University of Alberta

Evaluation of Weigh-In-Motion Systems in Alberta

by

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This thesis is proudly dedicated to

my mother who gave me the gift of happiness

and

my father who showed me the world of success

ABSTRACT

Weigh-In-Motion (WIM) systems are used for dynamic traffic data collection. These sensors are capable of collecting various truck characteristics such as weights, speed, and dimensions. Alberta Transportation (AT) installed 20 WIM sensors in six different highway sections across Alberta in 2004. The accuracy of these measurements and their effects on pavement design is evaluated in this thesis.

To investigate the accuracy of the WIM sensors a verification test was conducted on the sensors from 2004 to 2010. The errors in the WIM sensors' measurements were estimated. Statistical analysis was performed on the database of errors.

Statistical analysis on the verification test program database showed that WIM weight errors do not comply with current standards and there is a need to improve the system. The new predicted pavement performance results from the Mechanistic Empirical Pavement Design Guide (MEPDG) showed that local WIM traffic data inputs should be used for Alberta highways.

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GLOSSARY

AASHTO	American Association for State Highway and Transportation Officials
AGC	Automatic Gain Control
AT	Alberta Transportation
ATR	Automatic Traffic Recorder
EB	Eastbound
FHWA	Federal Highway Administration
GBC	Granular Base Course
HDF	Hourly Distribution Factor
HMA	Hot Mixed Asphalt
IRI	International Roughness Index
LTPP	Long Term Pavement Performance
MAF	Monthly Adjustment Factor
MEPDG	Mechanistic Empirical Pavement Design Guide
NB	Northbound
PC	Probability of Conformity
PMS	Pavement Management System
SB	Southbound

SUT	Single Unit Truck
TGF	Traffic Growth Factor
TTC	Truck Traffic Classification
TTC	Tractor Trailer Combination
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregates
WB	Westbound
WIM	Weigh In Motion

CHAPTER 1

1.0 Introduction

1.1 Weigh-in-Motion Systems

Weigh-in-Motion (WIM) sensors collect various traffic data as vehicles pass over them without any need for the vehicles to be stopped. Data recorded by a WIM sensor include vehicle's speed, axle loads, wheel loads, gross vehicle weights, distances between axles, and vehicle's class. The WIM sensors are used by highway agencies for: 1) Weight enforcement application and safety evaluation, 2) Pavement design purposes (1). "WIM technology was first introduced in Canada in 1982" (2). Other regions such as United States and European countries applied the WIM technology in a broader range to preserve their road network. In Europe, the application was enhanced to be used for pavements, bridges, and railways during the 1980s (2). Currently, "there are thousands of WM systems in use across the North America and around the world" (1).

Four main categories of parameters are measured using a WIM sensor: 1) Weight, 2) Speed, 3) Vehicle Classification, and 4) Identification. Wheel loads, axle loads, and Gross Vehicle Weights (GVW) are measured under the category of Weight. Wheel load is the amount of weight for each wheel assembly of a vehicle, at the end of each axle (3). Axle load is defined as the amount of load that each axle of tires (single, tandem, tridem, or quad) carries. Finally, GVW is defined as the total weight of the vehicle, which is equal to the summation of all of the axle loads for a specific vehicle (3).

The speed of the passing traffic is recorded as a result of the use of two or more sensors recording the elapsed time of tire and sensor contact and the distance between the two connected sensors in a WIM system.

WIM sensors are capable of classifying the passing vehicle based on a predetermined classification criterion such as the Federal Highway Administration (FHWA) vehicle classification. The distances between the axles are used as the basis for the classification.

WIM systems are able to record a large amount of traffic data in a single day. All the mentioned parameters should be calculated and assigned to each passing vehicle. The Identification parameters include site identification codes, lane and direction of travel, date and time of passage, and sequential vehicle record number (3).

A WIM sensor is embedded into the top layer of a flexible or rigid pavement and is aligned with this layer to provide a smooth surface. In this study, the term WIM system is used for a combination of WIM sensors in different lanes on one highway working under a computer program for interpreting the collected data.

The WIM sensors can work under three main groups of technologies. Each type is associated with different level of accuracy, sensitivity, installation and maintenance costs. The three main WIM types include: 1) Piezoelectric, 2) Bending plate, and 3) Load cell (2).

1.2 WIM Systems in Alberta

Alberta Transportation (AT) installed 20 piezoelectric WIM sensors in six different highway locations across Alberta in 2004. Based on the information provided by the AT personnel, the total installation costs for these sensors was approximated at $2.5 million at the year of installation (correspondence with AT via Email). These sensors started collecting the traffic data under the WIM verification test program in 2005 to present. The verification test program is explained in detail in the following sections. Concisely, this program was conducted to evaluate the accuracies of the WIM systems in Alberta. Figure 1.1 presents information on the locations of the WIM sensors in Alberta. There are six highway: control sections (Hwy:Cs) in this figure which contain the WIM systems. This figure provides information on how the WIM systems are distributed across the Province of Alberta. As seen in this figure, the majority of WIM systems are located in the central part of the Province near the City of Edmonton. Lethbridge and Red Deer are two other cities with WIM systems installed near them.

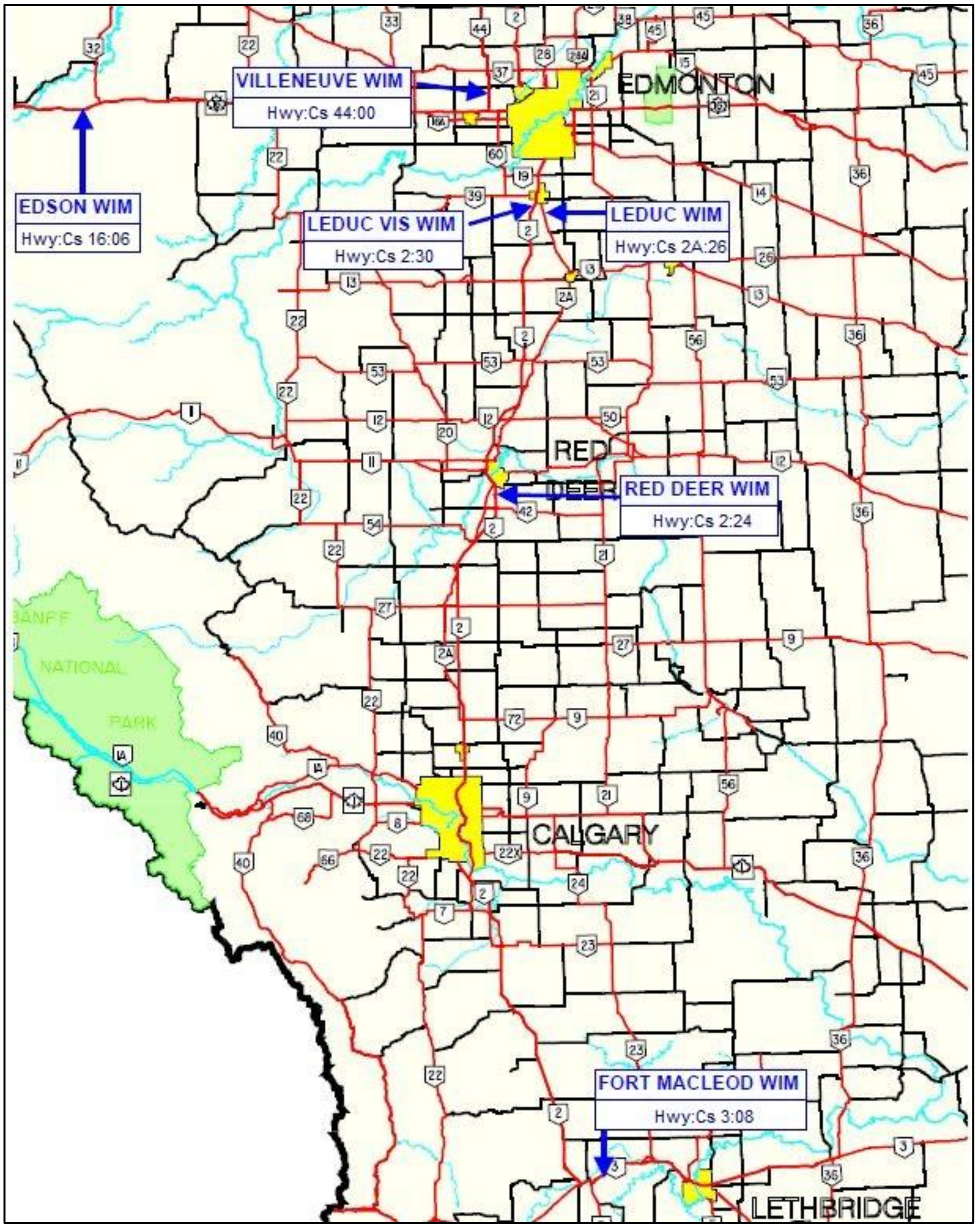

Figure 1.1 Geographical locations of the six WIM sites in Alberta – Source: Alberta Transportation website at http://www.transportation.alberta.ca

Table 1.1 provides more information on the location of the WIM systems in Alberta together with the traffic information such as Average Annual Daily Traffic (AADT) and Equivalent Single Axle Load per day per direction (ESAL/day/direction) calculated. The former is derived from Automated Traffic

Recorders (ATR) distributed in the entire highway network reflected in AT's 2010 Pavement Management Systems (PMS) report (4) and the latter is calculated using Equation 1.1 (5):

$$ESAL/day/direction = \frac{AADT}{2}\left[\frac{(\%SUT)}{100} \times 0.881 + \frac{(\%TTC)}{100} \times 2.073\right] \qquad \text{Eq. 1.1}$$

Where, %SUT and %TTC stand for the percentage of Single Unit Trucks (SUT) and Tractor Trailer Combinations (TTC).

As seen in Table 1.1, Highway Sections 2:24 and 2:30 have AADTs of 30,900 and 24,848, respectively, while Highway Sections 2A:26, 3:08, 16:06, and 44:00 show lower AADTs ranging from 6,970 to 8,130.

As seen in Table 1.1, four out of six locations have four lanes with WIM sensors (divided highways) and the remaining two locations have two lanes with WIM sensors (undivided highways) coming to a total of 20 sensors. The posted speed varies for the divided and undivided highways and is 110 km/h for the divided highways and 100 km/h for the undivided highways.

Table 1.1 – Information Regarding the WIM sites in Alberta

Highway: Control Section	Km at WIM	Number of Lanes with WIM	Geographical Location	Posted Speed (km/h)	AADT (2010)	ESAL/Day/ Direction (2010)
2:24	18	4	2.6 Km North of Hwy 2 and Hwy 42 Penhold (Red Deer)	110	30,900	3,741
2:30	30	4	2.0 Km South of Hwy 2 and Hwy 2A Leduc VIS (Leduc VIS)	110	24,848	3,584
2A:26	27	2	3.7 Km South of Hwy 2A and Hwy 2 Leduc (Leduc)	100	7,190	372
3:08	13	4	8.0 Km East of Fort Macleod (Ft. Macleaod)	110	7,260	975
16:06	39	4	5.8 Km West of Hwy 16 and Hwy 32 Edson (Edson)	110	8,130	1,550
44:00	6	2	3.4 Km South of Hwy 44 and Hwy 633 Villeneuve (Villeneuve)	100	6,970	1,881

1.2.1 WIM Sensor Calibration Procedure

All the 20 WIM sensors in Alberta are ECM Hestia-P – dual piezoelectric manufactured in France (correspondence with AT via Email). This type of sensor utilizes an Automatic Gain Control (AGC) algorithm to calibrate the WIM sensors. As the output of the piezoelectric sensors is sensitive to the changes in pavement and sensor temperature, a characteristic vehicle is selected for the calibration at different temperatures using the AGC algorithm. The following weight characteristics of the calibration vehicle are measured by the WIM sensor at different temperatures (6):

- The minimum weight of the characteristic vehicle
- The average weight value of the first axle, and
- The average total weight of the characteristic vehicle.

The Hestia will then adjust the AGC to compensate for the temperature drift, based on the recorded weights for the characteristic vehicle. The calibration procedure is not the concern of this study and it is more focused on the verification test program which will be explained in the following section.

1.2.2 WIM Verification Program

In order to verify the accuracy of the WIM systems in Alberta, AT runs a full repeatability and full environmental reproducibility verification test program. Every month, a FHWA Class 9 truck (Figure 1.2) passes 10 times over each of the 20 WIM sensors in Alberta. As seen in Figure 1.2, the FHWA Class 9 truck is a five-axle single trailer truck which is one of the dominant truck types in the Province of Alberta. The verification truck passes over the WIM sensors at the highway posted speeds of 100 km/h or 110 km/h depending on the highway type. The program started in 2005 and continued to 2010 providing a comprehensive database for the analysis and complete evaluation. Chapters 3 and 4 of this study focus on the analysis of the verification test program.

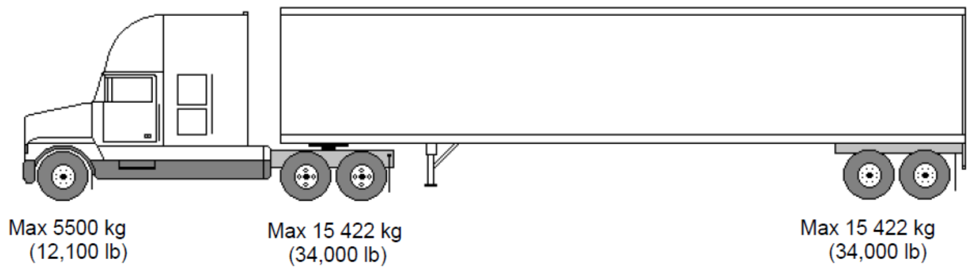

Figure 1.2 – Typical FHWA Class 9 truck used for WIM verification test program in Alberta (7)

1.3 Mechanistic Empirical Pavement Design Guide

Currently, pavement design is performed using the *Guide for Design of Pavement Structures* developed by the American Association of State Highway and Transportation Officials (AASHTO) revised for the last time in 1993. The AASHTO 1993 Guide uses empirical equations developed based on the serviceability loss of the AASHO Road Test during the tests in the late 1950s (8). A need for using a more mechanistic approach was recognized by transportation experts. As a result, AASHTO joined the National Cooperative Highway Research Program (NCHRP) and FHWA in 1997 to start developing a Mechanistic-Empirical pavement design (MEPDG) procedure under the NCHRP Project 1-37A (8).

The MEPDG software was released in 2004 preliminary for further discussion and commentary works (8) The MEPDG Software can be used for new pavement sections as well as rehabilitation sections. The MEPDG predicts pavement distresses such as longitudinal and transverse cracking, fatigue cracking, alligator cracking, rutting, and International Roughness Index (IRI) for flexible pavements.

The MEPDG is capable of performing both rigid and flexible pavement designs with various material properties. More than 100 design inputs are required for every pavement design scenario. Additionally, the MEPDG software accepts regional climate data to consider sophisticated environmental properties for its design procedure.

For its traffic inputs, which are the main focus of this study, the MEPDG accepts a wide range of traffic data, while the AASHTO 1993 uses a single input, the design ESAL, to characterize the traffic loads. In this approach the axle loads of two or three major truck classes are converted into a single standard axle load of 8.1 tons using load equivalency factors. The MEPDG, however, uses a more sophisticated approach. The distribution of all of the axle types is implemented into the design, as well as other traffic parameters. Based on different load groups defined in the MEPDG, the axle load distribution factor is calculated. The distribution frequency of each axle group having a known percentage of axle loads within that load group is defined as the axle load distribution factor. At the same time, more detailed traffic characteristics are included in the MEPDG such as the Hourly Distribution Factor (HDF), Monthly Adjustment Factor (MAF), Truck Traffic Classification (TTC), and traffic growth factor. More details are provided in Chapter 5 on above parameters.

The MEPDG traffic inputs such as HDF, MAF, vehicle class distributions, and axle load distributions can be defined at three levels: Level 1, 2 and 3 (8). Level 1 traffic inputs are those which are measured directly and specifically at each site. At Level 2 traffic inputs are estimated from correlation or regression equations with other sites (8). At Level 3 the default values in the MEPDG software are used. The default values in the Guide are based on the traffic input values for the LTPP test sections in North America. Using the AT's WIM traffic data for 2009 and 2010, a comparison is made between the default traffic inputs in the software and what is available from AT in Chapter 5.

For the final part of this thesis, the effect of extreme deviations of the AT's traffic data from the default values in the software on the predicted performance of the pavement is investigated.

1.4 Research Objectives

AT provided the University with a substantial amount of traffic data derived from its six WIM systems in Alberta. The first portion of this traffic data is related to the WIM verification test program. Since not all of the 20 WIM sensors were operational in 2005, year 2006 is selected as the starting year for this study. As a result, five years of data is considered for evaluation in this study. The second portion of the traffic data provided by AT consists of MEPDG traffic input values established based on two years of real traffic data derived from the WIM systems in the province. This data will be compared with the MEPDG default values incorporated in the software. The extreme cases will be entered into the software to get the performance predictions and to compare with those performances delivered by the default values in the software. In the following, the objectives of this study are provided:

- The accuracy of WIM systems in Alberta is evaluated statistically through analyzing the data from 2006 to 2010, derived from WIM verification test program.
- The effects of potential inaccuracies of generated WIM measurements on the pavement thickness design are performed.
- Predicted performance of the pavement is evaluated from two WIM sources of: 1) local traffic inputs and 2) MEPDG software default traffic inputs. These inputs are compared and the effects of potential differences on the performance level are presented.

1.5 Scope

- Basic statistics of the weight errors for the WIM systems, such as minimum, maximum, mean, first quartile, second quartile, and third quartile values are presented.
- Using the speed recordings of the WIM systems in Alberta, deviation of the recorded speeds from actual speeds of passing FHWA Class 9 truck is investigated.
- Regarding the WIM verification test program, the error levels are compared to the American Society for Testing and Materials (ASTM) E1318 requirements. The 95% compliance is checked in this part of the study to see whether or not the errors fall into the accepted limits by this compliance level.
- An outlier analysis is performed on the generated WIM weight errors.

