0.241 |
| | W2 | | | | | | 0.343 | | | |
| | λ3 | | | | | | 2.849 | | | |
| | ξ3 | | | | | | 0.097 | | | |
| | R2 | 0.991 | 1.000 | | 0.991 | 0.981 | 0.993 | | 0.997 | 0.998 |
| Tandem | W1 | 1.000 | | 0.112 | | 0.076 | 0.445 | 0.263 | | 0.176 |
| | λ1 | 3.214 | | 2.261 | | 1.718 | 2.677 | 2.778 | | 2.533 |
| | ξ1 | 0.194 | | 0.143 | | 0.212 | 0.331 | 0.246 | | 0.176 |
| | W2 | 0.000 | | 0.514 | | 0.064 | 0.294 | 0.174 | | 0.000 |
| | λ2 | | | 2.576 | | 2.518 | 3.323 | 2.987 | | 2.990 |
| | ξ2 | | | 0.387 | | 0.150 | 0.211 | 0.150 | | 0.185 |
| | λ3 | | | 3.459 | | 2.539 | 3.504 | 3.432 | | |
| | ξ3 | | | 0.238 | | 0.523 | 0.074 | 0.211 | | |
| | R2 | 0.990 | | 0.998 | | 0.999 | 0.999 | 0.997 | | 0.999 |

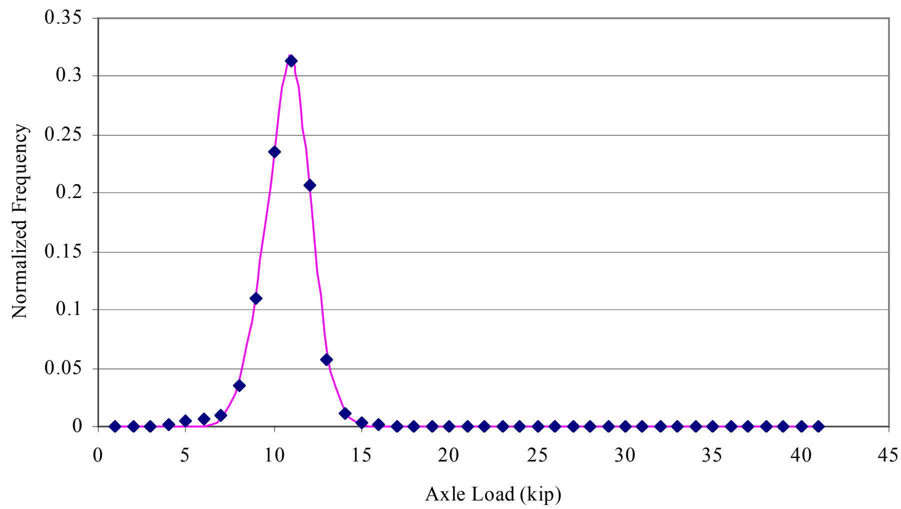


*Figure 4.13: Fit Function of Steering Axle Load on Class 10*

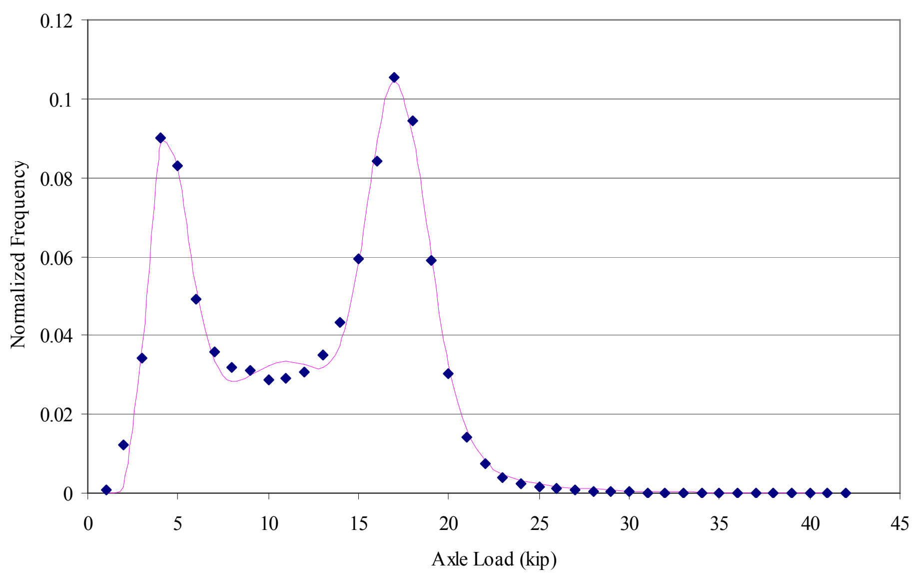


*Figure 4.14: Fit Function of Single Axle Load with Dual Wheels on Class 10*

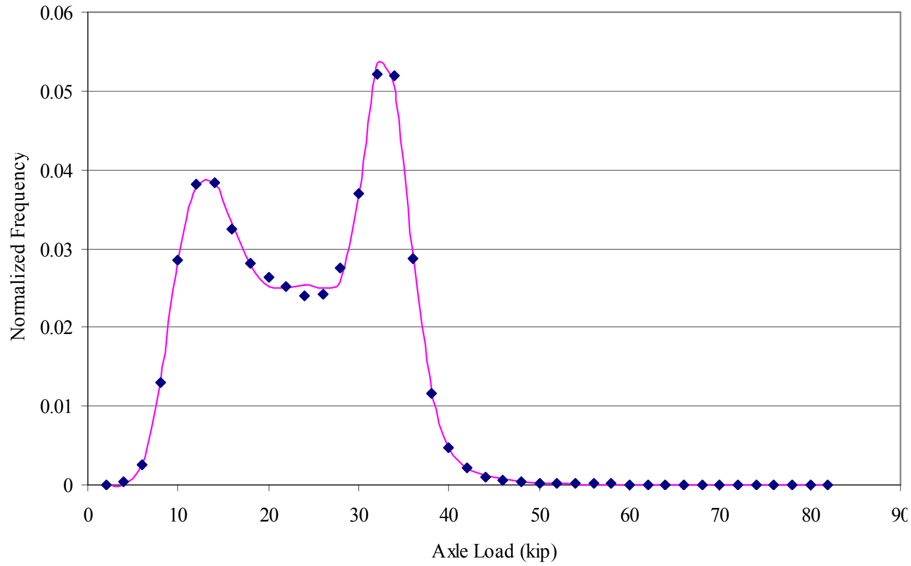


*Figure 4.15: Fit Function of Tandem Axle Load on Class 10*

The continuous distribution functions were based on the criterion of optimizing fit to the data. For pavement design, however, it is more convenient and equally valid to address the axle load-pavement impact by utilizing the population moment instead of the distributions, because it is the moments that are related to pavement damage, not the data. To this purpose, the summation part in Equation 4.9 can be re-expressed by its counterpart in the form of an integral with the width of each bin being sufficiently small. Thus:

$$LSF = \int \left( \frac{x}{L_s} \right)^m f(x)dx \tag{4.12}$$

$$LSF = \frac{\int x^m f(x)dx}{L_s^m} = \frac{E(X^m)}{L_s^m} = \frac{M^m}{C} \tag{4.13}$$

where x = axle load (kip); *f(x)* = axle load fit function for a given axle type; $L_s^m$ = constant, C; $M^m$ = m-th moment of a given probability density function (pdf).

The generalized m-th moment for a random variable X with lognormal distribution can be expressed as:

$$M^m = E(X^m) = \int x^m \frac{1}{\sqrt{2\pi}x\zeta} \exp\left( -\frac{1}{2}\left( \frac{Ln(x)-\lambda}{\zeta} \right)^2 \right) dx = \exp\left( m\lambda + \frac{m^2}{2}\zeta^2 \right) \tag{4.14}$$

With load spectra function fitted by mixing K lognormal distributions, the load-pavement impact by an axle type with given load spectrum, *LSF* is obtained as follows:

$$LSF = \sum_{k=1}^{K} W_k \exp\left( m\lambda_k + \frac{m^2}{2} \zeta_k^{\ 2} \right) \Big/ L_s^m \tag{4.15}$$

where $W_k$, $\lambda_k$ and $\zeta_k$, are as in Equation 4.11; m is the power.

Equation 4.15 shows that load-pavement impact based on axle load spectra under varying power conditions can be estimated provided the spectra fit functions are available (Hong et al., 2006). Therefore, the underlying approach provides an efficient and effective way to evaluate a specific axle load spectrum as well as quantitatively compare the load spectra obtained from different WIM sites.

# 5. Axle Load Spectra Specifications for Levels 2 and 3 (P3)

## 5.1 Background

The M-E Design Guide (developed under NCHRP 1-37A) requires the analysis of pavement performance based on the individual weights of the individual axle types. The four basic axle types proposed by the M-E Design Guide, according to their typical configurations, are single, tandem, tridem, and quad axles. It was recognized that through the M-E approach and by accounting for load-related pavement damage from the individual loads of each axle type, instead of the number of 18-kip equivalent axle loads, pavement performance could be estimated and forecast more accurately.

Regarding the loads for each type of axle, a series of load bins (ranges) is used to describe the axle load distribution (load spectrum). According to the M-E Design Guide, thirty-nine load bins ranging from 3 to 41 kips with an equal interval of 1 kip are used for single axles. Thirty-nine load bins are also required for tandem axles with a range from 6 to 82 kips, with an equal interval of 2 kips. Thirty-one load bins are utilized for both tridem and quad axles, ranging from 12 to 102 kips, with an equal interval of 3 kips.

Axle load spectra can be obtained from WIM data through the WIM system installed on the Texas highway network. For instance, for a given axle type, the number of axles corresponding to each load bin is the count of WIM-recorded loads falling into that bin interval. Axle load spectrum for any axle type is the normalized load distribution (the counts in each bin divided by the total counts). However, because of the limited number of WIM stations deployed in Texas (and most states), it is not possible to obtain the load spectra for many highway facilities. To some extent, this limitation also exists for the other factors in traffic input for pavement design, such as traffic volume. However, the number of vehicle classification systems and traffic counters is several times the number of WIM systems. Attempts to use classification and counts for estimating axle load spectra have been carried out with varied degree of success at the cost of significant assumptions (Papagiannakis and Jackson, 2006).

As was discussed earlier in this report, the M-E Design Guide proposes three levels of traffic input (Level 1, Level 2, and Level 3), mainly based on available resources and the significance of the underlying project. These levels represent the hierarchical accuracy of traffic information for the pavement being designed (NCHRP, 2005). The three levels are:

1. Level 1, which represents the highest accuracy level and includes very good knowledge of past and future traffic characteristics. This level should be applied to heavily trafficked pavements where early failures may cause important safety or economic consequences. Site-specific traffic volume and weight information is required to fulfill Level 1 design.

2. Level 2, which is the intermediate level and is consistent with the current version of the pavement design guide (AASHTO, 1993). This level should be applied when the resources necessary for Level 1 are not available, and represents a modest knowledge of past and future traffic characteristics. Level 2 requires site-specific traffic volume data, while regional axle load spectra are used to accommodate the pavement design.

3. Level 3, which offers the lowest level of accuracy for pavement design, represents the lowest level of knowledge of the traffic characteristics. Traffic input variables consist of

default values or averages for the particular state. Axle load spectra correspond to the state average of available WIM data. An evaluation of traffic characteristics, based on local experience, should also be considered in this level.

Currently, there are approximately twenty WIM stations deployed on the highway system in Texas. The number of stations in operation at a given time changes continuously because of pavement maintenance and rehabilitation activities, as well as equipment functionality. The majority of WIM stations are located on interstate highway facilities and the remaining ones are on the U.S. and state highway networks. Consequently, Level 1 design is restricted to the approximately twenty sites available because only these sites can provide the site-specific axle load spectra data. Levels 2 and 3 axle load spectra input need to be established based on the load data from the existing WIM stations complemented by more localized data provided by traffic counters and classifiers. In this regard, one of the major tasks of this research study was to provide the specifications for Level 2 (or regional) and Level 3 (or state default) load spectra to better characterize traffic loading in order to support the design of pavement structures in Texas.

Based on a thorough investigation of the M-E Design Guide and sound statistical analyses of the traffic data collected in Texas, the study has developed and proposed Level 2 and Level 3 load spectra input for Texas conditions. There are, however, some differences between the recommendations of the M-E Design Guide and the approach followed in this research study, which are discussed in the next paragraph.

Similarly, as in the M-E Design Guide, the assumptions on axle load distribution used for pavement analysis are: 1) the axle load spectrum for each axle type remains constant along the pavement design life. Axle load spectra changes caused by reasons such as political, legislative, or economic changes are beyond the scope of this research; and 2) the axle load spectra are assumed not to vary across the days of the week or months of the year. The types of variations, such as seasonal fluctuations and yearly growth, are captured through truck traffic volumes.

In this research study, there are some differences from the M-E Design Guide axle load recommended input. Considering the different load-associated pavement damages caused by single axles with single wheels and single axles with dual wheels, this study separated them as two different types of axles (albeit both belong to the single axle type in the M-E Design Guide). In addition, the aggregate axle load spectra of each axle type aforementioned for all truck classes are established, instead of for each truck class respectively. The main reasons for this consideration are:

1. Within the applicability of linear elasticity, only the axle/wheel loads are of interest for structural pavement design. The same magnitude of load of a specific axle type from a different vehicle class will be predicted to generate the same damage on a given pavement with a given set of environmental conditions.

2. Different vehicle classification schemes may be applied in a particular state so as to differ from the FHWA's thirteen-class scheme, which is adopted in the M-E Design Guide traffic input module. In Texas, traffic is recorded and reported on a fifteen-class basis in some WIM systems (e.g., the bending plate WIM with PAT system vehicle classification used in this report).

3. When performing volume forecasting, because of the limited sample size of some of the vehicle classes, the class-based traffic forecast brings more uncertainty than that from the

aggregate truck volumes. Alternatively, truck classes can be grouped into a smaller numbers of categories based on their significance to pavement design. For example, one such grouping could be the following four categories: Class 4 (busses), Class 5 (2-axle trucks), Class 10 (5-axle trucks or 18-wheelers), and all others.

4. Existing class-based traffic records are usually of relatively short duration. Hence, instead of adopting these class-based historical data, the relatively long duration truck traffic data (such as TLOG, which has more than 15 years of AADTT information), can be used to provide more reliable long-term traffic forecasts for pavement design.

As was shown earlier, in the M-E Design Guide, for each axle type, as many as thirty-nine parameters (normalized frequencies for the individual load bins) for single and tandem axles and thirty-one factors for tridem and quad axles are required for the pavement design engineers to conduct pavement analysis. This implies that a large amount of storage space and effort to input these parameters is needed. In addition, the statistical characteristics of axle load distributions are not easily and effectively reflected by the discrete data set, which causes difficulty in understanding traffic loading characteristics. These factors lead to challenges for pavement designers to adjust axle load spectra based on their knowledge and experience when it is necessary, as is common practice in most states. To address these issues, based on sound statistical analyses, relevant parameters for axle load spectrum functions of each individual axle type are obtained. Through these analyses, axle load spectra for Levels 2 and 3 pavement designs in Texas have been established in an effective and efficient manner.

## 5.2 Specification for Level 2 Axle Load Spectra (Regional)

Level 2 is used to design pavement for highways of importance with high traffic volumes when site-specific information is not available. This level roughly corresponds to that found in the current version of the design guide (AASHTO, 1993). To develop the axle load spectra for this level is of critical importance. Because Texas is one of the largest states in the U.S., axle load spectra can differ from region to region because of varying economic activities across the regions. In addition, load characterizations differ among highway facilities and locations. Hence, within this research study, the objective of developing Level 2 traffic data was to characterize and capture differences among highway functional groups in the individual regions so that the load characterization within those groups in each region is reasonably similar. When determining the regions, economic activities (mainly including industrial and agricultural factors) and environmental conditions were used as the predominant criteria. According to the recommendation by the Strategic Traffic Analysis and Reporting System (STARS), eight regions were established, which are:

1. Panhandle (PH),

2. West (WE),

3. North IH 35 Corridor (NI35),

4. Central Texas (CT),

5. South IH 35 Corridor (SI35),

6. Piney Woods (PW),
7. South Coastal (SC), and
8. North Coastal (NC).

The geographical region delineation is illustrated in Figure 5.1. Considering that there are no WIM systems available in Region 7 (North Coastal), it was assumed that the two coastal regions share similar traffic characterizations and Regions 7 and 8 were combined.

Regarding highway facility types, based on the available WIM data resources, most being on interstate highways and some on U.S. and state highways, two highway groups are proposed: interstate highways and non-interstate highways. However, this proposal does not mean that it is the researchers' opinion that all non-interstate facilities have similar traffic characteristics but instead reflects the limitations on the available data. In reality, the research team supports the idea that there are significant differences among non-interstate facilities, especially among FM and RM road types and the U.S. and state highway system.

For the interstate highway system, regional axle load spectra can be obtained from WIM stations in the individual regions. However, because only Regions 3 and 7 have WIM stations on their non-interstate highway systems, the axle load spectra for the non-interstate highways are estimated based on the existing information from other facilities or regions.

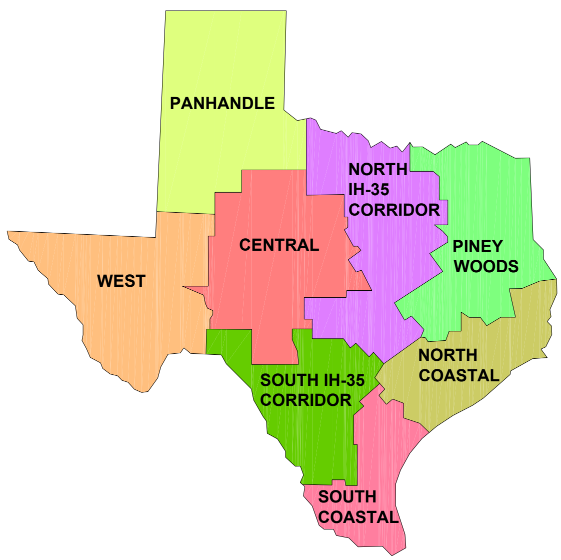


*Figure 5.1: Regions Used for Level 2 Axle Load Spectra Input in Texas*

### 5.2.1 Development of Level 2 Axle Load Spectra

Through integrating load weight data for each axle type for each highway group in an individual region, the regional axle load spectra were obtained in terms of normalized frequencies. It was found that all of the axle load distributions share the commonality of being multi-modal. By means of statistical analyses, the mixed lognormal distribution was established to effectively describe the axle load spectra (for details of characteristics of load spectrum function, see Chapter 4). The formulae for the mixed lognormal distribution are:

$$f(x) = W_1 \frac{1}{\sqrt{2\pi} x\sigma_1} e^{-\frac{1}{2}(\frac{\ln(x)-u_1}{\sigma_1})^2} + W_2 \frac{1}{\sqrt{2\pi} x\sigma_2} e^{-\frac{1}{2}(\frac{\ln(x)-u_2}{\sigma_2})^2} + (W_1 + W_2 + W_3) \frac{1}{\sqrt{2\pi} x\sigma_3} e^{-\frac{1}{2}(\frac{\ln(x)-u_3}{\sigma_3})^2} \quad (5.1)$$

where $\mu_k$ and $\sigma_k$ are the parameters for the lognormal distribution (k represents the kth mode); $W_k$ represents the weight of the kth mode ($W_1 + W_2 + W_3 = 1$).

The advantages of adopting mixed lognormal distribution are

1. it captures the basic distribution characterization of the loads, e.g., the peak location is directly reflected by the exponential of mean values; and

2. the load-related pavement damage in terms of load spectra can be easily accounted for through the succinct parameter sets, as will be shown later.

Because axle load spectra vary significantly across different regions in Texas, it is critical for pavement designers to understand the fundamental load spectra characteristics. After comprehensively examining the axle load spectra of each axle type across the various regions, their typical representatives were obtained, as illustrated in the following paragraphs. The axle load spectrum for quad axle is not presented here because of the large uncertainty resulting from the small sample size.

For steering axles, axle load spectra featured two pronounced peaks (or modes). However, the relationship between the two peak heights differs, leading to two typical load spectra. Type I-SS axle load spectrum shows almost the same peak heights (Figure 5.2) and Type II-SS axle load spectrum shows its right peak significantly higher than the left (Figure 5.3). It is implied that Type II-SS load spectrum is composed of a larger proportion of heavier loads than Type I-SS; therefore, it is more damaging to the pavement structure.

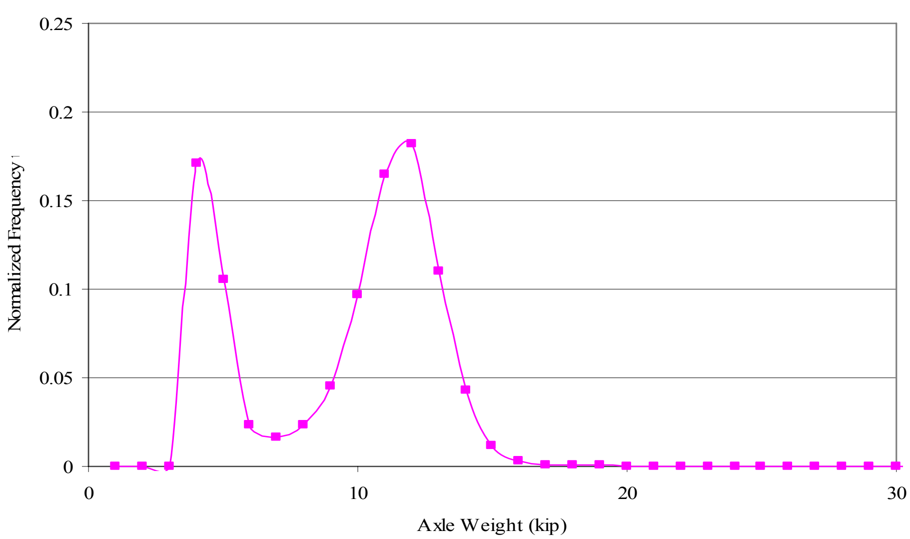


*Figure 5.2: Typical Steering Axle Load Spectrum (Type I-SS)*

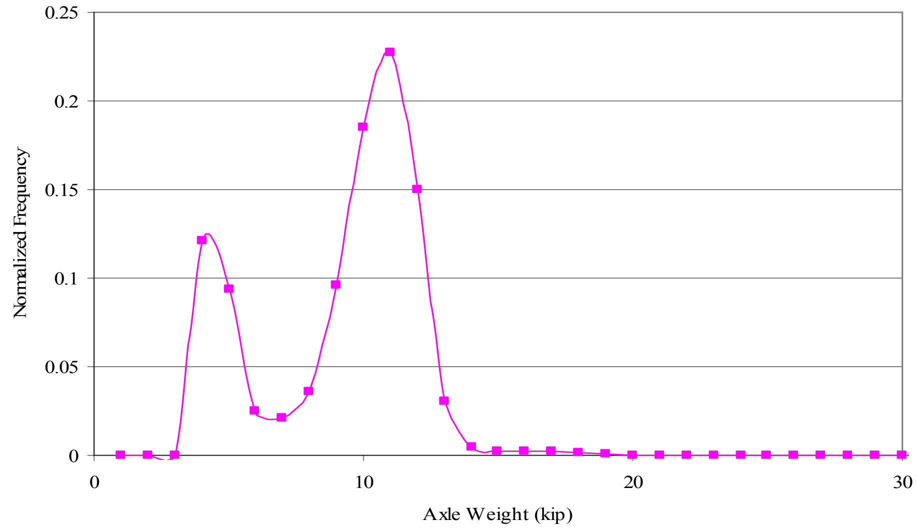


*Figure 5.3: Typical Steering Axle Load Spectrum (Type II-SS)*

For single axle with dual wheels, there are two typical axle load spectra, both with significantly higher left peaks (Figures 5.4 and 5.5). The difference between the two types of load spectra is that the right peak of Type II-SD load spectrum is more evident than that of Type I-SD. This suggests that, compared to the Type II-SD axle load spectrum, the relatively light loads account for the higher portion in the Type I-SD axle load spectrum, thus creating less damage.

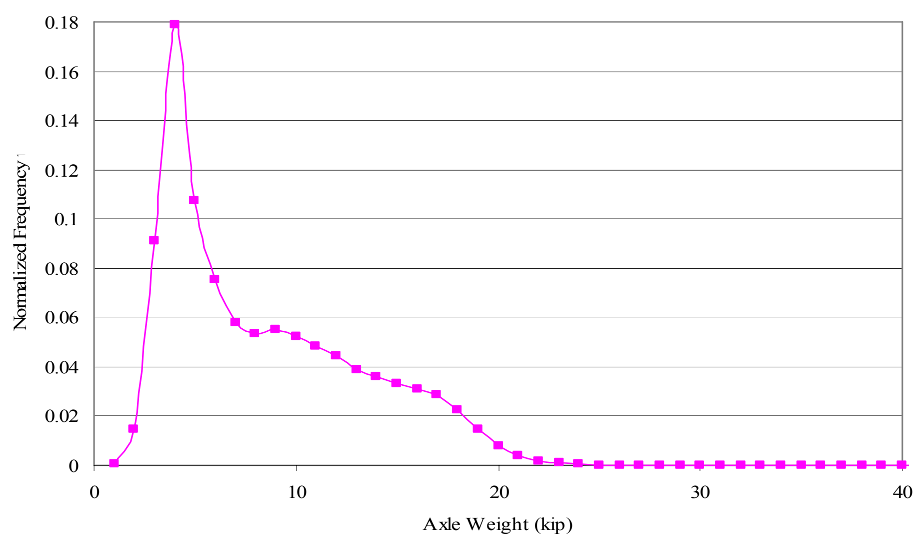


*Figure 5.4: Typical Single Axle (with Dual Wheels) Load Spectrum (Type I-SD)*

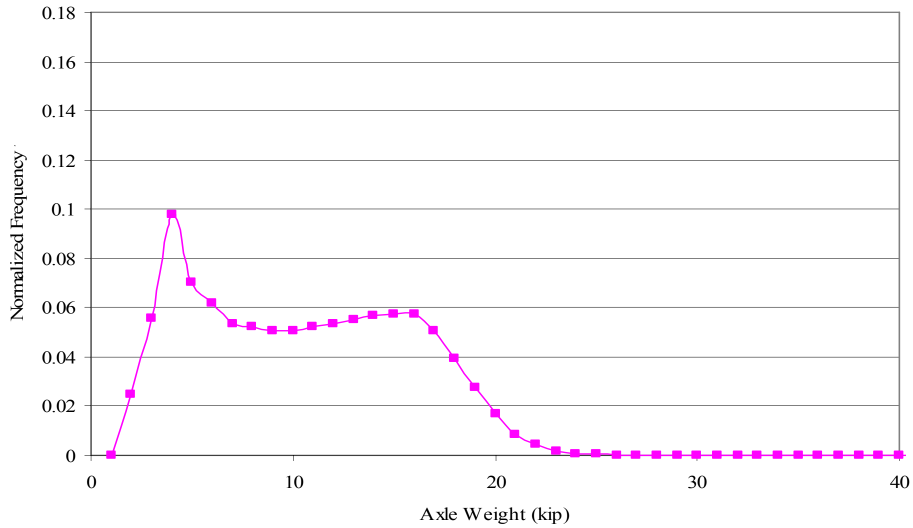


*Figure 5.5: Typical Single Axle (with Dual Wheels) Load Spectrum (Type II-SD)*

For tandem axle, the load spectra can be categorized into three typical representative groups, denoted as Type I-TA, II-TA, and III-TA, respectively. Figure 5.6 shows Type I-TA tandem axle load spectrum, featuring a left higher peak. Figure 5.8 shows Type III-TA tandem axle load spectrum, featuring a higher right peak. In between Types I-TA and III-TA, Type II-TA represents the tandem axle load spectrum with the two close-height peaks, as shown in Figure 5.7. It is implied that from Type I-TA to III-TA, the portion of relatively heavy load increases.

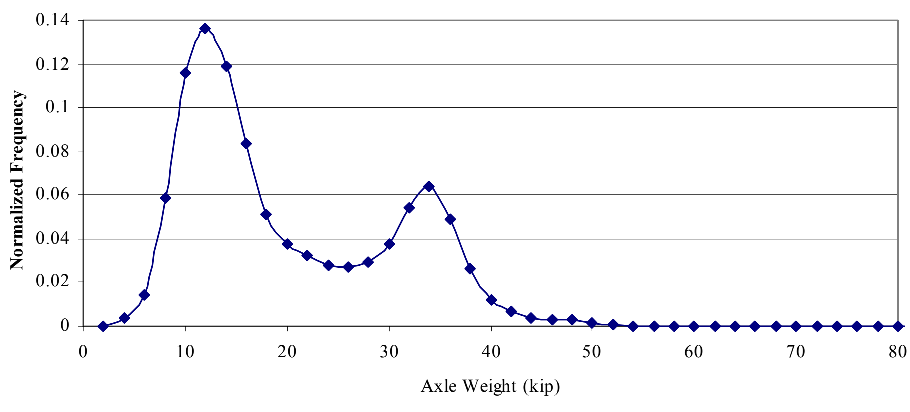


*Figure 5.6: Typical Tandem Axle Load Spectrum (Type I-TA)*

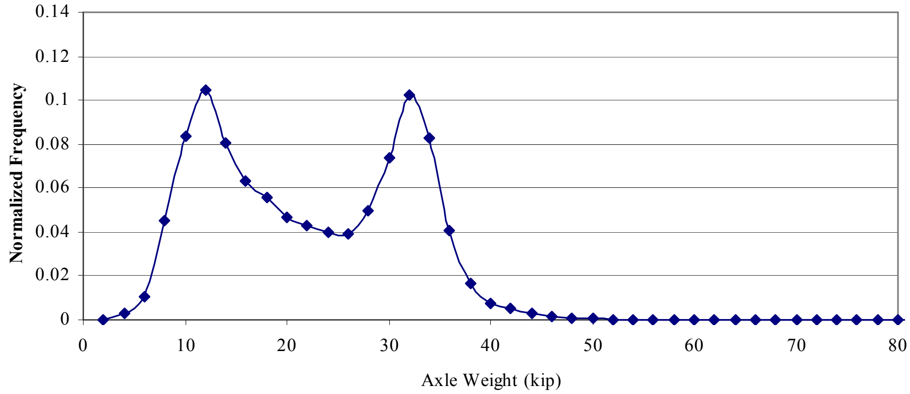


*Figure 5.7: Typical Tandem Axle Load Spectrum (Type II-TA)*

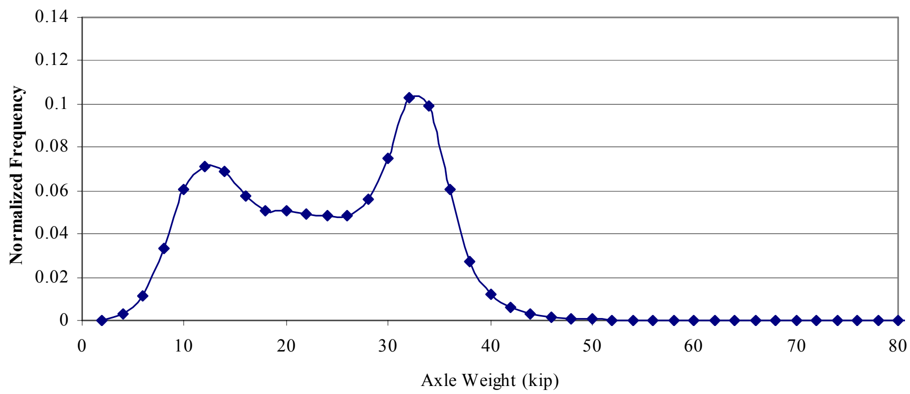


*Figure 5.8: Typical Tandem Axle Load Spectrum (Type III-TA)*

Compared with the previous three axle types, tridem axles were found to have less variation of load spectrum type. This may be due to the reduced number of tridem axles in Texas. The typical tridem axle load spectrum is shown in Figure 5.9, which illustrates that there is one pronounced peak on the relatively light load side, while the remaining peak(s) are less significant and lower in height. In addition, the load spectrum possesses a long tail on the heavier load side, which suggests the high variation of axle loads.

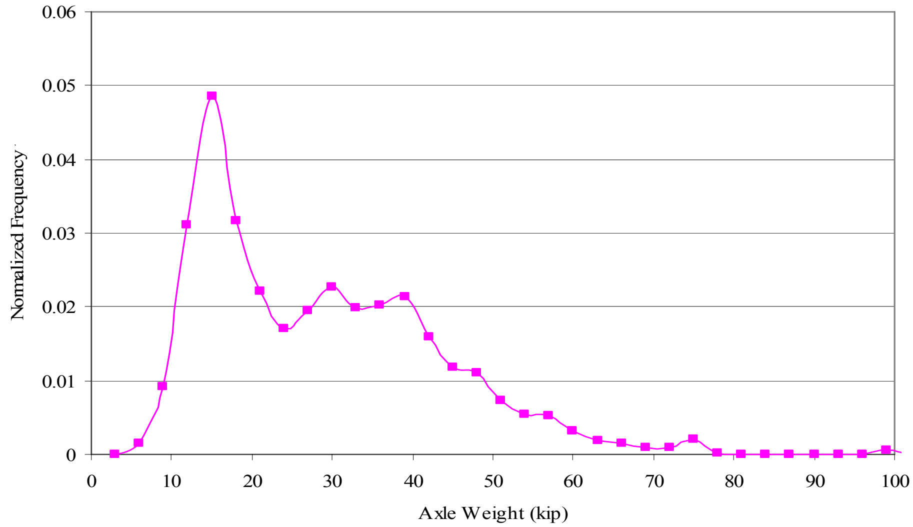


*Figure 5.9: Typical Tridem Axle Load Spectrum*

### 5.2.2 Specifications for Level 2 Axle Load Spectra

*5.2.2.1 Steering axle*

After fitting the normalized axle load frequencies with mixed lognormal distribution functions, it was observed that the data fit extremely well, with $R^2$ higher than 0.99. The statistics of the mixed lognormal distribution for the steering axle load spectra at each region are presented in Table 5.1. By applying load spectrum functions, the normalized frequencies can be easily obtained. For example, to obtain the normalized frequency corresponding to the load bin of 10 kips, one only needs to enter x = 10 into Equation 5.1 with the parameters in Table 5.1 for each region.

**Table 5.1: Parameters for Steering Axle Load Spectra of Level 2 Input**

| Facility | Region | $W_1$ | $W_2$ | $W_3$ | $\mu_1$ | $\mu_2$ | $\mu_3$ | $\sigma_1$ | $\sigma_2$ | $\sigma_3$ | LSF* |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Interstate | 1 | 0.288 | 0.165 | 0.547 | 1.477 | 2.245 | 2.465 | 0.134 | 0.237 | 0.110 | 0.131 |
| | 2 | 0.179 | 0.28 | 0.541 | 1.482 | 2.275 | 2.425 | 0.160 | 0.188 | 0.090 | 0.122 |
| | 3 | 0.203 | 0.279 | 0.518 | 1.494 | 2.248 | 2.399 | 0.165 | 0.168 | 0.095 | 0.106 |
| | 4 | 0.175 | 0.235 | 0.590 | 1.481 | 2.296 | 2.440 | 0.155 | 0.145 | 0.088 | 0.130 |
| | 5 | 0.197 | 0.357 | 0.446 | 1.511 | 2.292 | 2.402 | 0.178 | 0.168 | 0.091 | 0.109 |
| | 6 | 0.138 | 0.344 | 0.518 | 1.496 | 2.230 | 2.361 | 0.175 | 0.163 | 0.098 | 0.098 |
| | 7 | 0.248 | 0.308 | 0.444 | 1.501 | 2.275 | 2.411 | 0.162 | 0.139 | 0.089 | 0.101 |
| Non-Interstate | 3 | 0.217 | 0.198 | 0.585 | 1.491 | 2.238 | 2.411 | 0.152 | 0.173 | 0.084 | 0.110 |
| | 7 | 0.151 | 0.322 | 0.527 | 1.484 | 2.094 | 2.362 | 0.085 | 0.383 | 0.132 | 0.117 |

*LSF, the abbreviation of Load Spectrum Factor, is an index used to evaluate load-related pavement damage by each "unit" axle load spectrum according to the "4th power law." The larger the LSF, the more significant the load pavement damage on pavement. For the load spectrum with mixed lognormal distribution, the LSF is

$$\sum_k W_k \exp\left(8\left(\mu_k{}^2\right)+4\sigma_k\right)/L_s{}^4$$

where Ls is the standard axle load, i.e., 18 kip for single axle. For details, see Chapter 4.

As indicated by the LSF in Table 5.1, the load-associated pavement damage in terms of axle load spectrum in West Texas (including Regions 1, 2, and 4) is larger than that for the rest of the state. To obtain the estimation of axle load spectra for a non-interstate highway in the regions where there are no WIM data for the non-interstate highway, it is suggested that the weights for the two major peaks ($W_1$ and $W_3$) be adjusted ($W_2$ remains fixed) based on the axle load spectra on the non-interstate highways in Region 3 or 7. For the West Texas regions, the two weights are adjusted so that the LSF will be larger, while for the remaining regions, the LSF will be smaller. In order for pavement designers to better understand the effect of weight adjustment on the estimation of load pavement damage, a sensitivity analysis is provided to show how sensitive the LSF is to the change of $W_1$, as shown in Figure 5.10. "R" represents "Region" in the legend. Figure 5.10 implies that as $W_1$ increases, the load-associated pavement damage to that particular distribution decreases. This is natural because a larger $W_1$ implies a larger proportion of lighter axle loads in the specific spectrum.

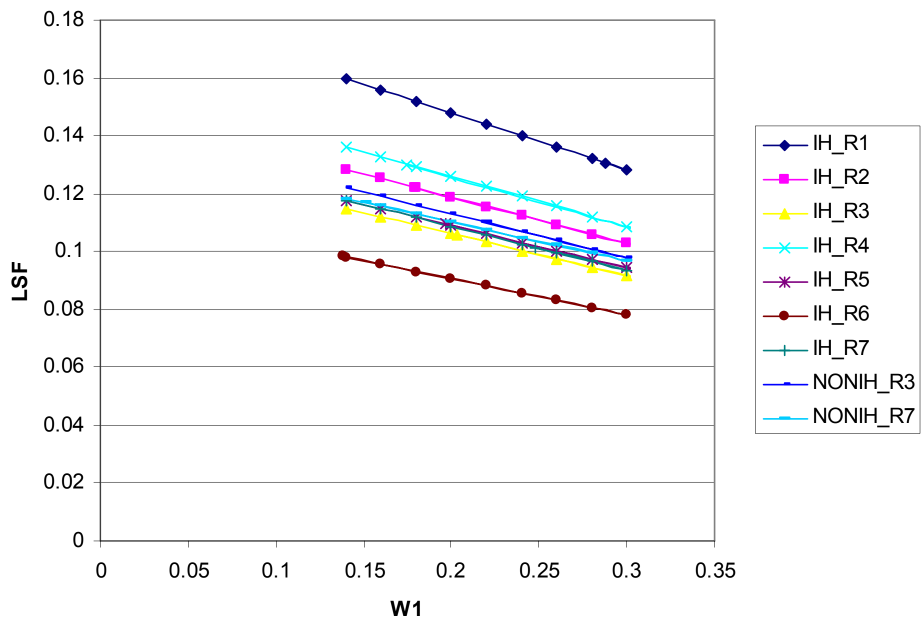


*Figure 5.10: Sensitivity of LSF to $W_1$ for Single Axles with Single Wheels*

### *5.2.2.2 Single axle with dual wheels*

Similar to the procedure for obtaining axle load spectra specification for the steering axle, the parameters for single axle load spectra are presented in Table 5.2. The sensitivity of LSF on change of axle load spectrum first peak weight $W_1$ is shown in Figure 5.11.