- Statistical distribution of the WIM weight errors are derived for 4 different scenarios using SPSS software Version 19.0.
- Probability of conformity (PC) checking for the WIM weight measurements for two sub-categories of verification years and site locations are performed.
- The effects of WIM weight errors on a typical AT flexible pavement section is studied using the current pavement design practice (equivalent single axle load concept). The extra asphalt layer needed due to existence of errors in capturing the weights are calculated.
- A comparison between AADTs captured by the current Automatic Traffic Recorder (ATR) installed in the interested control sections and the traffic data collected by WIM systems are presented to check the traffic counting task of current and future systems in place. Having the AADTs from both sources, the ESAL calculations are performed. The results from these two approaches are compared.
- Traffic inputs, based on WIM systems in Alberta and provided data from AT, are compared to the default values of MEPDG Software package.
- The effect of the deviations of AT's data with the default values of MEPDG software package on the pavement performance is accomplished. This sensitivity analysis shows how different traffic characteristics such as MAFs, HDFs, TTCs, and axle load distribution factors can affect the final performance of the pavement.

1.6 Thesis Organization

The organization and the contents of each chapter are provided below.

Chapter 1 provides an introduction to the WIM systems with a more detailed emphasis on the piezoelectric WIM sensors which are installed in Alberta. The AT’s WIM verification test program is introduced in this chapter. Additionally, the new Mechanistic Empirical Pavement Design Guide is introduced in this chapter. Finally, the need for evaluating the performance of a typical flexible pavement under default values of the software package and the AT’s traffic data is declared.

Chapter 2 is a review of past research works in the context of WIM. Different studies are covered in this chapter to provide information regarding the WIM systems and also to build up the existing knowledge in this area.

Chapter 3 focuses on the statistical analysis of the WIM errors with a more detailed focus on the WIM weight errors. On the basis of WIM verification test program, the accuracy of WIM systems in Alberta is investigated. ASTM E1318 requirements are used for this purpose. A sensitivity analysis on the WIM weight errors is implemented to evaluate the importance of years, months, locations, and axle types.

In Chapter 4, the effects of the WIM weight errors on the final pavement design are discussed. Current methodology for pavement design uses ESAL calculations from ATR data. Another way to capture traffic counting is using WIM systems. A comparison is made between these two sources of data and their related pavement designs.

Chapter 5 focuses on the MEPDG software package. With two years of real-time traffic data provided by AT, an evaluation on AT's traffic data is performed by checking them with default values, implemented into the software package. Pavement performances under default traffic characteristics introduced by the Guide and the extreme cases of AT's data in terms of these traffic parameters are evaluated.

Chapter 6 provides a summary and conclusion of this study and provide with future recommendations.

CHAPTER 2

2.0 Literature Review

2.1 Introduction

In this chapter, a summary of the studies conducted on the WIM systems are presented. First, different WIM technologies are described. In the next part of this chapter, the accuracy of the WIM sensors is investigated and practical ways to decrease the errors are discussed. The effects of these errors on the pavement structural performance are studied in the final part in this chapter.

2.2 WIM Technology

The main idea in using the WIM sensors is to embed sensors for recording the subjected weights. However, this technology does not capture the weight which is the product of mass and acceleration due to gravity (9). Rather it measures the "instantaneous impact of force *F* resulted from the various masses in the vehicle" (9). Different parameters such as the vehicle speed, the condition of the pavement, suspension parameters which are damping, friction, and stiffness affect this force *F* measured by the sensor (9).

Three main types of WIM sensor are used: 1) Piezoelectric sensors, 2) Bending plate, and 3) Load cell. The piezoelectric sensors can be divided into piezo-ceramic strips, piezopolymer strips and piezoquartz strips (10). It should be noted that AT used piezoelectric sensors for its WIM systems. Three types of WIM systems are described below (2):

2.2.1 Piezoelectric Sensors

"A Piezoelectric WIM system consists of at least one sensor and two inductive loops embedded in the road cut" (2). This type of sensor is installed perpendicular to the direction of traffic. Having a shape of a thin bar, the wheels of one axle of the vehicle touch the sensor at the same time. Figure 2.1 presents a schematic view of this type of sensor.

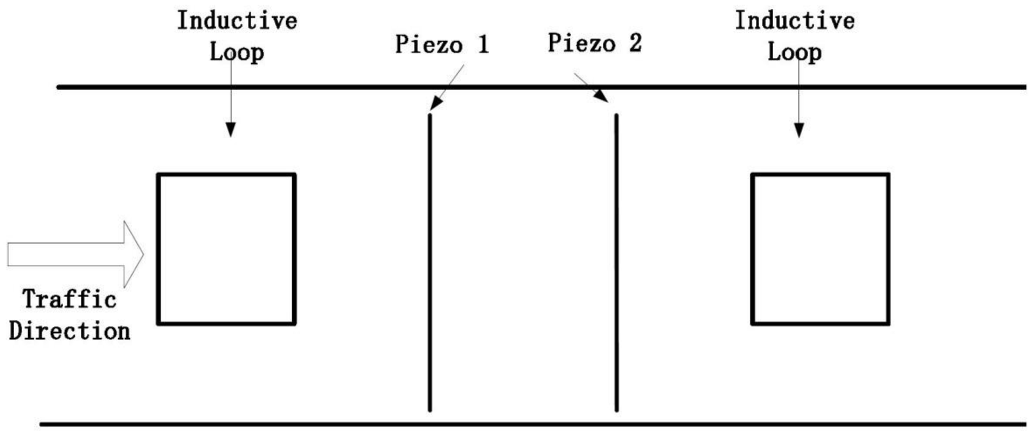

Figure 2.1 Schematic view of piezoelectric WIM system (2)

As seen in Figure 2.1, there are two piezoelectric sensors plus two inductive loops. The inductive loop at the upstream of the traffic helps the system to identify the approaching traffic. "It triggers a sequence of event including: the WIM sensor signal detection, amplification, and collection" (2). The other inductive loop at the downstream makes it possible to detect vehicles' speed and configuration by capturing the distances between different axles. For the purpose of vehicle classification, different parameters such as axle spacing, number of axles, length of the vehicle and its gross vehicle weight are required. In order to measure the amount of the weight subjected to the sensor, the piezoelectric sensors use the voltage generated by the impact of the axles which is proportional to this force.

Some of the advantages of a piezoelectric sensor are its low costs and quick installation in the existing pavement. As for its disadvantages, low repeatability, low accuracy, and the contact of tires with the strip could be mentioned (2).

2.2.2 Bending plate

The technology for bending plates is similar to piezoelectric systems. The only difference in this method is that the scale is not piezo-based. It utilizes strain gauges to detect and record the weights (2). The schematic of a bending plate system is shown in Figure 2.2.

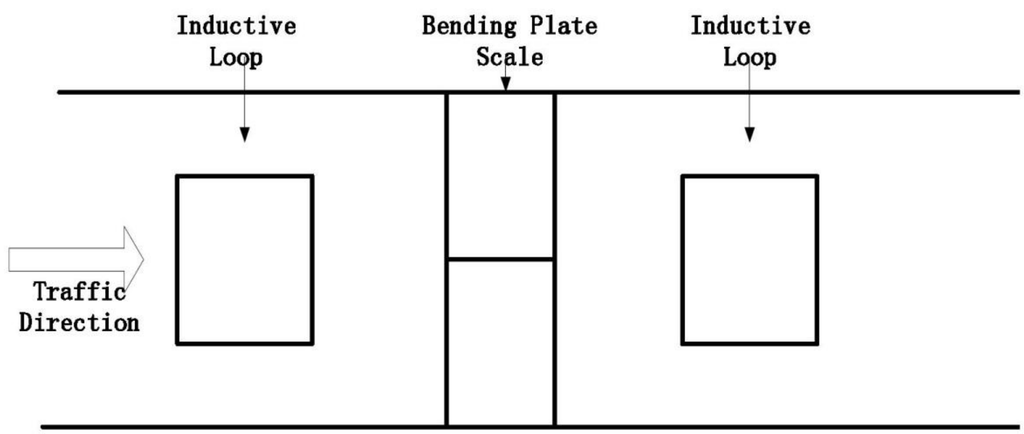

Figure 2.2 – Schematic view of a bending plate WIM system (2)

As seen in Figure 2.2, the only difference between the bending plate and piezoelectric WIMs is in the weight sensing technology. The inductive loops do the same task as in the piezoelectric WIM systems.

2.2.3 Load Cell

"A typical load cell WIM system consists of a single load cell that has two in-line scales, at least one inductive loop, and an axle sensor" (2). The schematic of this type of WIM sensor is shown in Figure 2.3.

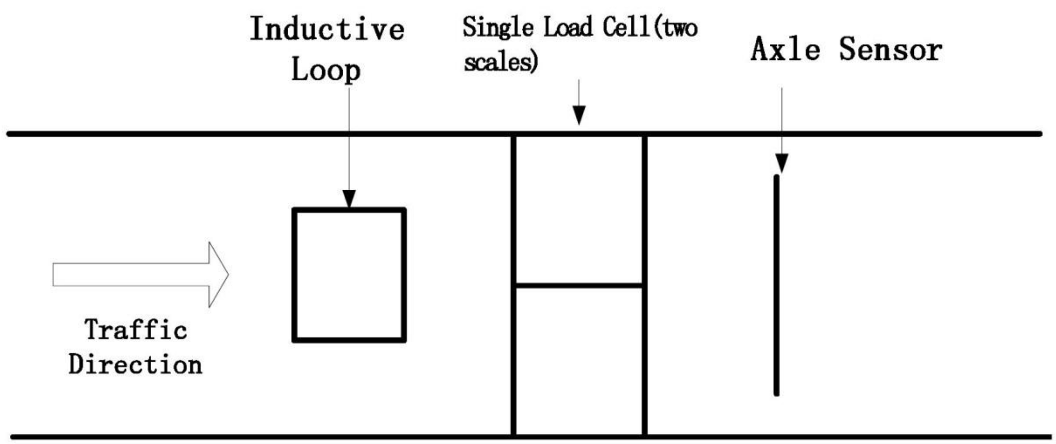

Figure 2.3 – Schematic view of a load cell WIM system (2)

In this system, the inductive loop detects the approaching traffic. Vehicle speed and axle spacing which leads to the classification of the passing vehicle are recoded by the axle sensor. The load cell utilizes a combination of strain gauge technology and resistance to pressure phenomenon. Opposite to the two previous types, the load cell collects the weight of each wheel separately and sums them up to form the axle weight.

2.3 Errors Associated with WIM

Regarding the accuracy and the errors associated with WIM sensors, research works have been done centering on the nature of the errors, the magnitude of these errors, and how they can affect the pavement design.

2.3.1 Nature of the Errors Associated with WIM Sensors

WIM errors are made up of three components (11):

- Actual static and dynamic force differences;
- Dynamic force measurement errors; and
- Static load measurement errors.

The first component consists of the errors that are caused by differences in the nature of any dynamic and static force capturing. The second and third components are related to the measuring errors that are generated for each type of measurement (dynamic or static). These errors are generally associated with any type of measuring tool.

Three main sources can cause these errors: 1) roadway characteristics such as the road's smoothness and water content of the pavement 2) vehicular characteristics such as speed, acceleration, suspension type, and tire condition, 3) environmental factors such as wind speed and direction, and ambient temperature (11). Adding to different factors, errors could be as a result of eccentric loading, bending, lateral forces, creep, or electromagnetic susceptibility. Some external factors include tilting of the vehicle and sensors, friction in suspension, vehicle oscillation, aerodynamic forces, sensor installation (leveling), the site and access evenness, and driver or operator behavior (12).

2.3.2 WIM Measurements' Errors – Statistical Point of View

The errors in the WIM weight measurements generally have a normal distribution (13). Standard deviation of the errors can be different and also the mean value can be shifted to the left or right. These variations are because of the nature of the random errors produced by the WIM system. “Random errors are caused by inherently unpredictable fluctuations in the readings of a measurement apparatus” (14). “Systematic error cannot be discovered this way because it always pushes the results in one direction”(14).

Figures 2.4 and 2.5 show different distributions of the WIM measurements’ errors. Figure 2.4 shows how the WIM measurement error can have zero bias while having different standard deviations from 1.5% to 10%. As the standard deviation gets closer to zero, the accuracy of the weight measurements increases.

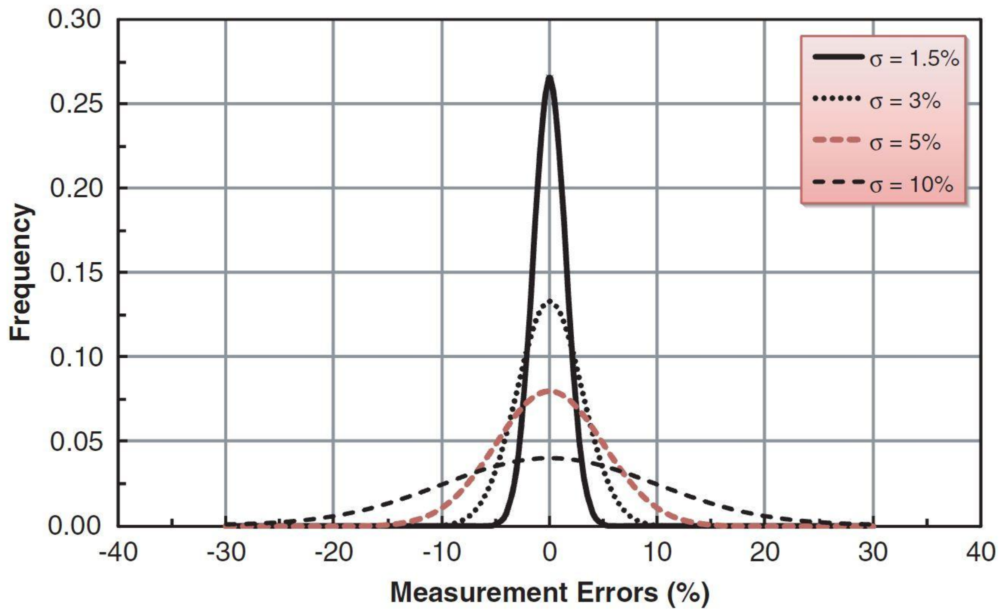

Figure 2.4 – Different distributions for the random error associated with WIM measurement with no bias (11)

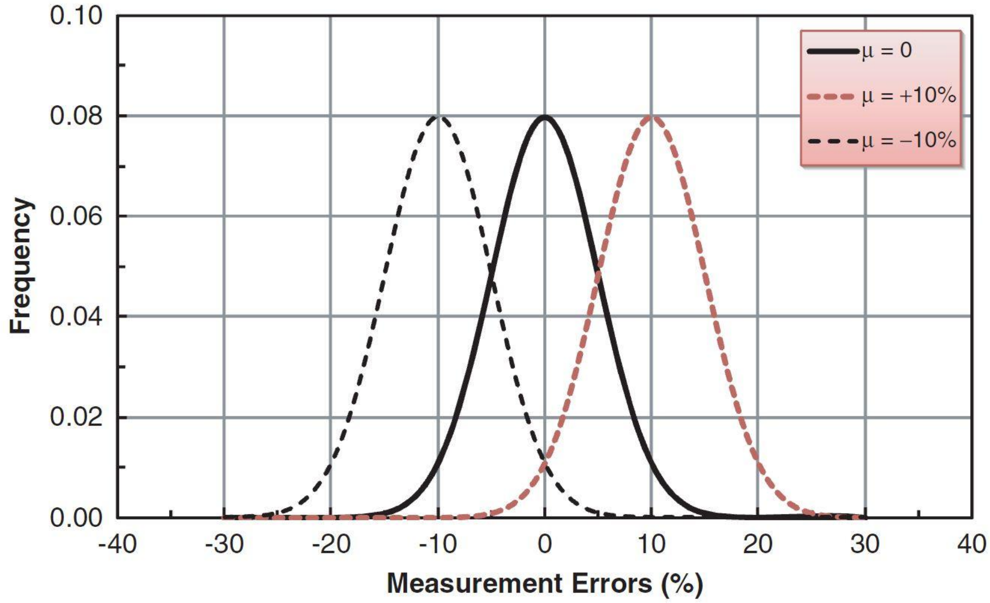

Figure 2.5 –Biased WIM measurement error distribution (11)

Figure 2.5 shows how the distribution of errors could be pushed to one side (positive or negative side. As seen in this figure, +10% and -10% of biased distribution for WIM errors exist (dashed line) along with zero mean (solid line). As noted previously in Section 2.3.1, many factors affect the intensity and frequency of the errors within different error intervals.

A case study was conducted in August 1997, which evaluated the accuracy of a WIM system in Manitoba. The WIM system underestimated approximately 90 percent of the truck weights during the test period. The degree of underestimation exceeded 50 percent of the corresponding static weights (1).

The ASTM E1318 sets confidence intervals for accepting the WIM systems accuracy. At 95% confidence interval, the allowable range for the errors is ±10.0% to ±20.0%. The quality of the WIM measurements, based on the steering axle of a five-axle semi-trailer 3-S2 truck was evaluated (15) and through doing that the confidence interval limits were established based on historic mean static loads. It was found by Ott et al. in 1996 (15) that the confidence interval limits could be a function of pavement roughness and vehicle speeds.

In Oregon, Ali et al. in 1993 (16) weighed the axle weights of five-axle tractor semi-trailer trucks using a piezoelectric WIM system and compared the values with the static axle weight measurements. Using a variance analysis and Dunnett's test, they compared the static and WIM axle weights and concluded that no significant difference existed for the steering and tandem axle weights; however, there was a significant difference for the trailing tandem axle.

Collop et al. in 2002 (17) used the data from 15 WIM systems in England. Three sources of error were identified: calibration errors, random sensor errors, and dynamic load effect errors. The 5-95th percentile range and the standard deviation of the calibration errors were found to be 30 and 11 percent, respectively. The average value of the error for a random sensor and a dynamic load effect was found to be 11 percent. The effect of these errors on the pavement design, using a fourth power law showed that sensor/dynamic and calibration errors are likely to over-predict the traffic by 15-20 percent. This would result in a 5-15 mm pavement thickness overdesign for a typical flexible pavement structure.

2.3.3 WIM Calibration

Once a WIM system is installed, it needs to be calibrated in order to generate unbiased measurements. It means that due to several factors that affect the accuracy of WIM sensors, some biased errors are probable to be generated. In the course of time, the sensor can be affected by either the axle impacts or the environmental factors mentioned earlier and become inaccurate. The calibration for WIM systems in Alberta are conducted by the contractor in charge of the WIM system, on a random basis. Unfortunately, reliable records of the calibration procedure was not available for the WIM systems in Alberta to investigate the effects of calibration on the quality of the measurements.

The evaluation of the effect of the WIM errors on the load-pavement impact estimation was done by Prozzi and Hong in 2007 (18). In their study, the traffic data from the WIM systems in Texas were used to demonstrate the varying load-pavement impact evaluation biases. It was found that for a WIM scale, 10 percent over-calibration resulted in as much as 51 percent overestimation of the load-pavement impact, while 10 percent under-calibration resulted in an underestimation of 31 percent. Thus, the load-pavement impact estimation error is more sensitive to the over-calibration than under-calibration of a WIM scale.

In the following section, studies covering the effects of traffic inputs and their inaccuracies on the pavement design using the MEPDG are discussed.

2.4 Effects of WIM Errors on Pavement Structural Performance Using MEPDG

various traffic inputs in the MEPDG are discussed below.

2.4.1 MEPDG Traffic Inputs

The definitions of the traffic inputs required for the design using the MEPDG are presented herein:

- **Axle-Load Spectra**: The axle-load spectrum is a histogram or distribution of axle loads for a specific axle type (single, tandem, tridem, and quad). In other words, the number of axle applications within a specific axle-load range (8).
- **Hourly Distribution Factors**: The percentage of trucks using a facility for each hour of the day. The sum of the hourly distribution factors must total 100 percent (8).
- **Monthly Distribution Factors**: This value defines the distribution of truck volumes on a monthly basis in a typical year. The sum of all monthly distribution factors for a specific truck class must equal 12 (8).
- **Normalized Truck Classification Distribution**: The normalized truck volume distribution is a normalized distribution of the different truck classes within the traffic stream. To determine the normalized truck class volume distribution, the number of trucks counted within a specific classification is divided by the total number of trucks counted. The cumulative sum of all incremental values for all of the truck classifications equals 100 percent (8).
- **Truck Classification Distribution**: The distribution of the number of truck applications for each truck classification for all trucks counted. Trucks are defined as vehicle classes 4 through 13 using the FHWA classifications (8).
- **Truck Traffic Classification (TTC) Group**: An index type number that defines a group of roadways with similar normalized axle-load spectra and

normalized truck volume distribution. Stated differently, the truck traffic classification (TTC) group is a value used to define the axle-load spectra and truck volume distribution from count data. In summary, it provides default values for the normalized axle-load spectra and normalized truck classification volume distributions (8).

2.4.2 Pavement Performance Indicators

In terms of the predicted pavement performances, the following definitions could be found in the Guide (8):

- **Alligator Cracking** − A form of fatigue or wheel load related cracking and is defined as a series of interconnected cracks (characteristically with a "chicken wire/alligator" pattern) that initiate at the bottom of the HMA layers. Alligator cracks initially show up as multiple short, longitudinal or transverse cracks in the wheel path that become interconnected laterally with continued truck loadings. Alligator cracking is calculated as a percent of total lane area in the MEPDG (8).
- **Longitudinal Cracking** − A form of fatigue or wheel load related cracking that occurs within the wheel path and is defined as cracks predominantly parallel to the pavement centerline. Longitudinal cracks initiate at the surface of the HMA pavement and initially show up as short longitudinal cracks that become connected longitudinally with continued truck loadings. Raveling or crack deterioration may occur along the edges of these cracks but they do not form an alligator cracking pattern. The unit

of longitudinal cracking calculated by the MEPDG is total feet per mile (meters per kilometer), including both wheel paths (8).

- **Transverse Cracking** − Non-wheel load related cracking that is predominately perpendicular to the pavement centerline and caused by low temperatures or thermal cycling. The unit of transverse cracking calculated by the MEPDG is feet per mile (meters per kilometer) (8).
- **Rutting or Rut Depth** − A longitudinal surface depression in the wheel path resulting from plastic or permanent deformation in each pavement layer. The rut depth is representative of the maximum vertical difference in elevation between the transverse profile of the HMA surface and a wire-line across the lane width. The unit of rutting calculated by the MEPDG is inches (millimeters), and represents the maximum mean rut depth between both wheel paths. The MEPDG also computes the rut depths within the HMA, unbound aggregate layers, and foundation (8).

MEPDG requires three levels for its traffic inputs: Level 1 to 3. “Level 1 is site-specific and directly related to each project while Level 3 is MEPDG default data for nationwide (United States of America) use” (8). “Level 2 is for state to use its own regionalized inputs when site-specific estimates are not available” (19). In this study Level 1 data provided by AT is compared to Level 3 default values of the Guide.

Different traffic inputs which are implemented into the design using the MEPDG include: Distribution of different classes of trucks, distribution of traffic over a day and a year, and also the axle load distribution of different axle types (single, tandem, tridem, and quad).

Li et al. in 2009 (19) evaluated the effect of different axle load distributions on the predicted performance of the pavement. Level 1 traffic data obtained from 12 WIM site locations in Washington State are considered for a sensitivity analysis on the pavement predicted performances such as rutting, longitudinal cracking, and alligator cracking. Different axle load distributions obtained from WIM systems were categorized in three scenarios of light, moderate, and heavy impacts. These levels were selected based on the location of frequency peaks. On the other hand, the MEPDG default values were considered as the fourth scenario. Predicted performances were plotted for a 12 year design life of a flexible pavement. As a conclusion, MEPDG was found to be moderately sensitive to the alternative axle load spectra.

Haider et al. in 2010 (11) evaluated the effects of axle load distribution errors on pavement predicted performance. The difference between static axle load distribution and WIM-based axle load distribution was considered for the analysis. The WIM-based axle load distribution was simulated through implementing biases in the source distribution (true axle load distribution). It was found that negative measurement bias in axle load have significant effects on the predicted performance (cracking). Tran et al. in 2007 performed a similar study and conluded that “the sensitivity of fatigue cracking to overestimated WIM data was the most pronounced” (20)

Zhang et al. in 2007 (2) simulated the single axle load distribution biases by forming 4 different scenarios:

- Overload = 0%, where all axle load factors are set to up to 20,000 lbs.
- Overload = 10%, where 90% of axle load factors are set up to 20,000 lbs.
- Overload = 20%, where 80% of axle load factors are set up to 20,000 lbs.
- Overload = 30%, where 70% of axle load factors are set up to 20,000 lbs.

Based on above scenarios, MEPDG Software was run to investigate the effects of these simulated biases on the pavement predicted performance. For a 20 year design life of a typical flexible pavement, IRI and rutting contrast graphs were plotted for these scenarios showing that 78% increase in IRI value exist for 30% overload. This increase is for 15 years of pavement life while this increase will be more sensed after 20 years of design life. Regarding the rutting, 52% increase in rutting was observed for the worst case scenario (overload = 30%) after 20 years of design life.

2.5 Summary

WIM technology is being used for different practices around the world. In this literature review chapter, the technology is being introduced by naming some main types of WIM sensors and their schematic views. Specifically, piezoelectric sensors which is being used by AT for its WIM systems is presented in this chapter. As the next part, the causes and sources of WIM errors are explained in detail. Moving forward, the WIM errors are divided into different types where each part has its own statistical behavior. Random errors versus systematic errors are named as two major groups of errors generated by any WIM system, in general. In the last part of this chapter the MEPDG traffic inputs and predicted performance indicators are explained in detail by giving technical term definitions used in this study and mainly included in Chapter 5. The effects of differences in traffic inputs for the MEPDG Software on the pavement predicted performances were investigated and it was found that simulated biases in axle load distribution could result in significant shifts in pavement distress levels.

CHAPTER 3

3.0 Accuracy of WIM Systems In Alberta

3.1 Introduction

The MEPDG uses dynamic traffic data for its flexible and rigid pavement design. These traffic data are collected in more detail than previous design practice to provide better characteristics of the traffic load subjected to the pavement. The WIM technology is used for this purpose. The accuracy of these systems, on the other hand, in measuring weight needs to be established. The deviation of the weight measurements from the actual weights can have considerable effects on the final pavement designs (2, 11, and 20). In order to evaluate the accuracy of the WIM sensors in Alberta, AT runs a monthly WIM verification test program on each WIM system.

Every month, a FHWA Class 9 truck with a known weight passes 10 times on the WIM sensors. For each WIM sensor and for each series of 10 passes, the truck load is kept the same. A large database was created over five years of testing (2006 – 2010). In this chapter, the weight measurements from the WIM systems in Alberta for the test truck are studied statistically.

3.2 Pavement Condition at the WIM Site Locations

WIM sensors should be placed on a smooth and even surface. The accuracy of the WIM sensors in terms of the generated values for both speed and weights is sensitive to the evenness of the asphalt layer. AT keeps a record of its highway network performance, in terms of rutting and IRI through automatic annual measurements.

Table 3.1 provides the information on pavement rutting and IRI at the beginning and the end of the study period (2006 and 2010) (4). For the divided highways, both lanes in one direction are included in the table while for the undivided highways only one lane in each direction is included in this table. The IRI trigger value (4) for rehabilitation is also presented in the table to compare the IRI values in 2006 and 2010. For all of the highways, recorded IRI is lower than the AT's trigger value which shows that during the study period the condition of the highways were within the acceptable ranges. Regarding the rutting, most of the highways have lower values in 2010 in comparison to 2006. Exceptionally for Highway 2:24, which has greater rutting values for 2010, other locations have lower or approximately similar values between these two years.

Regarding the differences between the two lanes in divided highways, Table 3.1 shows that in almost all of the highways, there is not a significant difference between the two lanes in the same location. To be noted that letters 'R' and 'L' stand for Right and Left lanes while standing toward North and East for North-South and East-West highways, respectively. Moreover, digits 1 and 2, for divided highways, indicate the outer and inner lanes, respectively.

Two rehabilitation procedures were performed during the study period. In 2010, Highway 2:30 (lane L2) was overlaid based on the AT's Pavement Management System (PMS) report (4). Also, Lane R2 on Highway 16:06 was overlaid in 2006.

The consequences of these rehabilitation procedures can be seen in Table 3.1, where both the IRI and maximum rutting depth decrease from 2006 to 2010.

Table 3.1 – Pavement condition at the WIM stations in 2006 and 2010

Site Location	Lane	IRI (m/km)		Max Rutting Depth (mm)		IRI Trigger for Rehabilitation (m/km)*
		2006	2010	2006	2010	
2:24	R2	1.23	1.46	8.48	19.35	1.9
	L2	1.35	1.70	10.25	21.70	
2:30	R2	1.53	1.68	10.15	11.95	1.9
	L2**	1.52	0.85	7.60	2.70	
2A:26	L1	0.83	1.06	4.15	6.50	2.1
3:08	R2	0.92	0.99	7.80	9.60	2.1
	L2	0.86	0.91	9.00	10.15	
16:06	R2***	2.38	0.77	18.63	4.43	1.9
	L1	1.78	1.80	16.02	9.6	
44:00	R1	1.12	1.33	7.30	8.73	2.1

* Based on Alberta Transportation Pavement Preservation Guide
** Based on Alberta Transportation PMS, road was overlaid in in 2010
*** Based on Alberta Transportation PMS, road was overlaid in 2006, IRI and rutting depth are related to pre-overly condition.

3.3 WIM Verification Test Program

As mentioned in Chapter 1, WIM verification test program is the basis for evaluation of WIM weight measurements in Alberta. This program consists of using a FHWA Class 9 verification truck with known properties such as speed, axle loads, gross vehicle weights, the distance between axles, and apparently the classification which is Class 9. For purpose of WIM weight measurement evaluation, the accuracy is defined by the deviation of the weights recorded by WIM systems from their stationary weights.

To evaluate the accuracy of the WIM weight measurements, the following equation is used (3 and 21):

$$E = 100 * \frac{W_{WIM} - W_{Static}}{W_{Static}} \qquad \text{Eq. 3.1}$$

Where:

E = WIM weight error (%)

W_{WIM} = Weight recorded by the WIM system (kg)

W_{Static} = Weight recorded by the static scale (kg)

The errors for the following weight parameters are calculated for each WIM sensor axles:

- Single Steering axle load;
- Tandem Drive axle load;
- Tandem Load axle load; and
- Gross Vehicle Weight.

3.3.1 WIM Weight Database

The WIM weight errors were estimated using Equation 3.1. Using the verification test program outputs, statistical analysis will be performed on the errors' database. AT provided the university with both the static scale and the WIM measurements recorded in a tabular format. Every single pass of the 10 passes has its own single steering axle load, tandem drive axle load, tandem load axle load, and gross vehicle weight recordings both for the static scale and the WIM measurements in one table. An example of the table can be found in Appendix A. A large number of tables are available for five years (60 months), 20 lanes, and 10 passes in each month. A total of 60 months × 20 lanes× 10 passes = 12,000 tables need to be used to estimate the errors.

The errors associated with each pass, and for each weight parameter, is calculated using Equation 3.1. There are four weight parameter for three different axle types plus one GVW. All the errors are stored in one database. The following properties are assigned to each error:

- **Year:** shows the year in which the error was generated;
- **Month:** provides information on which month the error occurred;
- **Weight Parameter:** shows which weight parameter the error is associated with; and
- **Location:** shows that the error is taken place in which location (out of six locations).

As a result, the errors can be categorized in different sub-categories under the above properties and be investigated thoroughly. For instance, it is possible to see which year has a higher range of errors or which weight parameter deviated more or less from the static measurements. The total number of expected errors equals:

$$N_{E-exp} = 60(months) * 20(lanes) * 10(passes) * 4(Weight\ Parameter) = 48{,}000$$

It is explained in the following section how some decreases were implemented to the expected number of errors.

Due to the AT's budget restraints, some verification files were missing during the study period. Table 3.2 summarizes the missing verification files. In 2008, the sensors were not able to record the traffic data (verification truck) in Highway

16:06 for 2 lanes in May and one lane in June. Furthermore, in 2010 and during the months of August, October, and December, the WIM system was not able to record the data (turned off sensor) for all the lanes of Highway 2:30 resulting in a total of 12 missing files from the database.

Due to budget restraints, instead of testing the WIM on a monthly basis, AT decided to do the verification program every other month, starting from May 2009 in all the WIM locations. Table 3.2 presents these missed data. Grey cells in this table represent the missing files due to temporarily turned off sensors rather than AT's decision to do the verification program with a lower overall cost (every-other-month basis). As seen in Table 3.2, a total of 200 files are missing due to this reason. 20 missing files for each month are for 20 different WIM lanes in all of the locations.

As a consequence, each missing file is equivalent to missing 40 number of weight data (errors). Therefore, a total of 40*215 = 8,600 numbers of errors are removed from the database. In the next section, another series of missing data is presented which are due to sensors' malfunction.

Table 3.2 – missing WIM verification files from 2006 to 2010

Year	Month	Highway	Number of Verification Missing Files
2008	May	16:06	2
	June	16:06	1
2009	May	All	20
	July	All	20
	September	All	20
	November	All	20
2010	January	All	20
	March	All	20
	May	All	20
	July	All	20
	August	2:30	4
	September	All	20
	October	2:30	4
	November	All	20
	December	2:30	4
Total			**215**

Due to sensors' malfunction during the study period, a series of data are not recorded in different months, years, and locations categorized in four different weight parameters mentioned in the preceding section. The distribution of these missing data is summarized in Table 3.3. This distribution is categorized based on the time of missing data (year and month) as well as the site location. At the end, the distribution of missing data on the weight parameter is also presented. These four categories are giving a similar number of missing data while each of them can be put into a more detailed way as Table 3.3 shows.

Regarding the months, December and October have the most number of data missing at the values of 56 and 53. No missing data is in place for the month of

August during the five year program in any location. However, other months have a range of missing data.

Different years have different number of missing data. A maximum number of 165 missing data exists in 2008. A minimum of 9 missing data in 2007 exists while 2006 has only 12 missing data. 2009 and 2010 with the values of 48 and 40 stand as the third and fourth years with highest number of missing data, respectively. The variation, with regard to years, is determined to be very high.

Between six different locations, Highway 2:24 has the highest number of missing data at 108 while Highway 2A:26 has only one. Other locations are ranked between these two locations in terms of the number of missing data.

The last category is WIM weight parameter. All the parameters are uniformly distributed in this group at around 69 number of missing data which is expected. The reason to this expectation is that as the sensor is unable to record one weight parameter, it is obvious that other weight parameters are not recorded as well while the sensor was not performing completely, at the time of recording.

In summary, the overall number of missing data (error) is 274 which is only 0.7% of the whole database.

Table 3.3 – Distribution of WIM malfunction sensor data

Category		Number of missing data
Verification Month	Jan	4
	Feb	28
	Mar	12
	Apr	4
	May	12
	Jun	40
	Jul	9
	Aug	0
	Sep	12
	Oct	53
	Nov	44
	Dec	56
Verification Year	2006	12
	2007	9
	2008	165
	2009	48
	2010	40
Site Location	2:24	108
	2:30	20
	2A:26	1
	3:08	69
	16:06	36
	44:00	40
WIM Weight Parameter	Single Steering Axle	66
	Tandem Drive Axle	69
	Tandem Load Axle	70
	Gross Vehicle Weight	69
Total		**274**

A total number of 8,600+274 = 8,874 errors are missed due to above reasons. As a result to these deductions from the database, *Total number of actual errors* ($N_{E\text{-}atc}$) can be derived by subtracting 8,874 from $N_{E\text{-}exp}$:

$$N_{E-act} = N_{E-exp} - 8{,}874$$

$$N_{E-act} = 39{,}126 \qquad \text{Eq. 3.2}$$

3.3.2 Stationary Weight Distribution

During the WIM verification test program, a large number of stationary weights were collected by static scales. These stationary weights were constant for each group of 10 passes as the truck is statically weighed once for each 10 passes of verification truck. It should be noted that these weights are either one of the axle types (single steering, tandem drive, or tandem load) or GVW. The distributions of the stationary weights along the whole program are illustrated in the followings using a box-plot graph.