LSF are larger in West Texas (Regions 1, 2 and 4) than in the other regions. Thus, when adjusting the weights to obtain the axle load spectra estimation for those regions without WIM data from the non-interstate highway system, a lower $W_1$ can be assigned to the West Texas regions and a higher $W_1$ assigned for the remaining regions.

**Table 5.2: Parameters for Single Axle (with Dual Wheels) Load Spectra of Level 2 Input**

| Facility | Region | $W_1$ | $W_2$ | $W_3$ | $\mu_1$ | $\mu_2$ | $\mu_3$ | $\sigma_1$ | $\sigma_2$ | $\sigma_3$ | LSF |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Interstate | 1 | 0.472 | 0.483 | 0.045 | 1.336 | 2.166 | 2.976 | 0.253 | 0.526 | 0.101 | 0.314 |
| | 2 | 0.389 | 0.367 | 0.244 | 1.571 | 2.444 | 2.793 | 0.405 | 0.286 | 0.149 | 0.323 |
| | 3 | 0.348 | 0.501 | 0.151 | 1.448 | 2.372 | 2.78 | 0.266 | 0.348 | 0.169 | 0.290 |
| | 4 | 0.288 | 0.420 | 0.292 | 1.430 | 2.318 | 2.736 | 0.248 | 0.340 | 0.194 | 0.321 |
| | 5 | 0.444 | 0.412 | 0.144 | 1.449 | 2.439 | 2.852 | 0.314 | 0.338 | 0.113 | 0.309 |
| | 6 | 0.273 | 0.510 | 0.217 | 1.496 | 2.367 | 2.757 | 0.316 | 0.318 | 0.181 | 0.309 |
| | 7 | 0.200 | 0.514 | 0.286 | 1.365 | 1.861 | 2.623 | 0.176 | 0.499 | 0.277 | 0.243 |
| Non-Interstate | 3 | 0.315 | 0.352 | 0.333 | 1.413 | 2.187 | 2.703 | 0.251 | 0.357 | 0.217 | 0.289 |
| | 7 | 0.500 | 0.224 | 0.276 | 1.592 | 2.236 | 2.711 | 0.338 | 0.248 | 0.263 | 0.268 |

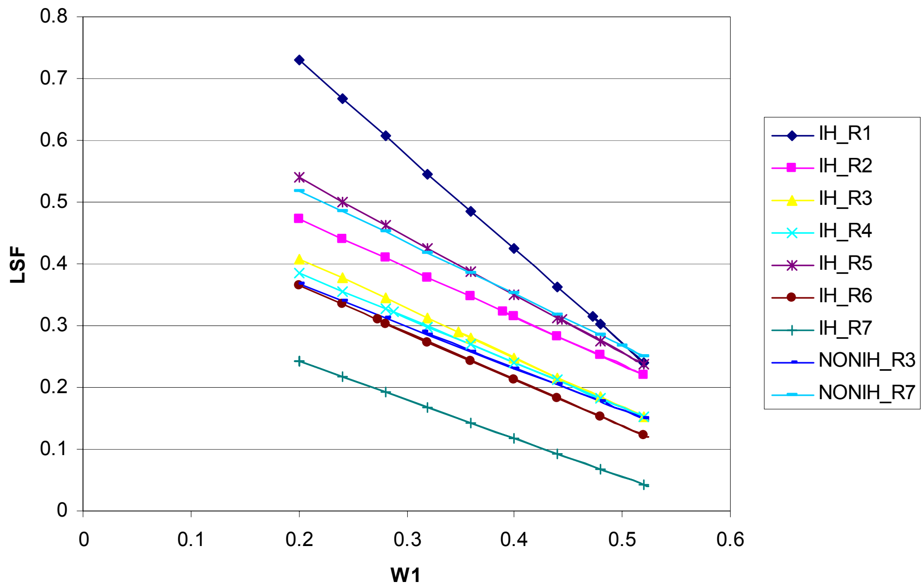


*Figure 5.11: Sensitivity of LSF to W1 for Single Axle with Dual Wheels*

*5.2.2.3 Tandem axle*

The specification for tandem axle load spectra is given in Table 5.3, and the sensitivity analysis of LSF to the change of $W_1$ is shown in Figure 5.12. Also, the LSF are larger in the three West Texas regions. Thus, lower weight $W_1$ should be considered when estimating the load spectra for those regions without a WIM station on the non-interstate highway systems.

**Table 5.3: Parameters for Tandem Axle Load Spectra of Level 2 Input**

| Facility | Region | $W_1$ | $W_2$ | $W_3$ | $\mu_1$ | $\mu_2$ | $\mu_3$ | $\sigma_1$ | $\sigma_2$ | $\sigma_3$ | LSF |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Interstate | 1 | 0.496 | 0.249 | 0.255 | 2.585 | 3.618 | 3.400 | 0.331 | 0.102 | 0.205 | 0.714 |
| | 2 | 0.513 | 0.205 | 0.282 | 2.858 | 3.320 | 3.496 | 0.426 | 0.175 | 0.085 | 0.598 |
| | 3 | 0.438 | 0.317 | 0.245 | 2.681 | 3.292 | 3.505 | 0.329 | 0.227 | 0.074 | 0.516 |
| | 4 | 0.449 | 0.240 | 0.311 | 2.820 | 3.304 | 3.520 | 0.396 | 0.191 | 0.068 | 0.608 |
| | 5 | 0.432 | 0.305 | 0.263 | 2.649 | 3.325 | 3.504 | 0.365 | 0.225 | 0.079 | 0.557 |
| | 6 | 0.500 | 0.271 | 0.229 | 2.693 | 3.329 | 3.500 | 0.333 | 0.211 | 0.075 | 0.490 |
| | 7 | 0.522 | 0.2085 | 0.270 | 2.633 | 3.334 | 3.512 | 0.339 | 0.182 | 0.059 | 0.478 |
| Non-Interstate | 3 | 0.423 | 0.249 | 0.328 | 2.728 | 3.358 | 3.521 | 0.384 | 0.212 | 0.060 | 0.641 |
| | 7 | 0.573 | 0.292 | 0.135 | 2.595 | 3.427 | 3.506 | 0.342 | 0.241 | 0.076 | 0.538 |

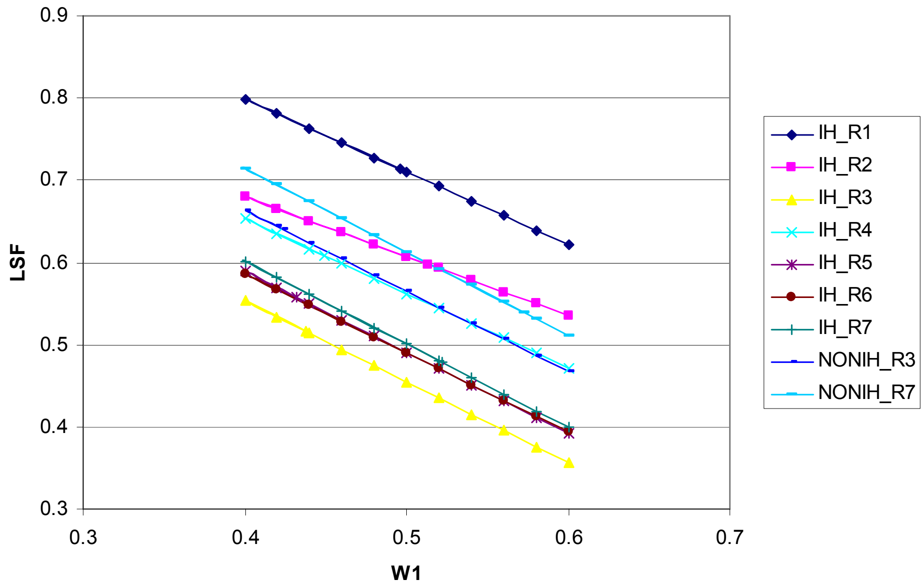


*Figure 5.12: Sensitivity of LSF to W1 for Tandem Axle Load Spectra*

Considering the interval for tandem axle load spectrum is 2 kips, the normalized frequency for each tandem load bin can be calculated by entering the load bin value in the identified mixed lognormal distribution (Equation 5.1) and multiplying the result by 2.

### *5.2.2.4 Tridem axle*

The specification for tridem axle load spectra is shown in Table 5.4, and the sensitivity analysis of LSF to the change of $W_1$ is shown in Figure 5.13. The LSF in the North Texas Regions (Regions 1, 3, and 6) are larger than in the other regions. When estimating axle load spectra for the regions without WIM data from the non-interstate highways, it is suggested that for the North Texas regions, the estimation be based on the load spectrum in Region 3; for the South Texas regions (Regions 2, 4, 5, and 7), the estimation should be based on the load spectrum in Region 7.

**Table 5.4: Parameters for Tridem Axle Load Spectra of Level 2 Input**

| Facility | Region | W1 | W2 | W3 | $\mu_1$ | $\mu_2$ | $\mu_3$ | $\sigma_1$ | $\sigma_2$ | $\sigma_3$ | LSF |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Interstate | 1 | 0.224 | 0.038 | 0.738 | 2.781 | 3.257 | 3.813 | 0.309 | 0.044 | 0.195 | 0.802 |
| | 2 | 0.159 | 0.571 | 0.270 | 2.779 | 3.202 | 3.645 | 0.121 | 0.434 | 0.150 | 0.310 |
| | 3 | 0.267 | 0.187 | 0.546 | 2.747 | 2.934 | 3.658 | 0.287 | 0.234 | 0.258 | 0.409 |
| | 4 | 0.439 | 0.529 | 0.032 | 2.853 | 3.573 | 3.584 | 0.265 | 0.258 | 0.048 | 0.297 |
| | 5 | 0.429 | 0.025 | 0.546 | 2.743 | 3.353 | 3.609 | 0.248 | 0.029 | 0.292 | 0.389 |
| | 6 | 0.241 | 0.485 | 0.274 | 2.720 | 3.425 | 3.645 | 0.165 | 0.525 | 0.085 | 0.859 |
| | 7 | 0.108 | 0.310 | 0.582 | 2.461 | 2.818 | 3.555 | 0.144 | 0.141 | 0.339 | 0.418 |
| Non-Interstate | 3 | 0.206 | 0.615 | 0.179 | 2.843 | 3.430 | 3.677 | 0.152 | 0.454 | 0.070 | 0.637 |
| | 7 | 0.179 | 0.340 | 0.481 | 2.591 | 2.866 | 3.684 | 0.254 | 0.177 | 0.261 | 0.402 |

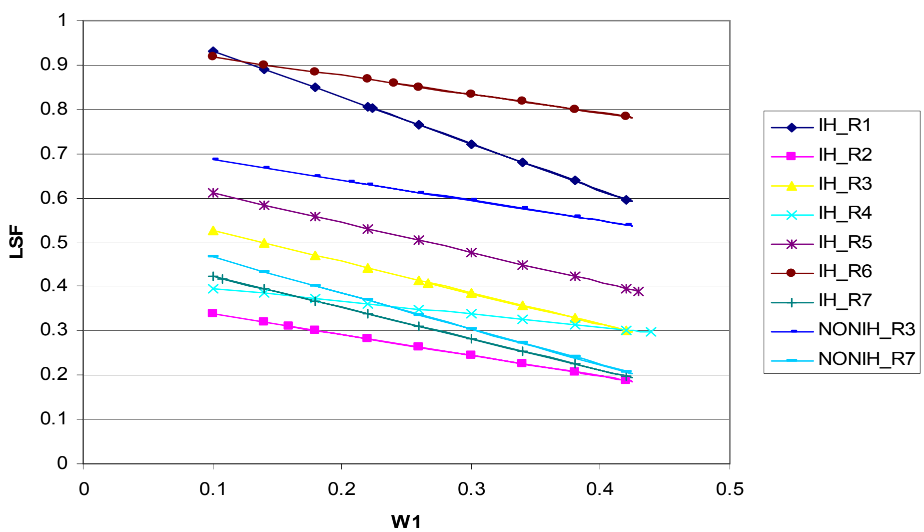


*Figure 5.13: Sensitivity of LSF to W1 for Tridem Axle Load Spectra*

## 5.3 Specification for Level 3 Axle Load Spectra (State Default)

Level 3 axle load spectra input should be used only when the designers have little knowledge of the load distribution of each type of axle. To estimate the axle load distribution of Level 3, the available axle load data in Texas are combined to establish a statewide axle load spectra. It is important to point out that Level 3 input from the available resources more closely represents the rural load characterization because the majority of WIM stations in Texas are located on rural highways. The statewide axle load spectra for all of the axle types are illustrated in Figures 5.14 to 5.17. The statistics for the distribution are presented in Table 5.5. For using the statewide load spectra, pavement designers should insert the statistics provided in Table 5.5 into Equation 5.1 and find the normalized frequency for each load bin by multiplying by the bin width.

**Table 5.5: Parameters for Steering Axle Load Spectra of Level 3 Input**

| | W1 | W2 | W3 | $\mu_1$ | $\mu_2$ | $\mu_3$ | $\sigma_1$ | $\sigma_2$ | $\sigma_3$ |
|---|---|---|---|---|---|---|---|---|---|
| Steering | 0.205 | 0.339 | 0.456 | 1.492 | 2.262 | 2.399 | 0.166 | 0.200 | 0.100 |
| Single | 0.394 | 0.480 | 0.126 | 1.475 | 2.379 | 2.806 | 0.302 | 0.358 | 0.157 |
| Tandem | 0.491 | 0.301 | 0.208 | 2.643 | 3.367 | 3.510 | 0.350 | 0.233 | 0.074 |
| Tridem | 0.287 | 0.370 | 0.343 | 2.790 | 3.232 | 3.683 | 0.207 | 0.566 | 0.201 |

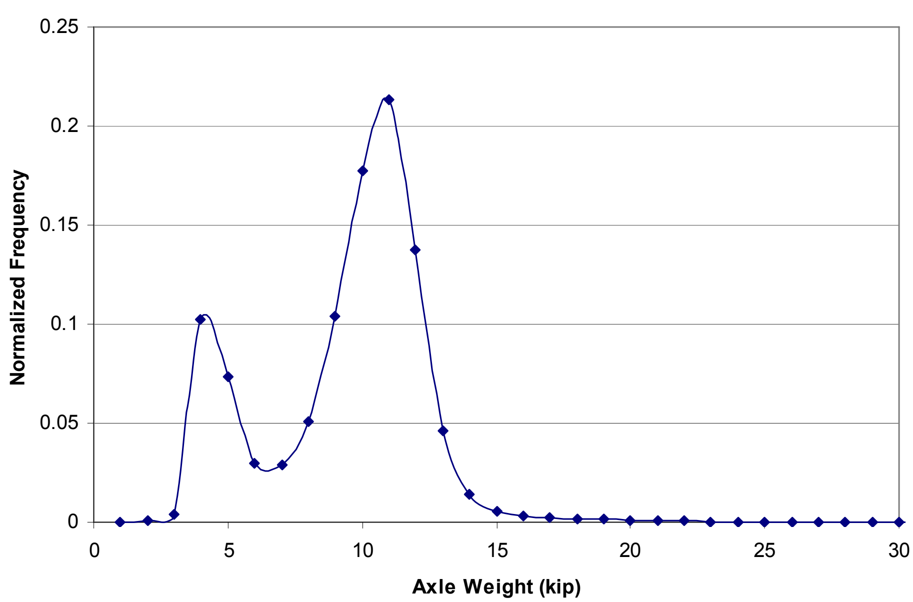


*Figure 5.14: Statewide Steering Axle Load Spectrum*

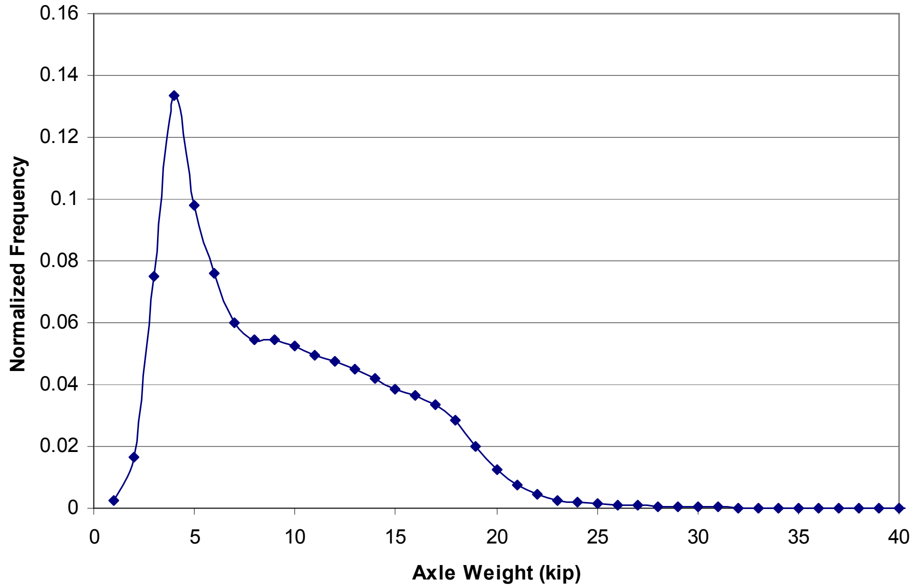


*Figure 5.15:  Statewide Single Axle (with Dual Wheels) Load Spectrum*

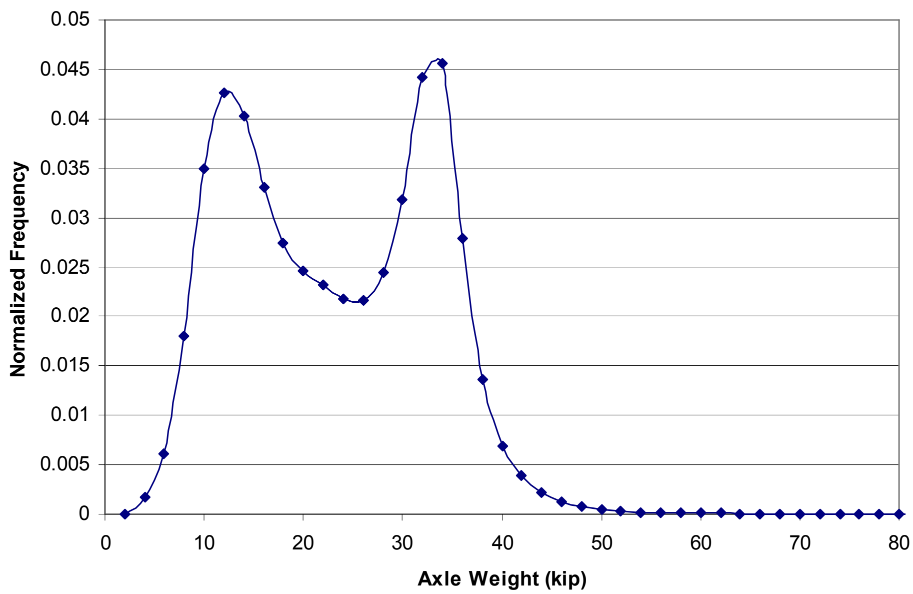


*Figure 5.16:  Statewide Tandem Axle Load Spectrum*

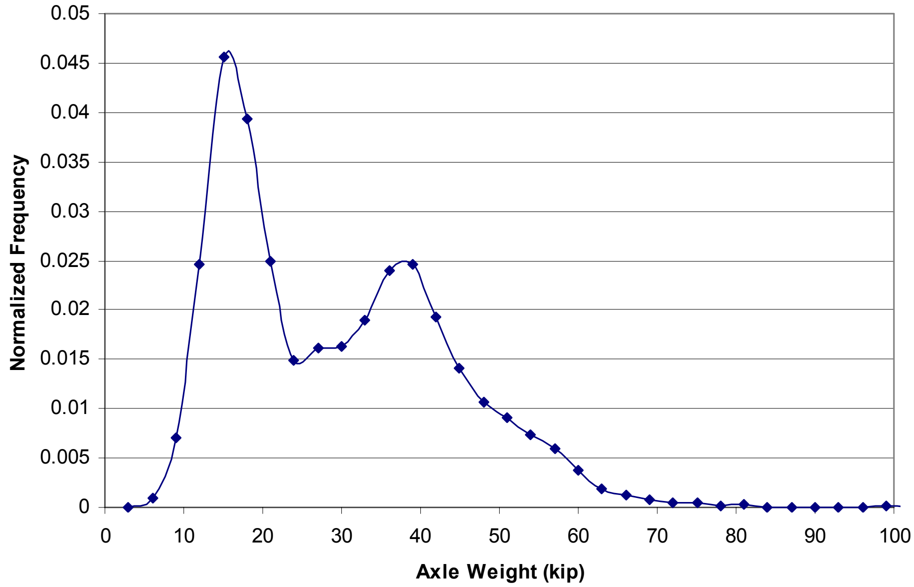


*Figure 5.17: Statewide Tridem Axle Load Spectrum*

# 6. Conclusions and Recommendations

## 6.1 Conclusions

The development of the M-E Design Guide has been one of the biggest research efforts ever addressed and accomplished by the highway community in recent years. It is the authors' opinion that, in its current status, the M-E Design Guide represents the most comprehensive pavement analysis tool ever developed. There are, however, numerous issues that should be further investigated at the individual state level before the M-E Design Guide can be implemented, including the need to evaluate the appropriateness of the performance prediction models and their calibration to local conditions. Another issue to address is improving the programming efficiency of the accompanying software to make it conducive to multiple runs in order to assess pavement performance variability and, consequently, reliability analysis by means of simulation. This aspect was an integral part of the original research but could not be adequately addressed because of the limited available resources.

More advanced models for the design and analysis of flexible pavements have been developed through research in the past 10 years, since the research study that produced the M-E Design Guide was initiated. Many of these advanced models make use of more complex mechanistic material models, such as viscosity and plasticity, which can be incorporated by integrating finite element analysis into the guide.

In terms of traffic and environmental characterization, the approach proposed in the current version of the guide is as systematic and comprehensive as can be achieved to date. In particular, the change from the practice of aggregating all traffic into a single statistic (ESAL) to considering actual axle load distribution represents one of the most significant improvements associated with moving from an empirical-based to a mechanistic-empirical approach. The goal of this research study was to assess and address the implications of the actual axle load distribution approach proposed by the M-E Design Guide. These implications have several dimensions. On one hand, the evaluation of current equipment and methodology for data collection and data management in Texas were addressed. Alternative ways for traffic data processing and delivering were developed and proposed. On the other hand, the implications on the structural design of pavement were evaluated. With these elements in hand, a number of recommendations were established and are presented in the next section for more effective and efficient traffic characterization for the structural design of pavements.

It is important to emphasize that TxDOT's TPP Division collects, processes, and delivers traffic data to many different users, the pavement design group being only one of the users. This research focuses on recommendations for traffic characterizations for the empirical and mechanistic design of pavements. Hence, the conclusions and recommendations presented in this chapter are only for this purpose and by no means are aimed at changing the systems that TPP has in place for data management. These recommendations represent the authors' opinions for guiding the data processing process to deliver traffic information to pavement designers only.

## 6.2 Recommendations

During this research study, several different versions of the M-E Design Guide were utilized because annual releases of the software have been necessary in order to improve functionality and to correct reported errors. During this period, several thousand examples were analyzed to asses the applicability of the current calibration to Texas conditions and to determine the sensitivity of the performance predictions to the most important traffic design variables. Through this extensive process, a number of recommendations were developed that could help TxDOT direct future developments in terms of empirical and mechanistic pavement design. Some of the most important recommendations are listed below.

1. Sensitivity analyses performed using the current version of the M-E Design Guide cannot be used to steer research efforts in Texas toward mechanistic-empirical design. Some of the performance models are known not to capture actual performance accurately, as observed in Texas. At this point, when in doubt it is preferable that local experience and engineering judgment guide the decision process, especially when the results of the analysis are counterintuitive. An example is the insensitivity of the performance of flexible pavements to the hourly traffic distribution.

2. This research study, as well as previous research, has indicated that there is a significant difference in the performance of a pavement when subjected to the action of a single axle with single wheels and a single axle with dual wheels of the same load. The current version of the M-E Design Guide does not account for this difference because all single axles are grouped into a single axle type. It is recommended that both axle types be treated separately. This can be done by assuming that each vehicle has, on average, one single axle with single wheels. This assumption, although not exact, introduces an error that is significantly lower than the error committed by assuming that both types of single axles produce the same damage.

3. It was determined that as many as 12,000 parameters are required by the M-E Design Guide to characterize traffic axle loads; however, only two parameters are used to characterize contact stresses. These are the average tire inflation pressures for single and dual tires. This is believed to be an unbalanced approach, and it is recommended that the distribution of actual tire inflation pressures be used. Several research projects already conducted in Texas have determined distributions that could be used as interim guidelines.

4. Significant attention is placed on the characterization of the thermo-rheological properties of the bituminous materials and the temperature conditions that affect those properties. However, the same properties vary as a result of the loading rate to which the material is subjected under real traffic conditions which, in turn, is correlated to the vehicle speed. Only one vehicle speed is used to characterize the traffic stream and, as before, this is considered unbalanced. For the sake of consistency, appropriate distribution of vehicle speed should be considered.

5. This research has demonstrated the advantages of characterizing axle load distribution by means of continuous function instead of discrete function (histograms). These advantages include the minimization of storage needs for data, the ease and simplicity for entering data into the design process, and the potential for improved computer efficiency. In addition, the specific family of distributions proposed has the advantage of a close form solution for estimating the various moments of the distribution. This desirable property facilitates the rapid estimation of the expected load-associated pavement damage of a given axle load spectrum.

6. Although the use of axle load spectra instead of ESALs is strongly supported and recommended for accurate pavement design, the need for summary statistics (such as ESALs) will remain. These summary statistics are necessary for facilitating rapid comparison among axle load spectra. In addition, these statistics can be used to compare spectra to historical traffic data, which in many cases is only available in terms of ESALs. To this effect, the moment statistics are proposed.

7. When using multi-layer linear-elastic modeling for estimating stresses and strains within a pavement structure, it should be remembered that the "pavement feels axles/wheels, not trucks." Hence, it is not necessary from the point of view of pavement design and performance analysis to characterize the traffic per class. It is therefore recommended that for mechanistic design, axle load spectra be developed for each axle type, i.e., single axles with single wheels, single axles with dual wheels, tandem axles, and tridem axles. These axle configurations cover the vast majority of highway traffic encountered on the Texas roadway network. Other axle configurations should be individually treated if heavily loaded; whereas, they could be ignored if lightly loaded. The error committed by this approach is well within the accuracy typical of current pavement design.

8. Through this research project, it was established that current WIM data availability, in terms of temporal and spatial distribution, are not adequate to support Level 1, or even Level 2, design as proposed by the M-E Design Guide. Recommendations for spatial distribution were presented highlighting the need for improved WIM coverage in different geographical regions, as well as different facility types. There is a particular need for establishing axle load spectra on the lower volume facilities in the state, especially on the FM and RM roadway networks. It should be noted that lower-volume facilities tend to be more sensitive to overloading than higher-volume facilities, such as interstate highways. Regarding temporal distribution, it was suggested that collecting WIM data on a 2-day-per-quarter basis has the potential to yield accurate data, especially if the 2-day periods are not the same from one year to another. In this regard, a 12-week interval is recommended. Such an interval will facilitate year-around coverage if it is strictly adhered to for several consecutive years.

9. At the higher level of all state and federal highway and transportation agencies, accurate data to support the M-E Design Guide is desirable but is not a low-priced alternative. Durable and accurate WIM systems are expensive and require sound pavement with high bearing capacity to provide durable support. The literature in this area indicates that accurate and durable systems are obtained when WIM equipment is installed into continuously reinforced concrete pavement (CRCP) structures. Because such a level of equipment installation is probably not economically viable to cover the needs associated with the M-E Design Guide, temporary WIM alternatives should be considered. To date, this research has not found cases of successful development and implementation of low-cost temporary WIM stations; however, it is the authors' belief that such an option is viable and deserves further consideration because it has the potential for complementing and enlarging the WIM network that is planned for Texas.

10. Finally, this research developed the methodology and demonstrated that the joint estimation of traffic growth (long-term volume changes) and seasonal variability (short-

term changes) is an effective and more efficient approach for traffic forecasting for structural pavement design. Not only does the approach capture both short- and long-term traffic volume changes, which are essential for pavement design, but does so by combining all available data into one analysis. Furthermore, the number of parameters needed is significantly reduced compared to those proposed by the M-E Design Guide. This aspect facilitates the input of data into the analysis, reduces data storage needs, and has the potential to shorten computer running time by optimizing software programming.

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# Appendix A. Effect of WIM Measurement Error on Pavement Performance Estimation

## A.1 Axle Load Distribution

Based on axle spacing and configurations, truck axles are divided into five types, steering axles, single axles with dual wheels (herein referred to as single axle), tandem axles, tridem axles, and quadruple (quads) axles. Tridem axles exist on vehicle Class 7 and Class 11. Quads exist only on vehicle Class 11. With the individual axle load magnitude obtained from each WIM scale, it is easy to obtain the number of loads falling in each bin (discrete load weight interval) for each axle type on a given truck class. The counts for each axle type are normalized in terms of percentages (normalized frequencies) to form the discrete axle load distribution. Axle load distribution is known as axle load spectrum. After examining all axle load distributions for an entire truck class across the twenty WIM stations, it is found that axle load spectra feature multi-modal patterns, with the number of peaks ranging from one to three.

The characteristics of multi-modal distribution of axle load spectra lead to adoption of mixed lognormal distribution to fit the data. The motivations to apply mixed lognormal distribution are:

1. The very nature of axle load being positive is represented by the fact that the variable in lognormal distribution is positive. Hence, the sum of the linear combination of lognormal distributions assigned with positive weights is guaranteed to be positive.

2. The individual peaks of a load distribution can easily be captured by lognormal distributions with sound parameters.

3. More importantly, it will be shown in the following discussion that load-related pavement damage can be described succinctly and effectively through the moment statistics of lognormal distribution.

### A.1.1 Mixed Lognormal Distribution Function

Assume a random variable *X* has a lognormal distribution

$$X \sim \ln(\lambda, \zeta) \qquad X > 0 \tag{A.1}$$

The probability density function (*pdf*) is,

$$f(x;\lambda,\zeta) = \frac{1}{\sqrt{2\pi}x\zeta}\exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\lambda}{\zeta}\right)^2\right) \tag{A.2}$$

where $\lambda$ and $\zeta$ are the parameters for lognormal distribution.

Thus, the mixed lognormal distribution *pdf* representing a multi-model load spectrum is,

$$f(x;\lambda_k,\zeta_k,W_k)=\sum_{k=1}^{K}\frac{W_k}{\sqrt{2\pi}x\zeta_k}\exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\lambda_k}{\zeta_k}\right)^2\right) \tag{A.3}$$

where $\lambda_k$ and $\zeta_k$ are the parameters for each lognormal distribution (*k* represents the *k*th piece of lognormal distribution, denoted as mode); $W_k$ represents the weight of the *k*th mode, and $\sum_{k=1}^{K}W_k=1$.

### A.1.2 Parameter Estimation

When it comes to fitting the data with mixed lognormal distributions, the process can be divided into two steps. The first step involves determining the number of modes, *K*. Because of the fact that the majority of load spectra exhibit a pronounced bi-modal pattern, two lognormal distributions are the minimum requirement for capturing the peaks. In the second step, parameter estimation, although the two mixed lognormal distributions can capture the pronounced peaks, the central segment (between the two peaks) is not well fitted. As a result, it may cause significant errors. Thus, a third lognormal distribution is added as the transition between the two pronounced peaks. If the error by fit function is sufficiently small (e.g., 1- $R^2$ < 1%) with two mixed lognormal distributions, the third peak is not necessary. The reason to impose a very high fit precision requirement is that load-related pavement damage is sensitive to the fit error.

The estimated parameters and data fit statistics for truck tandem axles within the classes that offer the four largest sample sizes are shown in Table 1. The four truck classes are 10, 6, 4, and 9 in descending order of sample size. In general, the parameters for the load spectrum based on tandem load data across all the truck classes are presented in the last row of Table A.1. Those cells without data mean the third peak is not adopted. The underlying functions fit the data very well with all $R^2$ more than 99 percent.

**Table A.1: Data Fit Parameters for Truck Tandem Axles**

| Class | Mixed Lognormal Distribution Parameters | | | | | | | | | $R^2$ | Performance Fit Error |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $W_1$ | $W_2$ | $W_3$ | $\lambda_1$ | $\lambda_2$ | $\lambda_3$ | $\zeta_1$ | $\zeta_2$ | $\zeta_3$ | | |
| 4 | 0.429 | 0.571 | –† | 3.192 | 3.386 | – | 0.160 | 0.083 | – | 0.997 | 3.12% |
| 6 | 0.147 | 0.565 | 0.288 | 2.231 | 2.462 | 3.404 | 0.090 | 0.347 | 0.173 | 0.992 | 1.90% |
| 9 | 0.269 | 0.731 | – | 2.486 | 2.841 | – | 0.164 | 0.350 | – | 0.998 | -4.33% |
| 10 | 0.424 | 0.292 | 0.285 | 2.733 | 3.264 | 3.488 | 0.325 | 0.189 | 0.065 | 0.995 | 3.11% |
| ALL | 0.433 | 0.296 | 0.270 | 2.714 | 3.265 | 3.487 | 0.335 | 0.189 | 0.065 | 0.995 | 2.19% |

†: Parameters not available because only bi-modal distribution is applied such as Classes 4 and 9.

It will be shown in the next section that load-related pavement damage can be estimated through the fourth moment statistic of axle load distribution. In this regard, the error is obtained as the relative difference between the fourth moment from fitted lognormal distribution and that from the discrete load distribution (observation). The corresponding results are presented in the last column in Table A.1. The errors are acceptable in pavement design context. For instance, the error for all truck tandems by fitted function is 2.19 percent.

In summary, it is proved that axle load spectra can be described effectively by mixed lognormal distributions.

## A.2 Methodology

### A.2.1 Load-related Damage on Pavement

It was established through the AASHO Road Test that the damage caused by each individual axle load on flexible pavement can be estimated according to the *fourth power law* (AASHTO, 1993; Huang, 2003). The *fourth power law* implies that pavement damage by passing vehicles increases exponentially with the increase of their axle load. This relationship is denoted by Load Equivalence Factor (LEF),

$$LEF = \left(\frac{x_r}{L_s}\right)^m \qquad \text{(A.4)}$$

where $x_r$ = weight of axle load in the *r*th bin, (lbs), (assume that axle loads within each bin are identical); $L_s$ = load weight on a standard axle with the same number of axles as $x_r$, usually 18 kip for the single axle and it is dependent on pavement structure for the tandem axle, usually 34 kip; and *m* = power denoting the relative damage to the pavement of a given load $x_r$, typically 4.

As a result, the load-related pavement damage based on a given axle load spectrum of truck Class *j* can be obtained by summing the contributions from all the loads $x_r$'s in the distribution, denoted as load spectra factor (LSF), $LSF_j$ (under the condition of power , m = 4) ,

$$LSF_j = \sum_{r=1}^{R}\left(\frac{x_r}{L_s}\right)^4 q_{r,j} \qquad \text{(A.5)}$$

where *R* = total number of load bins; and $q_{r,j}$ = normalized frequency of load in the *r*th bin of a given load spectrum of truck Class *j*.

It is shown that $LSF_j$ is the fourth sample moment statistic divided by $L_s^{\,4}$. Because the continuous distribution function of each axle load spectrum is available, it is more convenient and equally valid to address the axle-load-related pavement damage by employing the population moment from the *pdf*.

The fourth moment of load spectrum function *f(x)*, $M^4$, is defined as,

$$M^4 = E\left(X^4\right) = \int x^4 f(x)dx \qquad \text{(A.6)}$$

Integrating Equation (A.6) and the axle load spectra functions, as shown in Equation (A.3), i.e., summing the contribution from all of the axle loads according to their distribution, Equation A.6 is equivalent to,

$$LSF_j = \int \left(\tfrac{x}{L_s}\right)^4 f_j(x)dx$$

$$LSF_j = \frac{\int x^4 f_j(x)dx}{L_s^{\ 4}} = \frac{E\left(X^4\right)}{L_s^{\ 4}} = \frac{M^4}{C} \tag{A.7}$$

where $x$ = axle load weight (kip); $f_j(x)$ = axle load spectrum function of one axle type on truck Class $j$; $L_s^{\ 4}$ = constant, $C$.

In summary, on the basis of the axle load spectrum function *f(x)* as given in Equation (A.3), axle-load-related pavement damage can be equivalently estimated by applying Equation (A.7) in terms of the moment statistic. Thus, what remains to determine the underlying estimation is the fourth moment of load spectrum function *f(x)*.

### A.2.2 Moment for the Lognormal Distribution

As is shown in Equation (A.7), the moment of axle load spectrum function is the statistic governing the estimation of load-related pavement damage. The fourth moment for a random variable $X$ with lognormal distribution can be derived as:

$$M^4 = E\left(X^4\right) = \int x^4 \frac{1}{\sqrt{2\pi}x\zeta} \exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\lambda}{\zeta}\right)^2\right)dx = \exp\left(4\lambda + 8\zeta^2\right) \tag{A.8}$$

where, $\lambda$ and $\zeta$ are as in Equation (A.3.)

Thus, with load spectrum fitted by $K$ mixed lognormal distributions, the load-related pavement damage of an axle type on truck class $j$, $LSF_j$ is obtained as,

$$LSF_j = \sum_k W_k \exp\left(4\lambda_{k,j} + 8\zeta_{k,j}^{\ 2}\right) / L_s^{\ 4} \tag{A.9}$$

where $W_k$, $\lambda_{k,j}$ and $\zeta_{k,j}$, are as in Equation (A.3), for truck class $j$.

## A.3 Load-related Pavement Damage Estimation under Measurement Errors

Two scenarios are evaluated to study the effect of WIM measurement errors on pavement performance. First, the load-related pavement damage estimation is derived under the condition of ideal calibration (with zero calibration bias and involving random errors only) of a WIM scale. In the meantime, it is assumed that the axle load spectra with mixed lognormal distributions aforementioned are used as the reference (without measurement errors). A comparison is carried out between the estimated load-related pavement damage with random measurement error (normal distribution, $\sigma_\varepsilon$ not equal to zero) and the reference. The second scenario investigates the estimated load-related pavement damage with biased WIM calibration. In such cases, not only systematic error but also random errors are involved because the latter is unavoidable.