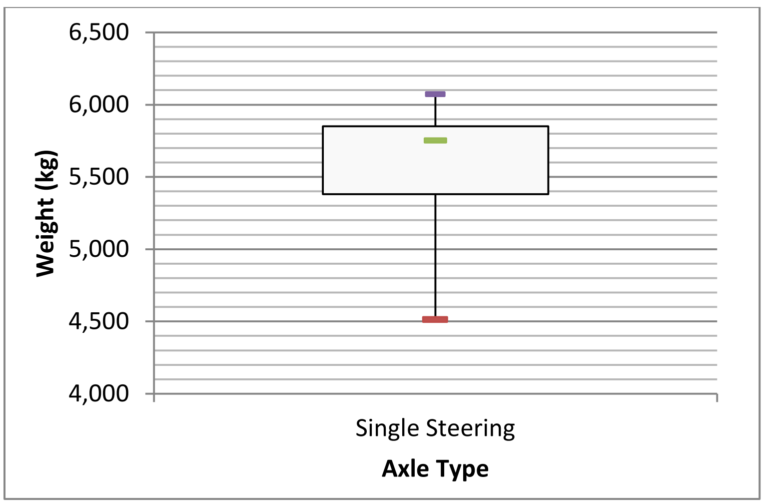

Figure 3.1 –Box-plot for stationary single steering axle

Figures 3.1 and 3.2 show how the stationary weights are distributed. As it can be seen in Figure 3.1, stationary single steering axle load ranges from a minimum of 4,500 kg to a maximum value of 6,100 kg. The first and third quartiles where the 25% and 75% data stand are at 5,400 kg and 5,850 kg, respectively with a median value at 5,750 kg. The stationary axle loads for single steering is not widely distributed while more concentrated around the median.

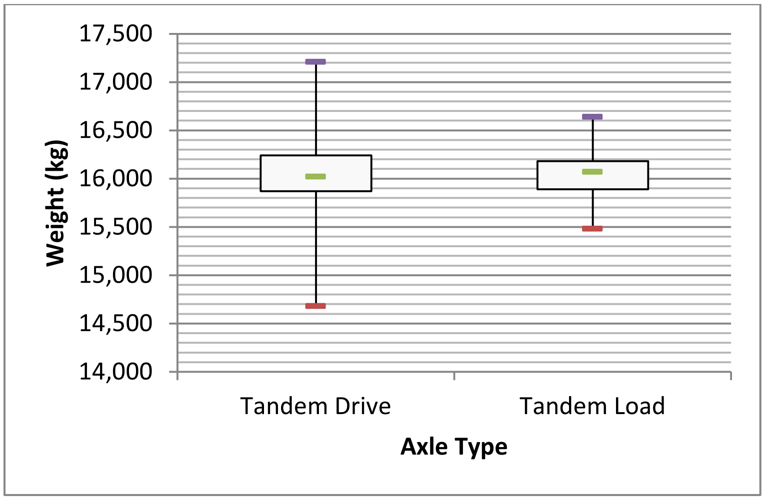

Figure 3.2 – Box-plot for stationary tandem axles

Stationary weights of tandem drive and tandem load axles are plotted in Figure 3.2. Both of them have similar median values at around 16,000 kg while tandem drive axle is widely distributed by having a minimum of 14,700 kg and a maximum of 17,200 kg. Tandem load axle has a narrower range with a minimum of 15,500 kg and a maximum of 16,650 kg.

3.4 ASTM E1318 Requirements

ASTM E1318 – Standard Specification for Highway Weigh-In-motion (WIM) Systems with User Requirements and Test Methods (3) – is the basis for the statistical analysis discussed in this chapter. It includes specifications for different types of WIM systems and the required properties that each type should have. It also provides requirements for testing the accuracy of WIM systems. In the following, a brief introduction to different types of WIM systems is provided and then the required limitations for accepting the WIM system, in terms of the accuracy of generated data, are explained.

Four types of WIM systems are defined in the ASTM E1318 (3) based on the vehicle speed, various features that each type should provide and also the application of WIM system such as weight-enforcement or pavement design purposes. For example, Type I is used for speeds between 16 and 130 km/h while type II operates at speed of 24 to 130 km/h. Type III is used for the lanes off the main highway at weight-enforcement stations. Type III also is not needed to produce ESAL calculations, wheelbase measurements, and classification. To provide one more example, Type IV WIM system is operating at a very low speed and for load-limit and weight-limit violations. More detailed information can be found in the code. As for this study only Type I is looked up in more details because all of the AT's WIM systems are Type I.

The speeds of the verification trucks in this study are either 100 km/h or 110 km/h for divided and undivided highways, respectively, with some tolerances around these two numbers. As a result, Type I WIM sensor can be applicable for the verification test program.

Table 3.4 provides information on various items that is required to be produced by WIM systems. Regarding Type I WIM system, all of these items are required to be provided by the system. Among different items shown in this table, axle load, axle-group load, and gross-vehicle weight are the weight parameters included in this study as well as speed. Other items such as site identification code, lane and direction of traffic, date and time of passage, sequential vehicle record number, and violation code are items which only help to organize the data to be able of being identified from other recordings. Other remaining items such as centre-to-centre spacing between axles, and wheelbase are used for the purpose of classification.

Table 3.4 – Data Items Produced by WIM System (3).

1	Wheel Load
2	Axle Load
3	Axle-Group Load
4	Gross-Vehicle Weight
5	Speed
6	Centre-to-Centre Spacing Between Axles
7	Vehicle Class (via axle arrangement)
8	Site Identification Code
9	Lane and Direction of Travel
10	Date and Time of Passage
11	Sequential Vehicle Record Number
12	Wheelbase (front-most to rear-most axle)
13	Equivalent Single-Axle Loads (ESALs)
14	Violation Code

The acceptable limits are provided in the ASTM E1318 for different WIM types. For each type of WIM systems (I to IV), the acceptable ranges for different parameters are provided in Table 3.5. For Type I, which is the focus of this study, tolerances of ±20 %, ±15 %, and ±10 % at 95% confidence are used for axle load, axle-group load, and gross vehicle weight, respectively.

Table 3.5 – Acceptable Tolerances for the WIM systems according to ASTM E1318 (3)

Function	Tolerance for 95% Compliance*				
	Type I	Type II	Type III	Type IV	
				Value ≥lb (kg)**	±lb (kg)
Wheel Load	±25 %		±20 %	5000 (2300)	300 (100)
Axle Load	±20 %	±30 %	±15 %	12 000 (5400)	500 (200)
Axle-Group Load	±15 %	±20 %	±10 %	25 000 (11 300)	1200 (500)
Gross-Vehicle Weight	±10 %	±15 %	±6 %	60 000 (27 200)	2500 (1100)
Speed	±1 mph (2 km/h)				
Axle-Spacing and Wheelbase	±0.5 ft (0.15m)				

* 95% of the respective data items produced by the WIM system must be within the tolerance.
** Lower values are not usually a concern in enforcement.

Table 3.5 also provides information on the acceptable tolerances for speed. The allowable error in measuring speed is ±1 mph (2 km/h) at 95% compliance. This range is required for all types of WIM systems. The acceptable tolerances for speeds is checked for WIM recorded data in 2010 and shown in Figure 3.3.

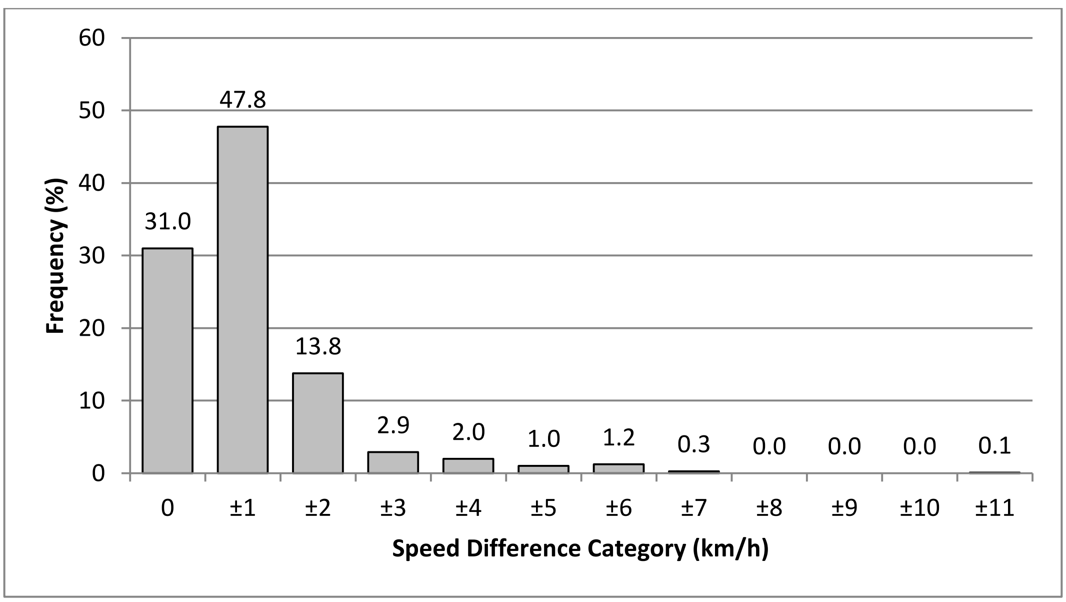

Figure 3.3 – distribution of verification truck speeds in 2010

The distribution of speed differences for the entire passes in 2010 is presented in Figure 3.3. As the main focus of this study is on the weight parameters, other years are not checked for this analysis. According to Figure 3.3, of all the passes of the verification truck, 31% are within the category of zero difference between the truck's speedometer and the WIM sensor record. Also, approximately 48% and 14% of the total passages were within the category of ±1 and ±2 km/h difference, respectively. A total of 7.5% of the passes was in the range of ±3 to ±11 km/h difference. According to the ASTM E1318 for sensor types I to IV, 5% of the entire recorded speeds can deviate from the actual speed by more than ±2 km/h. According to Figure 3.3, 92.5% of the passes show a speed difference of less than 2 km/h. As a result, speed recordings of the WIM systems in Alberta do not satisfy ASTM E1318's criterion (95% compliancy).

In order to investigate the accuracy of WIM systems in Alberta in terms of truck classification, a study was conducted on Highway 2:30. The WIM systems classify vehicles based on the number and weight of the axles and their spacing.

A camera was installed at the northbound roadside of Hwy 2:30 at the WIM site location in a way to record image of vehicles in both directions of this divided highway. Approximately 100 minutes of traffic passes was recorded from 10:20 to 12:08 a.m. on December 7, 2011. Video images were reviewed and matched with the records from the WIM sensors. A total of 2,189 vehicles were observed from the video images in both directions and matched with the WIM records.

Classification of only nine vehicles from the WIM records was different from the classification from the images. In all of these cases, the numbers of axles, for these vehicles, from WIM records were less than the number of axles observed in images by one axle. Also, eight vehicles were observed in the images, while there were not any records for them in the WIM records. Finally, there were eight vehicles which were recorded with the WIM, but they were not observed in video images. Considering limitations in visual review of the video images, it was concluded that the accuracy of the WIM sensors for truck classification at this site is acceptable. This short period of verification needs to be extended and also repeated for the other sites.

3.5 Statistical Analysis of the WIM Measurements' Errors

The database for the WIM weight measurement errors was generated as it was described in Section 3.3.1. This database is analyzed statistically in this section.

3.5.1 Descriptive Statistics for the WIM Weight Measurements' Errors

For four categories of WIM weight parameter, verification year, and site location, basic statistics such as minimum, maximum, mean and standard deviation for each category is provided in Table 3.6. Additionally, first, second, and third quartiles (Q1, Q2, and Q3) as well as the interquartile value (IQR) which is the distance between Q1 and Q3 (IQR=Q3-Q1) are provided in Table 3.6. The last column in Table 3.6 provides the number of the errors for each category. It should be noted that due to the missing records discussed previously, the number of data points within each sub-category is not equal. Furthermore, since Highway 2A:26 and 44:00 have one lane per direction, these two locations show lower number of errors in site location category in comparison to other highways. The number of errors for Highway 2A:26 and 44:00 are at 3,999 and 3,960, respectively while this number is in the order of approximately 8,000 for the other locations.

Table 3.6 – Basic statistics for the WIM weight measurements' errors

Category		Errors (%)								Count
		Min.	Q1	Q2	Q3	Max.	Mean	Standard Deviation	IQR (Q3-Q1)	
WIM Weight Parameter	Single Steering Axle	-64.8	-12.2	-2.1	10.3	114.7	0.1	17.8	22.6	9,784
	Tandem Drive Axle	-62.2	-9.9	-2.2	6.6	160.7	-0.6	15.5	16.5	9,781
	Tandem Load Axle	-75.4	-7.0	0.4	7.9	172.9	1.8	15.8	15.0	9,780
	Gross Vehicle Weight	-67.0	-7.2	-0.8	6.5	106.4	0.6	13.4	13.8	9,781
Test Year	2006	-64.2	-8.2	0.1	8.9	172.9	1.8	17.0	17.1	9,588
	2007	-48.4	-8.8	-1.3	7.1	74.5	-0.2	13.2	15.9	9591
	2008	-64.5	-8.7	-1.1	7.1	123.0	0.3	15.0	15.8	9,315
	2009	-74.3	-9.7	-1.5	7.8	139.9	0.0	16.7	17.5	6352
	2010	-75.4	-10.0	-1.8	7.1	160.7	0.3	17.5	17.2	4,280
Site Location	2:24	-64.5	-7.0	0.8	7.9	114.7	1.0	13.1	14.9	7,892
	2:30	-59.1	-8.6	-2.4	4.5	56.8	-1.6	10.8	13.0	7,500
	2A:26	-35.5	-4.6	2.9	12.2	73.6	4.9	15.0	16.7	3,999
	3:08	-47.0	-8.6	-1.4	8.2	97.5	1.0	15.2	16.9	7,931
	16:06	-74.3	-10.9	-2.5	6.8	172.9	-0.3	18.9	17.6	7,844
	44:00	-75.4	-14.8	-3.0	10.1	160.7	-0.8	21.3	24.8	3,960
Total		-75.4	-8.9	-1.0	7.7	172.9	0.5	15.7	16.6	39,126

The following results can be derived from Table 3.6:

- The comparison between the maximum and minimum values shows that a total maximum error of 173% occurs for tandem axle measurements in 2006 and for Highway 16:06. Also, the minimum value of -75.4% is seen for tandem axle for Highway 44:00 and in 2010.
- The mean values for different sub-categories range between -1.6% and 1.8%.
- The comparison of the median values (Q2) and the mean values shows that these two parameters are almost similar. The majority of the median values for different sub-categories are less than zero while the mean values are mostly positive and close to zero.
- With regard to interquartile values which is the representative of the boundaries of middle 50% of errors, the smaller value show a more concentration of errors around the median which is very close to zero as discussed earlier. A maximum number of 22.6% for single steering axle in the category of WIM weight parameter shows how widely errors are distributed in comparison to other weight parameters. For verification year, interquartile values are close to each other with maximum and minimum values of 17.5% and 15.8% for the years 2009 and 2008, respectively. Site locations have a wider range for IQR with a maximum of 24.8% for Highway 44:00 and a minimum of 13.0% for Highway 2:30. The overall interquartile value is equal to 16.6%.

- Standard deviation of errors is another indication of how well errors are distributed. A maximum of 17.8% happens for the category of WIM weight parameter for again the single steering axle. Regarding the verification year, the minimum value for standard deviation exists in sub-category of 2007. The maximum value for this indicator happens in 2010. Again, a wider range for different site locations exist with a maximum of 21.3% for Highway 44:00 and a minimum of 10.8% for Highway 2:30, similar to its interquartile value being the minimum. Totally, the standard deviation for all of the errors is equal to 15.7%.

3.5.2. Distribution of the WIM Weight Measurements' Errors

The distribution of the WIM weight measurements errors are presented in this section.

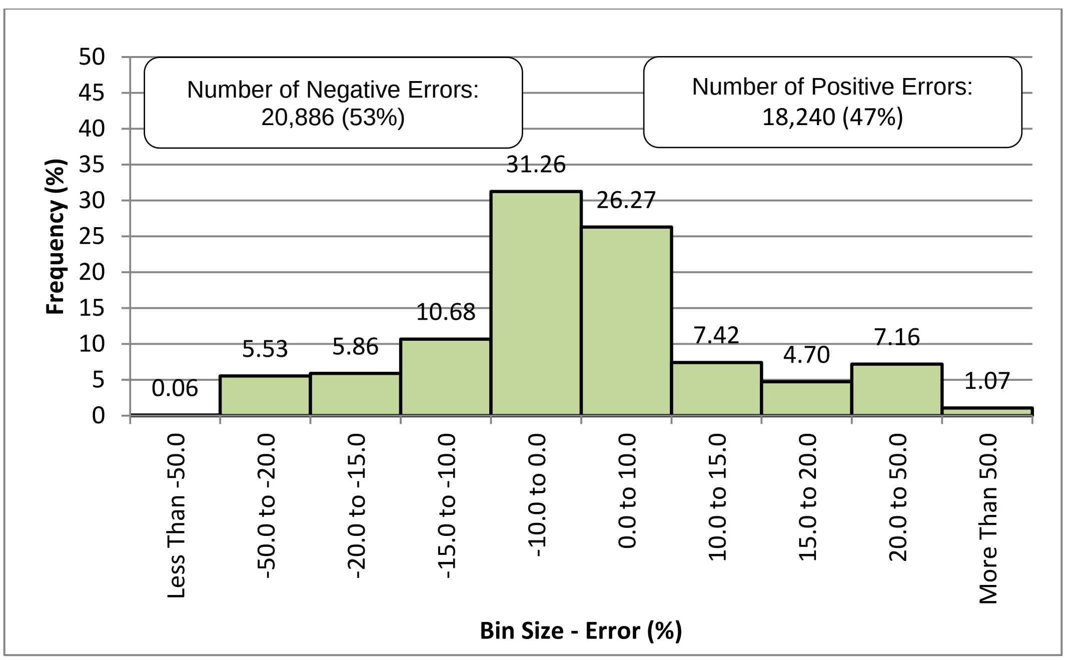

Figure 3.4 – Frequency distribution of the positive and negative WIM weight measurements' errors

Figure 3.4 shows the distribution of all the errors in different bin sizes. The bin sizes are selected in accordance with the ASTM E1318 requirements for 95% compliance discussed in Section 3.4. A total of 53% of the errors have a negative value while the remaining 47% of the errors are positive. Visually, the shape of the distribution is close to a normal distribution. The normality of the distribution of the errors is established in the following section.