### A.3.1 Scenario I

As mentioned previously, under sound conditions, WIM scale measurement errors exhibit a normal distribution (see the example in Figure 4.1). Assume for a given real axle load (of certain axle type on a truck class), $X = x$, the observation by a WIM scale with random errors is a random variable, denoted as $X'$.

$$\varepsilon_1 = \frac{X' - X}{X} \sim N\left(0, \sigma_\varepsilon^2\right) \qquad \text{(A.10)}$$

where $\varepsilon_1$ = axle load relative error under Scenario I; $\sigma_\varepsilon$ = indicator of WIM accuracy.

Hence, the variable $X'$ conditional on axle load $x$ also has a normal distribution,

$$X' \mid X = x \sim N\left(x, x^2 \sigma_\varepsilon^2\right) \qquad \text{(A.11)}$$

The estimated load-related pavement damage by observed load $X'$ conditional on $X$=$x$, denoted as $LSF_{X'|X=x}$, is

$$LF_{X'|X=x} = \frac{\int (x')^4 g_{X'|X}\left(x' \mid x\right) dx'}{L_s{}^4} = \frac{E\left(X' \mid X = x\right)^4}{L_s{}^4} \qquad \text{(A.12)}$$

where $g_{X'|X}\left(x' \mid x\right)$ = *pdf* of load observation $X'$ conditional on $X = x$, see Equation (A.11); $E\left(X' \mid X = x\right)^4$ = the fourth moment of $x'$ conditional on $x$; and $L_s$.

It is shown in Equation (A.12) that the moments for the normal distribution are required for the solution, which can be derived from the moment-generating function (*MGF*) (DeGroot and Schervish, 2002) of random variable $X'$,

$$\psi(t) = E_{X'}\left(\exp(tx')\right) = \int \exp(tx') \frac{1}{\sqrt{2\pi}\sigma} \exp\left(-\frac{(x' - \mu)^2}{2\sigma^2}\right) dx' \qquad \text{(A.13)}$$

where $\mu$ and $\sigma$ are the parameters of normal distribution of variable $X'$, as in Equation (A.11).

As a result, the *fourth* moments are presented as follows:

$$M_{normal}^4 = E(X'^4) = 3\sigma^4 + 6\sigma^2\mu^2 + \mu^4 \qquad \text{(A.14)}$$

Thereafter, the estimated conditional axle-load-related pavement damage factor $LF_{X'|X=x}$ can be determined by substituting Equation (A.14) in Equation (A.12),

$$LF_{X'|X=x}^4 = \frac{(3\sigma_\varepsilon^4 + 6\sigma_\varepsilon^2 + 1)x^4}{L_s{}^4} \qquad \text{(A.15)}$$

As previously established, the variable of axle load $X$ for each individual axle type follows a mixed lognormal distribution respectively; see Equation (A.3). Therefore, the estimated axle-load-related pavement damage (with random measurement errors occurring) based on load spectrum for a given axle type can be obtained by integrating the contribution from its overall axle loads, denoted as $LSF_j^E$

$$LSF_j^E = \iint x'^4 \, g_{X'|X}\left(x' \mid x\right) f_X{}^j(x) dx dx' \Big/ L_s{}^4 = \left(3\sigma_\varepsilon^4 + 6\sigma_\varepsilon^2 + 1\right) \sum_k W_k \exp\left(4\lambda_{k,j} + 8\zeta_{k,j}{}^2\right) \Big/ L_s{}^4 \qquad \text{(A.16)}$$

where $g_{X'|X}\left(x' \mid x\right)$ is the same as in Equation (A.12); $f_X{}^j(x)$ is the axle load (without measurement errors) distribution function of certain axle type on a truck class, see Equation (A.3); $\sigma_\varepsilon$, see Equation (A.10).

By comparing Equations (A.16) and (A.9), it is suggested that under random measurement error, an additive portion of $\left(3\sigma_{\varepsilon}^{4}+6\sigma_{\varepsilon}^{2}\right)\sum_{k} W_{k} \exp\left(4\lambda_{k,j}+8\zeta_{k,j}{}^{2}\right) \Big/ L_{s}{}^{4}$ is introduced into the load-related pavement damage estimation. Additionally, the always-positive value in the additive term implies that the random measurement error results in overestimation of the load-related pavement damage estimation. The extent of overestimation depends on the magnitude of the WIM accuracy indicator, $\sigma_{\varepsilon}$.

### A.3.2 Scenario II

When a WIM scale is not properly calibrated (biased), the axle load measurement is subject to systematic error and the measured weight differs from the actual value. In such cases, both systematic error and random error should be considered (see the example in Figure 4.2). Assume for a given actual axle load (of certain axle type on a truck class), $X = x$, the observation by a WIM scale with both errors is also a random variable, denoted as $X''$.

$$\varepsilon_{2}=\frac{X''-X}{X} \sim N\left(\alpha, \sigma_{\varepsilon}^{2}\right) \tag{A.17}$$

where $\varepsilon_{2}$ is axle load relative error under Scenario II; $\alpha$ is the calibration bias, which is 0 if ideally calibrated; and $\sigma_{\varepsilon}$ is as in Equation (A.10).

The variable $X''$ conditional on axle load $X = x$ has a normal distribution under biased WIM calibration condition,

$$X''\big|X=x \sim N\left((1+\alpha)x, x^{2}\sigma_{\varepsilon}^{2}\right) \tag{A.18}$$

Hence, the axle-load-related pavement damage by a given axle type on a certain truck class under the biased calibration condition is estimated using a similar approach as presented in Scenario I, denoted as, $LSF_{j}^{E(b)}$,

$$\begin{aligned} LSF_{j}^{E(b)} &= \iint x''^{4}\, g_{X''|X}(x''|x) f_{X}{}^{j}(x)\,dx\,dx'' \Big/ L_{s}{}^{4} \\ &= \left(3\sigma_{\varepsilon}^{4}+6\sigma_{\varepsilon}^{2}(1+\alpha)^{2}+(1+\alpha)^{4}\right)\sum_{k} W_{k} \exp\left(4\lambda_{k,j}+8\zeta_{k,j}{}^{2}\right) \Big/ L_{s}{}^{4} \end{aligned} \tag{A.19}$$

where $g_{X''|X}\left(x''|x\right)$ is conditional distribution of observation $X''$, see Equation (A.18).

The result in Equation (A.19) suggests that, similar to that in Scenario I, the estimated load-related pavement damage under the biased calibration situation is also comprised of two components: 1) a coefficient term, $(3\sigma_{\varepsilon}^{4}+6\sigma_{\varepsilon}^{2}(1+\alpha)^{2}+(1+\alpha)^{4})$, including both systematic and random errors; and 2) the load-related pavement damage obtained when no measurement errors occurred. Provided the coefficient term is greater than one, it implies an overestimation of load-related pavement damage; whereas if the term is less than one, the load-related pavement damage is underestimated. Furthermore, the extent of over- or underestimation is determined by both the magnitude of the WIM accuracy indicator, $\sigma_{\varepsilon}$ and calibration bias, $\alpha$.

The detailed results, as well as findings, are presented in Section 4.2 of Chapter 4.

# Appendix B: Traffic Growth Statistics Tables

**Table B1: Traffic Growth Statistics on IH 10**

| Section # | County # | Linear | | | | Compound Growth Factor |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | |
| | | | Y1 | Y8 | Y16 | |
| 1 | 72 | 192 | 5.57% | 4.01% | 3.03% | 3.72% |
| 2 | 72 | 201 | 5.43% | 3.93% | 2.99% | 3.65% |
| 3 | 72 | 204 | 5.07% | 3.74% | 2.88% | 3.46% |
| 4 | 72 | 220 | 5.17% | 3.79% | 2.91% | 3.51% |
| 5 | 72 | 312 | 7.39% | 4.87% | 3.51% | 4.44% |
| 6 | 72 | 256 | 4.34% | 3.33% | 2.63% | 3.01% |
| 7 | 72 | 310 | 4.73% | 3.56% | 2.77% | 3.22% |
| 8 | 72 | 317 | 4.37% | 3.35% | 2.64% | 3.04% |
| 9 | 72 | 286 | 3.55% | 2.85% | 2.32% | 2.53% |
| 10 | 72 | 262 | 3.21% | 2.62% | 2.17% | 2.38% |
| 11 | 72 | 518 | 8.90% | 5.48% | 3.81% | 5.52% |
| 12 | 72 | 521 | 8.49% | 5.33% | 3.73% | 5.35% |
| 13 | 72 | 521 | 8.49% | 5.33% | 3.73% | 5.35% |
| 14 | 72 | 521 | 8.45% | 5.31% | 3.73% | 5.34% |
| 15 | 72 | 538 | 9.48% | 5.70% | 3.91% | 5.65% |
| 16 | 72 | 596 | 9.05% | 5.54% | 3.84% | 5.65% |
| 17 | 72 | 583 | 9.29% | 5.63% | 3.88% | 5.72% |
| 18 | 72 | 602 | 9.58% | 5.73% | 3.93% | 5.86% |
| 19 | 72 | 613 | 10.09% | 5.91% | 4.02% | 5.98% |
| 20 | 72 | 549 | 10.53% | 6.06% | 4.08% | 6.09% |
| 21 | 72 | 559 | 10.96% | 6.20% | 4.14% | 6.26% |
| 22 | 72 | 593 | 12.28% | 6.60% | 4.32% | 6.76% |
| 23 | 72 | | | | | |
| 24 | 72 | 1068 | 38.89% | 10.45% | 5.69% | 10.25% |
| 25 | 72 | 592 | 15.45% | 7.42% | 4.66% | 7.39% |
| 26 | 72 | 344 | 9.24% | 5.61% | 3.87% | 5.42% |
| 27 | 72 | 294 | 8.28% | 5.24% | 3.69% | 5.03% |
| 28 | 72 | 241 | 7.01% | 4.70% | 3.42% | 4.42% |
| 29 | 72 | | | | | |
| 30 | 72 | | | | | |
| 31 | 116 | | | | | |
| 32 | 116 | | | | | |
| 33 | 116 | | | | | |
| 34 | 116 | 267 | 8.66% | 5.39% | 3.77% | 5.19% |
| 35 | 116 | 251 | 7.93% | 5.10% | 3.62% | 4.96% |
| 36 | 116 | | | | | |
| 37 | 116 | | | | | |
| 38 | 116 | | | | | |
| 39 | 116 | | | | | |
| 40 | 116 | | | | | |

| Section # | County # | Linear | | | | Compound |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | Growth Factor |
| | | | Y1 | Y8 | Y16 | |
| 41 | 116 | 267 | 9.04% | 5.54% | 3.84% | 5.31% |
| 42 | 116 | 267 | 9.04% | 5.54% | 3.84% | 5.31% |
| 43 | 116 | 249 | 8.33% | 5.26% | 3.70% | 4.96% |
| 44 | 116 | 293 | 9.65% | 5.76% | 3.94% | 5.56% |
| 45 | 116 | 267 | 8.66% | 5.39% | 3.77% | 5.19% |
| 46 | 55 | | | | | |
| 47 | 55 | | | | | |
| 48 | 55 | | | | | |
| 49 | 55 | 232 | 7.47% | 4.90% | 3.52% | 4.70% |
| 50 | 55 | 238 | 7.85% | 5.07% | 3.61% | 4.98% |
| 51 | 55 | 159 | 5.04% | 3.72% | 2.87% | 3.66% |
| 52 | 55 | 160 | 4.86% | 3.63% | 2.81% | 3.50% |
| 53 | 55 | 162 | 4.91% | 3.65% | 2.83% | 3.53% |
| 54 | 55 | 157 | 4.83% | 3.61% | 2.80% | 3.48% |
| 55 | 55 | 157 | 4.83% | 3.61% | 2.80% | 3.48% |
| 56 | 55 | 155 | 4.78% | 3.58% | 2.78% | 3.45% |
| 57 | 123 | 171 | 5.53% | 3.99% | 3.02% | 3.84% |
| 58 | 123 | 170 | 5.48% | 3.96% | 3.01% | 3.82% |
| 59 | 195 | | | | | |
| 60 | 195 | | | | | |
| 61 | 195 | 92 | 10.37% | 6.01% | 4.06% | 6.03% |
| 62 | 195 | 88 | 8.74% | 5.42% | 3.78% | 5.42% |
| 63 | 195 | 89 | 9.52% | 5.71% | 3.92% | 5.76% |
| 64 | 195 | 89 | 9.41% | 5.67% | 3.90% | 5.73% |
| 65 | 195 | 89 | 9.41% | 5.67% | 3.90% | 5.73% |
| 66 | 195 | 145 | 4.40% | 3.37% | 2.65% | 3.26% |
| 67 | 195 | 91 | 9.85% | 5.83% | 3.98% | 5.85% |
| 68 | 195 | 92 | 9.97% | 5.87% | 3.99% | 5.88% |
| 69 | 195 | 92 | 10.37% | 6.01% | 4.06% | 6.03% |
| 70 | 186 | | | | | |
| 71 | 186 | | | | | |
| 72 | 186 | 81 | 7.43% | 4.89% | 3.51% | 4.84% |
| 73 | 186 | 78 | 7.52% | 4.93% | 3.53% | 4.87% |
| 74 | 186 | 84 | 8.36% | 5.27% | 3.71% | 5.23% |
| 75 | 186 | 97 | 9.01% | 5.53% | 3.83% | 5.45% |
| 76 | 186 | 93 | 9.12% | 5.57% | 3.85% | 5.49% |
| 77 | 186 | 89 | 8.64% | 5.38% | 3.76% | 5.33% |
| 78 | 186 | 90 | 8.90% | 5.48% | 3.81% | 5.43% |
| 79 | 186 | 90 | 8.90% | 5.48% | 3.81% | 5.43% |
| 80 | 186 | 91 | 9.20% | 5.59% | 3.86% | 5.57% |
| 81 | 186 | 91 | 9.20% | 5.59% | 3.86% | 5.57% |
| 82 | 186 | 91 | 9.14% | 5.57% | 3.85% | 5.57% |
| 83 | 186 | 91 | 9.14% | 5.57% | 3.85% | 5.57% |
| 84 | 186 | 91 | 9.14% | 5.57% | 3.85% | 5.57% |
| 85 | 186 | | | | | |

| Section # | County # | Linear | | | | Compound |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | Growth Factor |
| | | | Y1 | Y8 | Y16 | |
| 86 | 186 | 90 | 9.46% | 5.69% | 3.91% | 5.75% |
| 87 | 186 | 87 | 8.77% | 5.43% | 3.79% | 5.46% |
| 88 | 186 | 79 | 7.05% | 4.72% | 3.43% | 4.67% |
| 89 | 186 | 79 | 7.05% | 4.72% | 3.43% | 4.67% |
| 90 | 186 | 66 | 7.07% | 4.73% | 3.43% | 4.64% |
| 91 | 186 | 81 | 7.49% | 4.91% | 3.53% | 4.87% |
| 92 | 53 | 108 | 10.80% | 6.15% | 4.12% | 6.06% |
| 93 | 53 | 103 | 10.00% | 5.88% | 4.00% | 5.81% |
| 94 | 53 | 103 | 10.00% | 5.88% | 4.00% | 5.81% |
| 95 | 53 | 100 | 9.49% | 5.70% | 3.92% | 5.59% |
| 96 | 53 | 95 | 9.19% | 5.59% | 3.86% | 5.58% |
| 97 | 53 | 92 | 8.96% | 5.50% | 3.82% | 5.53% |
| 98 | 53 | 76 | 6.09% | 4.27% | 3.18% | 4.03% |
| 99 | 53 | 116 | 9.56% | 5.73% | 3.93% | 5.51% |
| 100 | 53 | 109 | 8.81% | 5.45% | 3.79% | 5.24% |
| 101 | 218 | 109 | 8.75% | 5.43% | 3.78% | 5.24% |
| 102 | 218 | 109 | 8.75% | 5.43% | 3.78% | 5.24% |
| 103 | 218 | 106 | 8.87% | 5.47% | 3.81% | 5.23% |
| 104 | 218 | 107 | 9.14% | 5.57% | 3.86% | 5.38% |
| 105 | 218 | 105 | 8.80% | 5.45% | 3.79% | 5.26% |
| 106 | 218 | 95 | 7.62% | 4.97% | 3.56% | 4.80% |
| 107 | 218 | 95 | 7.62% | 4.97% | 3.56% | 4.80% |
| 108 | 218 | 95 | 7.62% | 4.97% | 3.56% | 4.80% |
| 109 | 218 | 99 | 8.15% | 5.19% | 3.67% | 5.02% |
| 110 | 218 | 99 | 8.15% | 5.19% | 3.67% | 5.02% |
| 111 | 134 | 97 | 7.87% | 5.07% | 3.61% | 4.89% |
| 112 | 134 | 99 | 7.99% | 5.13% | 3.63% | 4.95% |
| 113 | 134 | 99 | 7.99% | 5.13% | 3.63% | 4.95% |
| 114 | 134 | 103 | 8.25% | 5.23% | 3.69% | 5.05% |
| 115 | 134 | 100 | 7.81% | 5.05% | 3.60% | 4.89% |
| 116 | 134 | 31 | 1.60% | 1.44% | 1.29% | 1.49% |
| 117 | 134 | 31 | 1.60% | 1.44% | 1.29% | 1.49% |
| 118 | 134 | 40 | 2.09% | 1.83% | 1.59% | 1.91% |
| 119 | 134 | 36 | 1.78% | 1.58% | 1.40% | 1.61% |
| 120 | 134 | 36 | 1.86% | 1.65% | 1.45% | 1.69% |
| 121 | 134 | 41 | 2.17% | 1.88% | 1.64% | 1.95% |
| 122 | 134 | 42 | 2.33% | 2.00% | 1.73% | 2.09% |
| 123 | 133 | 41 | 2.29% | 1.97% | 1.70% | 2.05% |
| 124 | 133 | 41 | 2.29% | 1.97% | 1.70% | 2.05% |
| 125 | 133 | 42 | 2.37% | 2.03% | 1.75% | 2.11% |
| 126 | 133 | 46 | 2.52% | 2.14% | 1.83% | 2.23% |
| 127 | 133 | 43 | 2.26% | 1.95% | 1.69% | 2.03% |
| 128 | 133 | 45 | 2.43% | 2.07% | 1.78% | 2.14% |
| 129 | 133 | 57 | 2.55% | 2.16% | 1.84% | 2.21% |
| 130 | 133 | 37 | 1.64% | 1.47% | 1.32% | 1.49% |

| Section # | County # | Linear | | | | Compound |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | Growth Factor |
| | | | Y1 | Y8 | Y16 | |
| 131 | 131 | 104 | 5.07% | 3.74% | 2.88% | 3.29% |
| 132 | 131 | 110 | 4.65% | 3.51% | 2.74% | 3.10% |
| 133 | 131 | 115 | 4.84% | 3.61% | 2.80% | 3.24% |
| 134 | 131 | 113 | 4.70% | 3.53% | 2.75% | 3.12% |
| 135 | 131 | 120 | 5.32% | 3.88% | 2.96% | 3.44% |
| 136 | 131 | 120 | 5.32% | 3.88% | 2.96% | 3.44% |
| 137 | 131 | 132 | 5.63% | 4.04% | 3.05% | 3.63% |
| 138 | 131 | 129 | 5.11% | 3.76% | 2.89% | 3.36% |
| 139 | 131 | 180 | 7.70% | 5.00% | 3.57% | 4.75% |
| 140 | 131 | 198 | 7.85% | 5.06% | 3.60% | 4.74% |
| 141 | 131 | 62 | 2.87% | 2.39% | 2.01% | 2.42% |
| 142 | 131 | 96 | 4.16% | 3.22% | 2.56% | 3.26% |
| 143 | 15 | 227 | 1.85% | 1.64% | 1.45% | 1.54% |
| 144 | 15 | -479 | -3.60% | -4.81% | -7.82% | -4.56% |
| 145 | 15 | -292 | -3.14% | -4.03% | -5.95% | -3.86% |
| 146 | 15 | | | | | |
| 147 | 15 | | | | | |
| 148 | 15 | | | | | |
| 149 | 15 | | | | | |
| 150 | 15 | | | | | |
| 151 | 15 | | | | | |
| 152 | 15 | | | | | |
| 153 | 15 | | | | | |
| 154 | 15 | | | | | |
| 155 | 15 | 177 | 7.01% | 4.70% | 3.42% | 4.38% |
| 156 | 15 | 186 | 7.68% | 4.99% | 3.57% | 4.67% |
| 157 | 15 | 191 | 7.41% | 4.88% | 3.51% | 4.57% |
| 158 | 15 | 231 | 7.10% | 4.74% | 3.44% | 4.53% |
| 159 | 15 | | | | | |
| 160 | 15 | | | | | |
| 161 | 15 | | | | | |
| 162 | 15 | | | | | |
| 163 | 15 | 88 | 1.71% | 1.53% | 1.36% | 1.69% |
| 164 | 15 | 116 | 2.37% | 2.04% | 1.75% | 2.09% |
| 165 | 15 | -27 | -0.34% | -0.35% | -0.36% | 0.04% |
| 166 | 15 | -7 | -0.09% | -0.09% | -0.09% | 0.20% |
| 167 | 15 | | | | | |
| 168 | 95 | | | | | |
| 169 | 95 | 266 | 7.30% | 4.83% | 3.48% | 4.78% |
| 170 | 95 | 266 | 7.30% | 4.83% | 3.48% | 4.78% |
| 171 | 95 | 207 | 5.28% | 3.85% | 2.95% | 3.74% |
| 172 | 95 | 201 | 5.05% | 3.73% | 2.87% | 3.60% |
| 173 | 95 | 182 | 4.66% | 3.51% | 2.74% | 3.37% |
| 174 | 95 | 210 | 4.65% | 3.51% | 2.74% | 3.48% |
| 175 | 95 | 184 | 4.13% | 3.20% | 2.55% | 3.08% |

| Section # | County # | Linear | | | | Compound |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | Growth Factor |
| | | | Y1 | Y8 | Y16 | |
| 176 | 95 | 338 | 11.15% | 6.26% | 4.17% | 6.99% |
| 177 | 95 | 329 | 11.29% | 6.31% | 4.19% | 6.91% |
| 178 | 95 | 329 | 11.29% | 6.31% | 4.19% | 6.91% |
| 179 | 95 | 340 | 12.09% | 6.55% | 4.30% | 7.15% |
| 180 | 95 | 333 | 11.97% | 6.51% | 4.28% | 7.08% |
| 181 | 28 | 333 | 12.00% | 6.52% | 4.29% | 7.09% |
| 182 | 28 | 323 | 11.35% | 6.33% | 4.20% | 6.85% |
| 183 | 28 | 0 | -1.47% | -1.64% | -1.89% | -2.20% |
| 184 | 90 | 334 | 11.76% | 6.45% | 4.25% | 7.01% |
| 185 | 90 | 329 | 11.61% | 6.41% | 4.23% | 6.93% |
| 186 | 90 | 327 | 11.62% | 6.41% | 4.24% | 6.92% |
| 187 | 90 | 327 | 11.62% | 6.41% | 4.24% | 6.92% |
| 188 | 90 | 332 | 11.57% | 6.39% | 4.23% | 6.96% |
| 189 | 90 | 322 | 11.18% | 6.27% | 4.18% | 6.81% |
| 190 | 76 | 344 | 12.15% | 6.56% | 4.30% | 7.19% |
| 191 | 76 | 327 | 12.18% | 6.57% | 4.31% | 7.23% |
| 192 | 76 | 337 | 12.16% | 6.57% | 4.31% | 7.03% |
| 193 | 76 | 337 | 12.16% | 6.57% | 4.31% | 7.03% |
| 194 | 76 | 340 | 11.99% | 6.52% | 4.28% | 7.07% |
| 195 | 76 | 343 | 11.87% | 6.48% | 4.27% | 7.05% |
| 196 | 76 | 331 | 11.43% | 6.35% | 4.21% | 6.91% |
| 197 | 45 | 248 | 4.73% | 3.55% | 2.77% | 3.60% |
| 198 | 45 | 244 | 4.56% | 3.46% | 2.71% | 3.52% |
| 199 | 45 | 250 | 4.80% | 3.59% | 2.79% | 3.63% |
| 200 | 45 | 270 | 5.29% | 3.86% | 2.95% | 3.88% |
| 201 | 45 | 248 | 4.78% | 3.58% | 2.78% | 3.61% |
| 202 | 45 | 328 | 11.30% | 6.31% | 4.19% | 6.87% |
| 203 | 45 | 361 | 12.50% | 6.67% | 4.35% | 7.18% |
| 204 | 45 | 358 | 12.10% | 6.55% | 4.30% | 7.06% |
| 205 | 45 | 359 | 12.40% | 6.64% | 4.34% | 7.16% |
| 206 | 45 | 446 | 13.17% | 6.85% | 4.43% | 7.56% |
| 207 | 45 | 248 | 4.73% | 3.55% | 2.77% | 3.60% |
| 208 | 8 | 276 | 5.35% | 3.89% | 2.97% | 3.88% |
| 209 | 8 | 276 | 5.46% | 3.95% | 3.00% | 3.96% |
| 210 | 8 | 295 | 5.52% | 3.98% | 3.02% | 3.95% |
| 211 | 8 | 312 | 5.44% | 3.94% | 3.00% | 3.93% |
| 212 | 8 | 306 | 5.20% | 3.81% | 2.92% | 3.78% |
| 213 | 237 | 308 | 5.21% | 3.82% | 2.92% | 3.75% |
| 214 | 237 | 328 | 5.65% | 4.05% | 3.06% | 3.96% |
| 215 | 237 | 362 | 5.71% | 4.08% | 3.08% | 4.05% |
| 216 | 237 | 383 | 5.99% | 4.22% | 3.16% | 4.20% |
| 217 | 80 | 383 | 5.99% | 4.22% | 3.16% | 4.20% |
| 218 | 80 | 361 | 4.91% | 3.65% | 2.83% | 3.78% |
| 219 | 80 | | | | | |
| 220 | 80 | | | | | |

| Section # | County # | Linear | | | | Compound |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | Growth Factor |
| | | | Y1 | Y8 | Y16 | |
| 221 | 80 | | | | | |
| 222 | 102 | 51 | 0.61% | 0.59% | 0.56% | 0.62% |
| 223 | 102 | 143 | 1.60% | 1.44% | 1.29% | 1.50% |
| 224 | 102 | -15 | -0.10% | -0.10% | -0.10% | -0.09% |
| 225 | 102 | -105 | -0.70% | -0.73% | -0.78% | -0.71% |
| 226 | 102 | | | | | |
| 227 | 102 | | | | | |
| 228 | 102 | -688 | -2.73% | -3.37% | -4.61% | -3.57% |
| 229 | 102 | -676 | -2.58% | -3.15% | -4.20% | -3.29% |
| 230 | 102 | | | | | |
| 231 | 102 | | | | | |
| 232 | 102 | | | | | |
| 233 | 102 | | | | | |
| 234 | 102 | 61 | 0.44% | 0.42% | 0.41% | 0.45% |
| 235 | 102 | | | | | |
| 236 | 102 | | | | | |
| 237 | 102 | 104 | 0.68% | 0.65% | 0.61% | 0.69% |
| 238 | 102 | 371 | 2.79% | 2.34% | 1.97% | 2.29% |
| 239 | 102 | 252 | 2.10% | 1.83% | 1.60% | 1.77% |
| 240 | 102 | 342 | 2.93% | 2.43% | 2.03% | 2.39% |
| 241 | 102 | 349 | 3.35% | 2.71% | 2.23% | 2.65% |
| 242 | 102 | 528 | 5.93% | 4.19% | 3.14% | 3.88% |
| 243 | 102 | 451 | 5.81% | 4.13% | 3.10% | 4.09% |
| 244 | 102 | | | | | |
| 245 | 102 | | | | | |
| 246 | 102 | | | | | |
| 247 | 36 | 404 | 5.60% | 4.02% | 3.04% | 3.99% |
| 248 | 36 | 409 | 6.34% | 4.39% | 3.25% | 4.36% |
| 249 | 36 | 408 | 6.35% | 4.39% | 3.25% | 4.37% |
| 250 | 36 | 386 | 6.21% | 4.33% | 3.22% | 4.29% |
| 251 | 36 | 366 | 5.96% | 4.21% | 3.15% | 4.18% |
| 252 | 36 | 366 | 5.96% | 4.21% | 3.15% | 4.18% |
| 253 | 36 | 386 | 6.34% | 4.39% | 3.25% | 4.35% |
| 254 | 36 | 370 | 6.18% | 4.31% | 3.21% | 4.43% |
| 255 | 124 | 399 | 4.05% | 3.16% | 2.52% | 3.31% |
| 256 | 124 | | | | | |
| 257 | 124 | | | | | |
| 258 | 124 | | | | | |
| 259 | 124 | | | | | |
| 260 | 124 | | | | | |
| 261 | 124 | 381 | 6.29% | 4.37% | 3.24% | 4.51% |
| 262 | 124 | 406 | 6.36% | 4.40% | 3.26% | 4.56% |
| 263 | 124 | 529 | 6.61% | 4.52% | 3.32% | 4.80% |
| 264 | 181 | 442 | 6.36% | 4.40% | 3.25% | 4.62% |
| 265 | 181 | 407 | 6.00% | 4.22% | 3.16% | 4.40% |

| Section # | County # | Linear | | | | Compound Growth Factor |
|---|---|---|---|---|---|---|
| | | Growth Rate | Growth Factor | | | |
| | | | Y1 | Y8 | Y16 | |
| 266 | 181 | 394 | 6.37% | 4.41% | 3.26% | 4.57% |
| 267 | 181 | 353 | 5.81% | 4.13% | 3.10% | 4.21% |
| 268 | 181 | 352 | 5.98% | 4.21% | 3.15% | 4.24% |
| 269 | 181 | 339 | 5.71% | 4.08% | 3.08% | 4.09% |
| 270 | 181 | 377 | 5.94% | 4.19% | 3.14% | 4.25% |
| 271 | 181 | 378 | 6.40% | 4.42% | 3.27% | 4.51% |
| 272 | 181 | 355 | 5.99% | 4.22% | 3.16% | 4.28% |
| 273 | 181 | 168 | 2.65% | 2.23% | 1.90% | -0.70% |
| 274 | 181 | 345 | 6.24% | 4.34% | 3.22% | 4.49% |

## Table B2: IH 10 Traffic Growth in Percentiles

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 42 | 2.05% |
| 20 | 89 | 3.12% |
| 30 | 99 | 3.60% |
| 40 | 116 | 4.05% |
| 50 | 201 | 4.51% |
| 60 | 266 | 4.89% |
| 70 | 327 | 5.34% |
| 80 | 352 | 5.65% |
| 90 | 406 | 6.91% |
| 95 | 528 | 7.06% |
| 97.5 | 592 | 7.18% |
| mean | 223 | 4.40% |

## Table B3: IH 20 Traffic Growth in Percentiles

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 99 | 2.27% |
| 20 | 120 | 2.84% |
| 30 | 130 | 3.04% |
| 40 | 147 | 3.30% |
| 50 | 169 | 3.60% |
| 60 | 199 | 3.80% |
| 70 | 253 | 4.06% |
| 80 | 305 | 4.71% |
| 90 | 411 | 5.18% |
| 95 | 648 | 6.81% |
| 97.5 | 672 | 7.03% |
| mean | 223 | 3.74% |

## Table B4: IH35 Traffic Growth in Percentiles

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 238 | 4.53% |
| 20 | 333 | 4.99% |
| 30 | 415 | 5.41% |
| 40 | 496 | 5.95% |
| 50 | 542 | 6.53% |
| 60 | 581 | 7.16% |
| 70 | 620 | 7.43% |
| 80 | 645 | 7.67% |
| 90 | 735 | 8.42% |
| 95 | 859 | 11.68% |
| 97.5 | 936 | 11.95% |
| mean | 516 | 6.69% |

## Table B5: US 59 Traffic Growth in Percentiles

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 31 | 1.52% |
| 20 | 47 | 1.89% |
| 30 | 55 | 2.34% |
| 40 | 65 | 3.41% |
| 50 | 89 | 3.93% |
| 60 | 125 | 4.25% |
| 70 | 196 | 4.60% |
| 80 | 238 | 4.90% |
| 90 | 332 | 5.20% |
| 95 | 493 | 5.76% |
| 97.5 | 604 | 6.04% |
| mean | 153 | 3.57% |

**Table B6: US 82 Traffic Growth in Percentiles**

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 4 | 0.98% |
| 20 | 7 | 1.38% |
| 30 | 9 | 1.67% |
| 40 | 15 | 2.07% |
| 50 | 18 | 2.29% |
| 60 | 20 | 2.73% |
| 70 | 23 | 2.90% |
| 80 | 28 | 3.20% |
| 90 | 36 | 3.49% |
| 95 | 42 | 3.77% |
| 97.5 | 65 | 4.06% |
| Mean | 21 | 2.28% |

**Table B7: US 281 Traffic Growth in Percentiles**

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 7 | 0.93% |
| 20 | 10 | 1.60% |
| 30 | 16 | 2.05% |
| 40 | 21 | 3.05% |
| 50 | 23 | 3.78% |
| 60 | 27 | 4.75% |
| 70 | 48 | 5.23% |
| 80 | 60 | 5.76% |
| 90 | 94 | 6.56% |
| 95 | 134 | 6.95% |
| 97.5 | 246 | 7.86% |
| Mean | 44 | 3.81% |

**Table B8: US 290 Traffic Growth in Percentiles**

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 13 | 4.16% |
| 20 | 22 | 4.74% |
| 30 | 27 | 5.81% |
| 40 | 51 | 6.07% |
| 50 | 71 | 6.28% |
| 60 | 79 | 6.45% |
| 70 | 82 | 6.54% |
| 80 | 113 | 6.75% |
| 90 | 153 | 7.18% |
| 95 | 452 | 7.49% |
| 97.5 | 512 | 7.69% |
| Mean | 91 | 5.96% |

**Table B9: SH 16 Traffic Growth in Percentiles**

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 5 | 2.68% |
| 20 | 9 | 3.65% |
| 30 | 10 | 4.51% |
| 40 | 12 | 5.43% |
| 50 | 15 | 5.79% |
| 60 | 19 | 6.20% |
| 70 | 25 | 6.66% |
| 80 | 33 | 7.36% |
| 90 | 48 | 8.81% |
| 95 | 57 | 11.51% |
| 97.5 | 64 | 12.46% |
| Mean | 21 | 5.79% |

**Table B10: SH 71 Traffic Growth in Percentiles**

| percentile (nth) | GR | GF |
|---|---|---|
| 10 | 2 | 0.47% |
| 20 | 4 | 0.84% |
| 30 | 4 | 1.14% |
| 40 | 5 | 1.26% |
| 50 | 8 | 1.49% |
| 60 | 15 | 2.07% |
| 70 | 20 | 4.86% |
| 80 | 34 | 5.61% |
| 90 | 74 | 6.15% |
| 95 | 81 | 6.61% |
| 97.5 | 83 | 6.83% |
| mean | 22 | 2.78% |

# Appendix C: Traffic Growth Figures

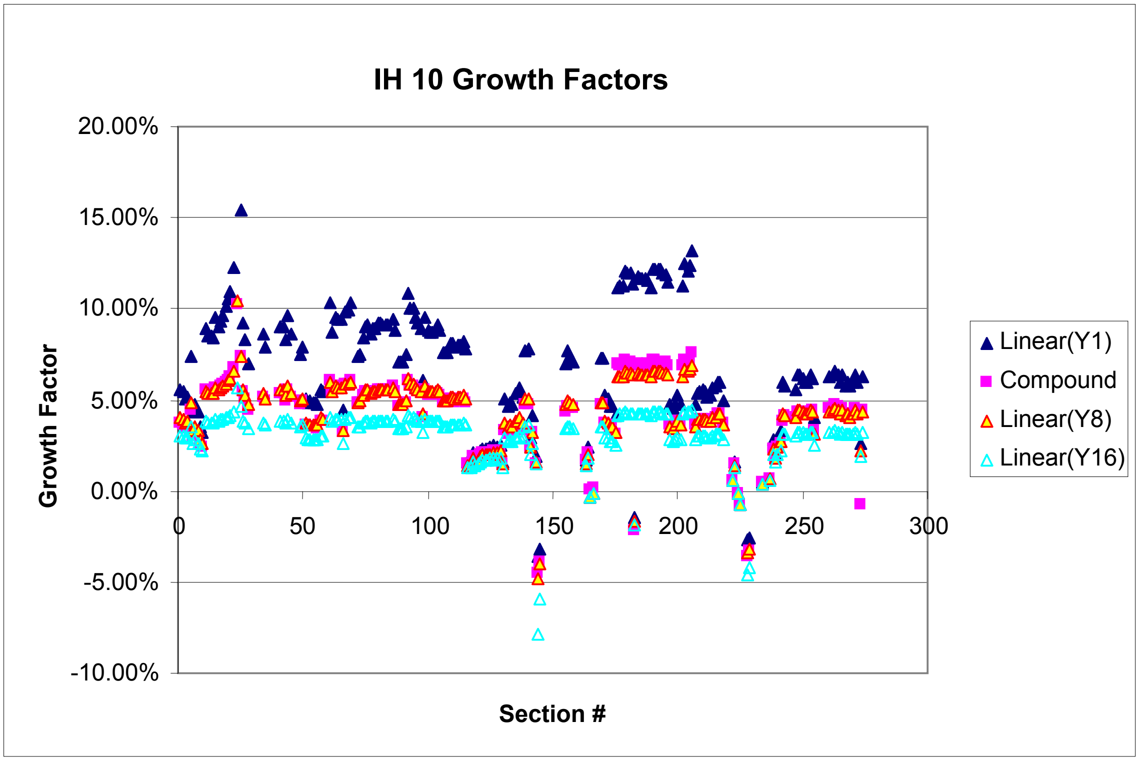


*Figure C1: IH 10 Growth Factors of Individual Sections from West to East*

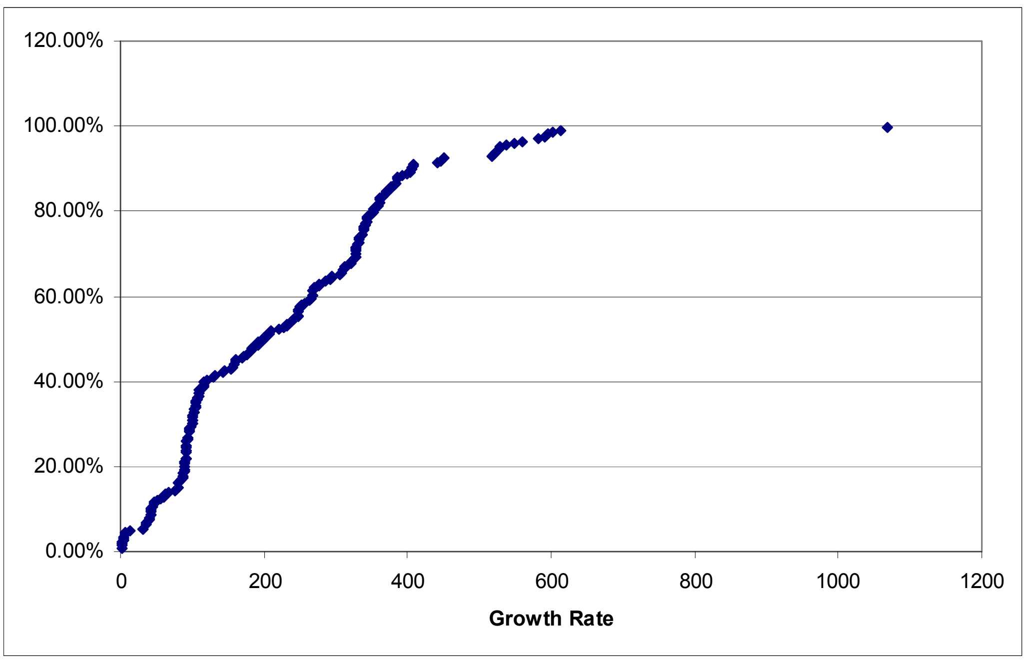


*Figure C2: IH 10 Growth Rate CDF*

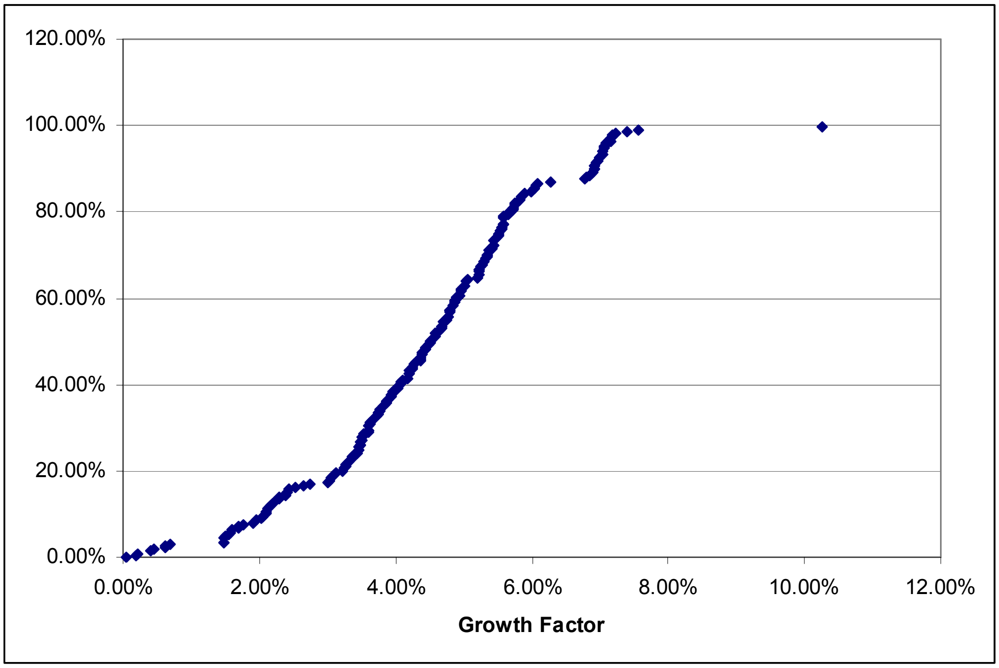


*Figure C3: IH 10 Growth Factor CDF*

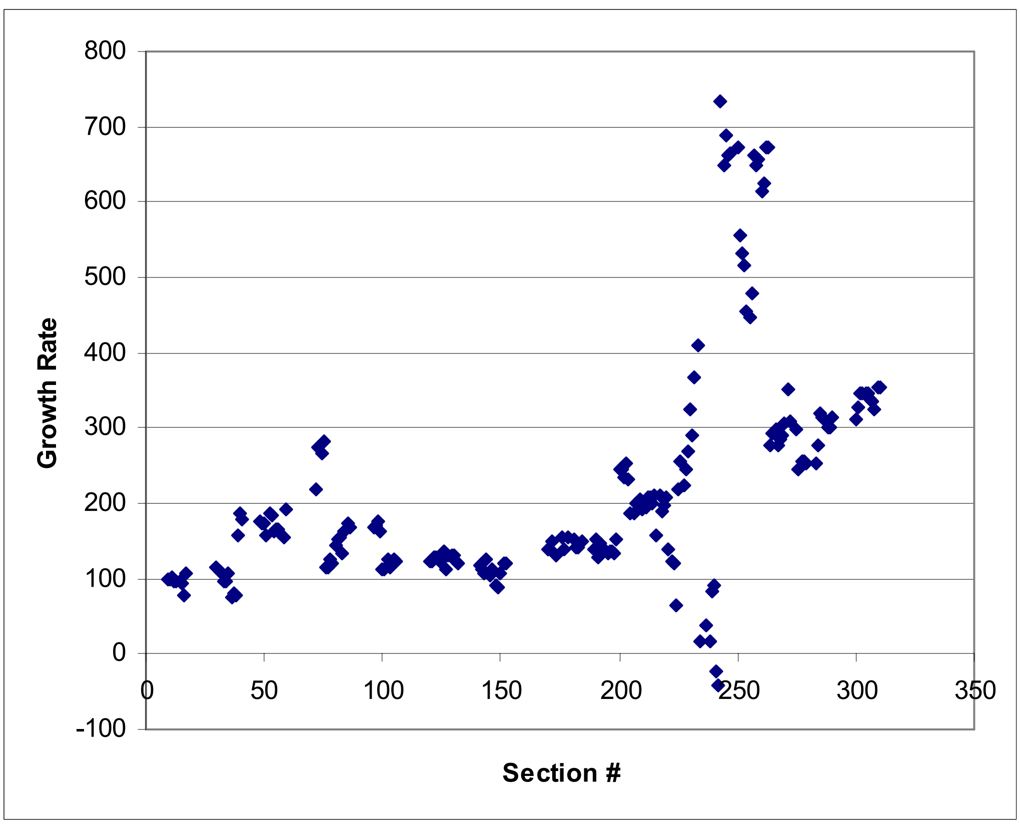


*Figure C4: Growth Rates of IH 20 along Highway from West to East*

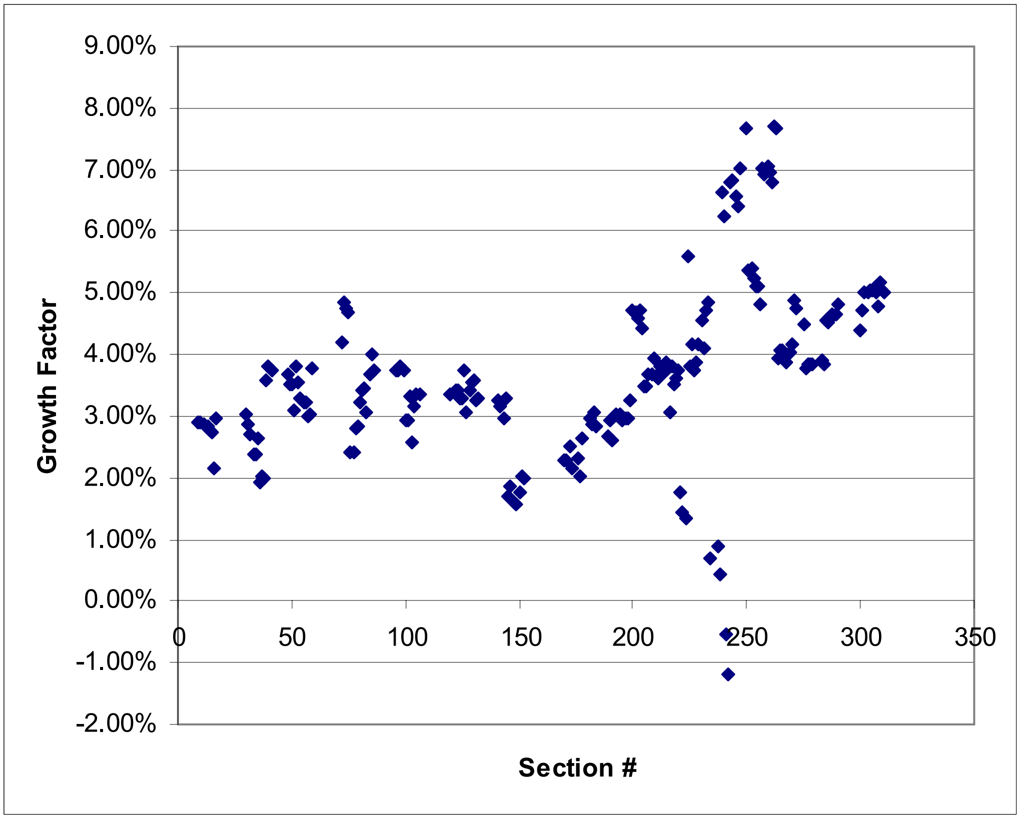


*Figure C5: Growth Factors of IH 20 along Highway from West to East*

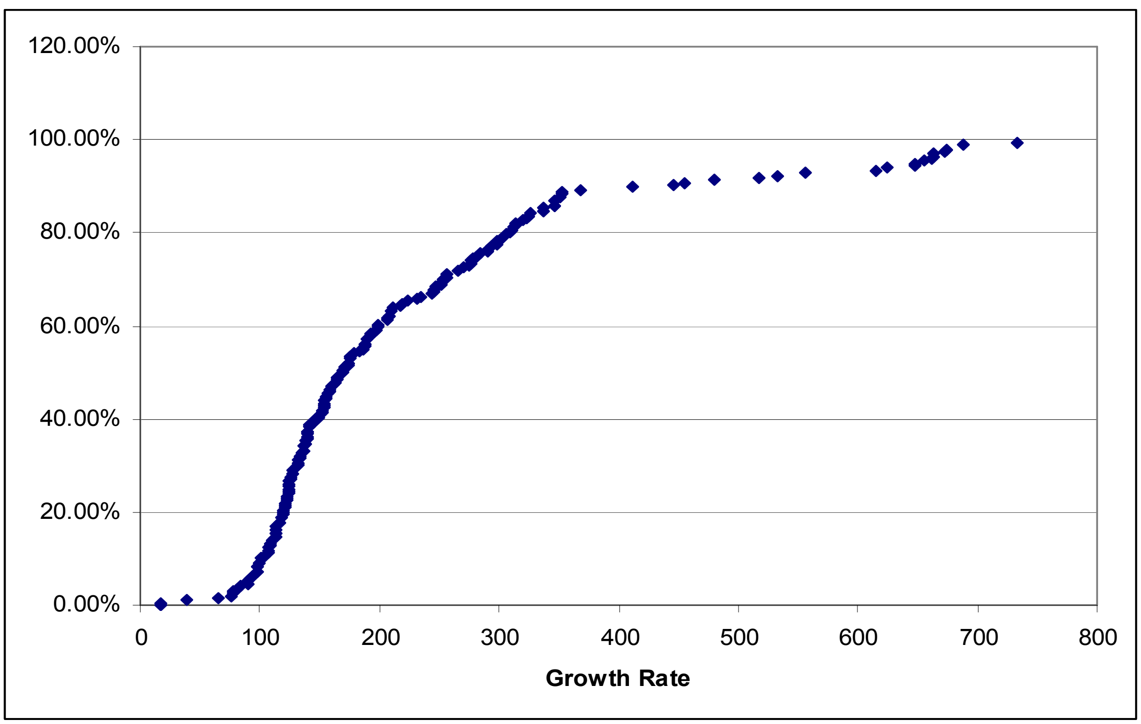


*Figure C6: Growth Rates of IH 20 CDF*

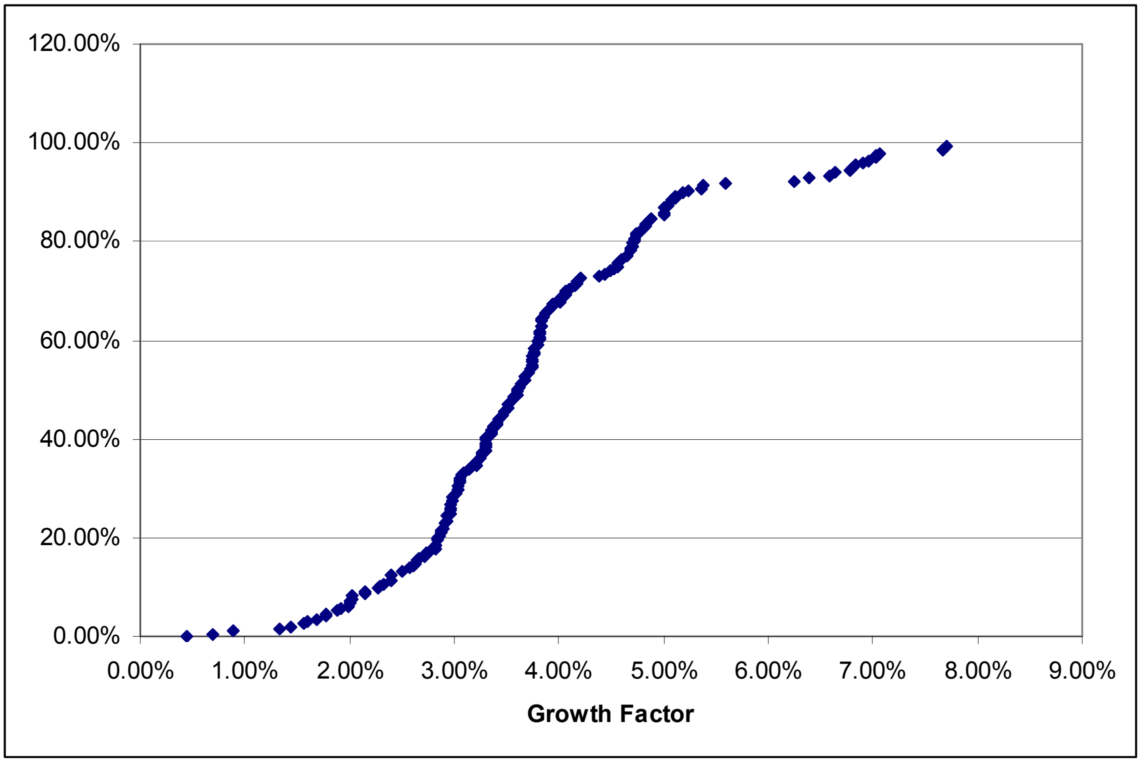


*Figure C7: Growth Factors of IH 20 CDF*

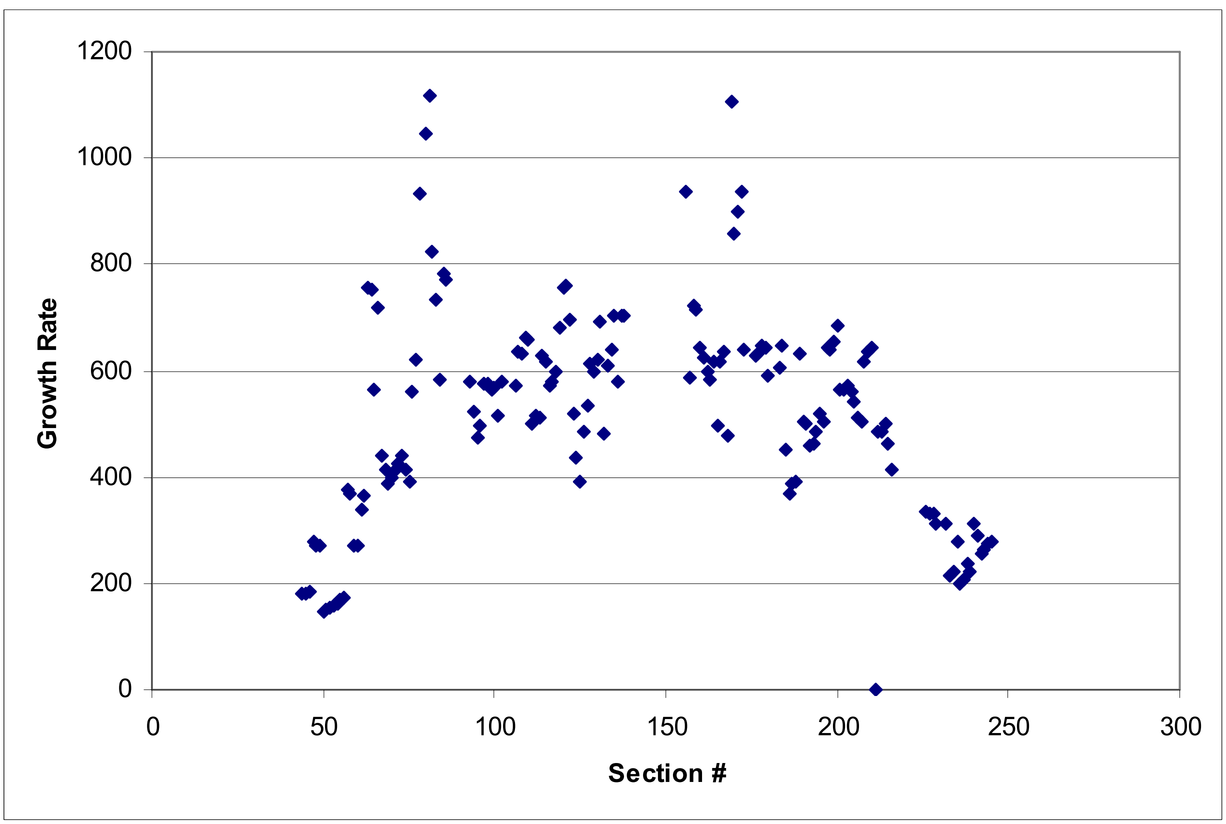


*Figure C8: Growth Rates of IH 35 along Highway from South to North*

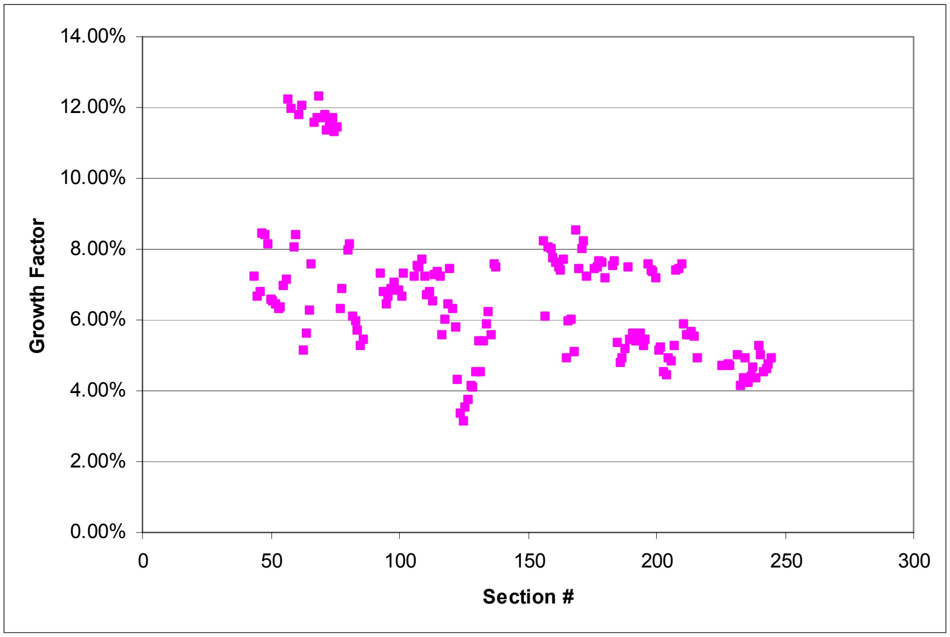


*Figure C9: Growth Factors of IH 35 along Highway from South to North*

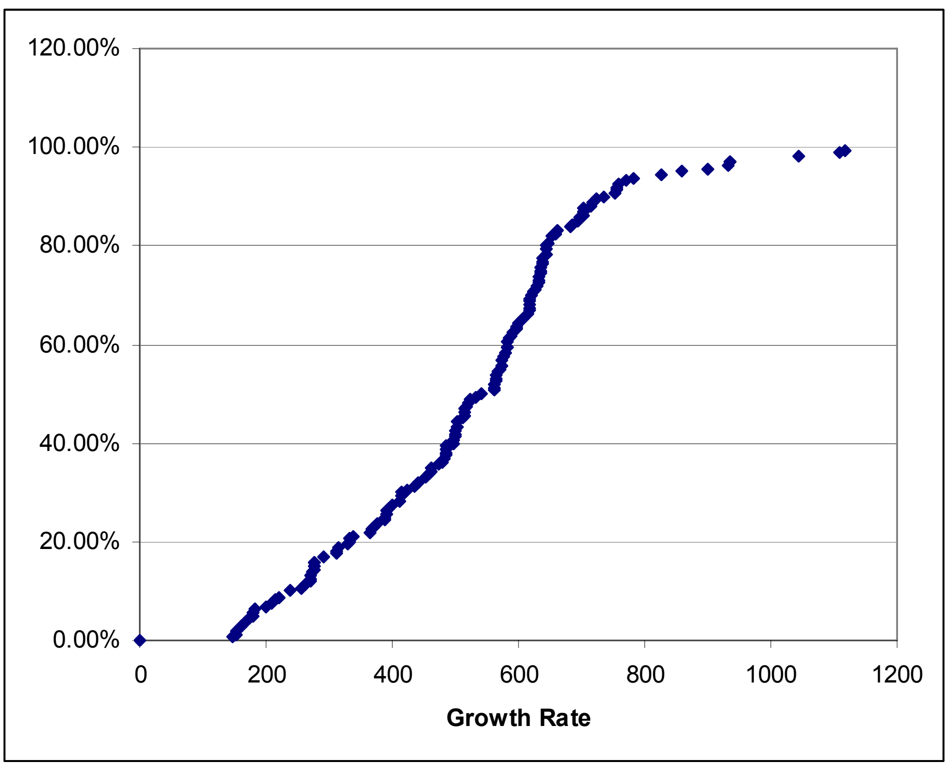


*Figure C10: IH 35 Growth Rate CDF*

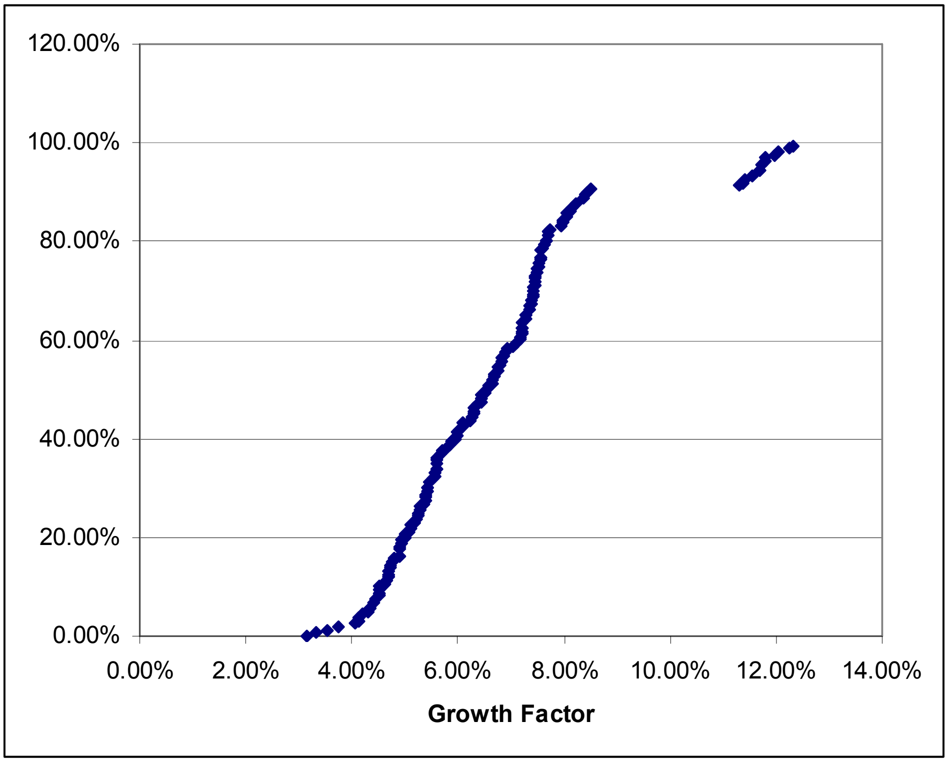


*Figure C11: IH 35 Growth Factor CDF*

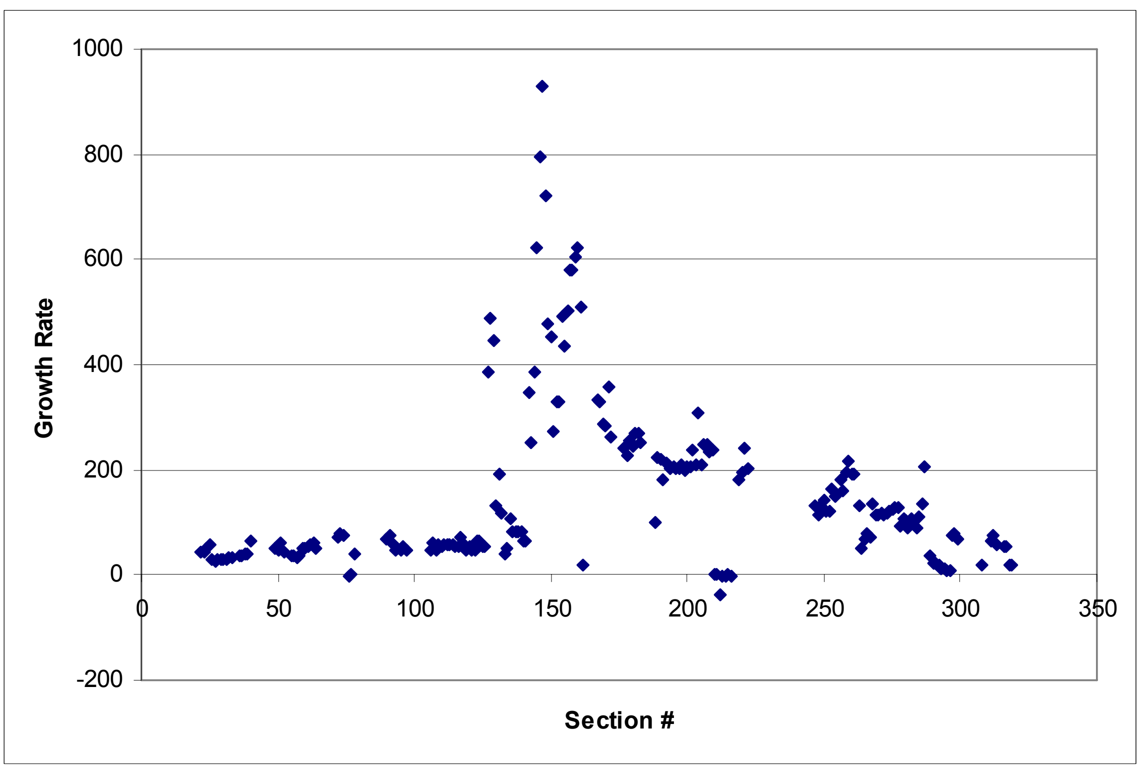


*Figure C12: Growth Rate of US 59 along Highway from South to North/Northeast*

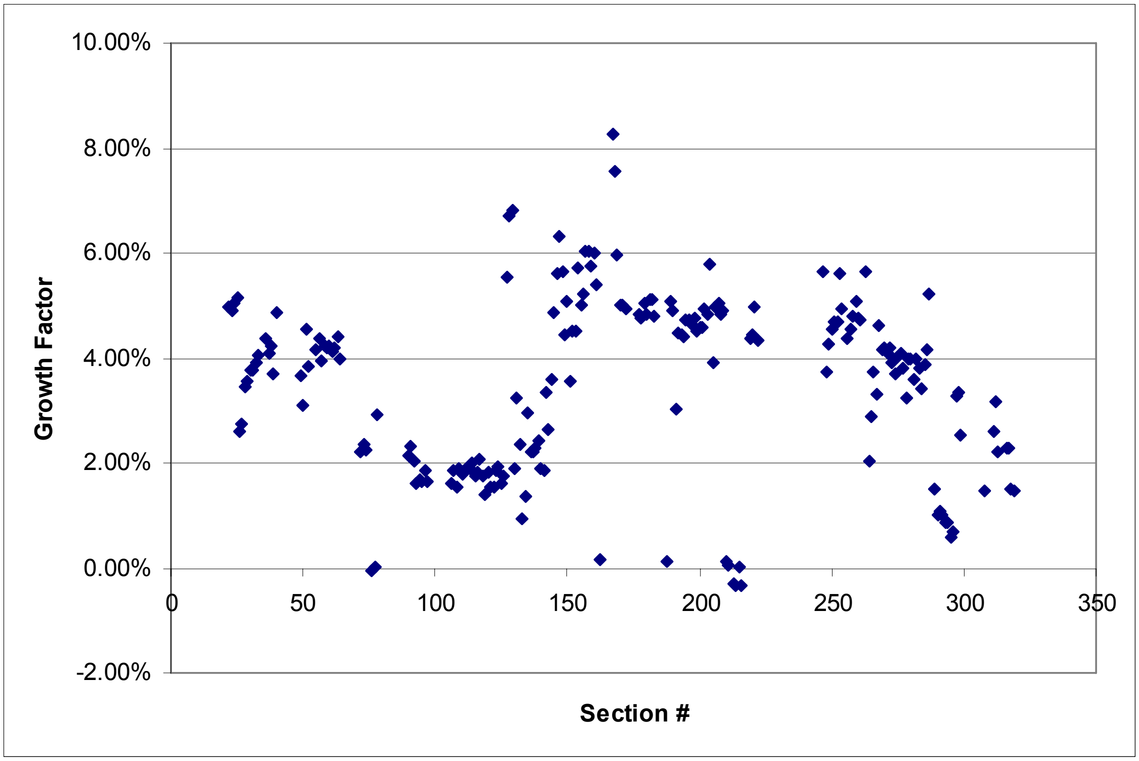


*Figure C13: Growth Factor of US 59 along Highway from South to North/Northeast*

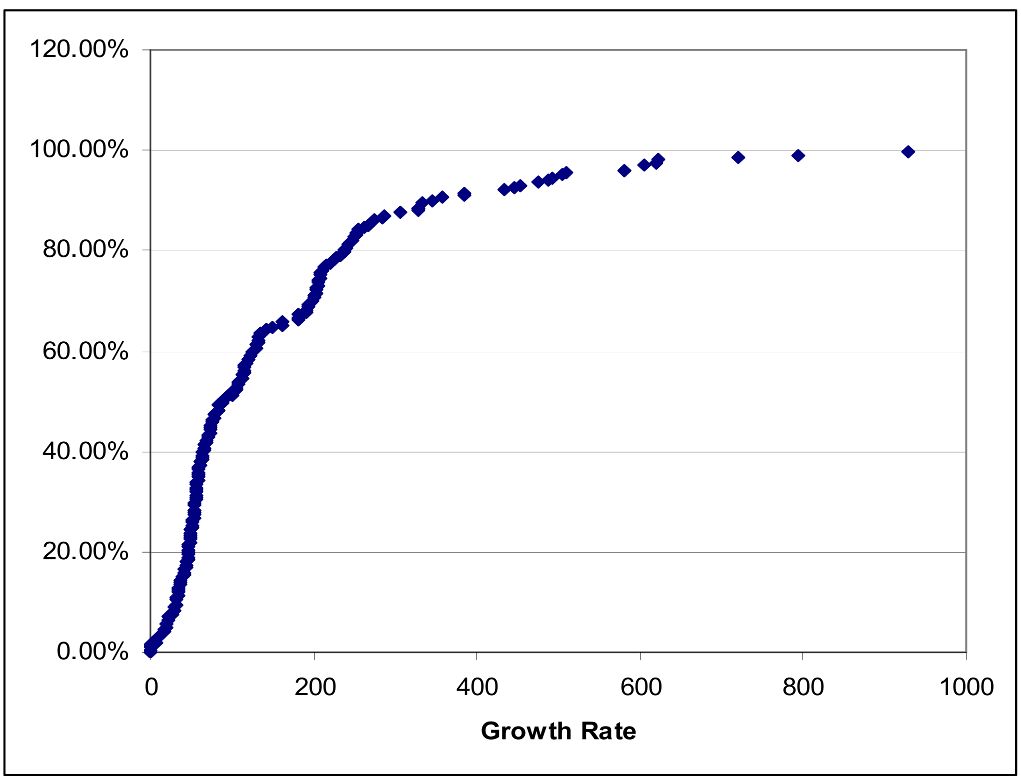


*Figure C14: US 59 Growth Rate CDF*

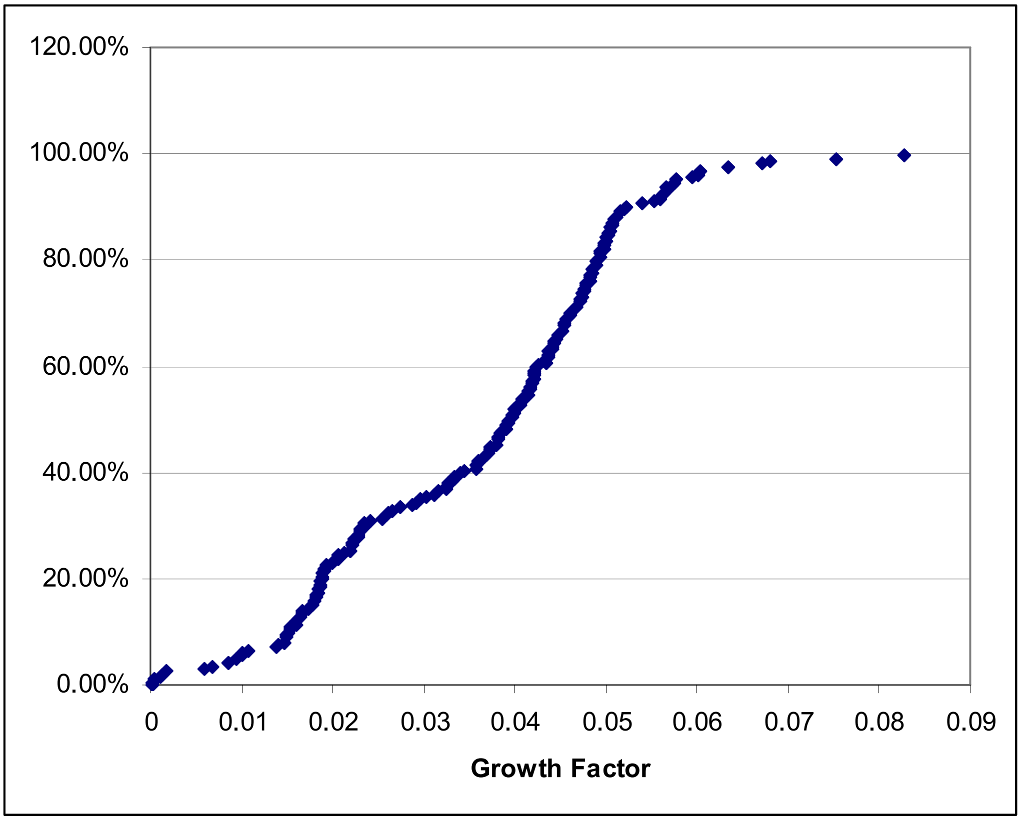


*Figure C15: US 59 Growth Factor CDF*

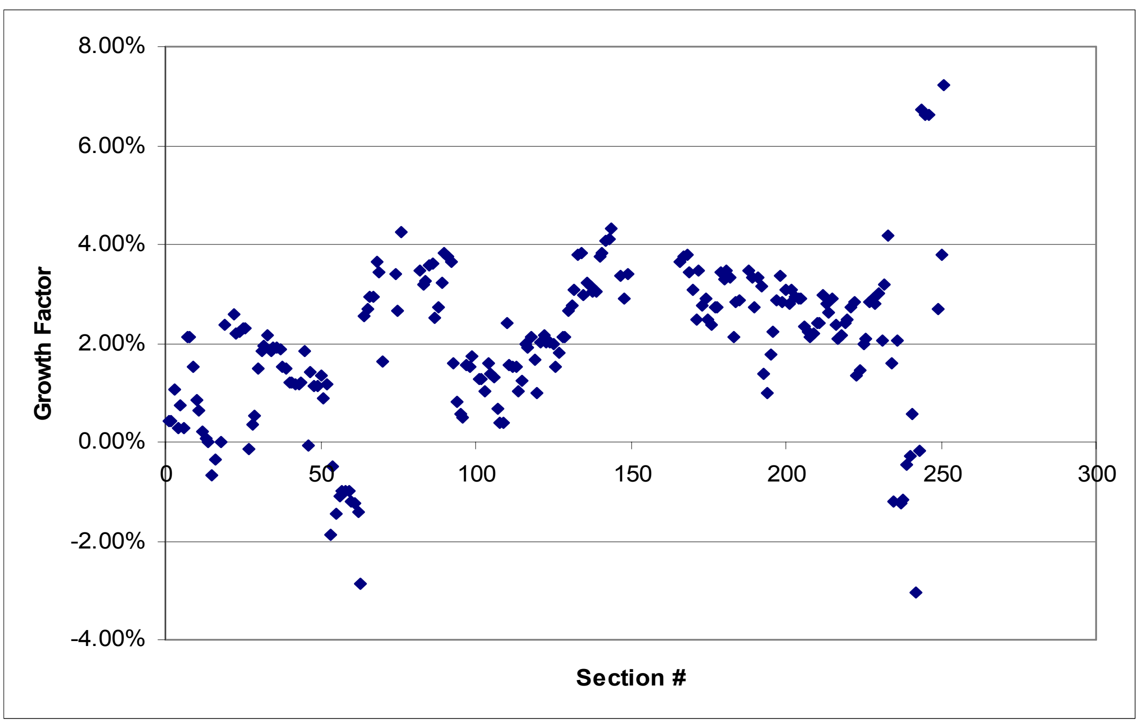


*Figure C16: Growth Factors of US 82 along Highway from South to Northeast*

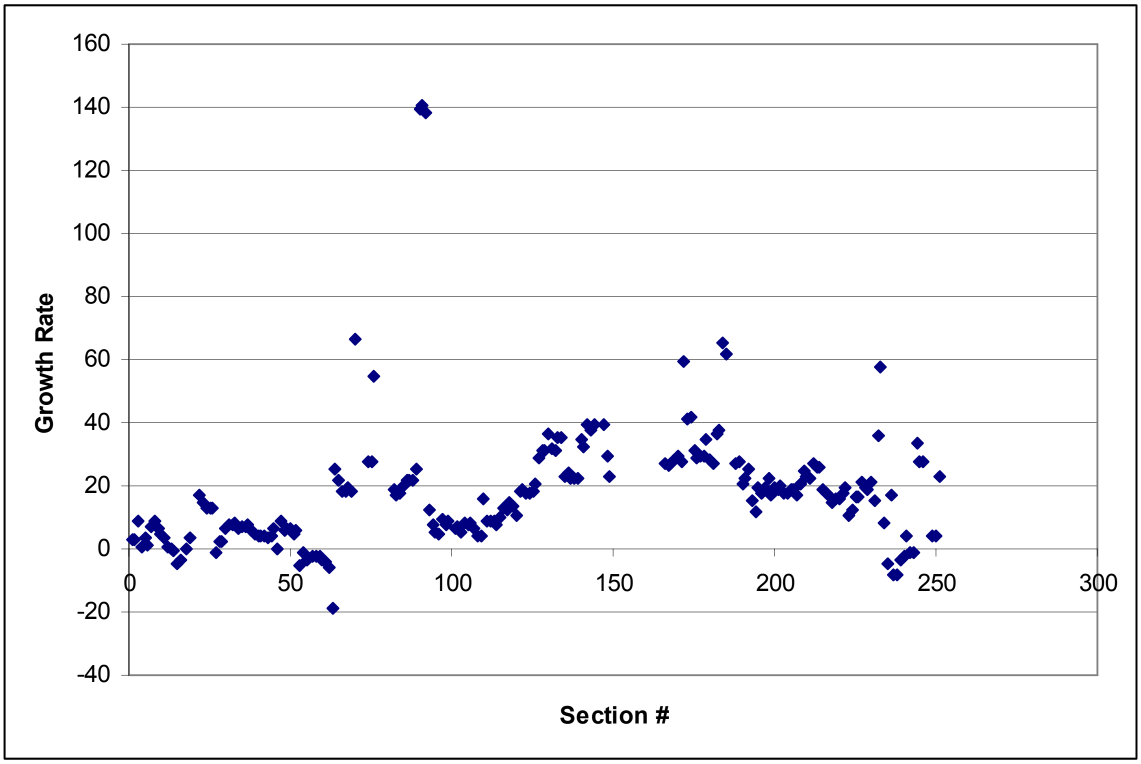


*Figure C17: Growth Rates of US 82 along Highway from South to Northeast*

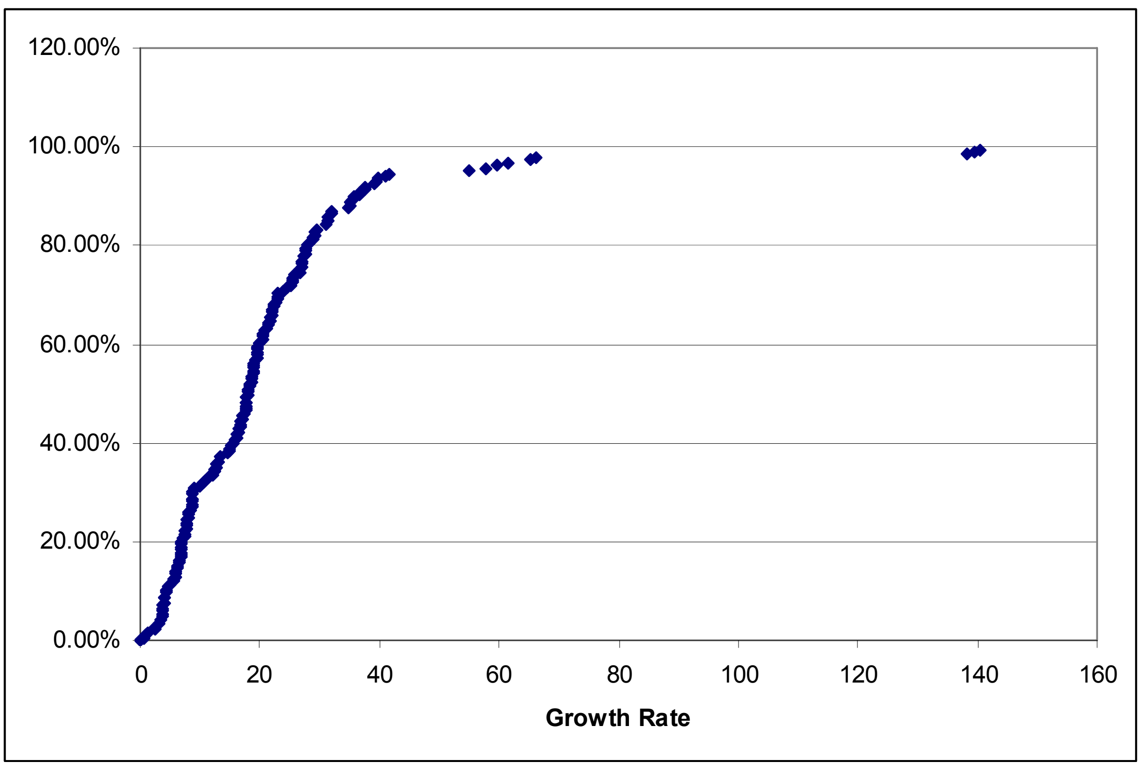


*Figure C18: US 82 Growth Rate CDF*

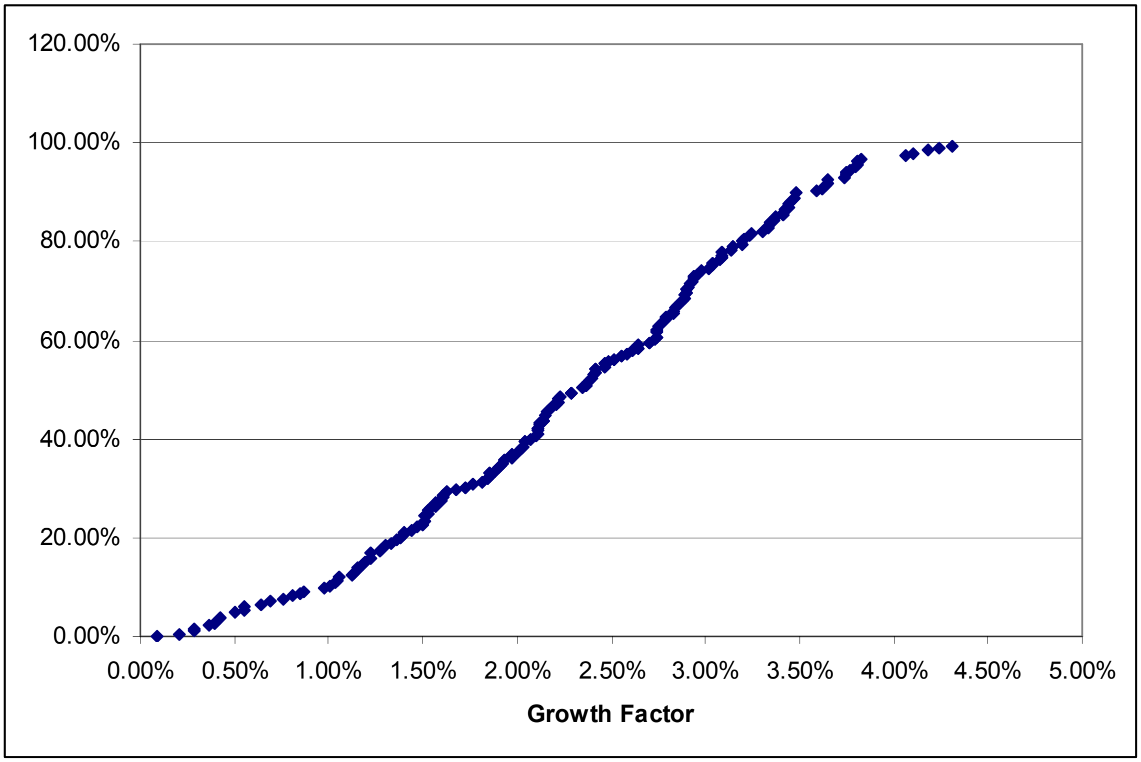


*Figure C19: US 82 Growth Factor CDF*

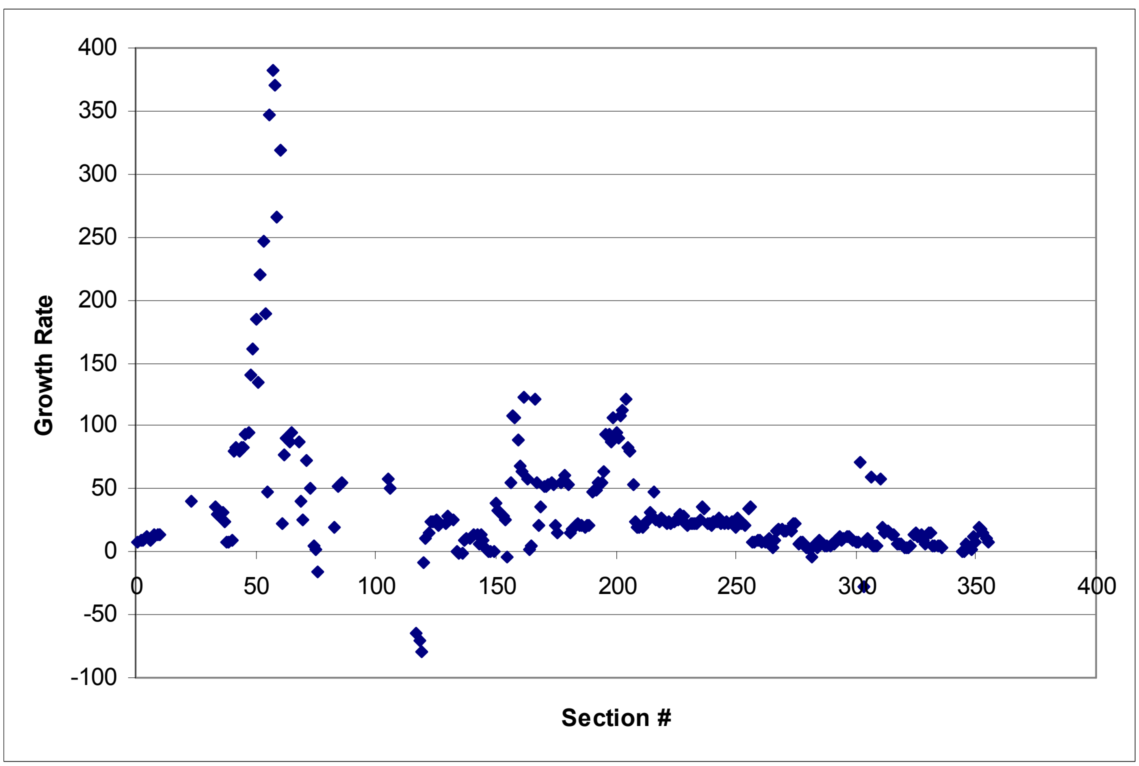


*Figure C20: US 281 Growth Rate along Highway from South to North*

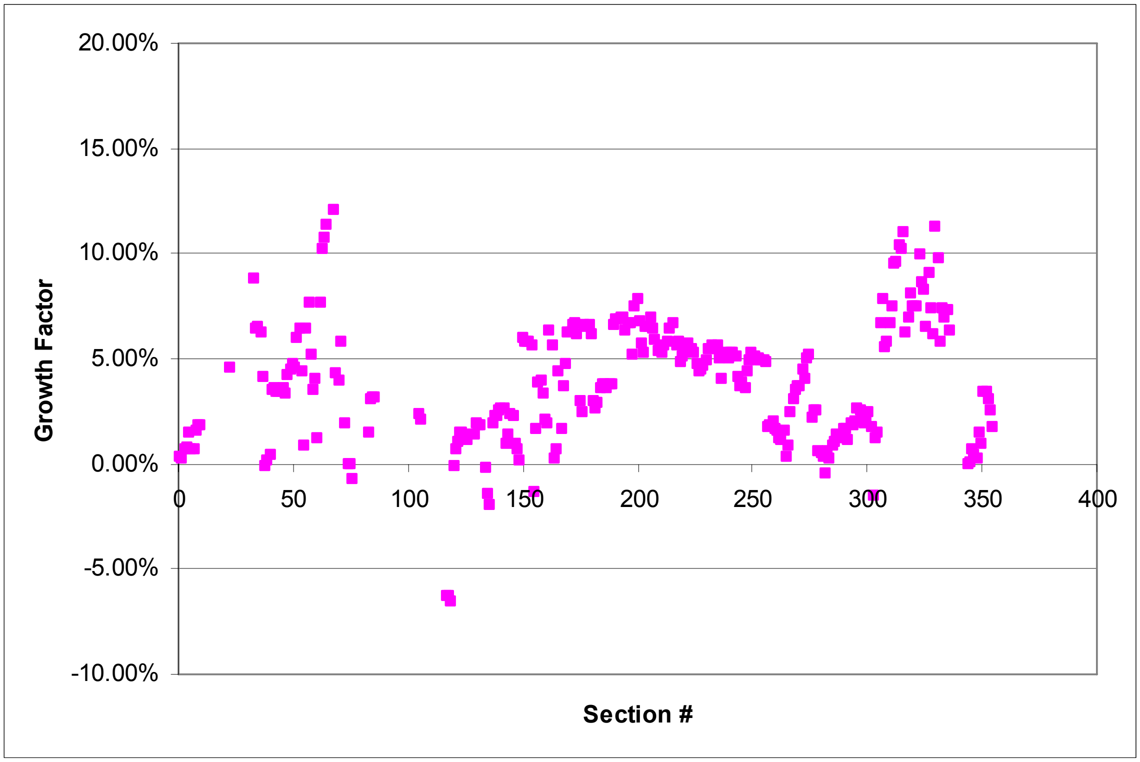


*Figure C21: US 281 Growth Factor along Highway from South to North*

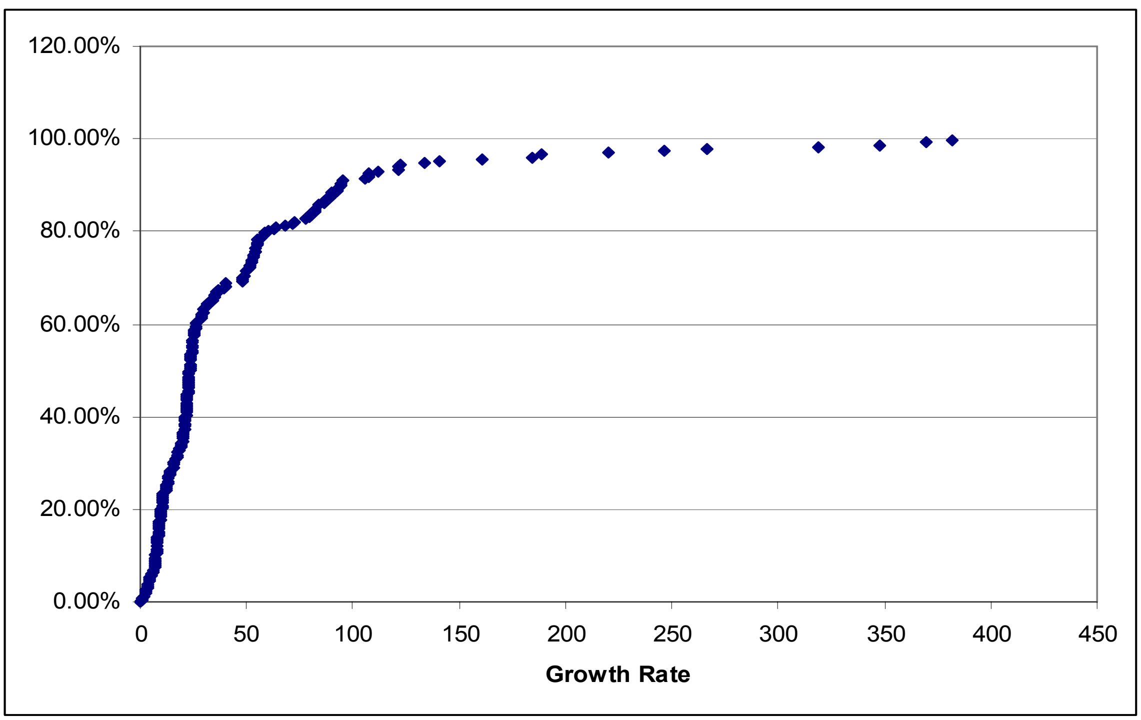


*Figure C22: US 281 Growth Rate CDF*

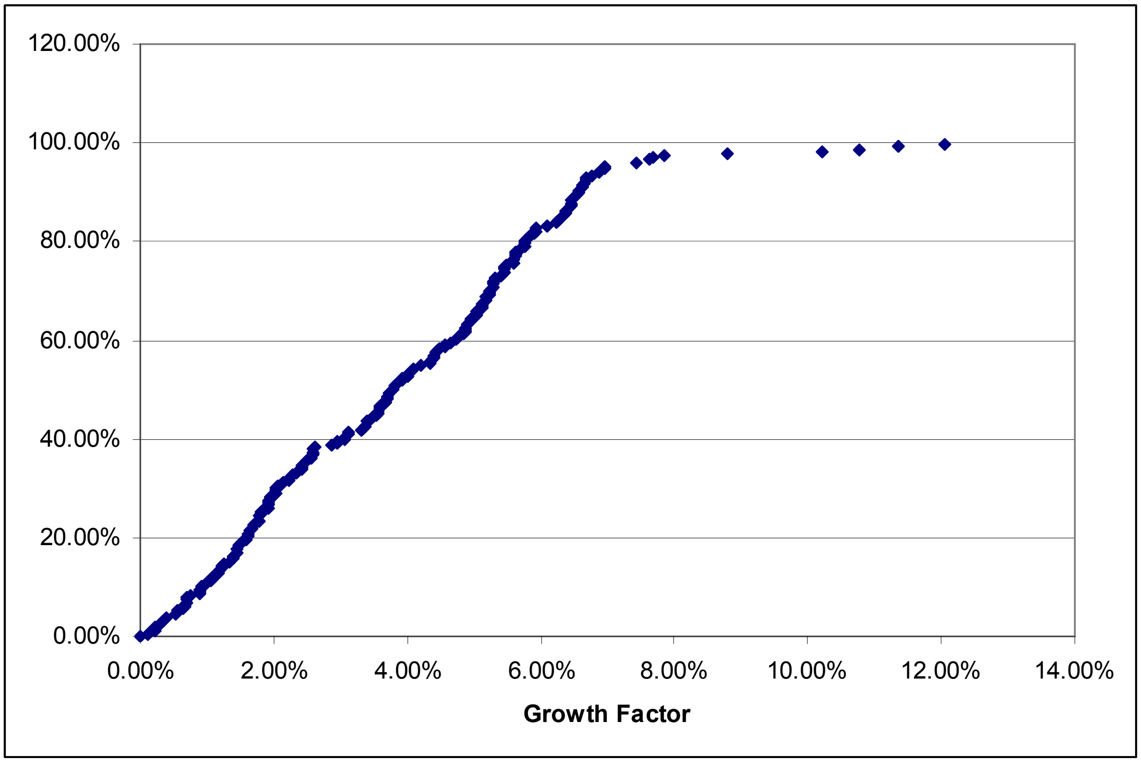