For further analysis, the distribution of the absolute values of the WIM errors in the four sub-categories of weight parameters is provided in in Table 3.7.

To satisfy the ASTM's requirements, 95% of the errors should be within the acceptable ranges. It should be noted that each axle type has a different acceptable limit in the ASTM. The ranges are ±20% for single steering axle, ±15% for tandem axles (load and drive), and ±10% for gross vehicle weight. In the following, different sub-categories for weight parameters are evaluated:

- Single steering axle: 44% of the errors are in Bin 0.0% to 10.0%. Next bin with the size of 10.0% to 15.0% has 19.0% of errors Moreover, 13.7% of errors are in the third bin of 15.0% to 20.0%. All of the other errors are beyond the ASTM acceptable range of 20%. These errors are highlighted in grey. The last column shows the sum of which should not exceed 5% according to the ASTM. As seen in Table 3.7, a Total of 23% of the errors are either more than 20% or less than -20%.
- Tandem drive axle: Errors for this parameter are more concentrated in the first bin at 57.3% in comparison to single steering axle. Moreover, 19.6% of errors are in the second bin of 10.0% to 15.0%. The errors more than

15.0% or less than -15.0% are out of ASTM range and they should be limited to only 5% while it shows that they are more than this value at 23.1%.

- Tandem load axle: A similar criterion for tandem load exists, as ASTM sets. A total of 19.8% of errors are out of ±15.0% range while it should be again limited to only 5%. However, 63.1% of errors are within the first bin comparable to two previous axle types.
- Gross vehicle weight: A large portion of errors at 65.8% in this sub-category are in the first bin while a total of 34.2% of data are either more than 10% or less than -10%. 95% compliance is not satisfied in this sub-category, either.

Table 3.7 – Distribution of the absolute WIM weight measurements' errors

Category		Bin Values for all Absolute Errors (%)					% of Rejected Errors based on ASTM E1318
		0.0 to 10.0	10.0 to 15.0	15.0 to 20.0	20.0 to 50.0	>50	
WIM Weight Parameter	Single Steering Axle (±20% tolerance)	43.9	19.3	13.7	22	1.1	23.1
	Tandem Drive Axle (±15% tolerance)	57.3	19.6	10.8	11.2	1.1	23.1
	Tandem Load Axle (±15% tolerance)	63.1	17.1	9.3	9.2	1.3	19.8
	Gross Vehicle Weight (±10% tolerance)	65.8	16.4	8.5	8.3	1	34.2

3.5.3 WIM Outliers

Grubbs has defined outliers as values that appear to deviate markedly from other members of the sample in which they occur (22). Table 3.8 provides information on the outliers of the WIM weight measurements errors. The errors below Q1-1.5×IQR and beyond Q3+1.5×IQR are identified as possible outliers according to ASTM E178 (23). Probable outliers are identified using the upper and lower limit values of Q1-3×IQR and beyond Q3+3×IQR. The outliers' boundaries for different categories of WIM weight parameters are shown in Table 3.8. The boundaries range on average from ±35% to ±60 for possible and probable outliers, respectively, when rounded to the nearest 5th. An average of ±50% was considered as the possible outlier boundary for problematic errors. The errors larger than approximately +50% or less than -50%, are likely generated due to different factors than the errors within the two boundaries. This observation is further investigated through fitting specific distributions to different portions of errors.

Last column provides information regarding the number of outliers for each sub-category where A=3.0. For WIM weight parameter, the minimum outlier count is recorded for single steering axle at 5 and the maximum for tandem load axle at 118. For different years, the distribution of outlier count has a maximum of 116 for year 2006 and a minimum of 12 for the year 2007. Among different highways, 2:30 generated 16 outlier errors while Highway 16:00 has a maximum of 133 outlier errors. Totally, 342 errors were calculated to be outliers.

Table 3.8 – Outliers of errors

Category		Q1-A×IQR		Q3+A×IQR		Outliers Count
		A=1.5	A=3.0	A=1.5	A=3.0	A=3.0
WIM Weight Parameter	Single Steering Axle	-46.1	-79.9	44.2	78.0	5
	Tandem Drive Axle	-34.6	-59.3	31.3	55.9	84
	Tandem Load Axle	-29.5	-51.9	30.4	52.8	118
	Gross Vehicle Weight	-27.9	-48.5	27.2	47.8	101
Verification Year	2006	-33.8	-59.4	34.5	60.1	116
	2007	-32.7	-56.5	31.0	54.9	12
	2008	-32.4	-56.1	30.8	54.5	60
	2009	-36.0	-62.3	34.1	60.4	46
	2010	-35.8	-61.5	32.9	58.6	52
Site Location	2:24	-29.4	-51.7	30.2	52.6	49
	2:30	-28.1	-47.6	24.0	43.5	16
	2A:26	-29.6	-54.7	37.3	62.4	39
	3:08	-33.9	-59.2	33.5	58.8	57
	16:06	-37.3	-63.8	33.2	59.7	133
	44:00	-52.0	-89.2	47.3	84.6	22
Total		-33.8	-54.7	32.5	54.4	342

3.5.4 Statistical Distribution of WIM Weight Errors

In this section, the behavior of errors, are more evaluated by fitting them to some specific statistical distributions. SPSS Version 19.0 software package was used for this purpose. All the errors were evaluated and fitted to various statistical distributions. The normality check was firstly conducted on the entire errors to see whether or not they follow the normal distribution. A pretty good portion of the errors were following the normal distribution while after a specific point they were not. Other statistical distributions were checked for the remaining part of errors. In the following, these checks are presented.

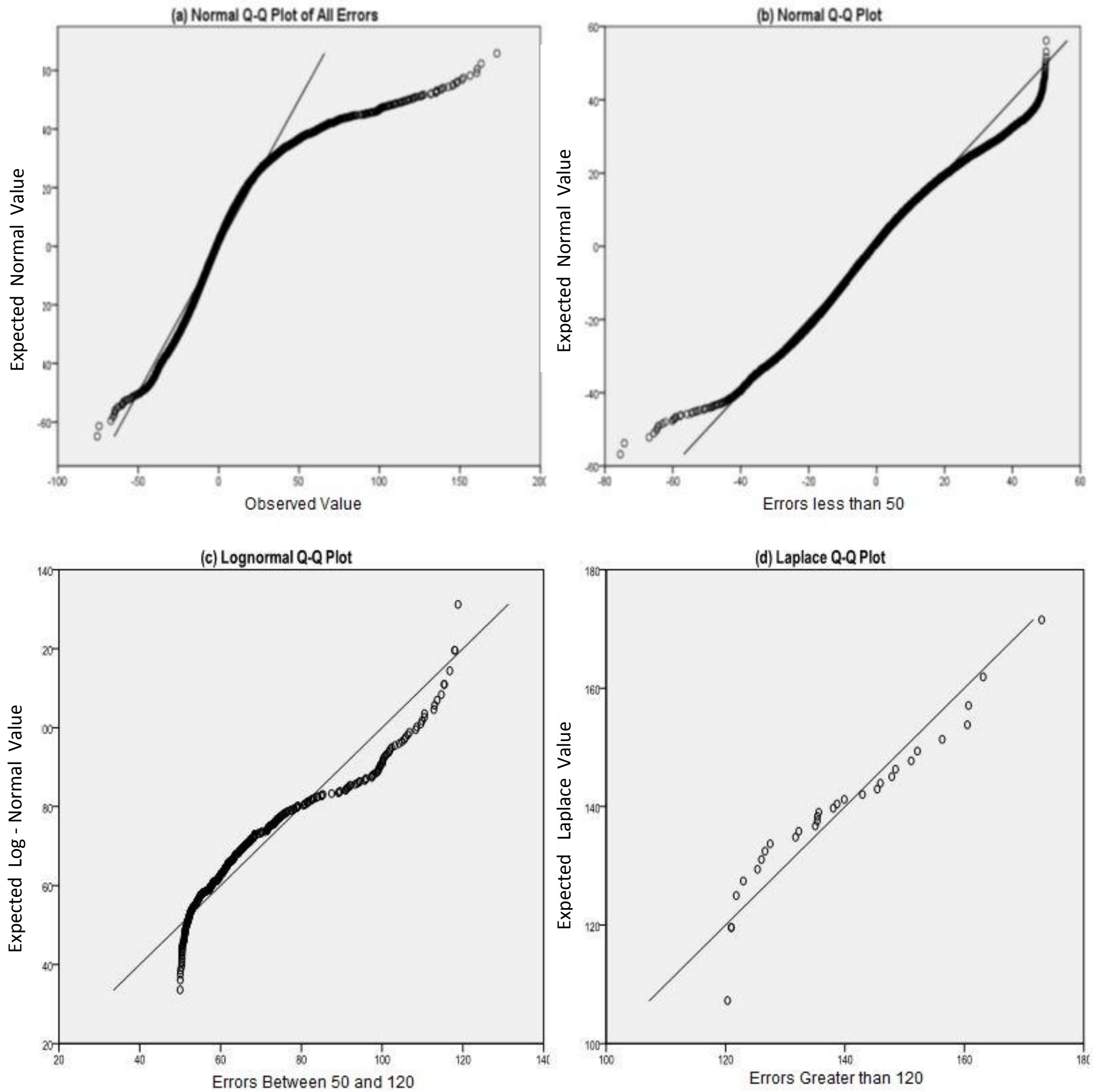

Figure 3.5 – Q-Q plots (a) Normal distribution for all errors (b) Normal distribution for errors less than 50% (c) Log-normal distribution of errors between 50% and 120% (d) Laplace distribution for errors greater than 120%.

Figure 3.5 illustrates how the errors are following specific distributions. Errors, based on outlier study, are divided into three groups of 1) errors less than 50% 2) errors between 50% and 120% 3) errors more than 120%. The normality check is being conducted on the errors to see whether or not they are following the normal distribution. For this purpose, Quantile Quantile (Q-Q) plots are drawn for all of the errors. Figure 3.5 (a) shows how the errors are following normal distribution. It is shown that up to the point of 40%, errors are following the normal distribution. Following the assumption of 50% boundary to be the point where problematic errors happen afterwards, Figure 3.5 (b) shows that for errors less than 50%, the normality fit is well satisfied with some deviations on the tails. Figure 3.5 (c) shows how the Q-Q plot is fitted to errors between 50% and 120% with a log-normal distribution. The level of fitness, visually, is considered to be not complete. However, it is the best description of distribution among other probability distributions such as Bernoulli, binomial, and Laplace. Figure 3.5 (d) describes the errors more than 120% to be fitted to a Laplace distribution. The reason that the boundary of 120% is selected is that for this group of errors, previous distribution – log-normal – was not fitted completely. The fitness degree is well for this few numbers of errors with some deviations around 120%.

3.5.5 Seasonal Variation of WIM Weight Errors

Up to this point, the errors were evaluated by looking at their behavior for different years during the study period. However, a monthly sensitivity analysis was not performed. In this section, the monthly variations of the errors are studied. Interesting trends are found and are presented in the followings.

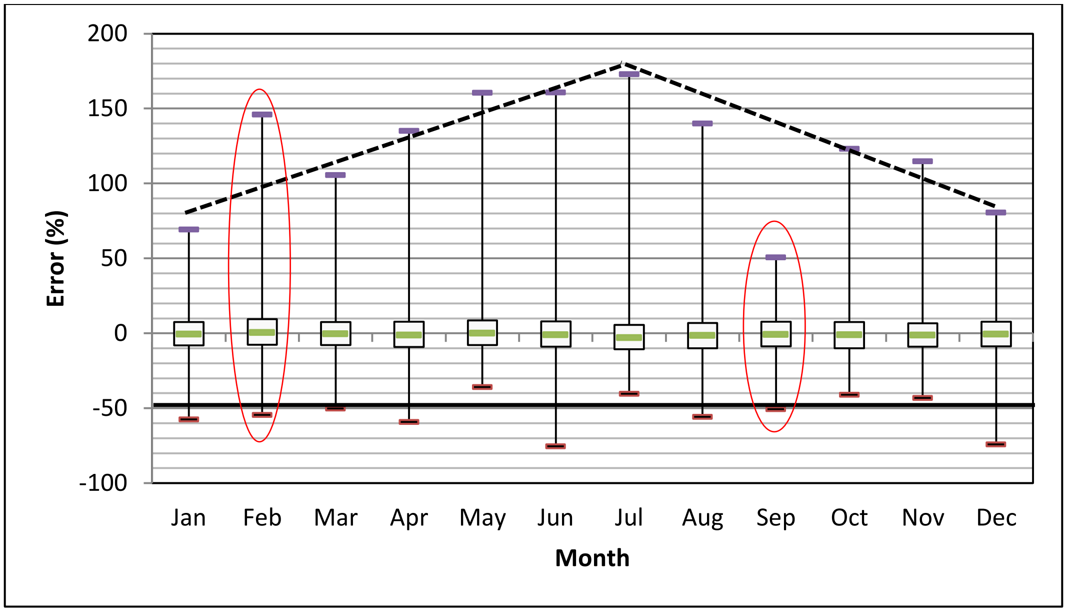

Figure 3.6 – Box-plots for WIM monthly errors

To go further into details about how the errors are distributed between different months for a period of 5 years, a box-plot is generated based on WIM errors. Figure 3.6 provides information on the maximum, minimum, and three quartiles of Q1, Q2 or median, and Q3. All the errors generated by the sensors are grouped based on the month of occurrence and above statistical parameters are calculated. The graph shows that for some months such as January and September, the deviation is not large in comparison to other months. Taking September and February, circled in the figure, as exceptions in the monthly analysis, an interesting trend can be considered for WIM monthly distribution. The minimum values of all of the months are approximately around the value of -50% while the maximum values are increasing starting from January to July and the decreasing from July to December. The solid lane represents the former observation and the dashed line shows the latter.

3.5.6 Probability of Conformity of WIM Weight Errors

The Probability of Conformity (PC) or the confidence level of errors within each WIM weight parameter is further studied in this section. ASTM E1318, as described earlier, requires a 95% of compliance with its limits for Type I WIM sensors. It means that based on the errors generated by any Type I WIM system, 95% of errors should be limited to a specific range. This specific range varies for different types of axles (weight parameters). These limits were discussed earlier in section 3.4.2 and also presented in Table 3.9. For three types of axles and also the gross vehicle weight, the PC value is calculated. The numbers in Table 3.9 show what portion of errors are within the ranges of each axle type, divided into two categories of years and site locations. As it can be seen in the table, there is not a presence of number 95% or more in any of sub-categories.

Regarding the years, the maximum PC happens in 2007 for tandem load axle with an acceptable limit of ±15% at 82.3% comparable to 95% compliance. The minimum amount for this category happens in 2009 for gross vehicle weight with ±10% at 55.3%. Most of the PCs are fluctuating between 70% and 80%. However, the PCs for gross vehicle weights are smaller and close to an average of 60%. This observation is expected as the limits for this weight parameter is narrower (±10%) in comparison to other weight parameters such as single axle load.

For site locations, the maximum PC happens in Highway 2:30 for single steering axle at 90.0%. The minimum PC in this sub-category happens in Highway 44:00 for gross vehicle weight at 43.5%. Generally, undivided highways of 2A:26 and

44:00 have smaller values of PC. Gross vehicle weight parameter have smaller amount of PC, similar to what happened for years.

Table 3.9 – Probability of conformity (PC) for WIM weight measurements

Category		Single Steering Axle (±20%)*	Tandem Drive Axle (±15%)*	Tandem Load Axle (±15%)*	GVW (±10%)*
Year	2006	80.2	73.4	74.5	59.1
	2007	78.1	76.1	82.3	64.8
	2008	74.4	78.6	82.0	65.7
	2009	74.8	72.0	70.7	55.3
	2010	75.9	76.8	71.1	56.8
Site Locations	2:24	87.7	78.1	78.4	65.2
	2:30	90.0	83.2	84.2	70.2
	2A:26	53.4	78.0	83.7	66.1
	3:08	69.4	79.4	81.8	68.2
	16:06	71.6	70.6	73.0	56.2
	44:00	67.2	60.6	60.1	43.5

CHAPTER 4

4.0 Effects of WIM Errors on Pavement Design

4.1 Introduction

In this chapter, the effects of the WIM weight measurements' errors from the WIM verification test program on pavement structural design are evaluated. The pavement design in this chapter refers to the AT's current design procedure which is based on the AASHTO 1993 design guide. Two approaches for flexible pavement thickness design are investigated in this chapter.

In the first analysis, WIM verification test program outputs and the calculated errors are used for the design process. From what was derived in Chapter 3, WIM weight errors had different distributions based on their magnitude. Four scenarios are formed based on these distributions. For the first one, all measurements are considered for the design process. As for the next one, a portion of outliers are removed from design inputs to investigate their absence. Next Scenario removes measurements generating errors more than 50% and the last one considers measurements generating errors within the ASTM E1318's tolerances. The design process considers stationary weights on one hand and WIM-based measurements on the other hand. The differences between these two design procedures in final asphalt thickness are in investigated in this chapter.