*Figure C23: US 281 Growth Factor CDF*

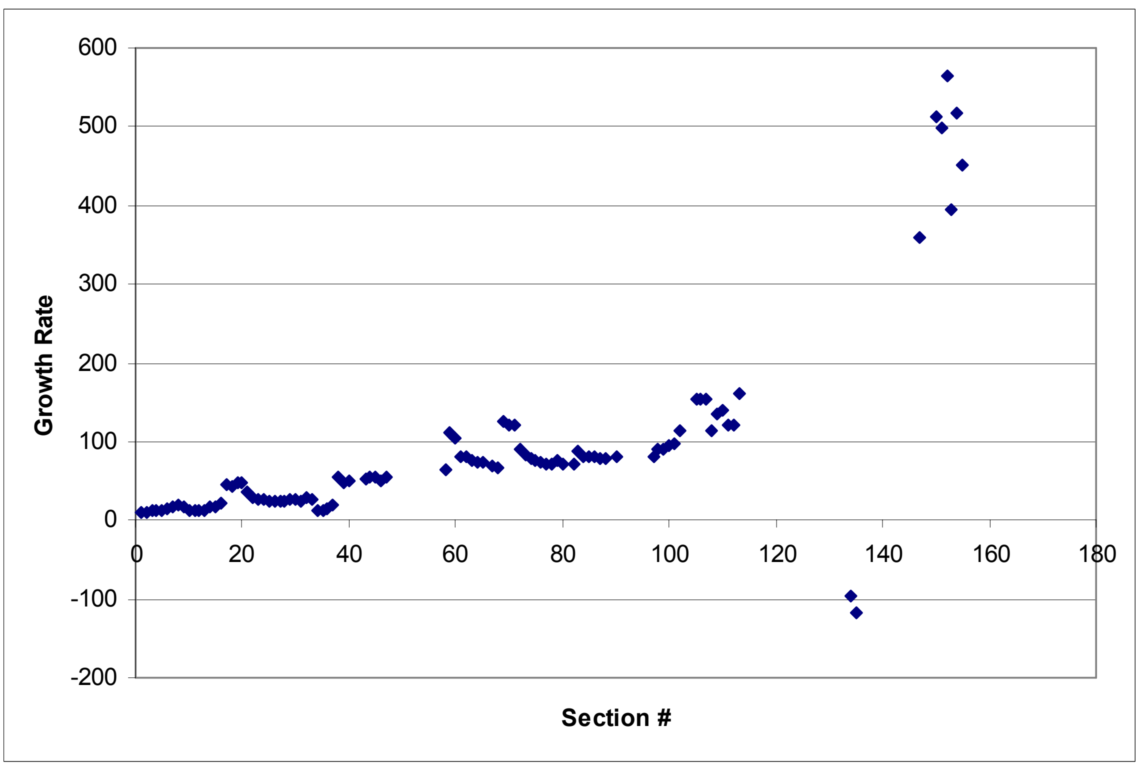


*Figure C24: US 290 Growth Rate along Highway from West to East*

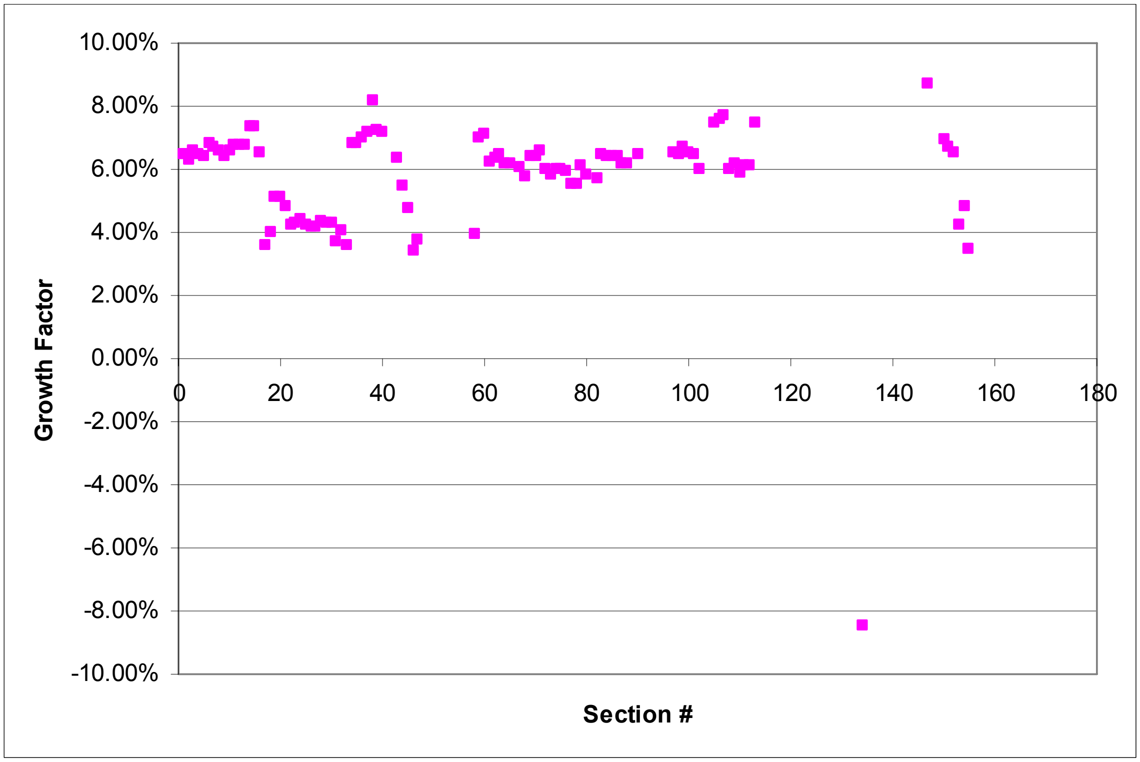


*Figure C25: US 290 Growth Factor along Highway from West to East*

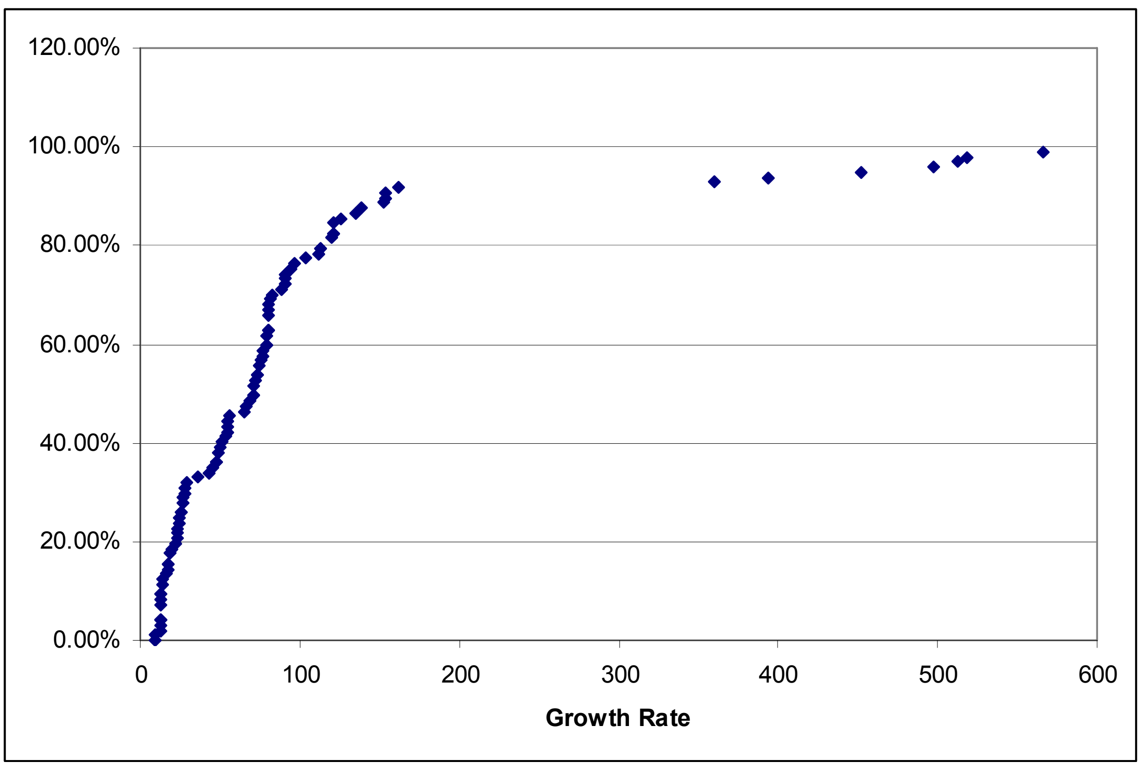


*Figure C26: US 290 Growth Rate CDF*

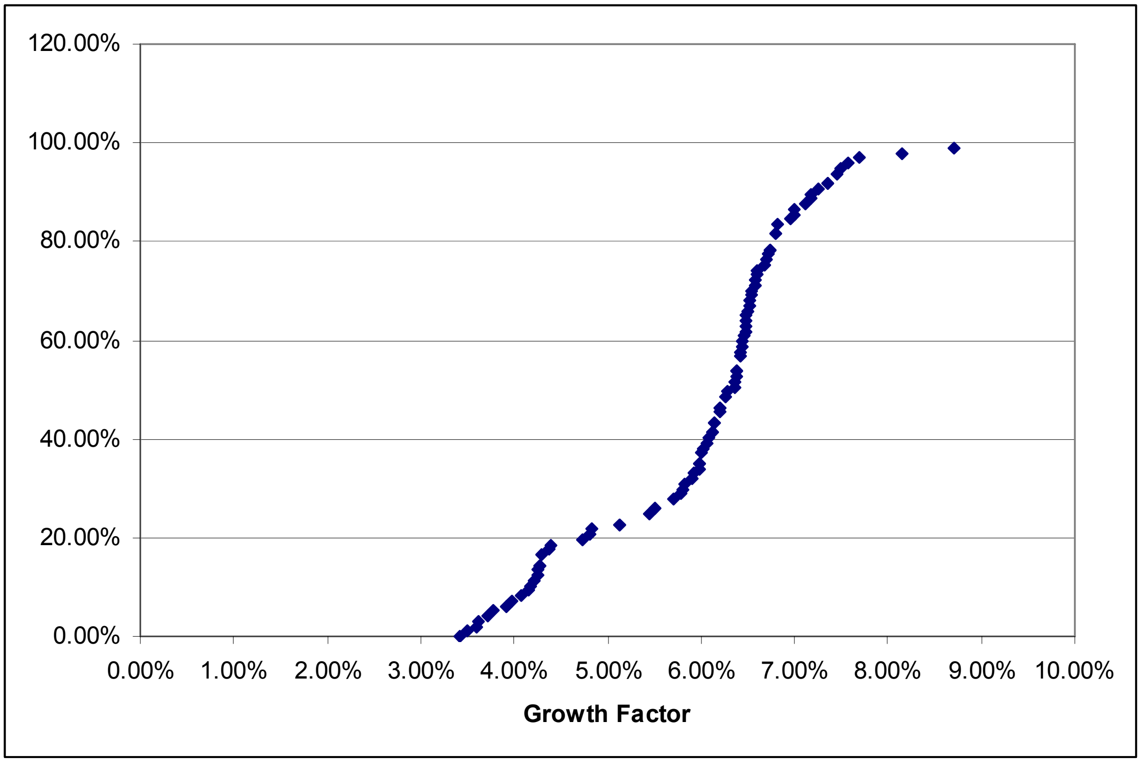


*Figure C27: US 290 Growth Factor CDF*

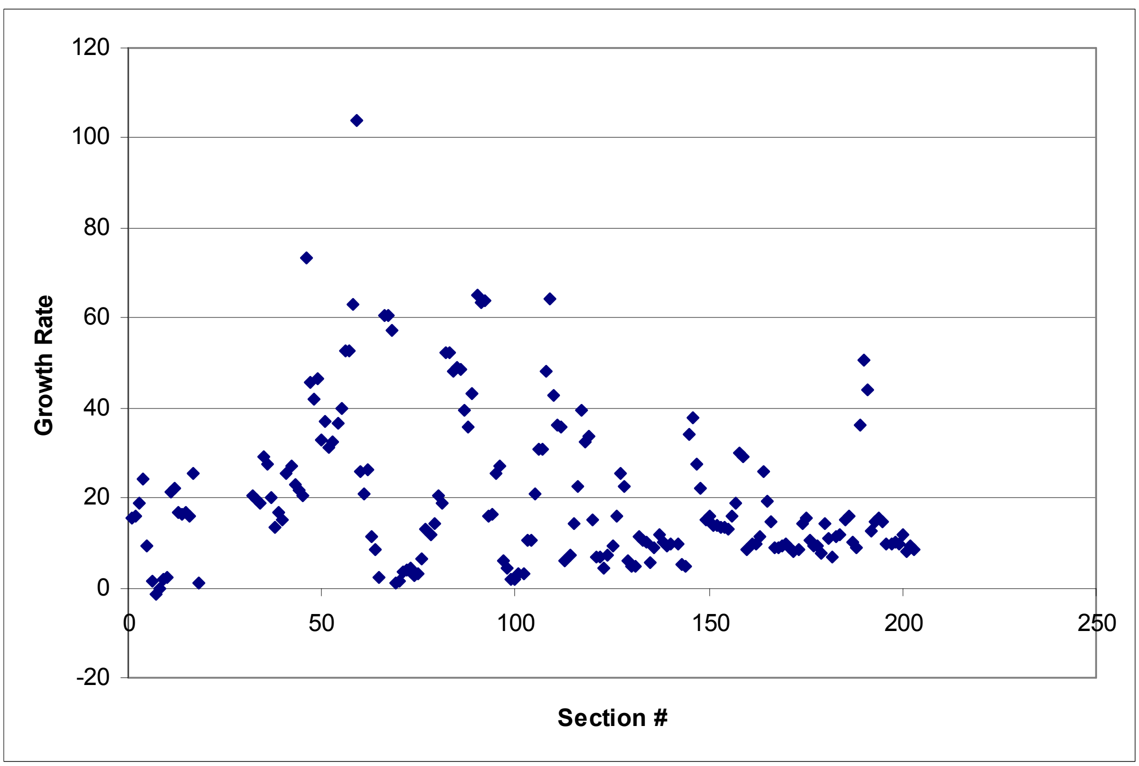


*Figure C28: SH 16 Growth Rate along Highway from South to North*

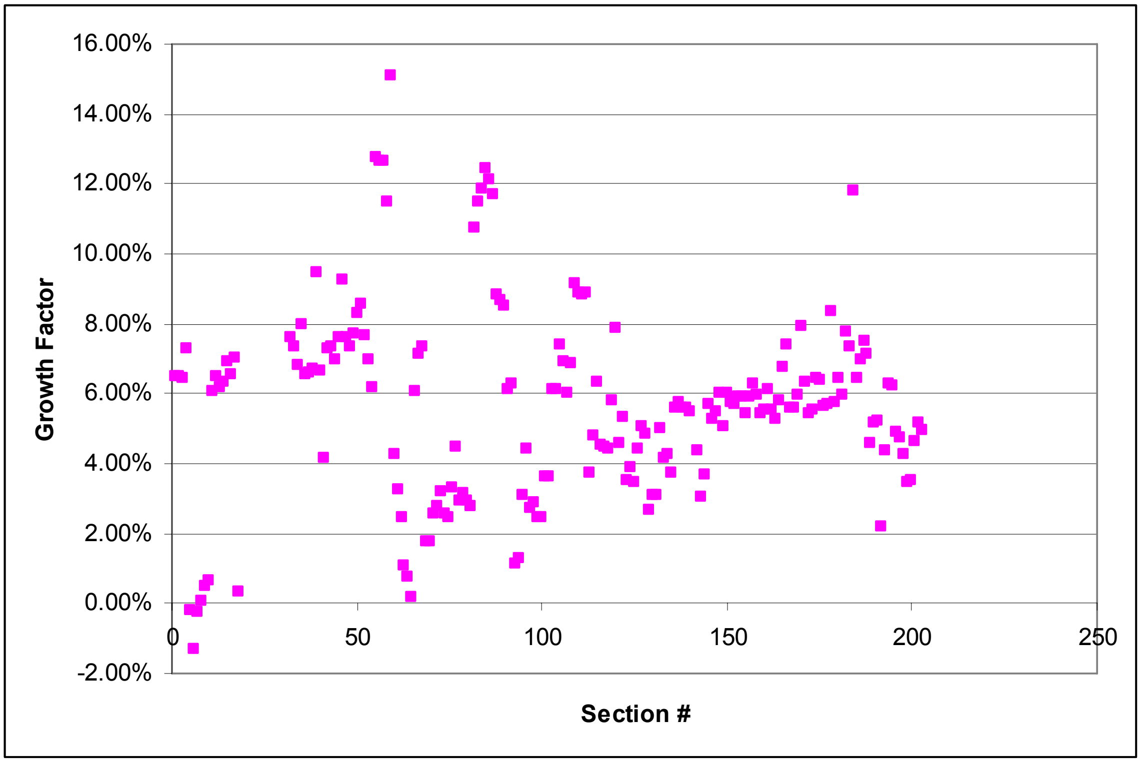


*Figure C29: SH 16 Growth Factor along Highway from South to North*

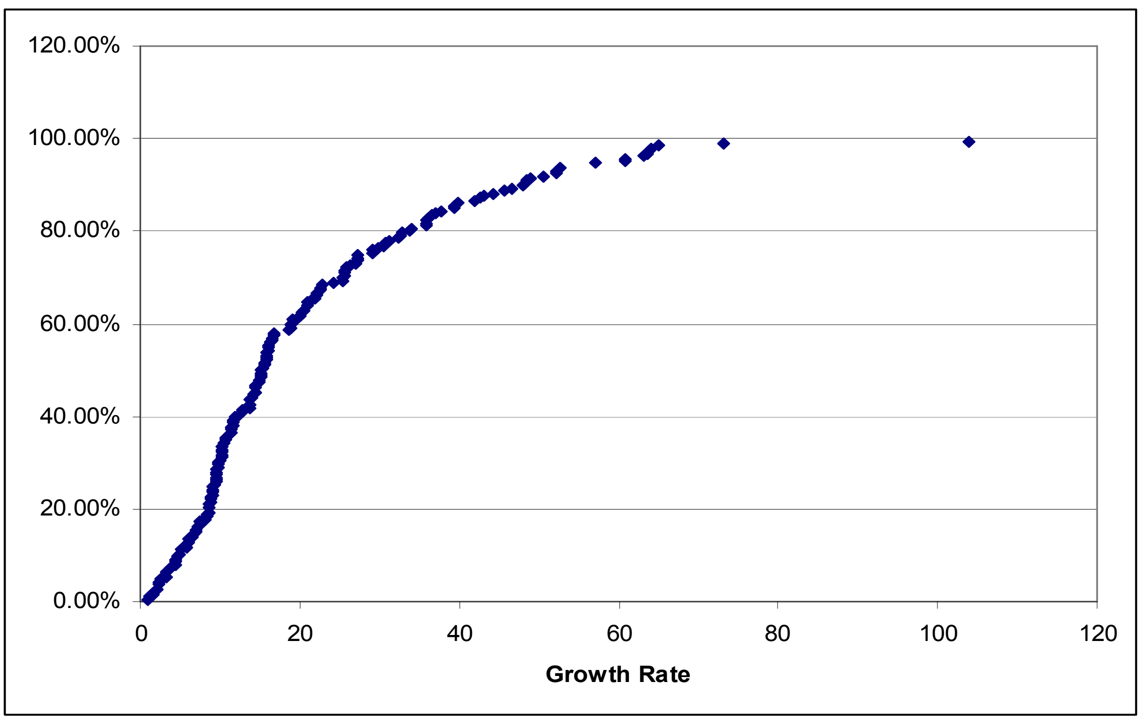


*Figure C30: SH 16 Growth Rate CDF*

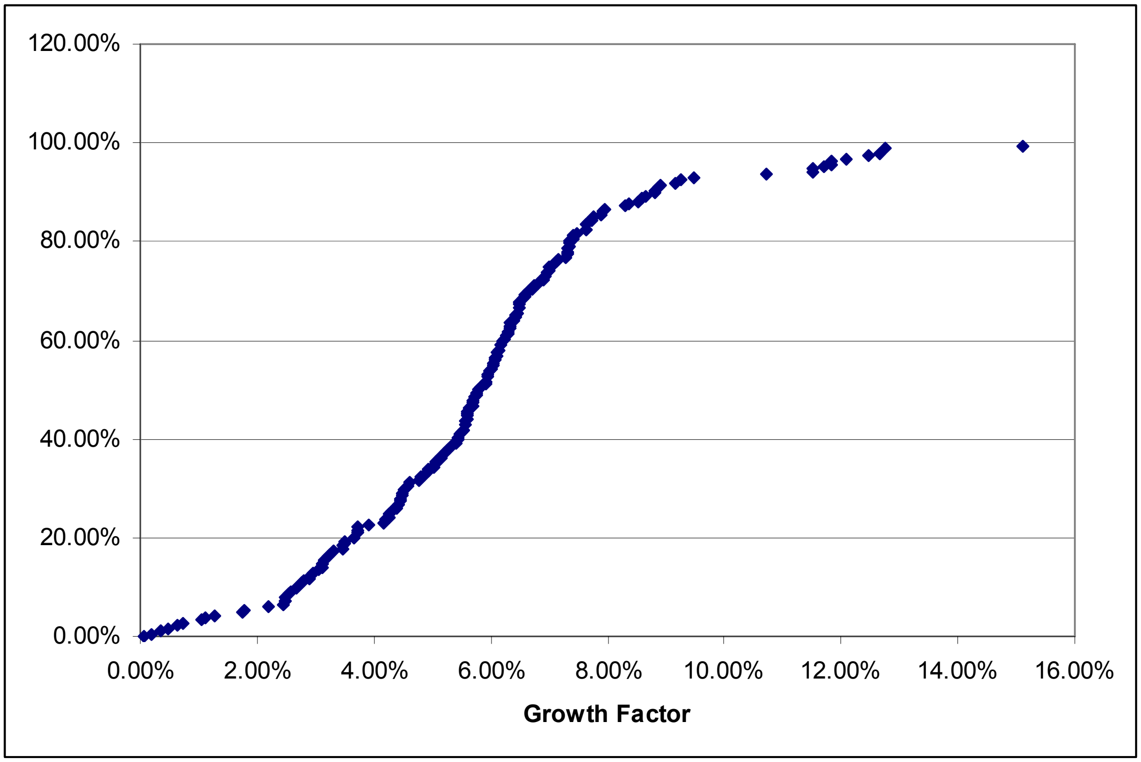


*Figure C31: SH 16 Growth Factor CDF*

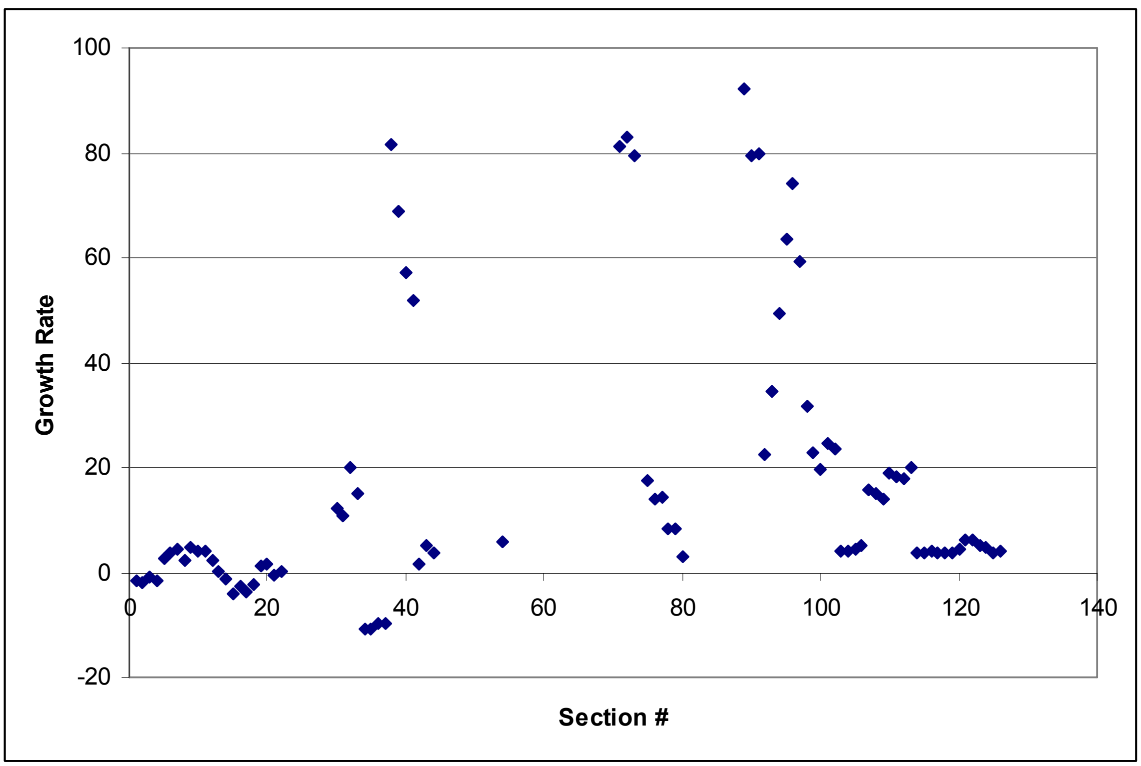


*Figure C32: SH 71 Growth Rate along Highway from West to East*

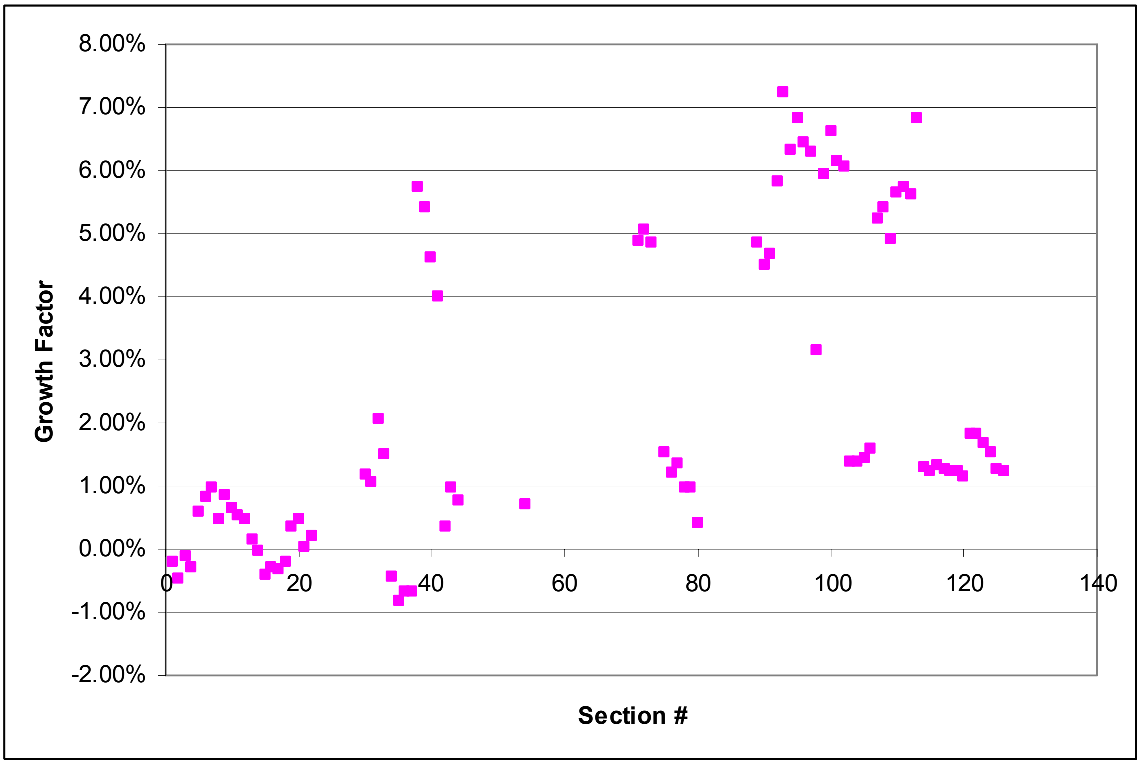


*Figure C33: SH 71 Growth Factor along Highway from West to East*

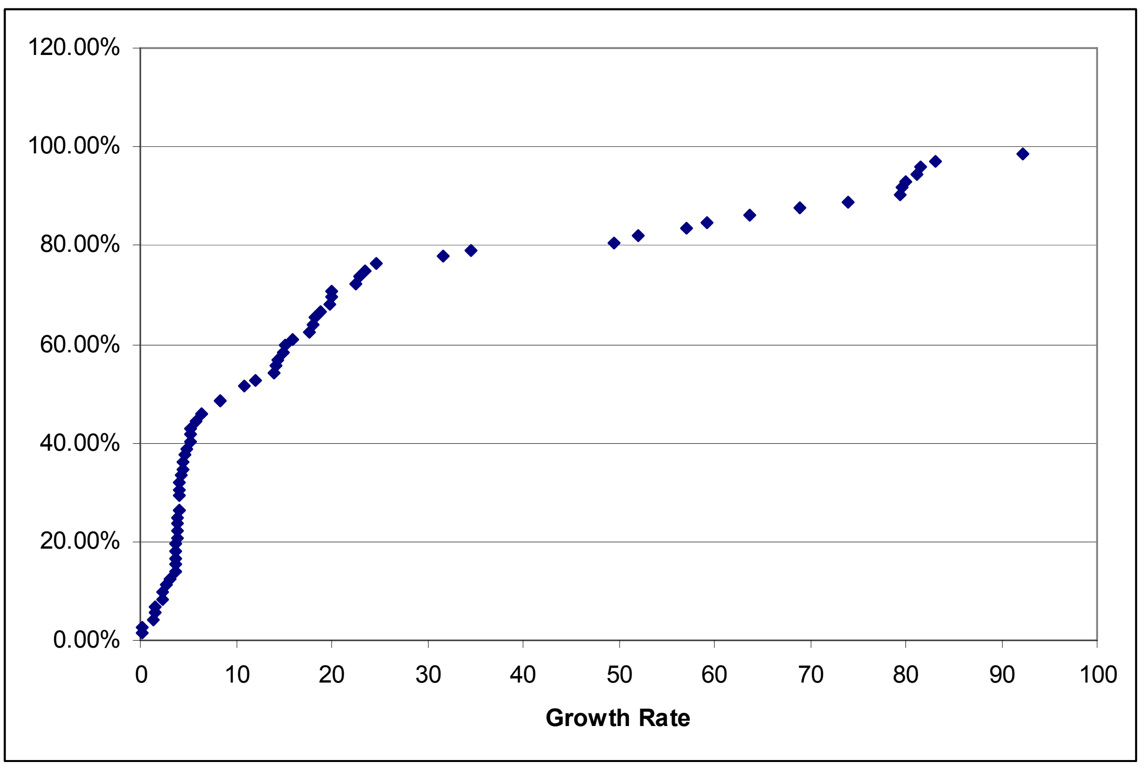


*Figure C34: SH 71 Growth Rate CDF*

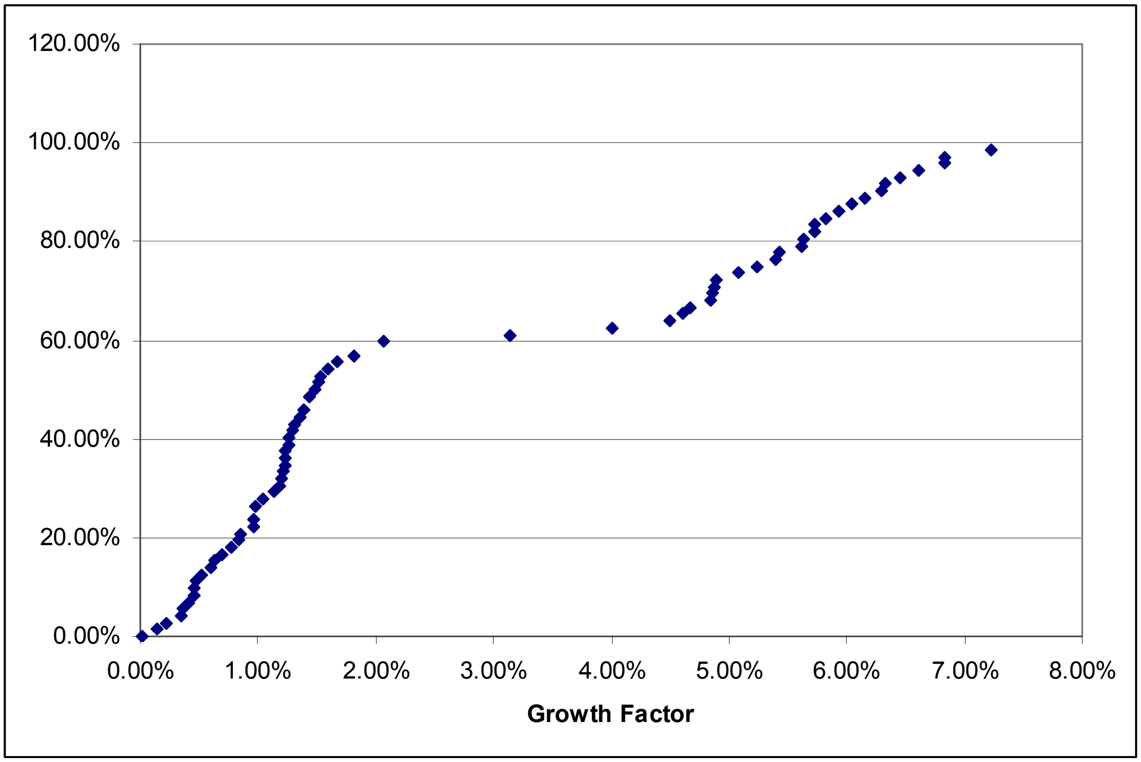


*Figure C35: SH 71 Growth Factor CDF*

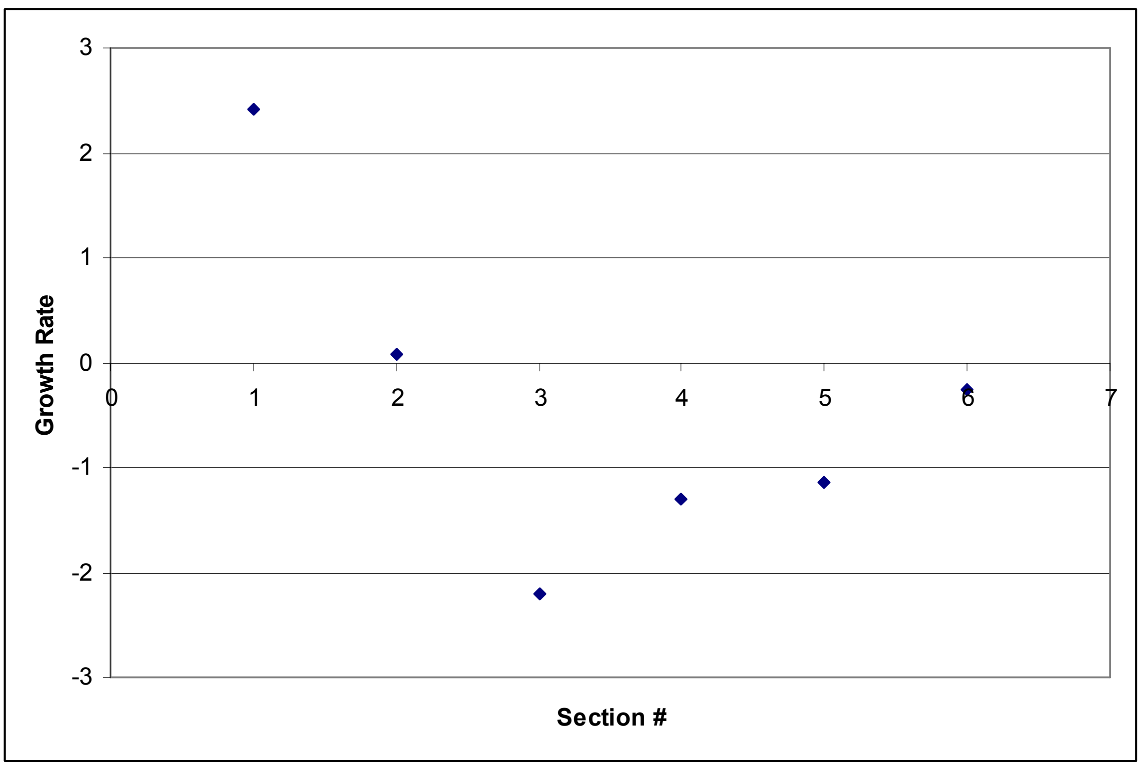


*Figure C36: FM 1329 Growth Rate*

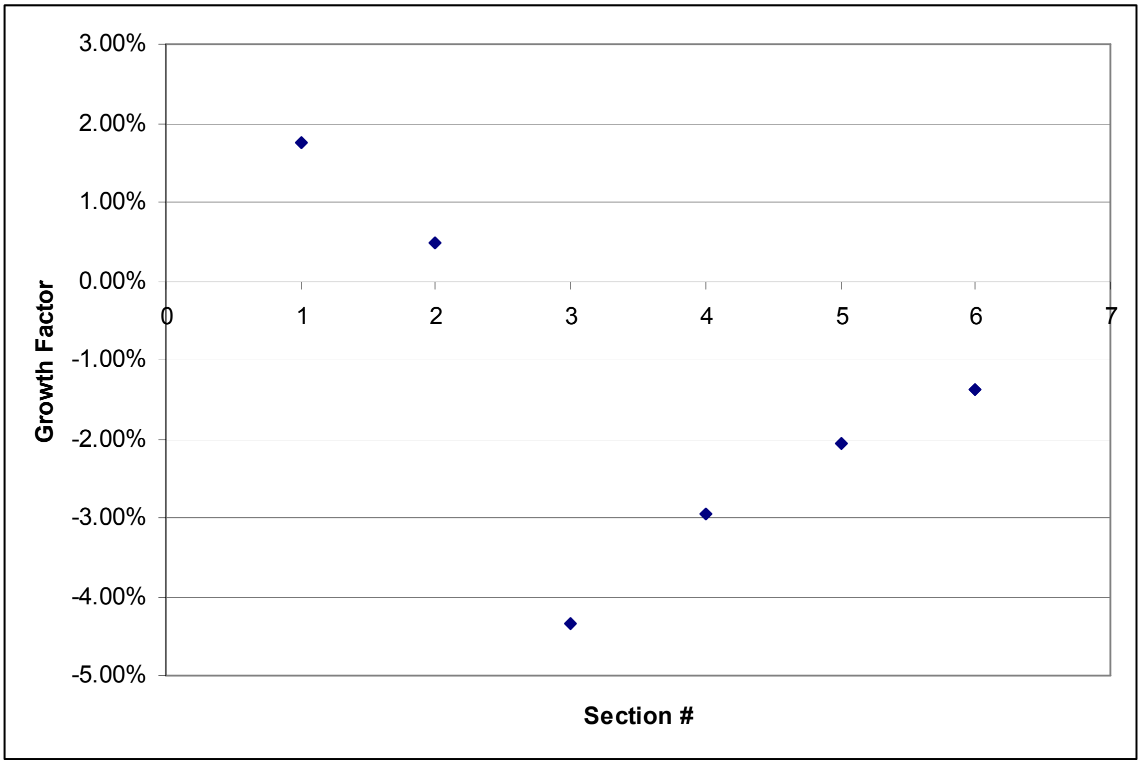


*Figure C37: FM 1329 Growth Factor*

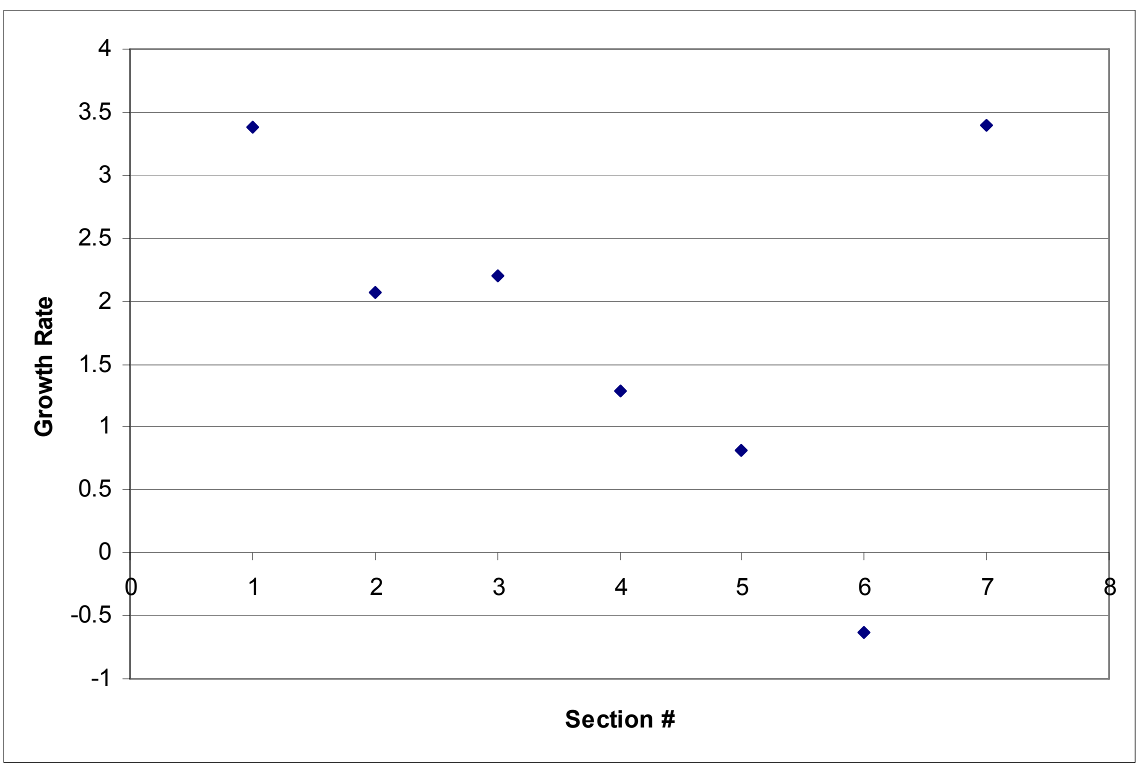


*Figure C38: FM 1450 Growth Rate*

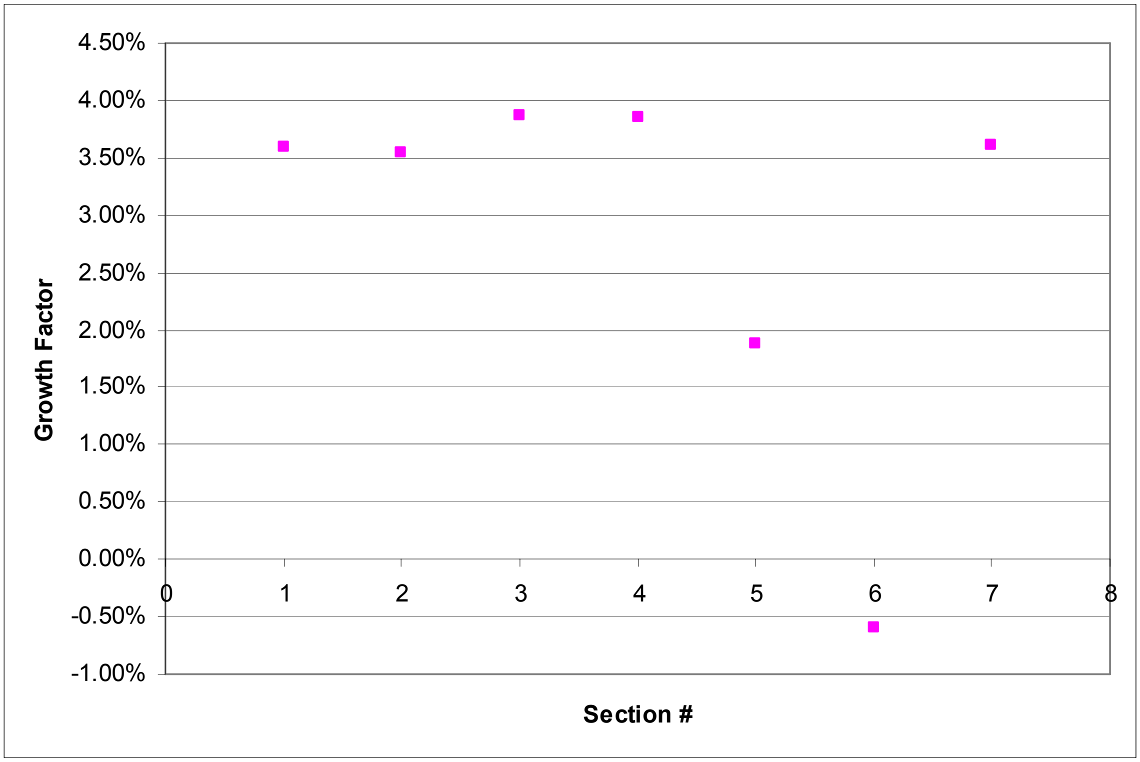


*Figure C39: FM 1450 Growth Factor*

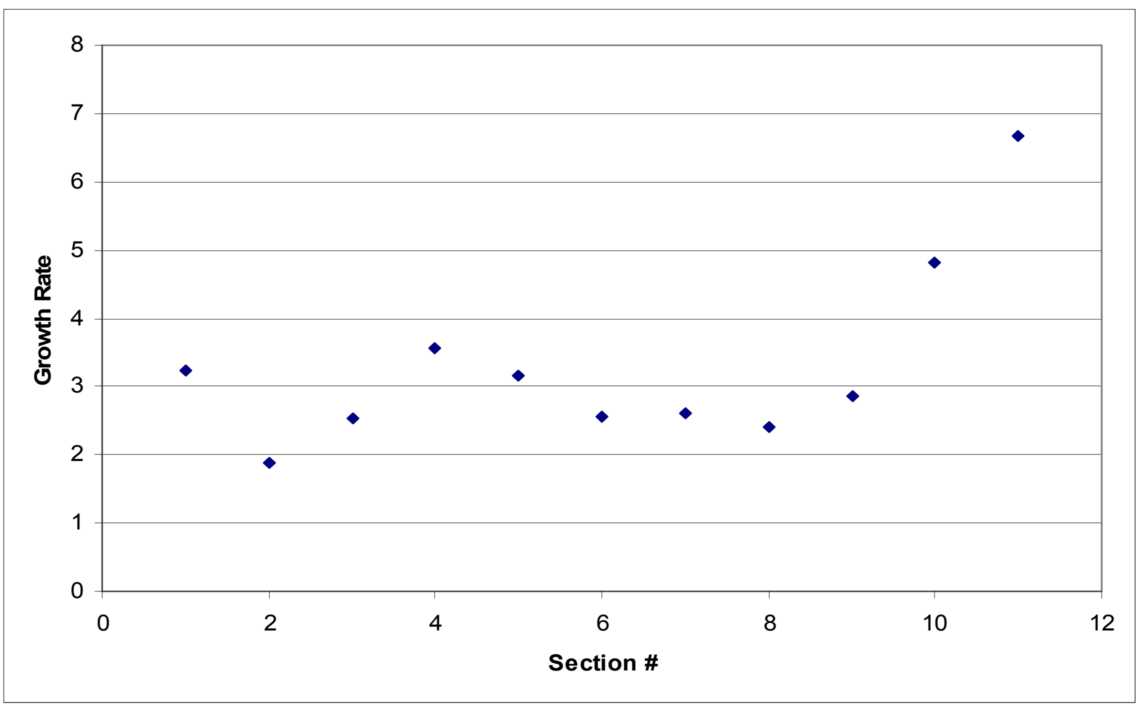


*Figure C40: FM 2088 Growth Rate*

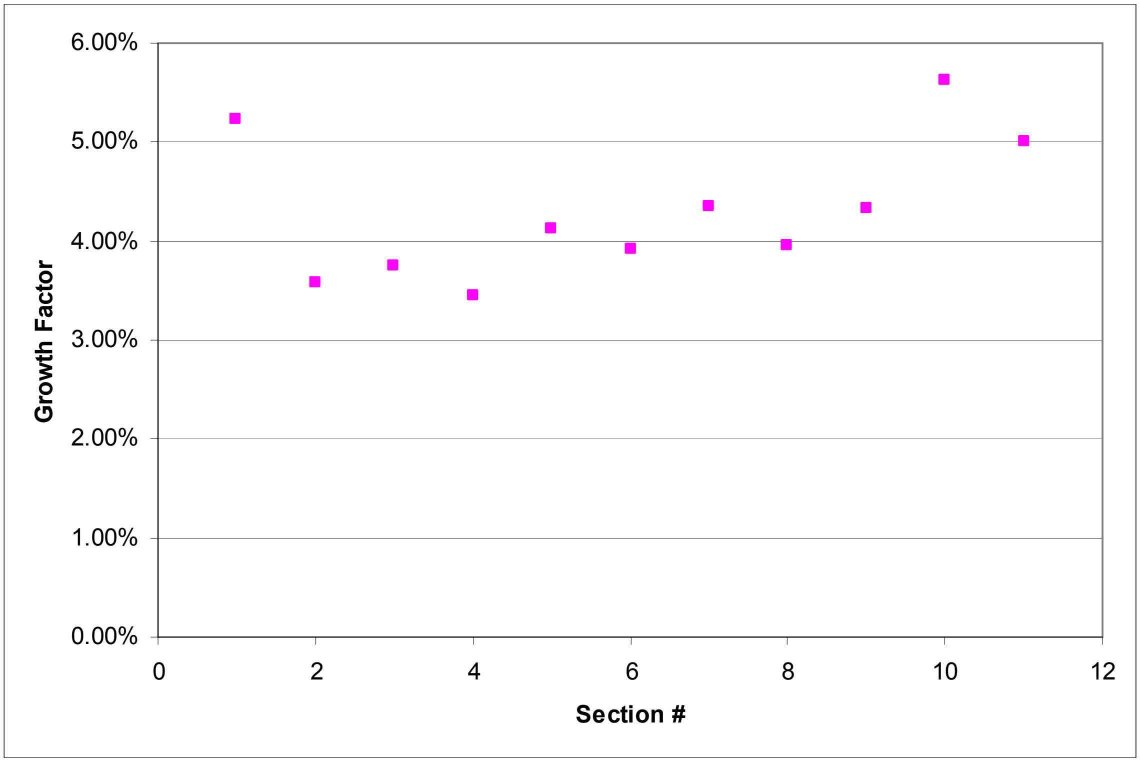


*Figure C41: FM 2088 Growth Factor*

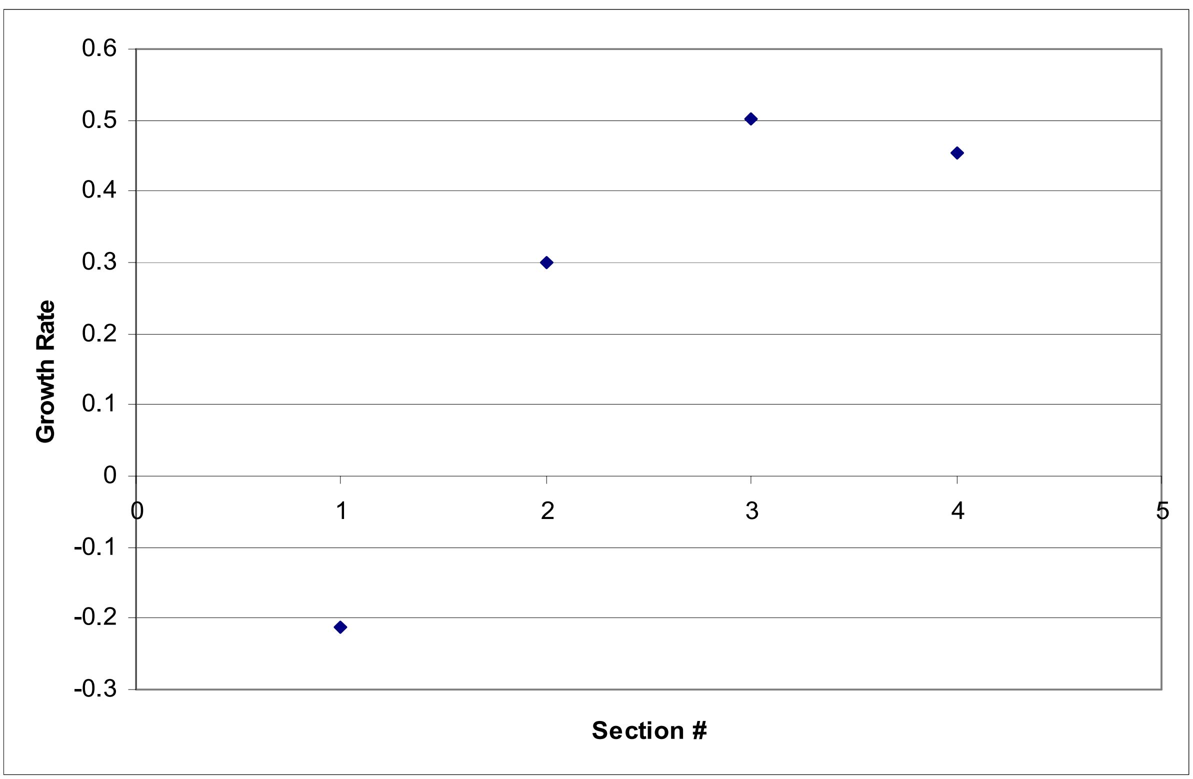


*Figure C42: FM 2111 Growth Rate*

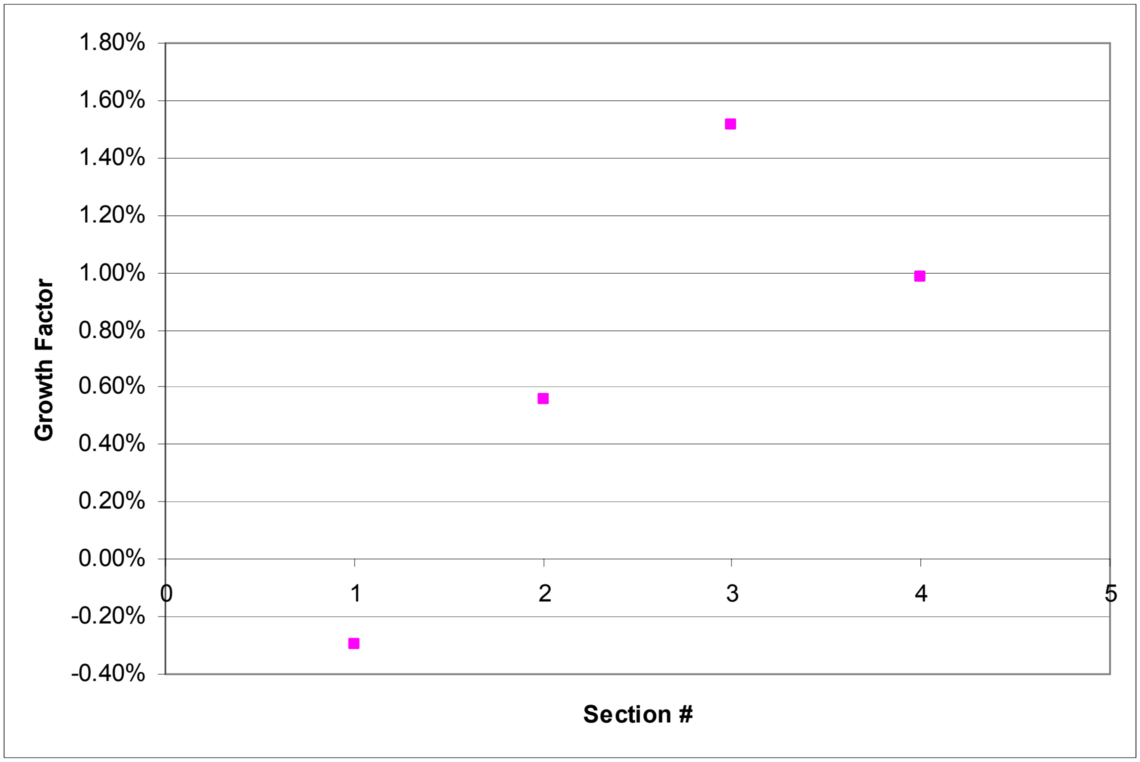


*Figure C43: FM 2111 Growth Factor*

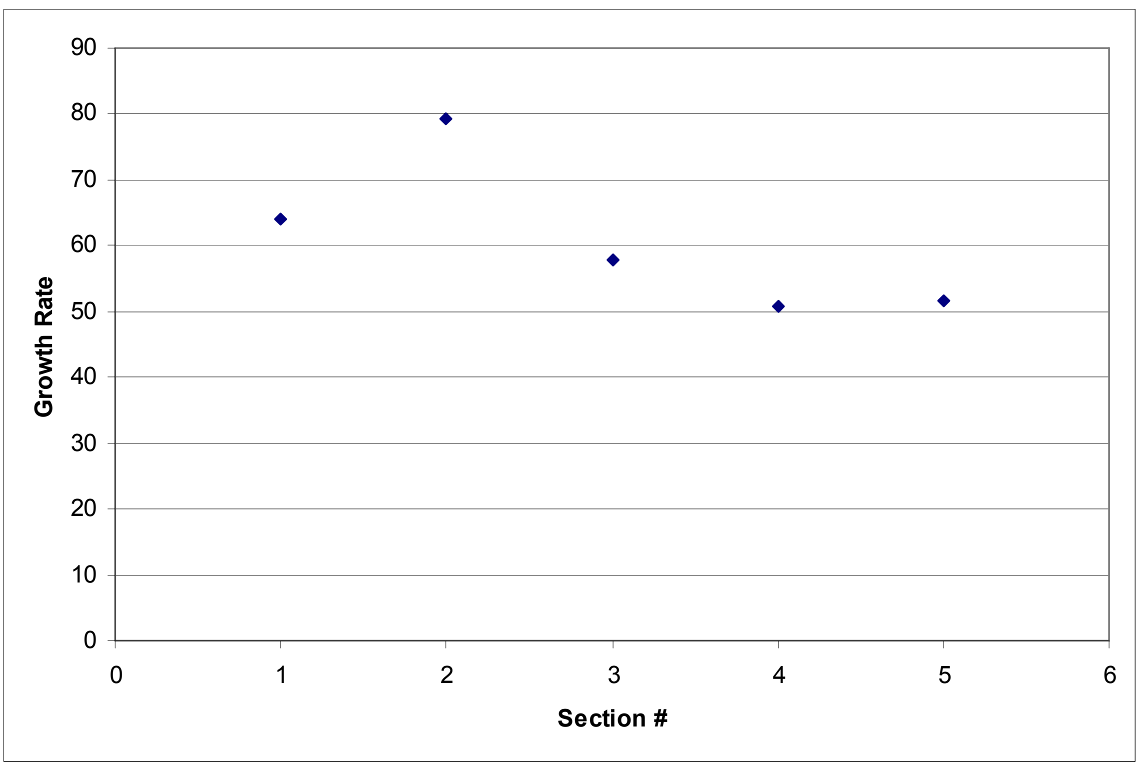


*Figure C44: FM 2222 Growth Rate*

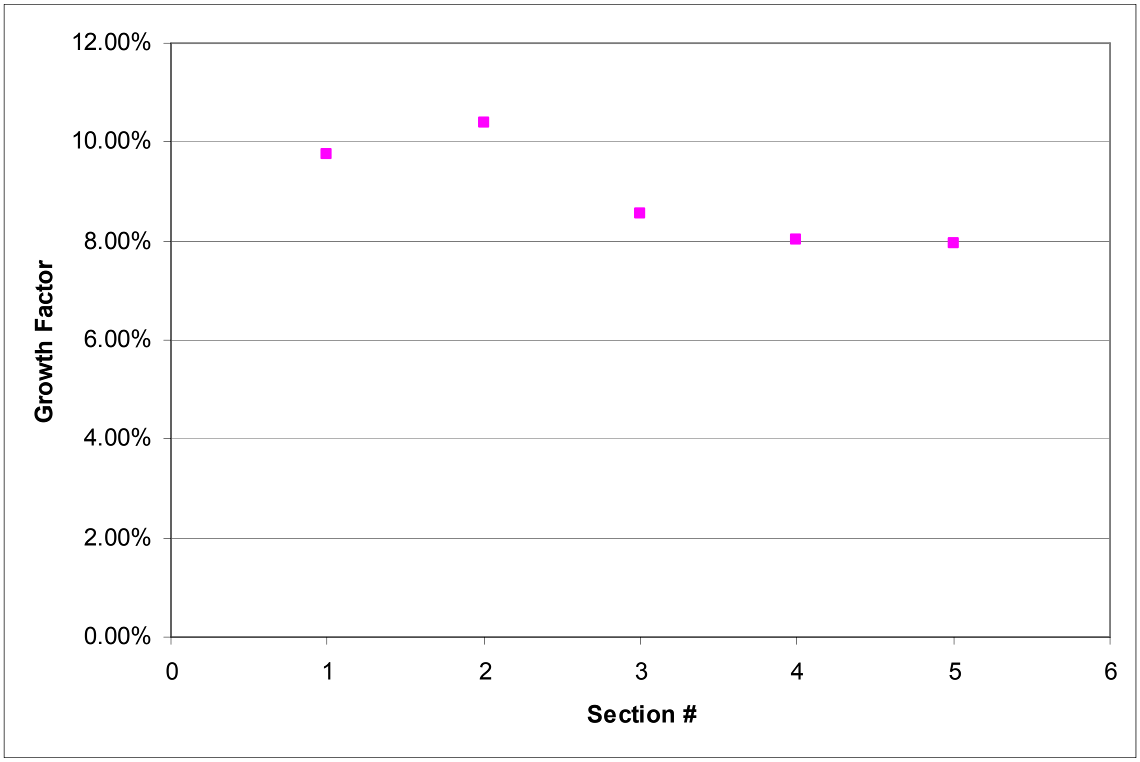


*Figure C45: FM 2222 Growth Factor*

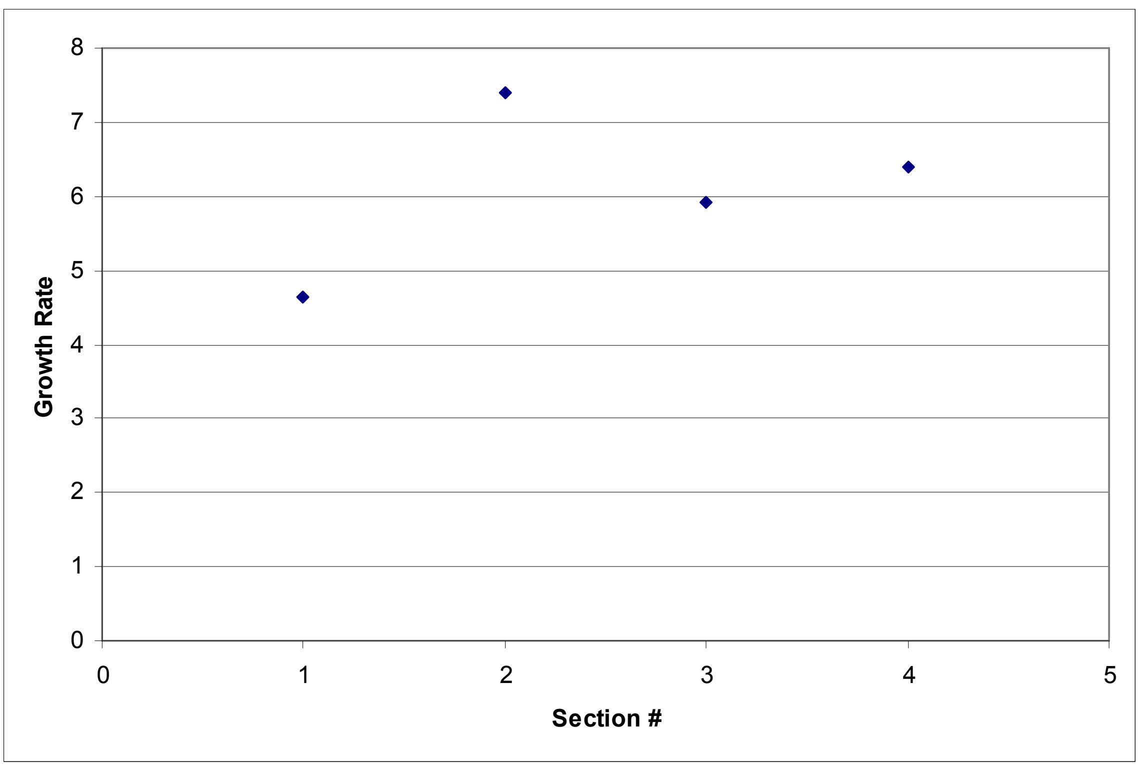


*Figure C46: FM 2917 Growth Rate*

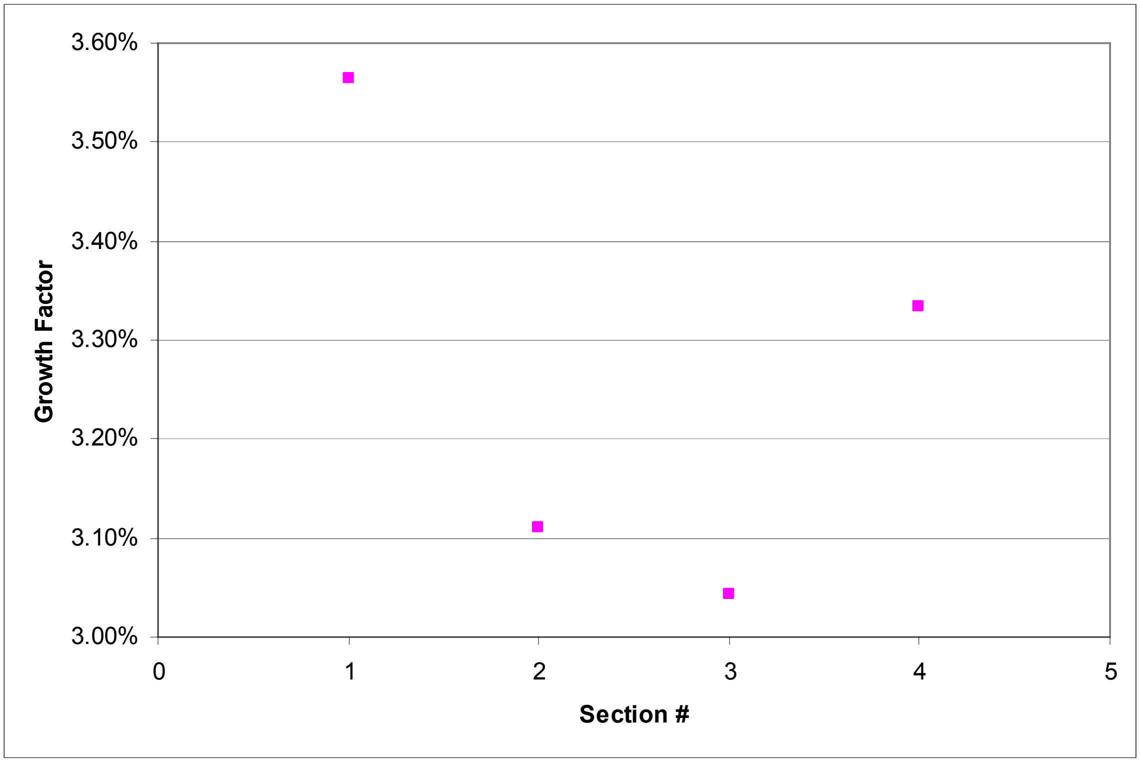


*Figure C47: FM 2917 Growth Factor*

# Appendix D: Monthly Traffic Volume Variability

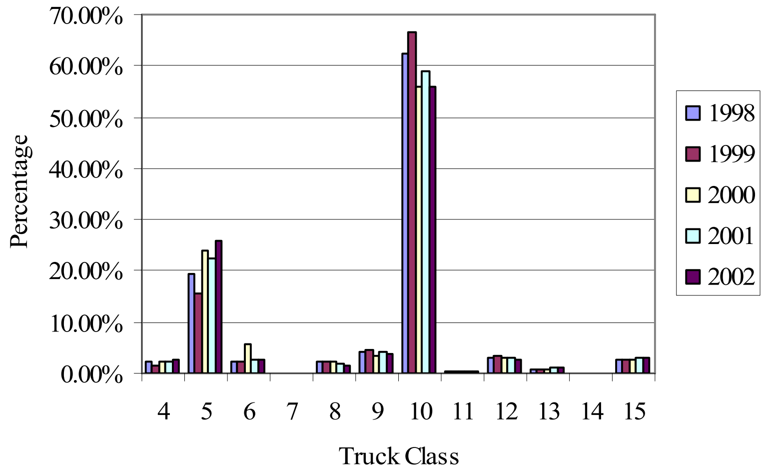


*Figure D1: Truck Volume Percentages in January*