In the second analysis, the differences between AADTs from WIM systems in Alberta and ATR traffic counts are evaluated. These differences and their effects on final pavement designs are investigated as the next section. Two years of real-life traffic counts (2009 and 2010) are considered for this analysis.

4.2 AT's Pavement Design Practice – Basic Traffic Information

Basic traffic counts are established by AT using the Turning movement traffic data collection. This program is conducted on a manual basis at major intersections across the entire highway network (4). In doing so, all vehicles are classified into five categories including: Passenger Vehicles (PV), Buses (BU), Recreation Vehicles (RV), Single Unit (SU), and Tractor Trailer (TT). The AADT, percentage of SU, TT, and truck factors of 0.881 for SU and 2.073 for TT are used to calculate the ESAL/day/direction for all traffic control sections. The latest ESAL information for all highways in Alberta is available on the AT's website (24).

4.3 Relative Pavement Damage Concept

The existing AT WIM weight measurement data collected during the last five years (2006 to 2010), errors from the WIM verification testing program, and the AT's traffic and ESAL data from the Turning movement program were used to investigate the effect of the WIM errors on the pavement structural design.

The ESAL is calculated using the relative damage power law. The concept of ESAL has been used for pavement design and is presented in the following Equation (25 and 26):

$$ESAL = \sum_{i=1}^{n} \left(\frac{W_{x_i}}{W_{8.1}}\right)^{m_i} \qquad \text{Eq. 4.1}$$

Where n corresponds to the number of axles per truck, W_{X_i} stands for the measured weight of axle i, $W_{8.1}$ stands for the weight of standard axle (8.1 metric ton), and m_i is defined by highway agencies based on the configuration of the axle. AT defines m to be 3.30 for single axles and 4.79 for tandem axles. ESALs for each truck are then summed up to form total ESALs.

Relative Pavement Damage (RPD) ratio is used to evaluate the effect of the WIM weight errors on pavement structural design. Equation 4.2 is used to calculate the RPD.

$$RPD = \frac{\sum_{j=1}^{J}\sum_{i=1}^{n}\left(\frac{W_{X_{ij}}}{W_{8.1}}\right)^{m_i}}{\sum_{j=1}^{J}\sum_{i=1}^{n}\left(\frac{W_{Y_{ij}}}{W_{8.1}}\right)^{m_i}} \qquad \text{Eq. 4.2}$$

Where J is the number of truck passes in the verification testing program equals to 11,785. $W_{X_{ij}}$ and $W_{Y_{ij}}$ are WIM and static axle weight measurements, respectively, for the i^{th} axle of the j^{th} truck.

This equation can be expanded for the FHWA Class 9 truck as follows:

$$RPD = \frac{\sum_{j=1}^{J}\left(\frac{W_{SS_j\,(WIM)}}{W_{8.1}}\right)^{3.30}+\left(\frac{W_{TD_j\,(WIM)}}{W_{8.1}}\right)^{4.79}+\left(\frac{W_{TL_j\,(WIM)}}{W_{8.1}}\right)^{4.79}}{\left(\frac{W_{SS_j\,(static)}}{W_{8.1}}\right)^{3.30}+\left(\frac{W_{TD_j\,(static)}}{W_{8.1}}\right)^{4.79}+\left(\frac{W_{TL_j\,(static)}}{W_{8.1}}\right)^{4.79}} \qquad \text{Eq. 4.3}$$

In Equation 4.3, *SS*, *TD*, and *TL* stand for three axle types of single steering, tandem drive, and tandem load, respectively.

A sensitivity analysis is conducted to assess the effect of different WIM weight error levels on the pavement design using the AT Pavement Design Manual (5).

RPD is calculated for all of the verification truck passes and the results are presented in a tabular format.

4.3.1 Effect of Alberta WIM weight Errors on Pavement Structural Design

Different scenarios of the WIM weight measurements' errors were provided and discussed previously in Chapter 3. The errors were categorized into four different groups of 1) all errors, 2) errors less than 50%, 3) errors between 50% and 120%, and 4) errors greater than 120%. Different distributions were found for the last three groups. A sensitivity analysis is performed on the errors associated with following four groups of measurements:

- Scenario 1: All axle weight measurements.
- Scenario 2: All axle weight measurements excluding those generating errors > 120%.
- Scenario 3: All axle weight measurements excluding those generating errors > 50%.
- Scenario 4: All axle weight measurements excluding those generating errors greater than the ASTM E1318 requirements.

Table 4.1 provides ESAL calculations for the above scenarios. For each scenario, ESALs are calculated using Equation 4.1 for both WIM weight measurements and static measurements. The RPDs were estimated in the next step for all the above scenarios. The WIM measurements have been done in six different locations. For the purpose of this analysis, these measurements are used for one design section instead of six sections.

Table 4.1 – Significance of Alberta WIM errors on pavement designs

WIM Error Scenarios	ESAL based on WIM*	ESAL based on Static Scale*	RPD** %	Extra Asphalt Layer(mm) based on RPD and for DESIGN ESAL ***			
				$1*10^6$	$5*10^6$	$10*10^6$	$30*10^6$
Scenario 1	699,350	505,750	33.7 to 44.3	12-15	13-18-14-20-17-21	14-20	17-21
Scenario 2	647,300	505,000	25.3 to 31.5	9-11	13-18	12-15	13-16
Scenario 3	569,800	499,350	12.7 to 15.5	5-6	5-7	6-8	6-8
Scenario 4	400,450	396,150	1.1	Not Significant			

* Rounded to the nearest 50.
** Ranges of relative pavement damage were obtained by bootstrapping statistical method
*** Pavement designs were based on AT Pavement Design Manual and considering 300 mm granular base thickness in all cases.

Bootstrapping method was used to identify the uncertainty in relative damage (27). The upper and lower bounds were derived for the RPD. Bootstrapping is a computer-based statistical method which estimates the sampling distribution of a statistic using sampling method. "To use the simplest bootstrap technique, the original data set of size *N* is taken and using a computer, a new sample (called a bootstrap sample) that is also of size N is created. This new sample is taken from the original using sampling with replacement so it is not identical with the original "real" sample. The procedure is repeated many times (1000 times for example), and for each of these bootstrap samples its mean is computed" (28).

Regarding the WIM errors, this is done by replacing the observed errors with a known distribution to acquire a confidence level (95% in this case) for the statistic in question (weight measurements in this case).

Regarding these upper and lower bounds for the RPD, four different traffic levels is considered for each scenario. These traffic levels are then used for final pavement thickness design using the AASHTO 1993 method. Design ESALs for these traffic levels are considered based on typical practices of pavement design: 1) one million ESALs 2) five million ESALs 3) ten million ESALs, and 4) thirty million ESALs. These four traffic levels are considered in order to cover almost all the possible traffic passes over the 20-year pavement design life. For each of these traffic levels (design ESALs) the extra asphalt layer is calculated using the AASHTO 1993 method. In this study, the procedure for obtaining the thickness required based on this method is not presented as it is not the concern of this study.

The differences between the necessary asphalt layer for each ESAL type (WIM-based or static-based) and for every traffic level is estimated and presented in Table 4.1

Regarding the first scenario, when all the WIM data are accepted as the basis for the thickness design, ESALs from both WIM measured weights and stationary weights have the higher values in comparison to other scenarios. This is because more weight measurements are in place despite the fact that no measurement is removed. RPD is calculated to range from 33.7% to 44.3%. Consequently, the extra asphalt layer needed range from a minimum of 12 mm and a maximum of 21 mm for one million and thirty million design ESALs, respectively.

Second scenario considers that all the measurements are accepted and implemented into the design except for those which generate WIM weight errors more than 120%. ESALs from WIM measurements and stationary measurements decreased slightly with a RPD of 25.3% to 31.5%. The minimum and maximum extra asphalt layer needed to compensate the deviation of measurements are 9 mm and 16 mm, respectively.

RPD decreases substantially to approximately 13% for the third scenario where the measurements which produce errors less than 50% are considered for the design. Extra asphalt layer also decreases to a minimum of 5 mm and a maximum of 8 mm for the smallest and largest 20 years traffic.

The last scenario removes all the weight measurements which produce errors beyond ASTM E1318. Weight measurements, both for stationary and WIM in this scenario, are those which are really close to each other satisfying ASTM E1318's limits. To be reminded that for single steering axles, a limit of 20% is considered for accepting the errors while the tandem axles have a limit of 15%. ESALs calculated are almost the same in this scenario generating a RPD of only 1.1%. As a consequence, no significant extra asphalt layer and extra money are needed in this case.

4.4 Comparison between ATR and WIM Based Traffic Data

In order to compare existing traffic data from the AT's turning movement program and the traffic data derived from the WIM systems, a study is conducted to evaluate the differences between these two sources.

Table 4.2 provides information on AADTs derived from two sources of ATR and WIM systems in Alberta for the six WIM site locations. The AADT from the

ATR is based on the 2010 traffic data from the AT's PMS records. The AADT from the WIM systems are derived from the information provided to University in 2009. The ATR data is only available for 2010 while the WIM data was only available for the year 2009. The comparison is made between these two AADTs and is reflected in the last column of Table 4.2.

Of the six locations, Highways 2:24 and 2:30 have the highest AADTs between approximately 31,000 and 25,000, respectively. Four other highways have similar AADTs at approximately 7,000. The differences between the ATR records and the WIM systems' records show a minimum difference of 0.9% for Highway 2:30 and a maximum difference of -14.5% for Highway 16:06.

Table 4.2 – AADT from the AT's ATR and WIM systems

Highway	AADT from ATR (2010)	AADT from WIM (2009)	Difference (%)
2:24	30,900	32,275	4.4
2:30	24,848	25,084	0.9
2A:26	7,190	7,540	4.9
3:08	7,260	6,858	-5.5
16:06	8,130	6,950	-14.5
44:00	6,970	6,715	-3.7

The axle weight measurements for all vehicles recorded by the WIM system at each site in 2009 were converted to ESAL/year, based on the average ESAL factors provided by AT and is presented in Table 4.3. These load equivalency factors were obtained by the AT from all WIM measurements from 2005 to 2009 (29). Equations that were used to convert axle weights to ESAL for the verification truck are shown below (29):

$$ESAL_{Single\ Steering} = \left[\frac{Axle\ Weight\ (lbs)}{11{,}500}\right]^{3.30} \quad \text{Eq. 4.4}$$

$$ESAL_{Tandem\ Carry} = \frac{\left\{\left[\frac{Axle\ Weight\ (lbs)}{1{,}000}\right]+2\right\}^{4.79}}{26{,}829{,}693} \quad \text{Eq. 4.5}$$

Using Equations 4.4 and 4.5, average ESAL loads based on the AT's WIM systems for different configuration of axles are shown in Table 4.3. These ESAL calculations based on the WIM data are used to compare the existing method of ESAL calculations by AT which ATR system has recorded and those derived by implementing WIM data.

Table 4.3 – AT average equivalent single axle load from WIM

Axle	Configuration	Average Equivalent Single Axle Load					
		2005	2006	2007	2008	2009	Average
2	oo	0.01	0.01	0.01	0.01	0.01	0.01
2	o o	0.66	0.47	0.48	0.48	0.46	0.51
3	o oo	1.58	1.51	1.50	1.43	1.45	1.50
3	3 axle misc	0.17	0.09	0.08	0.09	0.09	0.10
4	o o oo	0.16	0.16	0.08	0.08	0.08	0.09
4	4 axle misc	1.07	1.07	1.06	1.08	1.05	1.07
5	o oo oo	1.94	1.59	1.49	1.55	1.63	1.64
5	5 axle misc	0.72	0.46	0.45	0.51	0.59	0.55
6	o oo ooo	2.42	2.13	2.16	2.13	2.16	2.20
6	6 axle misc	2.74	2.66	2.62	2.49	2.02	2.51
7	7 axle misc	2.72	2.61	2.69	2.76	2.53	2.66
8	o oo ooo oo	3.53	3.27	3.23	3.25	3.46	3.35
8+	8+ axle misc	2.71	2.27	2.21	2.44	2.43	2.41

Table 4.4 provides information regarding the ESAL calculations from the two mentioned sources of ATR records available in AT's PMS report and WIM records. The ESAL per year per direction from AT's PMS report are available for every control section for the entire highway network in Alberta. These data are

derived for six WIM site locations and reflected in Table 4.4. For WIM related ESAL per year per day, the equivalent single axle load factors for each vehicle configuration, from Table 4.3, is multiplied by the equivalent AADT of those vehicles. These products are then summed up for each site location and are presented in Table 4.4.

The design ESALs for 20 years traffic and for these two sources of data are calculated using the following equation (5):

$$Design\ ESAL = ESAL * TGF * 0.85 \qquad \text{Eq. 4.6}$$

Where ESAL stands for the first year ESAL per year per direction. 0.85 is the percentage of traffic in design lane for highways with two lanes per direction. Additionally, Traffic Growth Factor (TGF) is calculated based on the following equation (5):

$$TGF = [(1+g)^n - 1]/g \qquad \text{Eq. 4.7}$$

Where n is equal to the design year which is 20 in this case. g is equal to the growth factor for a new construction equal to 5% (5). After calculation, TGF is equal to 33.06%.

The design ESAL for six WIM site locations are calculated and presented in Table 4.4. The differences of design ESALs are calculated in the next step. In the remaining columns, the calculations for asphalt thickness based on a 300 mm Granular Base Course (GBC) are presented. These calculations are based on the current AT's design practice from AASHTO 1993 manual (5). Two subgrade moduli, 25 MPa and 50 MPa, are selected to cover the verity of subgrade soil types. Now, there are four asphalt layer thicknesses for each location. Two of them for the first subgrade modulus selection and for AT's data, based on ATR recordings and WIM data.

A similar calculation happens for the second subgrade modulus at 50 MPa. Maximum differences of asphalt layer between these four thicknesses are presented for each location.

The 20 year design ESALs are presented for six site locations. The numbers for this parameter are higher for AT PMS in almost all of the locations. The highest difference happens at 78.8% for Highway 44:00. The next largest difference happens for Highway 2A:26 at 62.9%. These two locations are undivided highways with one lane per direction. A portion of the large gap could be because of the increase in traffic for these two undivided highways in 2010, where AT PMS data were available in that year. Other highways have smaller differences with a minimum of -2.2% for Highway 16:06 in which the WIM based calculated ESAL is slightly larger than ESAL from AT PMS (ATR based).

Table 4.4 – Comparison between ESAL from AT and WIM and impacts on pavement design

HWY	ESAL/year/Dir.		20-year Design ESAL**		Design ESAL Difference (%)	Pavement Design Reliability% *		Asphalt Thickness based on 300 mm GBC (mm)				Max. Difference of asphalt layer thickness (mm)
								Subgrade Modulus (MPa)				
								25		50		
	from AT's PMS (2010)	from WIM (2009)	AT	WIM		AT	WIM	AT	WIM	AT	WIM	
2:24	1,265,236	1,218,846	35,554,400	34,250,800	3.8	95	95	383	381	293	291	2
2:30	1,317,431	1,079,181	37,021,100	30,326,100	22.1	95	95	386	374	295	285	12
2A:26	141,875	87,076	3,986,800	2,446,900	62.9	85	85	233	211	162	143	23
3:08	350,801	263,351	9,857,900	7,400,400	33.2	95	90	310	276	230	200	35
16:06	596,811	609,940	16,771,000	1,7139,900	-2.2	95	95	340	341	256	256	-2
44:00	726,934	406,595	20,427,600	11,425,700	78.8	95	95	351	319	265	237	31

*Pavement design reliability base don AT pavement Design Manual for new pavement design and function of design ESAL
** Deign ESALs were rounded to nearest 100

CHAPTER 5

5.0 Comparison of MEPDG Nationally Calibrated Traffic Input Values with Weigh-In-Motion Measurements in Alberta

5.1 Introduction

The MEPDG provides the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures, based on mechanistic principles. (8)

For MEPDG input data, three different levels are defined based on the quality of the data available. Specifically for traffic inputs, At Level 1, a very good knowledge of traffic characteristics is available. This includes “counting and classifying the number of trucks travelling over the roadway, along with the breakdown by lane and direction which are measured at or near the site” (8). At Level 2, “a modest knowledge of traffic characteristics is available” (8), requires vehicle weights clustered into heavy (loaded) and light (unloaded) truck weights. Moreover, Level 2 data is more regional than site specific. Regression estimations are used in this case. At Level 3, a poor knowledge of traffic characteristics including nationally average load distribution is available (8).

AT used the data from six WIM stations to establish the MEPDG different traffic input variables. In this chapter, the default values in the MEPDG Software at Level 3 are compared with the AT’s traffic data extracted from the WIM measurements. AT provided the University with the WIM data for two consecutive years; 2009 and 2010. An example of the data can be found in

Appendix B. The comparison is made for both years to provide more confidence in the analysis. It should be noted that, for all the locations, the design lane (the outer truck lane) is considered in the study. The following parameters are included in the study:

- Truck Traffic Classification
- Truck Hourly Distribution Factors
- Monthly Adjustment Factors
- Axle Load Distribution Factors

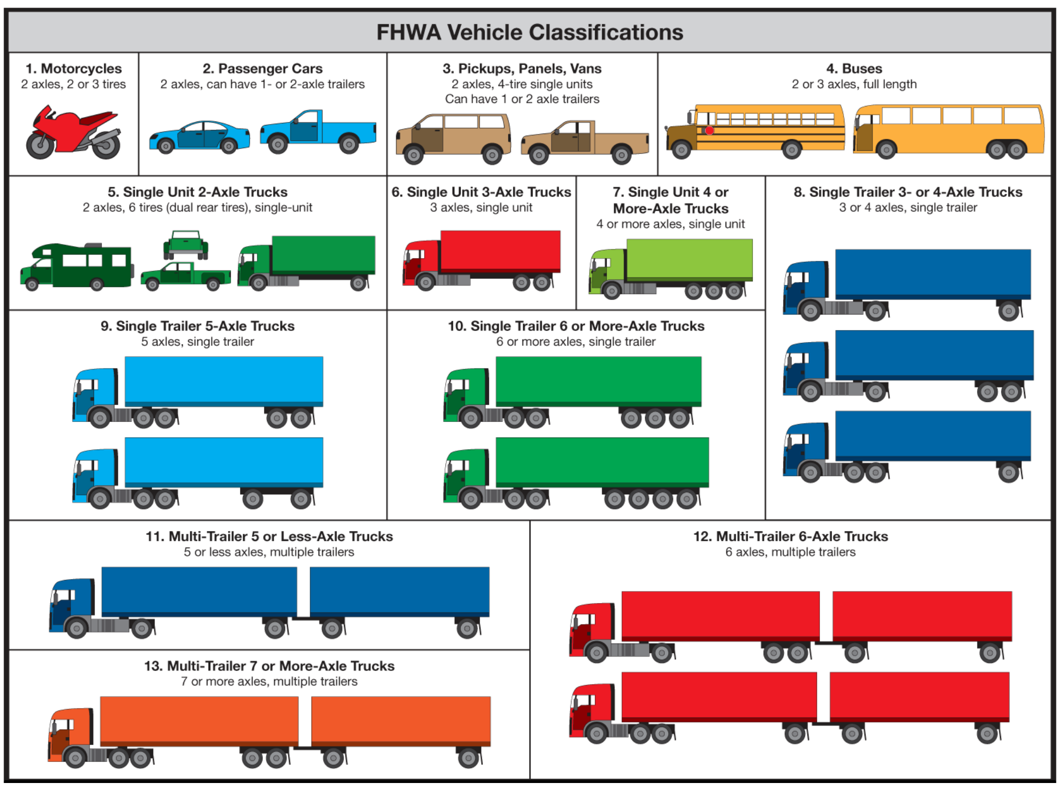

Figure 5.1 – FHWA vehicle classification. Source: http://onlinemanuals.txdot.gov/txdotmanuals/tri/images/FHWA_Classification_Chart_FINAL.png

In order to compare the above traffic factors, the distributions of different classes of truck in the AADTT should be first determined. The FHWA classifies vehicles into 13 different groups. Figure 5.1 represents the FHWA vehicle classifications. AT's provided data consists of the traffic information for these truck classes.

In the second part of this chapter, the effects of these differences on a typical pavement project are evaluated because of a sensitivity analysis on the predicted performance of the subject pavement.

5.2 Comparative Study

In this section, the comparison is performed between default values of traffic characteristics in MEPDG with their counterparts from WIM data.

5.2.1 Truck Traffic Classification

Truck Traffic Classification (TTC) is the distribution of different classes of trucks (4 to 13, based on the FHWA classification) in the AADTT. The MEPDG default values at Level 3 are categorized in 17 different groups – TTC 1 to TTC 17. The default values for different TTCs in the Design Guide software are established based on the traffic data from the Long Term Pavement Performance (LTPP) sections across the United States. For this study the interstate-principal arterial highways was selected for all the six site locations. Table 5.1 presents different TTC groups, as Software's default values and their truck class distributions.

As seen in Table 5.1, Classes 5 and 9 and 10 are the most populated classes.

Table 5.1 - Default TTC embedded in the MEPDG. (6)

TTC Group	Vehicle/Truck Class Distribution (%)									
	Clas s 4	Clas s 5	Clas s 6	Clas s 7	Clas s 8	Clas s 9	Clas s 10	Clas s 11	Clas s 12	Clas s 13
1	1.3	8.5	2.8	0.3	7.6	74.0	1.2	3.4	0.6	0.3
2	2.4	14.1	4.5	0.7	7.9	66.3	1.4	2.2	0.3	0.2
3	0.9	11.6	3.6	0.2	6.7	62.0	4.8	2.6	1.4	6.2
4	2.4	22.7	5.7	1.4	8.1	55.2	1.7	2.2	0.2	0.4
5	0.9	14.2	3.5	0.6	6.9	54.0	5.0	2.7	1.2	11.0
6	2.8	31.0	7.3	0.8	9.3	44.8	2.3	1.0	0.4	0.3
7	1.0	23.8	4.2	0.5	10.2	42.2	5.8	2.6	1.3	8.4
8	1.7	19.3	4.6	0.9	6.7	44.8	6.0	2.6	1.6	11.8
9	3.3	34.0	11.7	1.6	9.9	36.2	1.0	1.8	0.2	0.3
10	0.8	30.8	6.9	0.1	7.8	37.5	3.7	1.2	4.5	6.7
11	1.8	24.6	7.6	0.5	5.0	31.3	9.8	0.8	3.3	15.3
12	3.9	40.8	11.7	1.5	12.2	25.0	2.7	0.6	0.3	1.3
13	0.8	33.6	6.2	0.1	7.9	26.0	10.5	1.4	3.2	10.3
14	2.9	56.9	10.4	3.7	9.2	15.3	0.6	0.3	0.4	0.3
15	1.8	56.5	8.5	1.8	6.2	14.1	5.4	0.0	0.0	5.7
16	1.3	48.4	10.8	1.9	6.7	13.4	4.3	0.5	0.1	12.6
17	36.2	14.6	13.4	0.5	14.6	17.8	0.5	0.8	0.1	1.5

Figures 5.2 to 5.7 present the TTCs based on both the WIM measurements and the best match from the MEPDG for all the six highway locations. Solid lines represent similar directions (North Bound or East Bound, depending on the direction of the highway), while dash lines represent the TTC for the lanes in the opposite direction. In addition, the black lines present the TTC based on the 2010 data, while the grey lines correspond to 2009.

Highways 2:24 and 2:30 in Figures 5.2 and 5.3 show a similar trend for truck classification. For both highways, the FHWA Class 5 truck has a distribution of between 9 to 20 percent, and Classes 6, 7, and 8 show the lowest occurrence in the AADTT. For both highways, approximately 30 percent of the trucks are Class 9.

Truck Class 10 tends to show a high number of approximately 27 percent as well for both the highways. Truck Classes 11 and 12 have a close to zero distribution, while for, Class 13 has a 20 percent distribution. For these two control sections, TTC 11 from the MEPDG is found to best fit the existing trend. As seen Figures 5.2 and 5.3, while Class 10 seems not to follow the TTC11 trend, it is still the best match among the available TTC groups in the MEPDG.

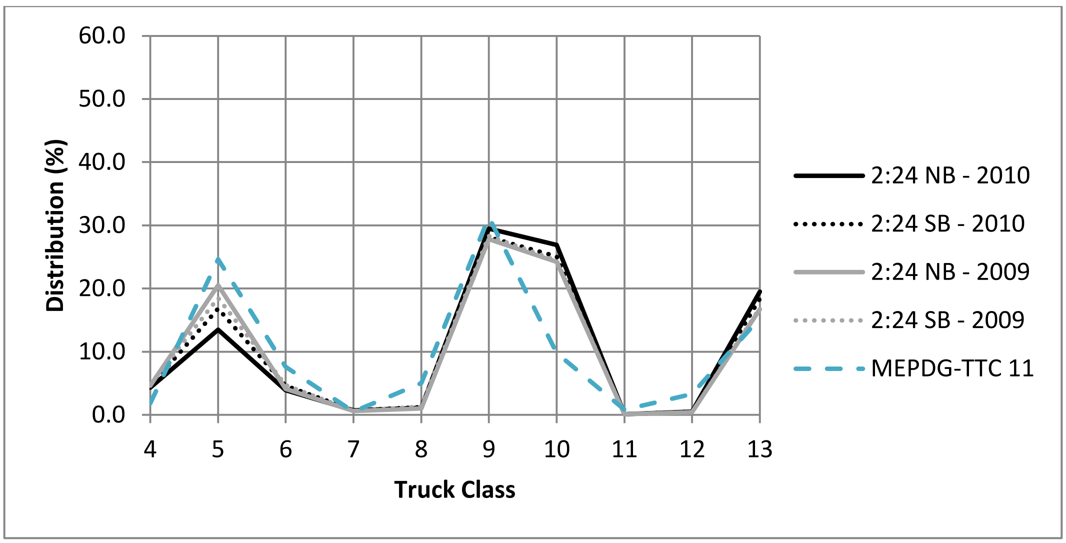

Figure 5.2 - Truck Class distribution – Highway: Control Section 2:24 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

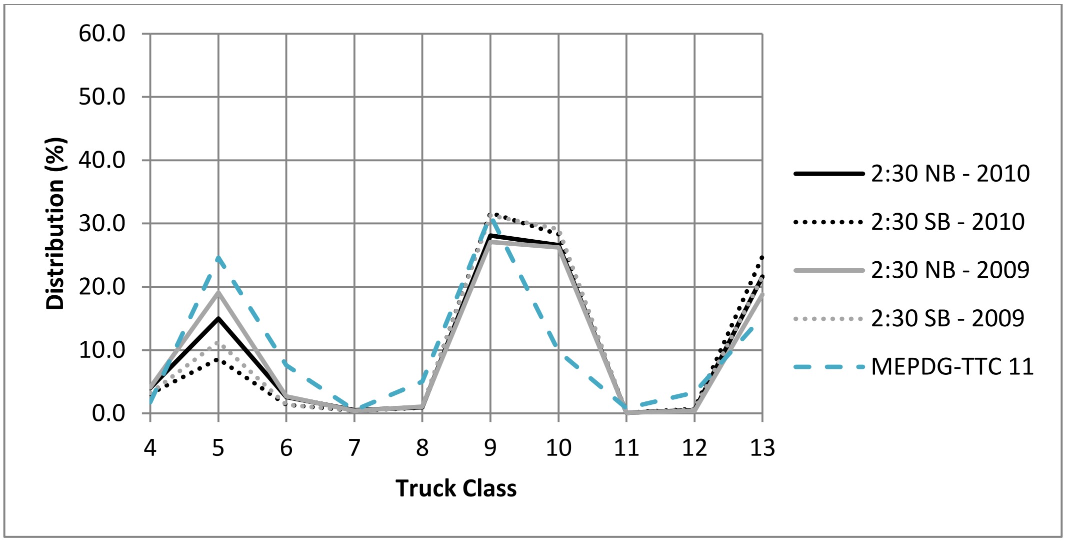

Figure 5.3 - Truck Class distribution – Highway: Control Section 2:30 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

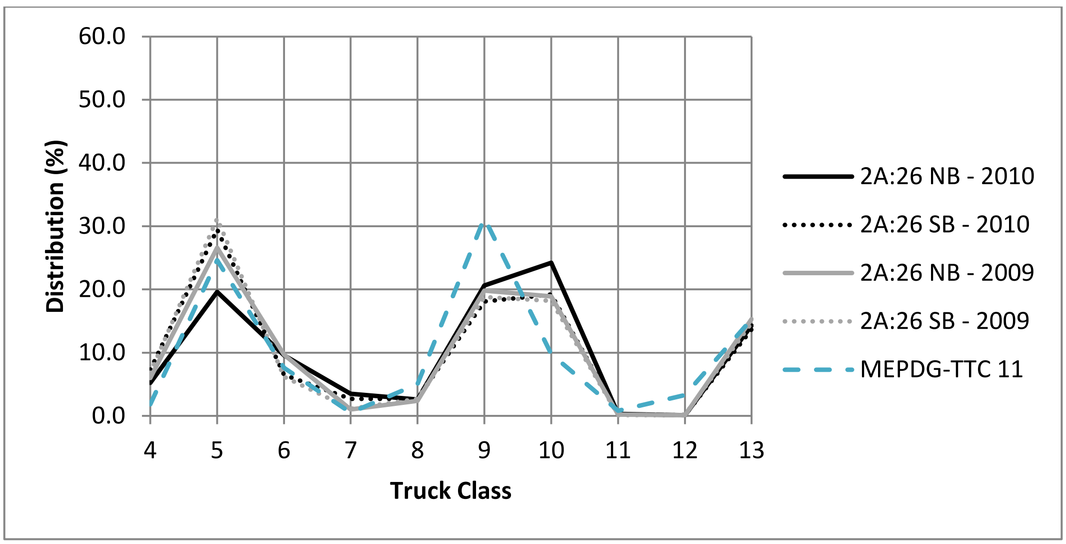

Figure 5.4 - Truck Class distribution – Highway: Control Section 2A:26 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

Figure 5.5 presents the TTC for Highway 3:08 which shows a slightly different trend in comparison to Highway 2A:26. Truck Class 5 has a 15% distribution. Truck Classes 6, 7 and 8 have almost zero distributions. The difference emerges in Class 9 with a high occurrence of 36 percent which is the largest occurrence for Class 9 among all the six highway sections. As seen in Figure 5.5, again TTC 11 from the MEPDG is selected as the best fit for this highway section.

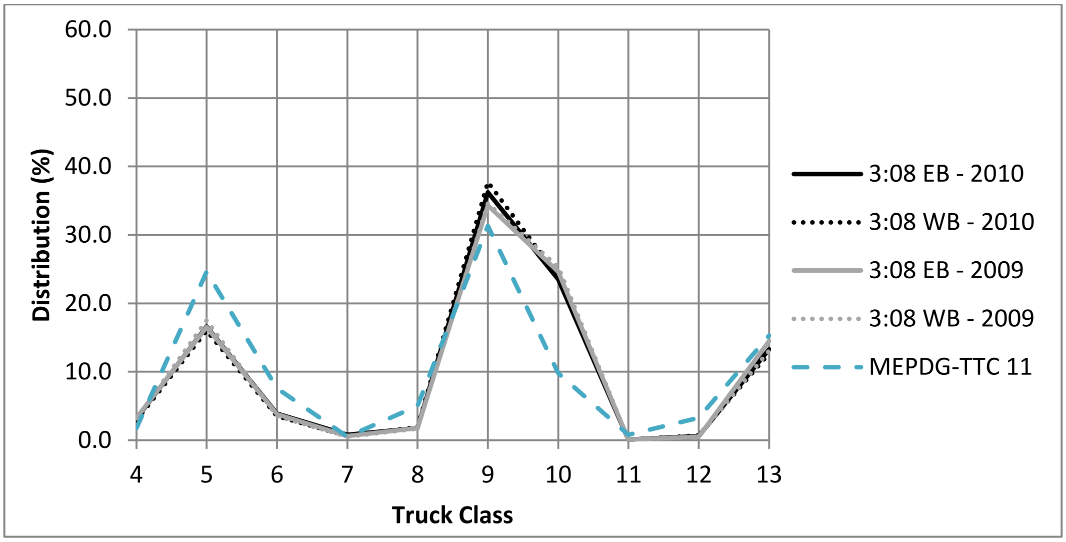

Figure 5.5 - Truck Class distribution – Highway: Control Section 3:08 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

Highway 16:06 and 44:00 in Figures 5.6 and 5.7 have the lowest number of Class 9 trucks in comparison to other highways under study. The distribution for Class 9 is 19 and 10 percent for Highways 16:06 and 44:00, respectively. At the same time, a large number of trucks are recorded for Class 10 for both locations. Especially for Highway 44:00, the distribution of truck Class 10 is as high as 35 percent. For these two locations, TTC11 seems not to follow the WIM trend for all classes of trucks. It appears that another TTC with a lower distribution for Class 9 and a higher distribution for Class 10 should be selected, However, such type of TTC does not exist in the MEPDG. Again, the best fit for these two highways is selected to be TTC11.

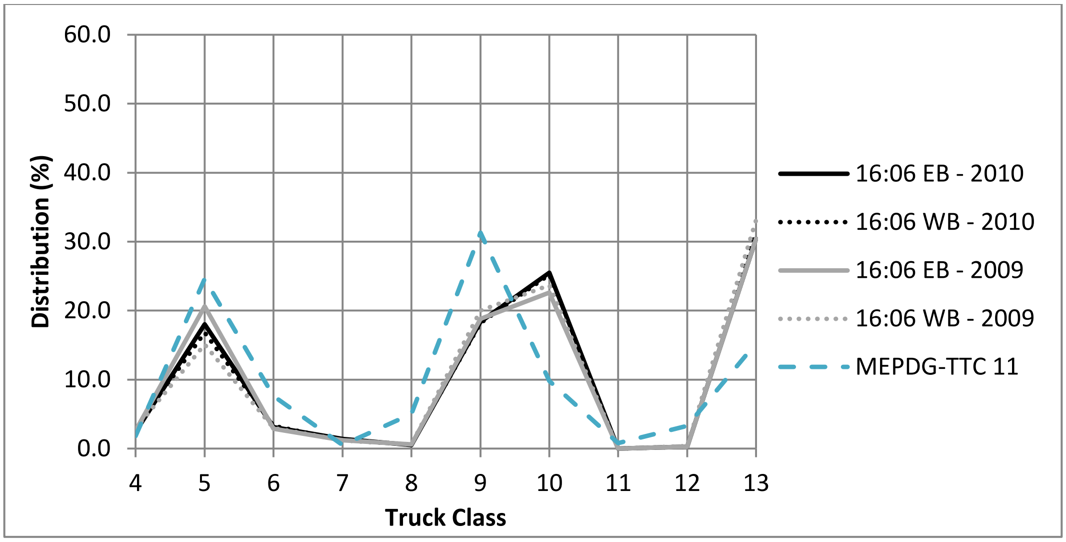

Figure 5.6 - Truck Class distribution – Highway: Control Section 16:06 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

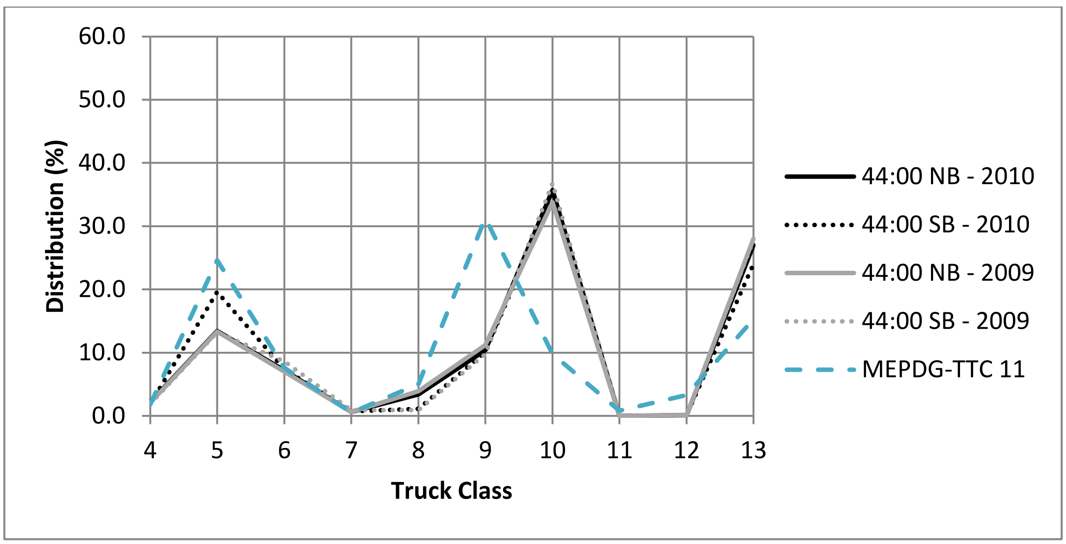

Figure 5.7 - Truck Class distribution – Highway: Control Section 44:00 TTC based on WIM data for 2009 and 2010 versus MEPDG TTC 11.