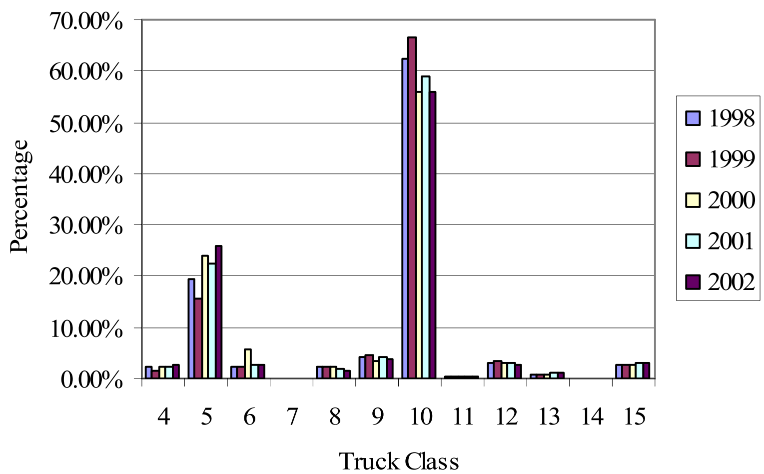


*Figure D2: Truck Volume Percentages in February*

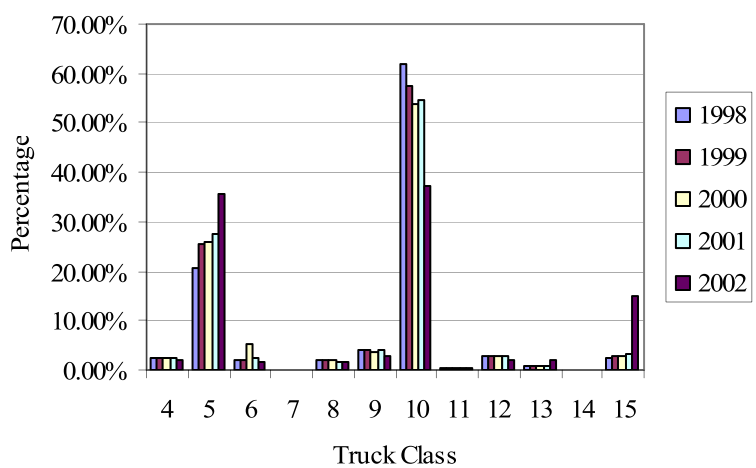


*Figure D3: Truck Volume Percentages in March*

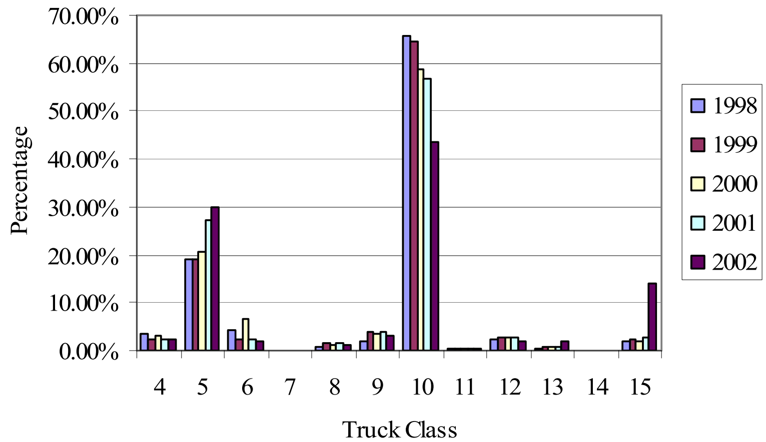


*Figure D4: Truck Volume Percentages in April*

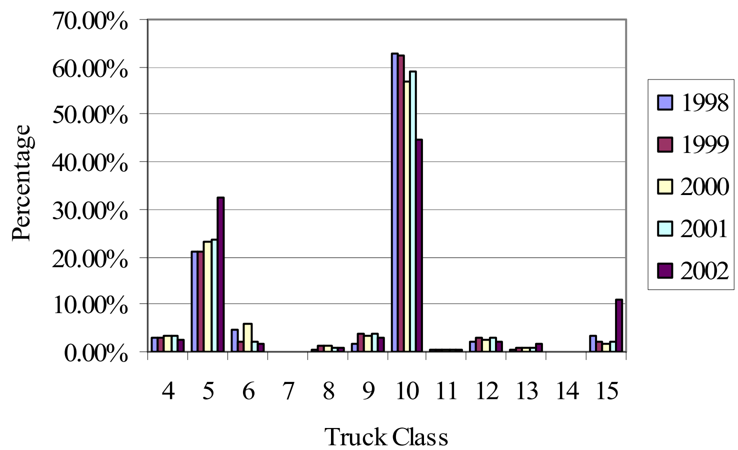


*Figure D5: Truck Volume Percentages in May*

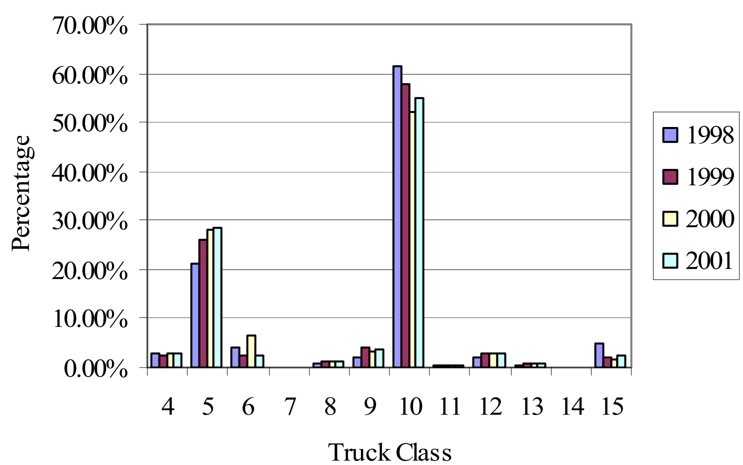


*Figure D6: Truck Volume Percentages in June*

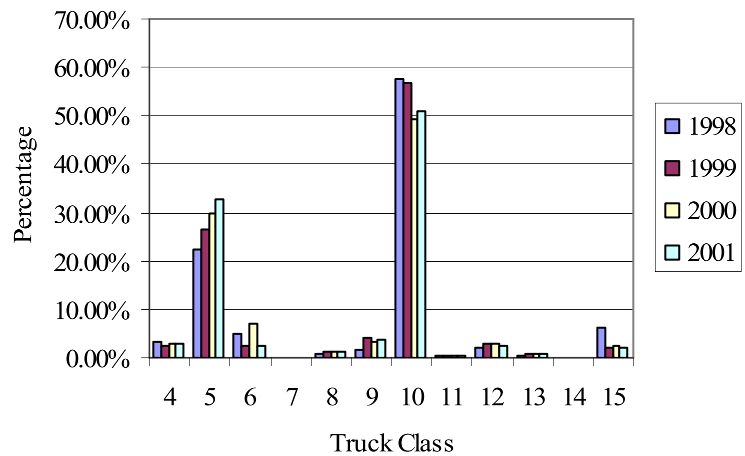


*Figure D7: Truck Volume Percentages in July*

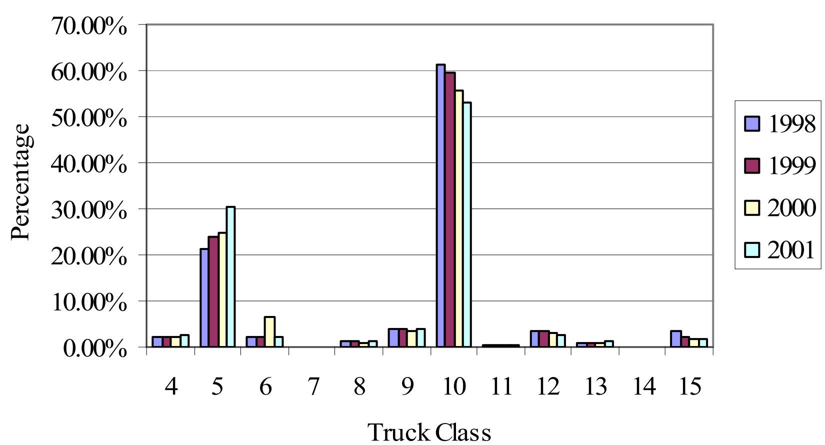


*Figure D8: Truck Volume Percentages in August*

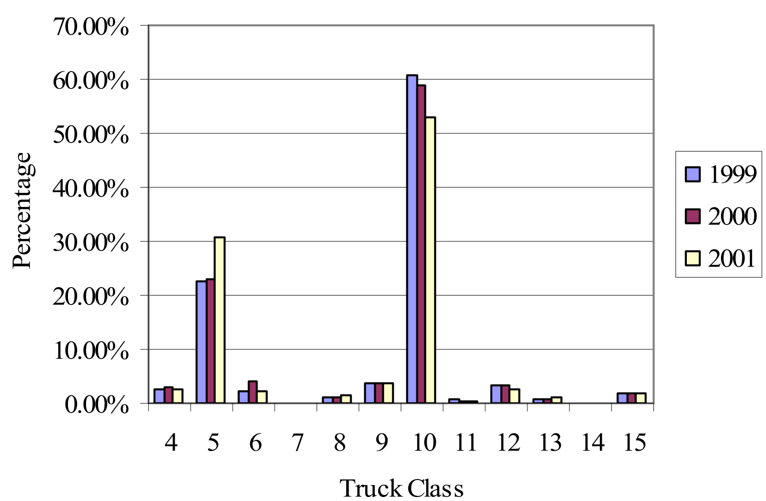


*Figure D9: Truck Volume Percentages in September*

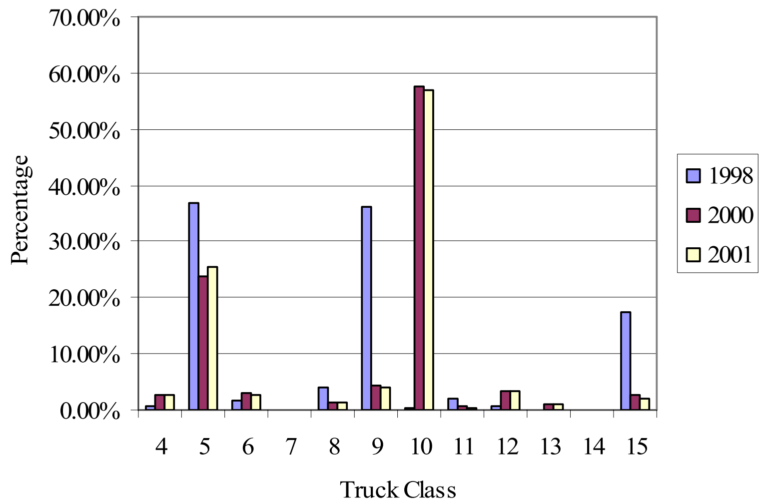


*Figure D10: Truck Volume Percentages in October*

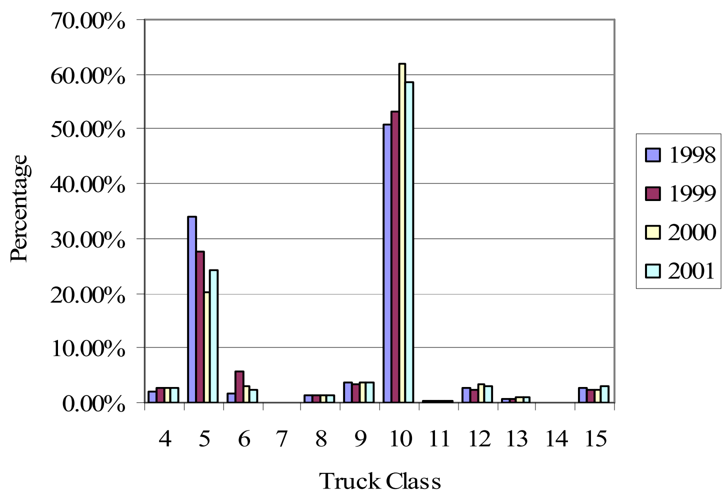


*Figure D11: Truck Volume Percentages in November*

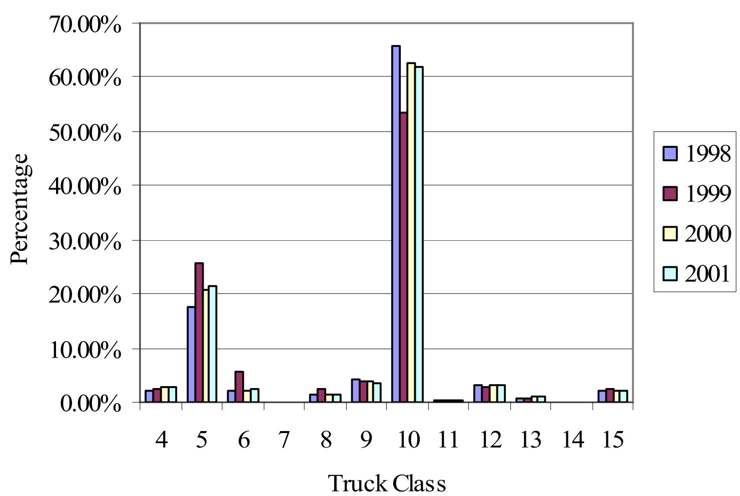


*Figure D12: Truck Volume Percentages in December*

# Appendix E: Monthly Traffic Volume Variability per Class

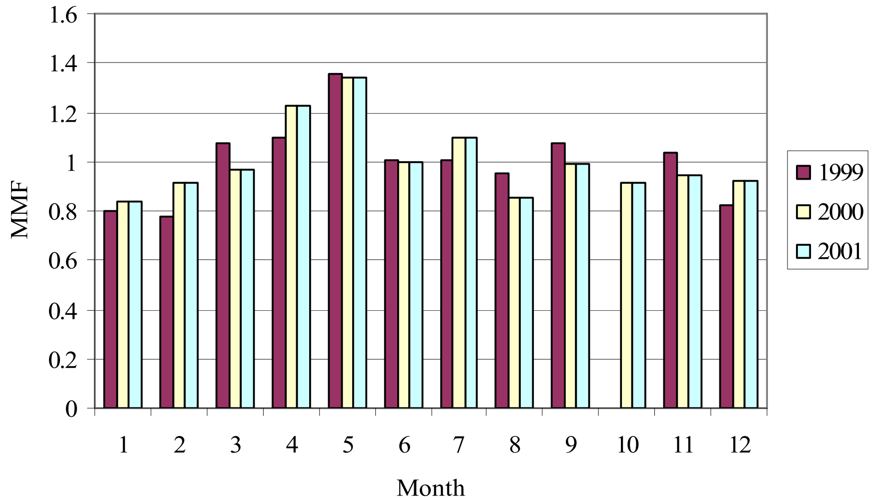


*Figure E1: Seasonal Fluctuation of Truck Class 4*

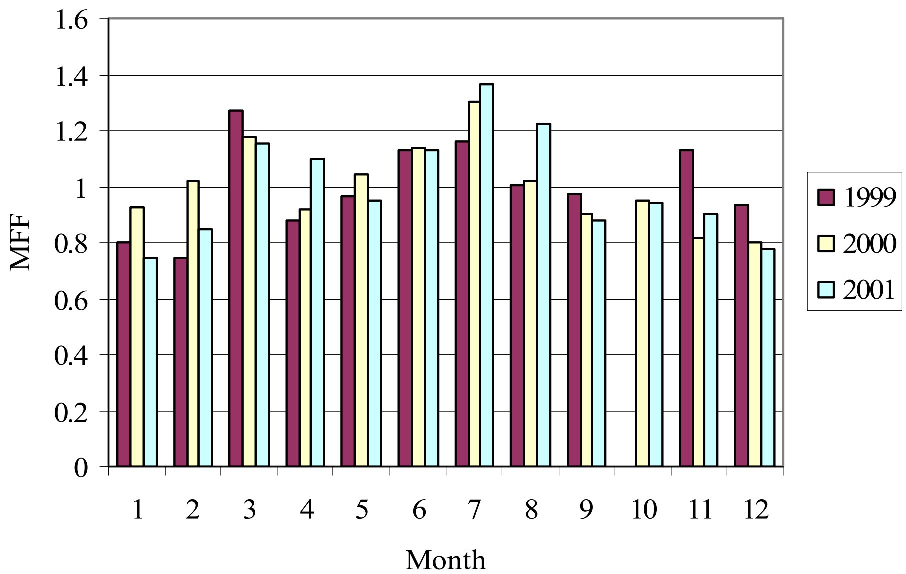


*Figure E2: Seasonal Fluctuation of Truck Class 5*

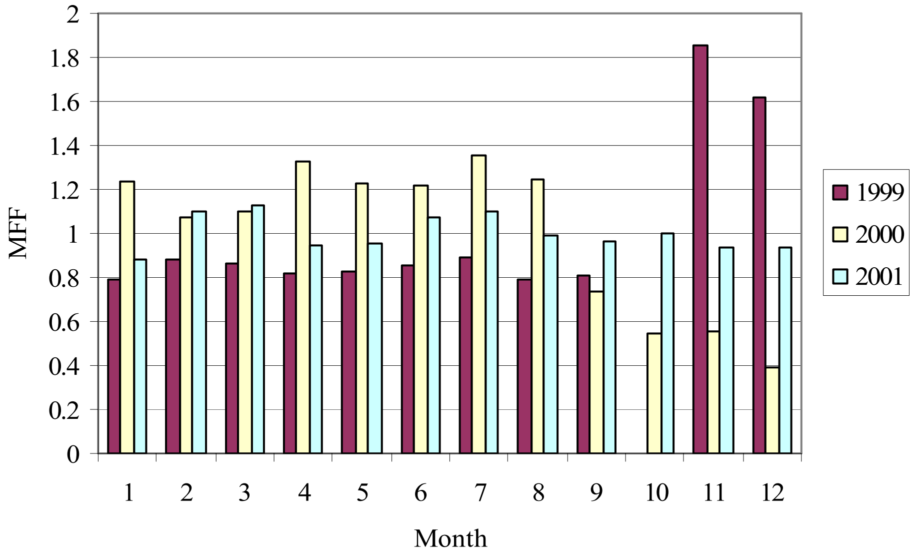


*Figure E3: Seasonal Fluctuation of Truck Class 6*

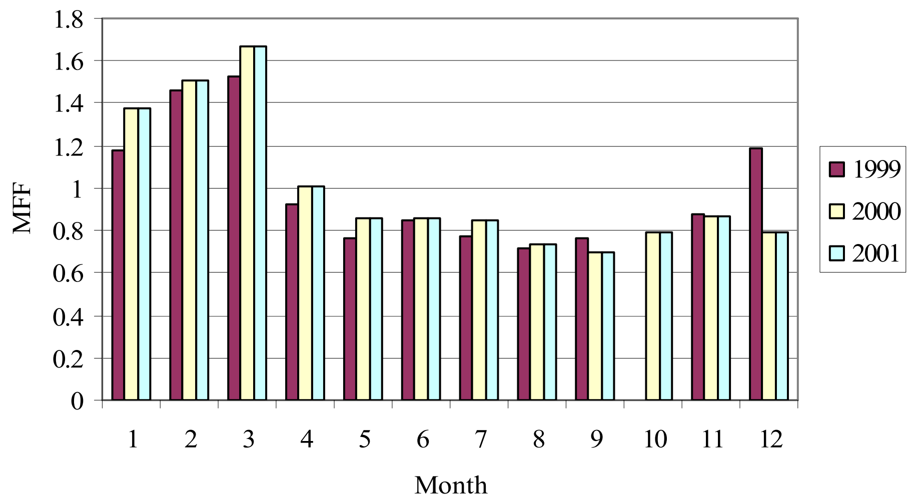


*Figure E4: Seasonal Fluctuation of Truck Class 8*

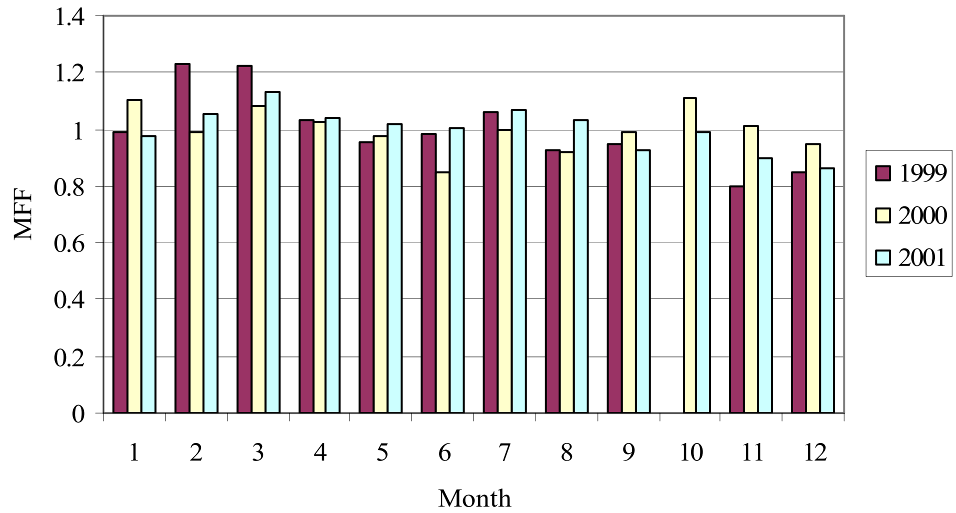


*Figure E5: Seasonal Fluctuation of Truck Class 9*

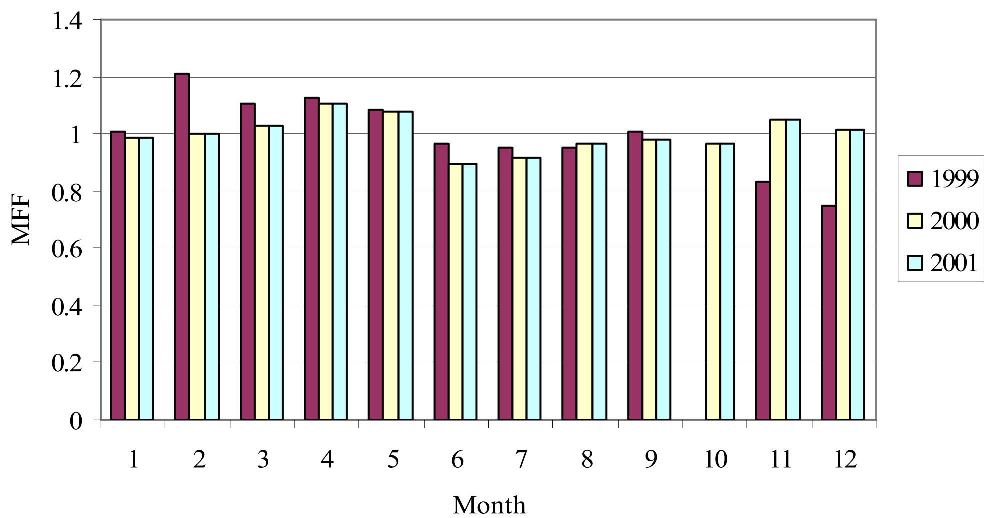


*Figure E6: Seasonal Fluctuation of Truck Class 10*

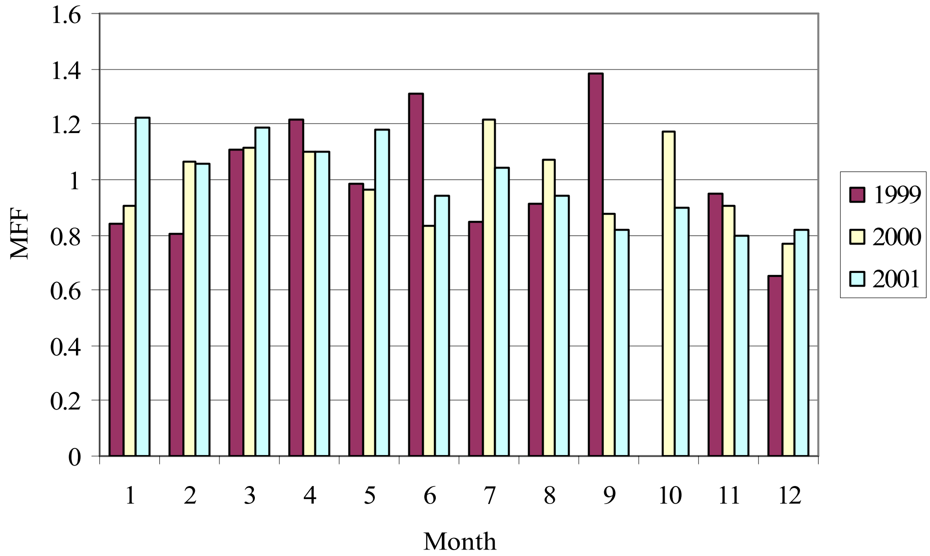


*Figure E7: Seasonal Fluctuation of Truck Class 11*

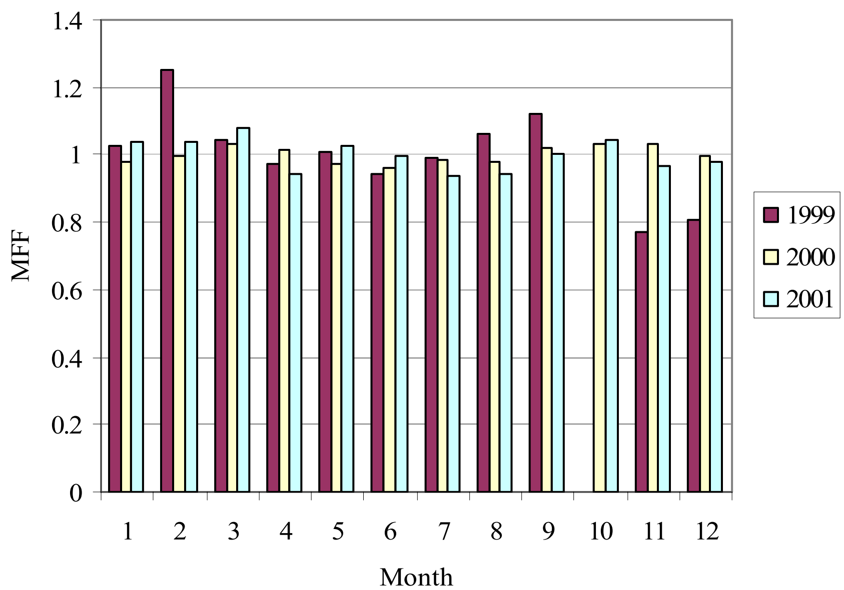


*Figure E8: Seasonal Fluctuation of Truck Class 12*

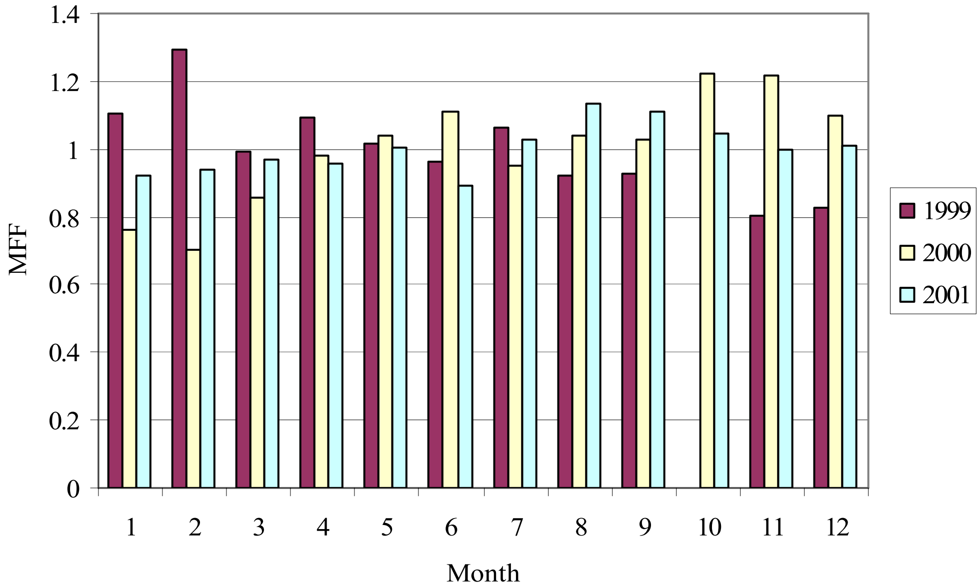


*Figure E9: Seasonal Fluctuation of Truck Class 13*

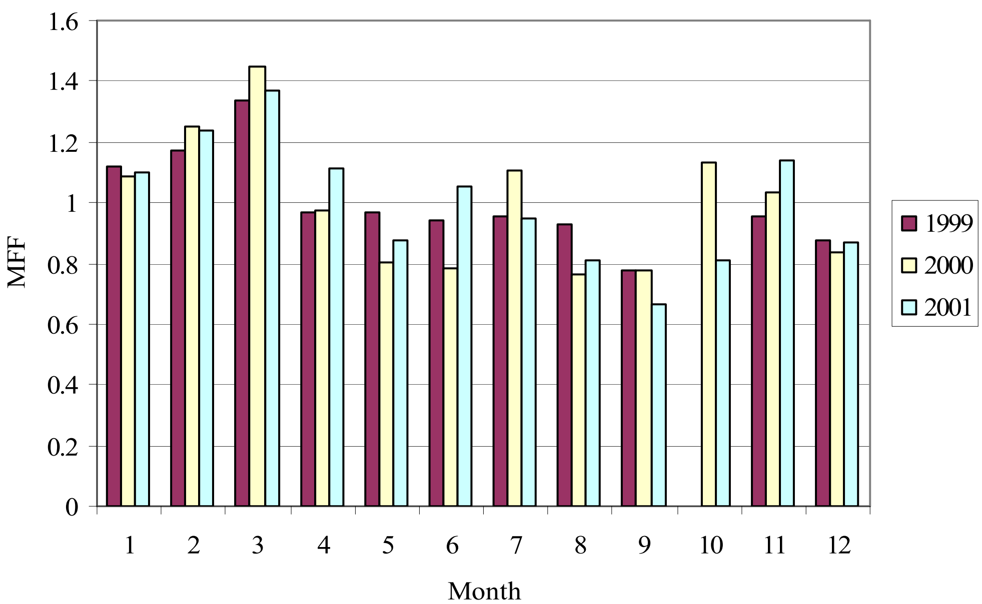


*Figure E10: Seasonal Fluctuation of Truck Class 15*

# Appendix F: Level 2 Axle Load Spectra Input for the Mechanistic-Empirical Pavement Design Guide

**Table F1: Region 1 - Level 2 Axle Load Spectra Input for Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.38% | 4 | 0.01% | 6 | 0.02% |
| 3 | 3 | 0.07% | 3 | 8.86% | 6 | 0.40% | 9 | 0.64% |
| 4 | 4 | 7.11% | 4 | 21.50% | 8 | 2.73% | 12 | 3.13% |
| 5 | 5 | 17.03% | 5 | 16.94% | 10 | 6.62% | 15 | 5.32% |
| 6 | 6 | 4.78% | 6 | 9.42% | 12 | 9.15% | 18 | 5.19% |
| 7 | 7 | 1.51% | 7 | 6.12% | 14 | 9.11% | 21 | 3.69% |
| 8 | 8 | 2.31% | 8 | 4.97% | 16 | 7.45% | 24 | 2.36% |
| 9 | 9 | 3.33% | 9 | 4.36% | 18 | 5.46% | 27 | 4.35% |
| 10 | 10 | 6.32% | 10 | 3.82% | 20 | 3.97% | 30 | 2.27% |
| 11 | 11 | 13.38% | 11 | 3.28% | 22 | 3.25% | 33 | 2.85% |
| 12 | 12 | 18.17% | 12 | 2.78% | 24 | 3.19% | 36 | 5.10% |
| 13 | 13 | 14.59% | 13 | 2.32% | 26 | 3.45% | 39 | 7.59% |
| 14 | 14 | 7.50% | 14 | 1.93% | 28 | 3.72% | 42 | 9.47% |
| 15 | 15 | 2.74% | 15 | 1.60% | 30 | 3.97% | 45 | 10.15% |
| 16 | 16 | 0.80% | 16 | 1.38% | 32 | 4.56% | 48 | 9.62% |
| 17 | 17 | 0.23% | 17 | 1.32% | 34 | 5.68% | 51 | 8.23% |
| 18 | 18 | 0.07% | 18 | 1.41% | 36 | 6.74% | 54 | 6.48% |
| 19 | 19 | 0.03% | 19 | 1.52% | 38 | 6.80% | 57 | 4.75% |
| 20 | 20 | 0.01% | 20 | 1.48% | 40 | 5.62% | 60 | 3.29% |
| 21 | 21 | 0.01% | 21 | 1.26% | 42 | 3.83% | 63 | 2.16% |
| 22 | 22 | 0.00% | 22 | 0.94% | 44 | 2.21% | 66 | 1.37% |
| 23 | 23 | 0.00% | 23 | 0.64% | 46 | 1.11% | 69 | 0.83% |
| 24 | 24 | 0.00% | 24 | 0.42% | 48 | 0.51% | 72 | 0.49% |
| 25 | 25 | 0.00% | 25 | 0.28% | 50 | 0.23% | 75 | 0.29% |
| 26 | 26 | 0.00% | 26 | 0.20% | 52 | 0.10% | 78 | 0.16% |
| 27 | 27 | 0.00% | 27 | 0.16% | 54 | 0.05% | 81 | 0.09% |
| 28 | 28 | 0.00% | 28 | 0.13% | 56 | 0.03% | 84 | 0.05% |
| 29 | 29 | 0.00% | 29 | 0.10% | 58 | 0.01% | 87 | 0.03% |
| 30 | 30 | 0.00% | 30 | 0.09% | 60 | 0.01% | 90 | 0.01% |
| 31 | 31 | 0.00% | 31 | 0.07% | 62 | 0.00% | 93 | 0.01% |
| 32 | 32 | 0.00% | 32 | 0.06% | 64 | 0.00% | 96 | 0.00% |
| 33 | 33 | 0.00% | 33 | 0.05% | 66 | 0.00% | 99 | 0.00% |
| 34 | 34 | 0.00% | 34 | 0.04% | 68 | 0.00% | 102 | 0.00% |
| 35 | 35 | 0.00% | 35 | 0.03% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.03% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.02% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.02% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.02% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.02% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.01% | 82 | 0.00% | | |

**Table F2: Region 2 - Level 2 Axle Load Spectra Input for Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.59% | 4 | 0.01% | 6 | 0.03% |
| 3 | 3 | 0.15% | 3 | 4.15% | 6 | 0.30% | 9 | 0.56% |
| 4 | 4 | 4.77% | 4 | 7.88% | 8 | 1.42% | 12 | 2.34% |
| 5 | 5 | 9.18% | 5 | 8.37% | 10 | 3.20% | 15 | 8.78% |
| 6 | 6 | 3.47% | 6 | 6.94% | 12 | 4.84% | 18 | 14.88% |
| 7 | 7 | 1.42% | 7 | 5.58% | 14 | 5.80% | 21 | 9.58% |
| 8 | 8 | 3.09% | 8 | 5.04% | 16 | 6.01% | 24 | 7.07% |
| 9 | 9 | 5.64% | 9 | 5.11% | 18 | 5.76% | 27 | 6.39% |
| 10 | 10 | 10.52% | 10 | 5.28% | 20 | 5.46% | 30 | 6.35% |
| 11 | 11 | 21.13% | 11 | 5.27% | 22 | 5.46% | 33 | 7.19% |
| 12 | 12 | 23.26% | 12 | 5.14% | 24 | 5.71% | 36 | 8.23% |
| 13 | 13 | 12.41% | 13 | 5.18% | 26 | 5.90% | 39 | 8.24% |
| 14 | 14 | 3.76% | 14 | 5.51% | 28 | 6.20% | 42 | 6.94% |
| 15 | 15 | 0.87% | 15 | 5.88% | 30 | 7.69% | 45 | 4.98% |
| 16 | 16 | 0.23% | 16 | 5.90% | 32 | 10.13% | 48 | 3.17% |
| 17 | 17 | 0.07% | 17 | 5.36% | 34 | 10.52% | 51 | 1.89% |
| 18 | 18 | 0.03% | 18 | 4.36% | 36 | 7.70% | 54 | 1.11% |
| 19 | 19 | 0.01% | 19 | 3.20% | 38 | 4.11% | 57 | 0.68% |
| 20 | 20 | 0.00% | 20 | 2.15% | 40 | 1.79% | 60 | 0.44% |
| 21 | 21 | 0.00% | 21 | 1.34% | 42 | 0.76% | 63 | 0.30% |
| 22 | 22 | 0.00% | 22 | 0.78% | 44 | 0.38% | 66 | 0.22% |
| 23 | 23 | 0.00% | 23 | 0.44% | 46 | 0.23% | 69 | 0.16% |
| 24 | 24 | 0.00% | 24 | 0.24% | 48 | 0.16% | 72 | 0.12% |
| 25 | 25 | 0.00% | 25 | 0.13% | 50 | 0.11% | 75 | 0.09% |
| 26 | 26 | 0.00% | 26 | 0.07% | 52 | 0.08% | 78 | 0.07% |
| 27 | 27 | 0.00% | 27 | 0.04% | 54 | 0.06% | 81 | 0.05% |
| 28 | 28 | 0.00% | 28 | 0.02% | 56 | 0.05% | 84 | 0.04% |
| 29 | 29 | 0.00% | 29 | 0.01% | 58 | 0.04% | 87 | 0.03% |
| 30 | 30 | 0.00% | 30 | 0.01% | 60 | 0.03% | 90 | 0.02% |
| 31 | 31 | 0.00% | 31 | 0.01% | 62 | 0.02% | 93 | 0.02% |
| 32 | 32 | 0.00% | 32 | 0.00% | 64 | 0.02% | 96 | 0.01% |
| 33 | 33 | 0.00% | 33 | 0.00% | 66 | 0.01% | 99 | 0.01% |
| 34 | 34 | 0.00% | 34 | 0.00% | 68 | 0.01% | 102 | 0.01% |
| 35 | 35 | 0.00% | 35 | 0.00% | 70 | 0.01% | | |
| 36 | 36 | 0.00% | 36 | 0.00% | 72 | 0.01% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

**Table F3: Region 3 - Level 2 Axle Load Spectra Input for Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.08% | 4 | 0.00% | 6 | 0.01% |
| 3 | 3 | 0.17% | 3 | 3.22% | 6 | 0.15% | 9 | 0.74% |
| 4 | 4 | 5.05% | 4 | 11.03% | 8 | 1.33% | 12 | 4.58% |
| 5 | 5 | 10.17% | 5 | 11.73% | 10 | 4.00% | 15 | 9.71% |
| 6 | 6 | 4.28% | 6 | 7.73% | 12 | 6.60% | 18 | 11.46% |
| 7 | 7 | 1.57% | 7 | 5.49% | 14 | 7.67% | 21 | 9.41% |
| 8 | 8 | 3.48% | 8 | 5.26% | 16 | 7.31% | 24 | 6.64% |
| 9 | 9 | 7.09% | 9 | 5.61% | 18 | 6.48% | 27 | 5.33% |
| 10 | 10 | 14.03% | 10 | 5.76% | 20 | 5.89% | 30 | 5.50% |
| 11 | 11 | 22.83% | 11 | 5.61% | 22 | 5.68% | 33 | 6.24% |
| 12 | 12 | 19.65% | 12 | 5.32% | 24 | 5.59% | 36 | 6.77% |
| 13 | 13 | 8.77% | 13 | 5.09% | 26 | 5.35% | 39 | 6.73% |
| 14 | 14 | 2.34% | 14 | 4.94% | 28 | 5.03% | 42 | 6.17% |
| 15 | 15 | 0.46% | 15 | 4.73% | 30 | 5.78% | 45 | 5.28% |
| 16 | 16 | 0.09% | 16 | 4.34% | 32 | 8.54% | 48 | 4.26% |
| 17 | 17 | 0.02% | 17 | 3.75% | 34 | 10.14% | 51 | 3.28% |
| 18 | 18 | 0.01% | 18 | 3.03% | 36 | 7.65% | 54 | 2.43% |
| 19 | 19 | 0.00% | 19 | 2.29% | 38 | 3.82% | 57 | 1.75% |
| 20 | 20 | 0.00% | 20 | 1.65% | 40 | 1.52% | 60 | 1.22% |
| 21 | 21 | 0.00% | 21 | 1.13% | 42 | 0.65% | 63 | 0.84% |
| 22 | 22 | 0.00% | 22 | 0.75% | 44 | 0.34% | 66 | 0.57% |
| 23 | 23 | 0.00% | 23 | 0.49% | 46 | 0.20% | 69 | 0.38% |
| 24 | 24 | 0.00% | 24 | 0.32% | 48 | 0.12% | 72 | 0.25% |
| 25 | 25 | 0.00% | 25 | 0.21% | 50 | 0.07% | 75 | 0.16% |
| 26 | 26 | 0.00% | 26 | 0.14% | 52 | 0.04% | 78 | 0.10% |
| 27 | 27 | 0.00% | 27 | 0.09% | 54 | 0.03% | 81 | 0.07% |
| 28 | 28 | 0.00% | 28 | 0.06% | 56 | 0.01% | 84 | 0.04% |
| 29 | 29 | 0.00% | 29 | 0.04% | 58 | 0.01% | 87 | 0.03% |
| 30 | 30 | 0.00% | 30 | 0.03% | 60 | 0.01% | 90 | 0.02% |
| 31 | 31 | 0.00% | 31 | 0.02% | 62 | 0.00% | 93 | 0.01% |
| 32 | 32 | 0.00% | 32 | 0.02% | 64 | 0.00% | 96 | 0.01% |
| 33 | 33 | 0.00% | 33 | 0.01% | 66 | 0.00% | 99 | 0.00% |
| 34 | 34 | 0.00% | 34 | 0.01% | 68 | 0.00% | 102 | 0.00% |
| 35 | 35 | 0.00% | 35 | 0.01% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.00% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

## Table F4: Region 4 - Level 2 Axle Load Spectra Input for Interstate Highway

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.04% | 4 | 0.01% | 6 | 0.00% |
| 3 | 3 | 0.12% | 3 | 2.58% | 6 | 0.20% | 9 | 0.29% |
| 4 | 4 | 4.62% | 4 | 9.89% | 8 | 1.17% | 12 | 3.33% |
| 5 | 5 | 9.20% | 5 | 10.31% | 10 | 2.92% | 15 | 9.23% |
| 6 | 6 | 3.18% | 6 | 6.45% | 12 | 4.63% | 18 | 11.78% |
| 7 | 7 | 0.55% | 7 | 4.74% | 14 | 5.62% | 21 | 10.03% |
| 8 | 8 | 1.43% | 8 | 4.81% | 16 | 5.83% | 24 | 7.74% |
| 9 | 9 | 4.41% | 9 | 5.20% | 18 | 5.61% | 27 | 6.90% |
| 10 | 10 | 9.67% | 10 | 5.38% | 20 | 5.44% | 30 | 7.12% |
| 11 | 11 | 20.82% | 11 | 5.48% | 22 | 5.57% | 33 | 7.54% |
| 12 | 12 | 25.76% | 12 | 5.67% | 24 | 5.81% | 36 | 8.71% |
| 13 | 13 | 14.91% | 13 | 5.94% | 26 | 5.79% | 39 | 7.93% |
| 14 | 14 | 4.43% | 14 | 6.07% | 28 | 5.41% | 42 | 5.53% |
| 15 | 15 | 0.79% | 15 | 5.88% | 30 | 5.63% | 45 | 4.20% |
| 16 | 16 | 0.10% | 16 | 5.33% | 32 | 8.80% | 48 | 3.11% |
| 17 | 17 | 0.01% | 17 | 4.50% | 34 | 12.58% | 51 | 2.21% |
| 18 | 18 | 0.00% | 18 | 3.56% | 36 | 10.62% | 54 | 1.52% |
| 19 | 19 | 0.00% | 19 | 2.66% | 38 | 5.23% | 57 | 1.02% |
| 20 | 20 | 0.00% | 20 | 1.89% | 40 | 1.78% | 60 | 0.67% |
| 21 | 21 | 0.00% | 21 | 1.29% | 42 | 0.60% | 63 | 0.43% |
| 22 | 22 | 0.00% | 22 | 0.85% | 44 | 0.28% | 66 | 0.27% |
| 23 | 23 | 0.00% | 23 | 0.55% | 46 | 0.16% | 69 | 0.17% |
| 24 | 24 | 0.00% | 24 | 0.35% | 48 | 0.10% | 72 | 0.11% |
| 25 | 25 | 0.00% | 25 | 0.21% | 50 | 0.07% | 75 | 0.07% |
| 26 | 26 | 0.00% | 26 | 0.13% | 52 | 0.04% | 78 | 0.04% |
| 27 | 27 | 0.00% | 27 | 0.08% | 54 | 0.03% | 81 | 0.02% |
| 28 | 28 | 0.00% | 28 | 0.05% | 56 | 0.02% | 84 | 0.02% |
| 29 | 29 | 0.00% | 29 | 0.03% | 58 | 0.01% | 87 | 0.01% |
| 30 | 30 | 0.00% | 30 | 0.02% | 60 | 0.01% | 90 | 0.01% |
| 31 | 31 | 0.00% | 31 | 0.01% | 62 | 0.01% | 93 | 0.00% |
| 32 | 32 | 0.00% | 32 | 0.01% | 64 | 0.01% | 96 | 0.00% |
| 33 | 33 | 0.00% | 33 | 0.01% | 66 | 0.00% | 99 | 0.00% |
| 34 | 34 | 0.00% | 34 | 0.00% | 68 | 0.00% | 102 | 0.00% |
| 35 | 35 | 0.00% | 35 | 0.00% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.00% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

**Table F5: Region 5 - Level 2 Axle Load Spectra Input for Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.36% | 4 | 0.01% | 6 | 0.00% |
| 3 | 3 | 0.20% | 3 | 5.52% | 6 | 0.40% | 9 | 0.59% |
| 4 | 4 | 4.56% | 4 | 12.85% | 8 | 2.16% | 12 | 5.80% |
| 5 | 5 | 9.22% | 5 | 12.44% | 10 | 4.84% | 15 | 12.72% |
| 6 | 6 | 4.64% | 6 | 8.28% | 12 | 6.71% | 18 | 12.33% |
| 7 | 7 | 1.64% | 7 | 5.42% | 14 | 7.06% | 21 | 8.12% |
| 8 | 8 | 3.11% | 8 | 4.47% | 16 | 6.38% | 24 | 5.47% |
| 9 | 9 | 7.10% | 9 | 4.46% | 18 | 5.49% | 27 | 5.13% |
| 10 | 10 | 14.11% | 10 | 4.60% | 20 | 4.96% | 30 | 7.97% |
| 11 | 11 | 22.89% | 11 | 4.55% | 22 | 4.88% | 33 | 6.34% |
| 12 | 12 | 19.98% | 12 | 4.26% | 24 | 4.98% | 36 | 6.33% |
| 13 | 13 | 9.06% | 13 | 3.84% | 26 | 5.00% | 39 | 5.96% |
| 14 | 14 | 2.60% | 14 | 3.56% | 28 | 5.05% | 42 | 5.25% |
| 15 | 15 | 0.64% | 15 | 3.67% | 30 | 6.28% | 45 | 4.40% |
| 16 | 16 | 0.18% | 16 | 4.12% | 32 | 9.12% | 48 | 3.53% |
| 17 | 17 | 0.05% | 17 | 4.43% | 34 | 10.47% | 51 | 2.74% |
| 18 | 18 | 0.02% | 18 | 4.14% | 36 | 8.05% | 54 | 2.07% |
| 19 | 19 | 0.00% | 19 | 3.28% | 38 | 4.32% | 57 | 1.53% |
| 20 | 20 | 0.00% | 20 | 2.24% | 40 | 1.89% | 60 | 1.11% |
| 21 | 21 | 0.00% | 21 | 1.37% | 42 | 0.84% | 63 | 0.79% |
| 22 | 22 | 0.00% | 22 | 0.79% | 44 | 0.44% | 66 | 0.56% |
| 23 | 23 | 0.00% | 23 | 0.46% | 46 | 0.26% | 69 | 0.39% |
| 24 | 24 | 0.00% | 24 | 0.28% | 48 | 0.16% | 72 | 0.27% |
| 25 | 25 | 0.00% | 25 | 0.18% | 50 | 0.10% | 75 | 0.19% |
| 26 | 26 | 0.00% | 26 | 0.12% | 52 | 0.06% | 78 | 0.13% |
| 27 | 27 | 0.00% | 27 | 0.09% | 54 | 0.04% | 81 | 0.09% |
| 28 | 28 | 0.00% | 28 | 0.06% | 56 | 0.02% | 84 | 0.06% |
| 29 | 29 | 0.00% | 29 | 0.05% | 58 | 0.01% | 87 | 0.04% |
| 30 | 30 | 0.00% | 30 | 0.03% | 60 | 0.01% | 90 | 0.03% |
| 31 | 31 | 0.00% | 31 | 0.02% | 62 | 0.00% | 93 | 0.02% |
| 32 | 32 | 0.00% | 32 | 0.02% | 64 | 0.00% | 96 | 0.01% |
| 33 | 33 | 0.00% | 33 | 0.01% | 66 | 0.00% | 99 | 0.01% |
| 34 | 34 | 0.00% | 34 | 0.01% | 68 | 0.00% | 102 | 0.01% |
| 35 | 35 | 0.00% | 35 | 0.01% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.01% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

## Table F6: Region 6 - Level 2 Axle Load Spectra Input for Interstate Highway

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.15% | 4 | 0.00% | 6 | 0.04% |
| 3 | 3 | 0.16% | 3 | 2.70% | 6 | 0.17% | 9 | 0.44% |
| 4 | 4 | 3.50% | 4 | 7.15% | 8 | 1.46% | 12 | 3.15% |
| 5 | 5 | 6.57% | 5 | 7.92% | 10 | 4.39% | 15 | 11.89% |
| 6 | 6 | 3.06% | 6 | 6.41% | 12 | 7.28% | 18 | 12.96% |
| 7 | 7 | 1.83% | 7 | 5.59% | 14 | 8.50% | 21 | 6.93% |
| 8 | 8 | 4.89% | 8 | 5.83% | 16 | 8.04% | 24 | 4.64% |
| 9 | 9 | 10.69% | 9 | 6.35% | 18 | 6.84% | 27 | 4.12% |
| 10 | 10 | 20.48% | 10 | 6.60% | 20 | 5.79% | 30 | 3.90% |
| 11 | 11 | 25.30% | 11 | 6.54% | 22 | 5.25% | 33 | 4.55% |
| 12 | 12 | 16.15% | 12 | 6.36% | 24 | 5.08% | 36 | 8.45% |
| 13 | 13 | 5.71% | 13 | 6.22% | 26 | 4.96% | 39 | 12.38% |
| 14 | 14 | 1.33% | 14 | 6.06% | 28 | 4.87% | 42 | 9.93% |
| 15 | 15 | 0.26% | 15 | 5.73% | 30 | 5.86% | 45 | 5.04% |
| 16 | 16 | 0.05% | 16 | 5.13% | 32 | 8.51% | 48 | 2.42% |
| 17 | 17 | 0.01% | 17 | 4.31% | 34 | 9.65% | 51 | 1.58% |
| 18 | 18 | 0.00% | 18 | 3.40% | 36 | 7.04% | 54 | 1.26% |
| 19 | 19 | 0.00% | 19 | 2.52% | 38 | 3.49% | 57 | 1.06% |
| 20 | 20 | 0.00% | 20 | 1.77% | 40 | 1.42% | 60 | 0.89% |
| 21 | 21 | 0.00% | 21 | 1.19% | 42 | 0.62% | 63 | 0.75% |
| 22 | 22 | 0.00% | 22 | 0.78% | 44 | 0.33% | 66 | 0.63% |
| 23 | 23 | 0.00% | 23 | 0.49% | 46 | 0.19% | 69 | 0.53% |
| 24 | 24 | 0.00% | 24 | 0.30% | 48 | 0.11% | 72 | 0.45% |
| 25 | 25 | 0.00% | 25 | 0.19% | 50 | 0.06% | 75 | 0.38% |
| 26 | 26 | 0.00% | 26 | 0.11% | 52 | 0.04% | 78 | 0.32% |
| 27 | 27 | 0.00% | 27 | 0.07% | 54 | 0.02% | 81 | 0.27% |
| 28 | 28 | 0.00% | 28 | 0.04% | 56 | 0.01% | 84 | 0.23% |
| 29 | 29 | 0.00% | 29 | 0.03% | 58 | 0.01% | 87 | 0.19% |
| 30 | 30 | 0.00% | 30 | 0.02% | 60 | 0.00% | 90 | 0.16% |
| 31 | 31 | 0.00% | 31 | 0.01% | 62 | 0.00% | 93 | 0.14% |
| 32 | 32 | 0.00% | 32 | 0.01% | 64 | 0.00% | 96 | 0.12% |
| 33 | 33 | 0.00% | 33 | 0.00% | 66 | 0.00% | 99 | 0.10% |
| 34 | 34 | 0.00% | 34 | 0.00% | 68 | 0.00% | 102 | 0.09% |
| 35 | 35 | 0.00% | 35 | 0.00% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.00% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

**Table F7: Region 7 - Level 2 Axle Load Spectra Input for Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.49% | 4 | 0.01% | 6 | 0.00% |
| 3 | 3 | 0.16% | 3 | 4.06% | 6 | 0.34% | 9 | 0.36% |
| 4 | 4 | 5.78% | 4 | 15.19% | 8 | 2.33% | 12 | 6.08% |
| 5 | 5 | 12.62% | 5 | 14.40% | 10 | 5.93% | 15 | 11.01% |
| 6 | 6 | 5.35% | 6 | 8.61% | 12 | 8.68% | 18 | 16.42% |
| 7 | 7 | 1.09% | 7 | 6.62% | 14 | 9.19% | 21 | 10.15% |
| 8 | 8 | 2.25% | 8 | 5.74% | 16 | 7.99% | 24 | 5.41% |
| 9 | 9 | 6.77% | 9 | 5.21% | 18 | 6.21% | 27 | 5.35% |
| 10 | 10 | 13.55% | 10 | 4.95% | 20 | 4.76% | 30 | 5.99% |
| 11 | 11 | 21.82% | 11 | 4.84% | 22 | 4.06% | 33 | 6.20% |
| 12 | 12 | 19.56% | 12 | 4.70% | 24 | 3.98% | 36 | 5.94% |
| 13 | 13 | 8.61% | 13 | 4.45% | 26 | 4.09% | 39 | 5.36% |
| 14 | 14 | 2.06% | 14 | 4.05% | 28 | 4.00% | 42 | 4.62% |
| 15 | 15 | 0.32% | 15 | 3.54% | 30 | 4.28% | 45 | 3.84% |
| 16 | 16 | 0.04% | 16 | 2.99% | 32 | 7.81% | 48 | 3.11% |
| 17 | 17 | 0.01% | 17 | 2.44% | 34 | 12.21% | 51 | 2.46% |
| 18 | 18 | 0.00% | 18 | 1.94% | 36 | 9.20% | 54 | 1.92% |
| 19 | 19 | 0.00% | 19 | 1.50% | 38 | 3.43% | 57 | 1.47% |
| 20 | 20 | 0.00% | 20 | 1.14% | 40 | 0.90% | 60 | 1.12% |
| 21 | 21 | 0.00% | 21 | 0.86% | 42 | 0.31% | 63 | 0.84% |
| 22 | 22 | 0.00% | 22 | 0.63% | 44 | 0.15% | 66 | 0.63% |
| 23 | 23 | 0.00% | 23 | 0.46% | 46 | 0.08% | 69 | 0.47% |
| 24 | 24 | 0.00% | 24 | 0.33% | 48 | 0.04% | 72 | 0.35% |
| 25 | 25 | 0.00% | 25 | 0.24% | 50 | 0.02% | 75 | 0.26% |
| 26 | 26 | 0.00% | 26 | 0.17% | 52 | 0.01% | 78 | 0.19% |
| 27 | 27 | 0.00% | 27 | 0.12% | 54 | 0.00% | 81 | 0.14% |
| 28 | 28 | 0.00% | 28 | 0.09% | 56 | 0.00% | 84 | 0.10% |
| 29 | 29 | 0.00% | 29 | 0.06% | 58 | 0.00% | 87 | 0.07% |
| 30 | 30 | 0.00% | 30 | 0.05% | 60 | 0.00% | 90 | 0.05% |
| 31 | 31 | 0.00% | 31 | 0.03% | 62 | 0.00% | 93 | 0.04% |
| 32 | 32 | 0.00% | 32 | 0.02% | 64 | 0.00% | 96 | 0.03% |
| 33 | 33 | 0.00% | 33 | 0.02% | 66 | 0.00% | 99 | 0.02% |
| 34 | 34 | 0.00% | 34 | 0.01% | 68 | 0.00% | 102 | 0.02% |
| 35 | 35 | 0.00% | 35 | 0.01% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.01% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

**Table F8: Region 3 - Level 2 Axle Load Spectra Input for Non-Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.07% | 4 | 0.01% | 6 | 0.01% |
| 3 | 3 | 0.11% | 3 | 3.29% | 6 | 0.30% | 9 | 0.19% |
| 4 | 4 | 5.22% | 4 | 11.50% | 8 | 1.62% | 12 | 1.14% |
| 5 | 5 | 11.65% | 5 | 11.67% | 10 | 3.74% | 15 | 5.96% |
| 6 | 6 | 4.31% | 6 | 7.63% | 12 | 5.47% | 18 | 13.00% |
| 7 | 7 | 1.30% | 7 | 5.61% | 14 | 6.14% | 21 | 10.83% |
| 8 | 8 | 2.68% | 8 | 5.11% | 16 | 5.89% | 24 | 7.25% |
| 9 | 9 | 4.82% | 9 | 4.93% | 18 | 5.24% | 27 | 6.06% |
| 10 | 10 | 10.16% | 10 | 4.84% | 20 | 4.70% | 30 | 5.62% |
| 11 | 11 | 23.37% | 11 | 4.95% | 22 | 4.49% | 33 | 5.23% |
| 12 | 12 | 23.79% | 12 | 5.24% | 24 | 4.52% | 36 | 6.10% |
| 13 | 13 | 10.07% | 13 | 5.50% | 26 | 4.57% | 39 | 9.91% |
| 14 | 14 | 2.14% | 14 | 5.54% | 28 | 4.43% | 42 | 10.21% |
| 15 | 15 | 0.32% | 15 | 5.24% | 30 | 4.70% | 45 | 5.69% |
| 16 | 16 | 0.06% | 16 | 4.64% | 32 | 8.46% | 48 | 2.85% |
| 17 | 17 | 0.01% | 17 | 3.87% | 34 | 14.37% | 51 | 1.96% |
| 18 | 18 | 0.00% | 18 | 3.05% | 36 | 12.34% | 54 | 1.56% |
| 19 | 19 | 0.00% | 19 | 2.29% | 38 | 5.44% | 57 | 1.27% |
| 20 | 20 | 0.00% | 20 | 1.66% | 40 | 1.74% | 60 | 1.03% |
| 21 | 21 | 0.00% | 21 | 1.16% | 42 | 0.72% | 63 | 0.83% |
| 22 | 22 | 0.00% | 22 | 0.78% | 44 | 0.42% | 66 | 0.68% |
| 23 | 23 | 0.00% | 23 | 0.52% | 46 | 0.26% | 69 | 0.55% |
| 24 | 24 | 0.00% | 24 | 0.34% | 48 | 0.16% | 72 | 0.44% |
| 25 | 25 | 0.00% | 25 | 0.21% | 50 | 0.10% | 75 | 0.36% |
| 26 | 26 | 0.00% | 26 | 0.14% | 52 | 0.06% | 78 | 0.29% |
| 27 | 27 | 0.00% | 27 | 0.08% | 54 | 0.04% | 81 | 0.23% |
| 28 | 28 | 0.00% | 28 | 0.05% | 56 | 0.02% | 84 | 0.19% |
| 29 | 29 | 0.00% | 29 | 0.03% | 58 | 0.01% | 87 | 0.15% |
| 30 | 30 | 0.00% | 30 | 0.02% | 60 | 0.01% | 90 | 0.12% |
| 31 | 31 | 0.00% | 31 | 0.01% | 62 | 0.01% | 93 | 0.10% |
| 32 | 32 | 0.00% | 32 | 0.01% | 64 | 0.00% | 96 | 0.08% |
| 33 | 33 | 0.00% | 33 | 0.00% | 66 | 0.00% | 99 | 0.07% |
| 34 | 34 | 0.00% | 34 | 0.00% | 68 | 0.00% | 102 | 0.06% |
| 35 | 35 | 0.00% | 35 | 0.00% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.00% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

**Table F9: Region 7 - Level 2 Axle Load Spectra Input for Non-Interstate Highway**

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.20% | 4 | 0.01% | 6 | 0.01% |
| 3 | 3 | 0.15% | 3 | 3.41% | 6 | 0.53% | 9 | 1.07% |
| 4 | 4 | 2.78% | 4 | 9.97% | 8 | 3.23% | 12 | 5.50% |
| 5 | 5 | 14.43% | 5 | 12.58% | 10 | 7.47% | 15 | 11.88% |
| 6 | 6 | 4.67% | 6 | 10.81% | 12 | 10.17% | 18 | 16.23% |
| 7 | 7 | 4.38% | 7 | 8.42% | 14 | 10.19% | 21 | 11.57% |
| 8 | 8 | 5.17% | 8 | 7.02% | 16 | 8.50% | 24 | 5.40% |
| 9 | 9 | 8.64% | 9 | 6.27% | 18 | 6.47% | 27 | 3.23% |
| 10 | 10 | 14.86% | 10 | 5.67% | 20 | 4.89% | 30 | 3.66% |
| 11 | 11 | 17.35% | 11 | 5.10% | 22 | 4.01% | 33 | 4.71% |
| 12 | 12 | 13.37% | 12 | 4.60% | 24 | 3.70% | 36 | 5.48% |
| 13 | 13 | 7.42% | 13 | 4.18% | 26 | 3.68% | 39 | 5.71% |
| 14 | 14 | 3.36% | 14 | 3.79% | 28 | 3.83% | 42 | 5.42% |
| 15 | 15 | 1.45% | 15 | 3.40% | 30 | 4.57% | 45 | 4.78% |
| 16 | 16 | 0.70% | 16 | 2.99% | 32 | 6.22% | 48 | 3.97% |
| 17 | 17 | 0.41% | 17 | 2.55% | 34 | 7.08% | 51 | 3.15% |
| 18 | 18 | 0.27% | 18 | 2.12% | 36 | 5.72% | 54 | 2.40% |
| 19 | 19 | 0.18% | 19 | 1.71% | 38 | 3.51% | 57 | 1.77% |
| 20 | 20 | 0.13% | 20 | 1.35% | 40 | 2.01% | 60 | 1.28% |
| 21 | 21 | 0.09% | 21 | 1.04% | 42 | 1.27% | 63 | 0.90% |
| 22 | 22 | 0.06% | 22 | 0.78% | 44 | 0.89% | 66 | 0.62% |
| 23 | 23 | 0.04% | 23 | 0.58% | 46 | 0.63% | 69 | 0.42% |
| 24 | 24 | 0.03% | 24 | 0.43% | 48 | 0.45% | 72 | 0.29% |
| 25 | 25 | 0.02% | 25 | 0.31% | 50 | 0.31% | 75 | 0.19% |
| 26 | 26 | 0.02% | 26 | 0.22% | 52 | 0.21% | 78 | 0.13% |
| 27 | 27 | 0.01% | 27 | 0.16% | 54 | 0.15% | 81 | 0.08% |
| 28 | 28 | 0.01% | 28 | 0.11% | 56 | 0.10% | 84 | 0.05% |
| 29 | 29 | 0.01% | 29 | 0.08% | 58 | 0.07% | 87 | 0.04% |
| 30 | 30 | 0.00% | 30 | 0.05% | 60 | 0.04% | 90 | 0.02% |
| 31 | 31 | 0.00% | 31 | 0.04% | 62 | 0.03% | 93 | 0.02% |
| 32 | 32 | 0.00% | 32 | 0.03% | 64 | 0.02% | 96 | 0.01% |
| 33 | 33 | 0.00% | 33 | 0.02% | 66 | 0.01% | 99 | 0.01% |
| 34 | 34 | 0.00% | 34 | 0.01% | 68 | 0.01% | 102 | 0.00% |
| 35 | 35 | 0.00% | 35 | 0.01% | 70 | 0.01% | | |
| 36 | 36 | 0.00% | 36 | 0.01% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

# Appendix G: Level 3 Axle Load Spectra Input for the Mechanistic-Empirical Pavement Design Guide

## Table G1: Level 3 - Axle Load Spectra Input (from Fitted Functions)

| | Steering | | Single w/ Dual Wheels | | Tandem | | Tridem | |
|---|---|---|---|---|---|---|---|---|
| Number | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency | Bin(kip) | Frequency |
| 1 | 1 | 0.00% | 1 | 0.00% | 2 | 0.00% | 3 | 0.00% |
| 2 | 2 | 0.00% | 2 | 0.19% | 4 | 0.01% | 6 | 0.20% |
| 3 | 3 | 0.18% | 3 | 4.01% | 6 | 0.36% | 9 | 1.11% |
| 4 | 4 | 5.19% | 4 | 11.08% | 8 | 2.27% | 12 | 4.16% |
| 5 | 5 | 10.23% | 5 | 11.95% | 10 | 5.48% | 15 | 11.02% |
| 6 | 6 | 4.48% | 6 | 8.80% | 12 | 7.88% | 18 | 13.56% |
| 7 | 7 | 2.28% | 7 | 6.46% | 14 | 8.36% | 21 | 9.49% |
| 8 | 8 | 4.28% | 8 | 5.68% | 16 | 7.45% | 24 | 5.84% |
| 9 | 9 | 7.49% | 9 | 5.58% | 18 | 6.14% | 27 | 4.45% |
| 10 | 10 | 13.67% | 10 | 5.48% | 20 | 5.14% | 30 | 4.68% |
| 11 | 11 | 20.76% | 11 | 5.19% | 22 | 4.66% | 33 | 5.65% |
| 12 | 12 | 18.03% | 12 | 4.80% | 24 | 4.55% | 36 | 6.54% |
| 13 | 13 | 8.98% | 13 | 4.47% | 26 | 4.53% | 39 | 6.83% |
| 14 | 14 | 3.03% | 14 | 4.29% | 28 | 4.52% | 42 | 6.37% |
| 15 | 15 | 0.90% | 15 | 4.17% | 30 | 5.30% | 45 | 5.38% |
| 16 | 16 | 0.30% | 16 | 3.96% | 32 | 7.71% | 48 | 4.19% |
| 17 | 17 | 0.11% | 17 | 3.54% | 34 | 9.35% | 51 | 3.07% |
| 18 | 18 | 0.04% | 18 | 2.94% | 36 | 7.48% | 54 | 2.16% |
| 19 | 19 | 0.02% | 19 | 2.28% | 38 | 4.13% | 57 | 1.48% |
| 20 | 20 | 0.01% | 20 | 1.65% | 40 | 1.94% | 60 | 1.01% |
| 21 | 21 | 0.00% | 21 | 1.14% | 42 | 1.00% | 63 | 0.69% |
| 22 | 22 | 0.00% | 22 | 0.77% | 44 | 0.61% | 66 | 0.49% |
| 23 | 23 | 0.00% | 23 | 0.50% | 46 | 0.40% | 69 | 0.36% |
| 24 | 24 | 0.00% | 24 | 0.33% | 48 | 0.26% | 72 | 0.27% |
| 25 | 25 | 0.00% | 25 | 0.22% | 50 | 0.17% | 75 | 0.21% |
| 26 | 26 | 0.00% | 26 | 0.15% | 52 | 0.11% | 78 | 0.17% |
| 27 | 27 | 0.00% | 27 | 0.10% | 54 | 0.07% | 81 | 0.13% |
| 28 | 28 | 0.00% | 28 | 0.07% | 56 | 0.04% | 84 | 0.11% |
| 29 | 29 | 0.00% | 29 | 0.05% | 58 | 0.03% | 87 | 0.09% |
| 30 | 30 | 0.00% | 30 | 0.04% | 60 | 0.02% | 90 | 0.08% |
| 31 | 31 | 0.00% | 31 | 0.03% | 62 | 0.01% | 93 | 0.07% |
| 32 | 32 | 0.00% | 32 | 0.02% | 64 | 0.01% | 96 | 0.06% |
| 33 | 33 | 0.00% | 33 | 0.01% | 66 | 0.00% | 99 | 0.05% |
| 34 | 34 | 0.00% | 34 | 0.01% | 68 | 0.00% | 102 | 0.04% |
| 35 | 35 | 0.00% | 35 | 0.01% | 70 | 0.00% | | |
| 36 | 36 | 0.00% | 36 | 0.01% | 72 | 0.00% | | |
| 37 | 37 | 0.00% | 37 | 0.00% | 74 | 0.00% | | |
| 38 | 38 | 0.00% | 38 | 0.00% | 76 | 0.00% | | |
| 39 | 39 | 0.00% | 39 | 0.00% | 78 | 0.00% | | |
| 40 | 40 | 0.00% | 40 | 0.00% | 80 | 0.00% | | |
| 41 | 41 | 0.00% | 41 | 0.00% | 82 | 0.00% | | |

# Appendix References

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