Comparing different truck distributions available in the MEPDG Software, TTC 11 is found to have the closest distribution to AT's truck distribution for all six highways. In almost all locations, the trend for the truck distribution is similar with two peaks at Classes 5 and 9 and in some cases Class 10. The outer truck lanes in both directions show a similar trend for almost all highways with exceptions in some Highways and truck classes such as Highway 2:24 and 2:30, for Class 5. These slight differences happen due to some arbitrary reasons.

5.2.2 Hourly Distribution Factors

The Hourly Distribution Factor (HDF) represents the percentage of the AADTT within each hour of the day (8). It simply shows how the trucks are distributed in one day. Figures 5.8 through 5.13 illustrate the HDFs for the six different WIM locations for 2009 and 2010 and for both directions (outer truck lane). It should be noted that the MEPDG value for HDF are independent of the TTC group selection. In other words, any other selection of TTC groups would result in a similar HDF in the Design Guide.

The dashed line represents the MEPDG default HDF. The solid lines represent the Northbound (NB) direction while the dotted lines represent the Southbound (SB). Furthermore, Black lines are used to show the 2010 HDF and the grey lines show 2009.

Figure 5.8 presents the HDF for Highway 2:24. Generally, the HDF for this highway section follows the default HDF from the MEPDG very well. From mid-night to 9:00 AM, the MEPDG overestimates the HDF for this highway at one percent difference. On the other hand, from 9:00 AM to mid-night, the MEPDG default HDF underestimates the actual HDF by 2.0 percent at most

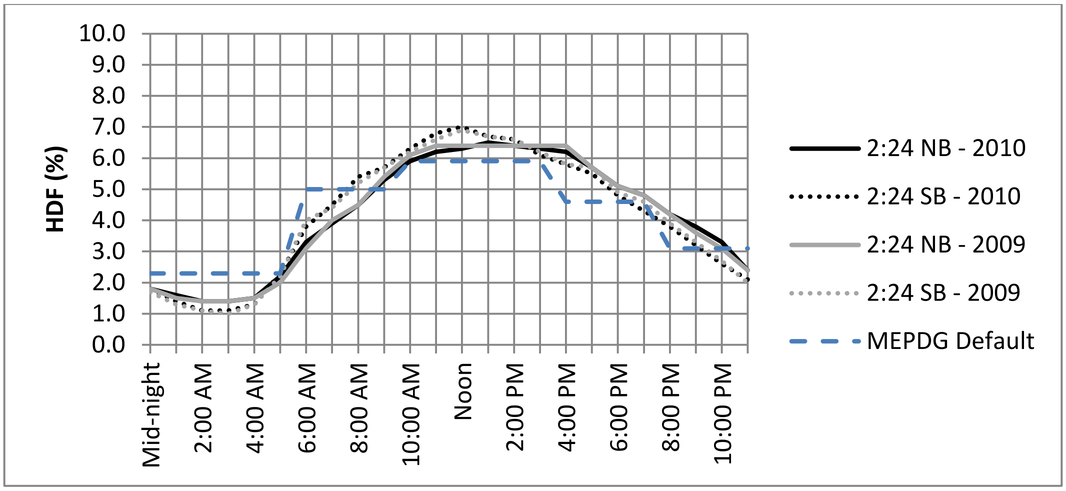

Figure 5.8 - Hourly Distribution Factor - Highway: Control Section 2:24 based on WIM data for 2009 and 2010 versus MEPDG default values.

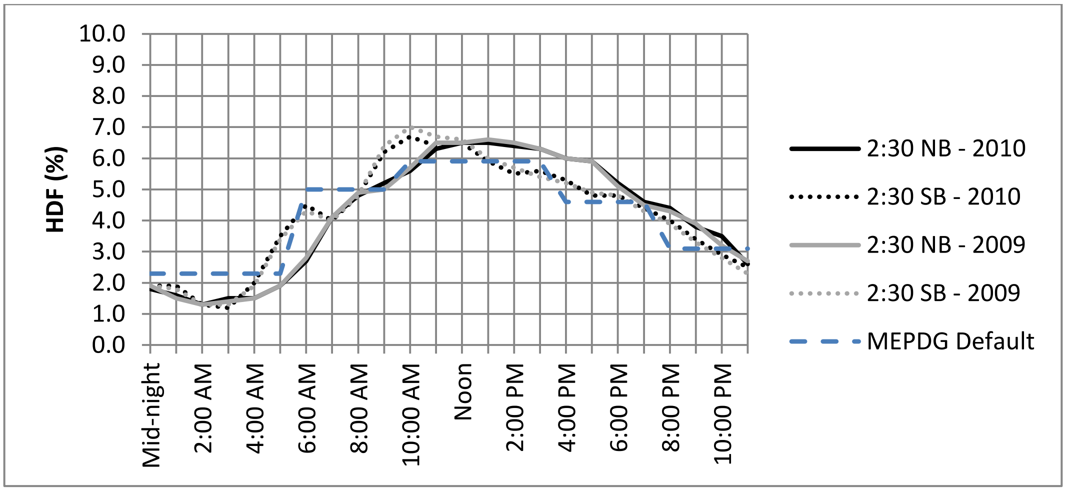

Figure 5.9 - Hourly Distribution Factor - Highway: Control Section 2:30 based on WIM data for 2009 and 2010 versus MEPDG default values.

Figure 5.10 presents the HDF for Highway 2A:26. The MEPDG HDF overestimates the actual HDF in a large scale at a maximum of 2.5 percent from mid-night to 5:00 AM and from 4:00 PM to mid-night. The rush hour lies between 8:00 AM to 2:00 PM at around 8.0 percent.

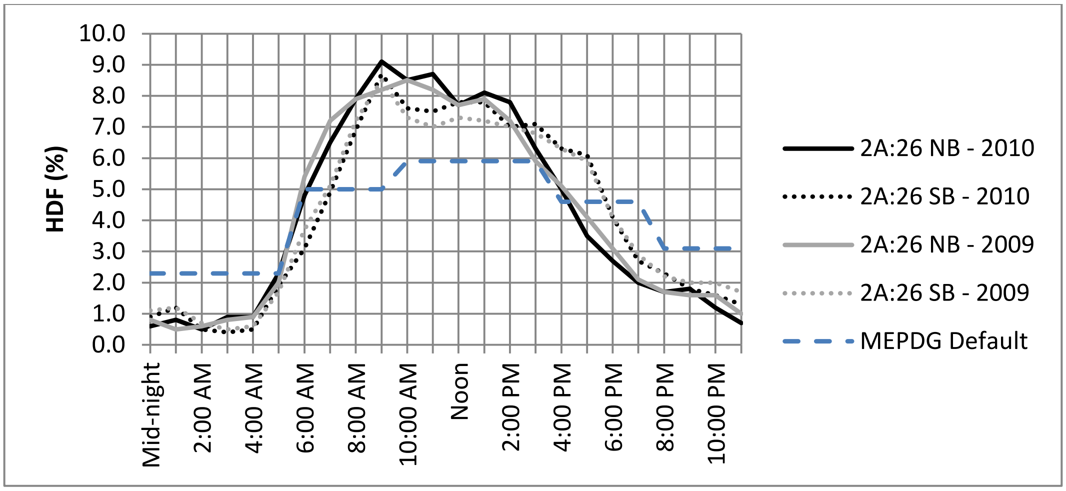

Figure 5.10 – Hourly Distribution Factor – Highway: Control Section 2A:26 based on WIM data for 2009 and 2010 versus MEPDG default values.

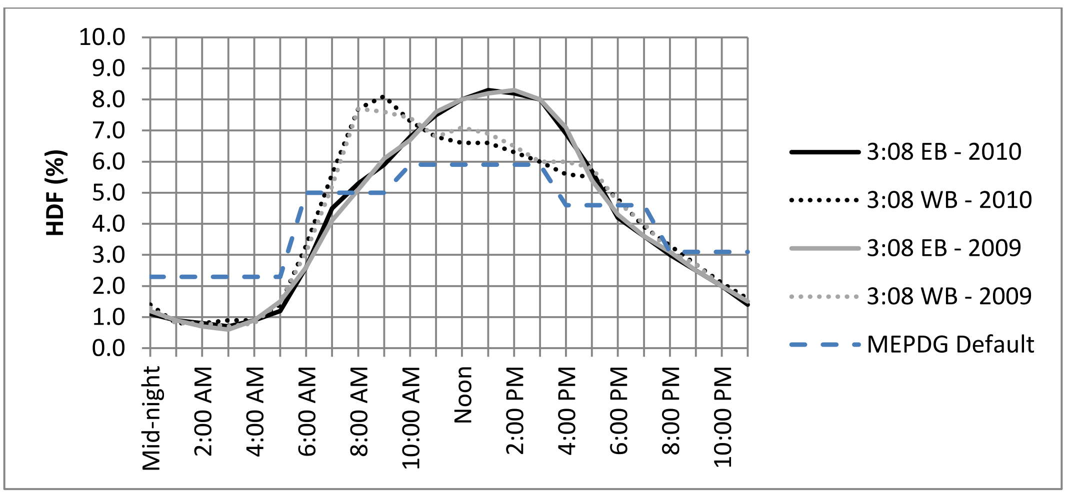

Figure 5.11 – Hourly Distribution Factor - Highway: Control Section 3:08 based on WIM data for 2009 and 2010 versus MEPDG default values

Figure 5.12 provides information on Highway 16:06 about the HDF trends. In this analysis, this section follows the MEPDG very well with a very strong level of acceptance. A minor overestimation occurs from mid-night to 8:00 AM at one percent difference. On the other side of the graph, from 3:00 PM to midnight, a minor underestimation happens from 3:00 PM to 10:00 PM at 1.5 percent.

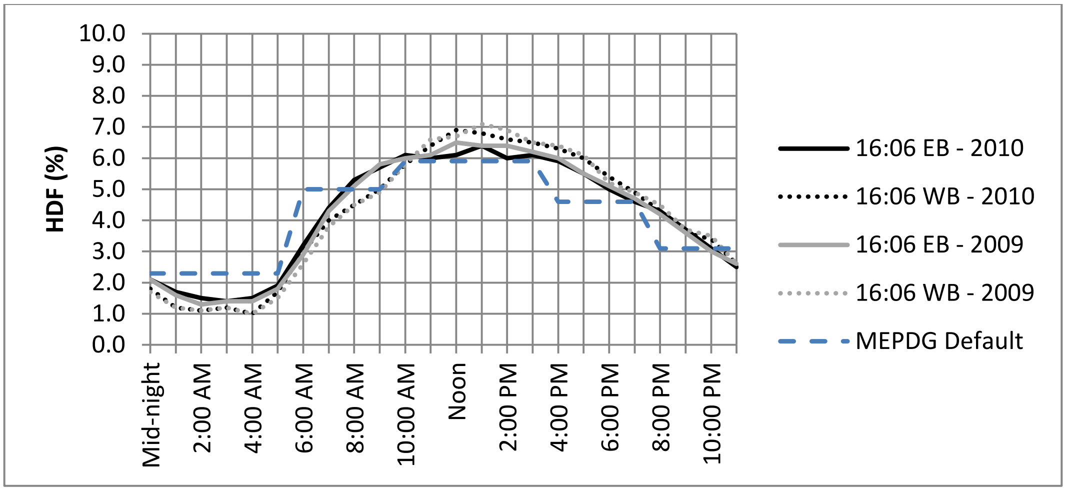

Figure 5.12 – Hourly Distribution Factor - Highway: Control Section 16:06 based on WIM data for 2009 and 2010 versus MEPDG default values

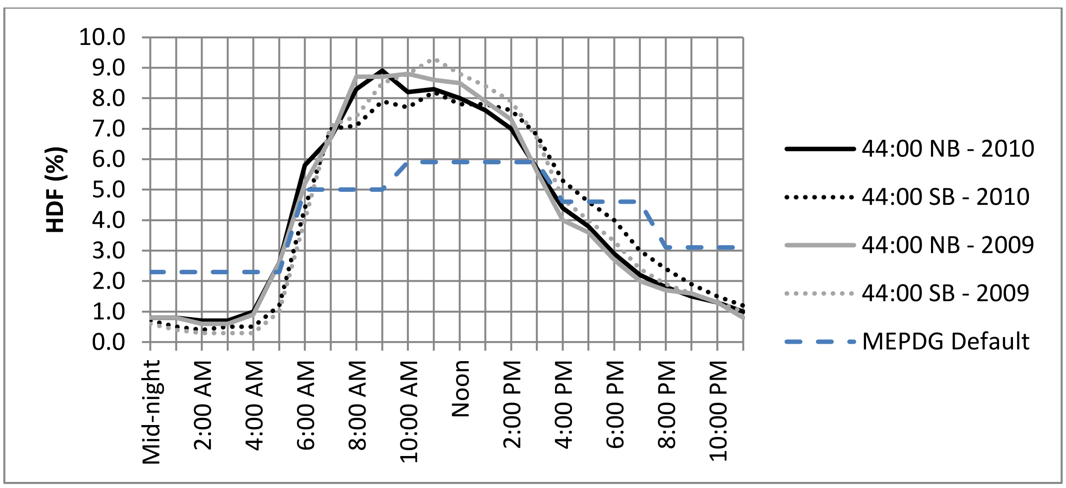

Figure 5.13 – Hourly Distribution Factor - Highway: Control Section 44:00 based on WIM data for 2009 and 2010 versus MEPDG default values

A general for all the locations, the trend is similar to what is set as a default in the MEPDG. For two highways, 2A:26 and 44:00 which are the highways with only one lane in each direction, the deviation from the default values increase to a difference of 4.0 percent from 8:00 AM to 2:00PM. It implies that most of the trucks travel in the day times rather than the evening times.

With regard to the differences between the two years of 2009 and 2010, almost all the locations have a similar trend with minor differences in both positive and negative sides. In other words, in some cases, 2009 HDFs are slightly greater than default values of MEPDG during the day while for some other cases the 2010 HDF lies above 2009.

Another conclusion from these figures is that for all the cases, the rush hour happens at around 9:00 AM to 3:00 PM at a HDF of seven percent. During this period, Highways 2:24, 2:30, and 16:06 have a lower HDF at six percent. On the other hand, Highways 2A:26, 3:08, and 44:00 have a higher HDF at nine percent. The MEPDG HDF sets the rush hour at between 10:00 AM to 3:00 PM at approximately six percent.

Overall, the MEPDG default HDF stay in the range of 2.2 to 5.9 percent while the actual HDF for the six different highways in Alberta show a wider range of 0.8 to 9.1 percent during different hours of the day. The MEPDG overestimates the actual HDF in the afternoon and night time from 4:00 PM to 5:00/6:00 AM. The HDF is underestimated by the MEPDG from 9:00 AM to 3:00 PM.

The comparison of the actual TTC and HDF with the MEPDG defaults shows that the lane direction does not have a considerable influence on the analysis. As a result, in the next comparison which is centered on the monthly adjustment factors only the lanes in one direction per highway location for two consecutive years of 2009 and 2010 are considered. For North-South highway locations, the NB lane is selected and for East-West highways, the EB lane is selected.

5.2.3 Monthly Adjustment Factors (MAF)

Monthly adjustment factors (MAF) is calculated from the following equation (6):

$$MAF_i = \frac{AMDTT_i}{\sum_{i=1}^{12} AMDTT_i} * 12 \qquad \text{Eq. 5.1}$$

MAF_i = monthly adjustment factor for month i.

$AMDTT_i$ = average monthly daily truck traffic for month i.

This factor shows how the truck traffic is distributed over one year period. The default value in MEPDG for all months is equal to one. The WIM data show different values for MAF from a minimum value of zero to a maximum value of 12. However the main purpose of this chapter is to compare default values with AT data, in this part, the comparison is conducted between 2009 and 2010 since the MAFs for the Guide is set to 1 for all situations.

For each truck class (Class 4 to 13), the MAF is derived from recorded WIM data over 20 sensors in place. In this part, as it was discussed before, only the truck lanes in NB and EB directions are considered. The comparison in this section is only between 2009 and 2010 to investigate how different they are from each other.

For this purpose, the MAF for a specific truck class in 2009 and 2010 for a specific month is considered. The differences of MAFs between these 2009 and 2010 MAFs are derived for each class which can be seen in Tables 5.2 to 5.7. Each table is related to one location.

Table 5.2 provides information on recorded MAFs for years 2009 and 2010 in highway 2:24. The differences between MAFs for two consecutive years can be seen in this table. For all other locations, Tables 5.3 to 5.7 provide the same information. It should be noted that in some locations and for some classes of trucks, the MAFs reach a maximum of 12 and a minimum of zero. The former phenomenon is because that specific truck class has passed over the section in only that particular month over one year and the latter is explained in the same way that the truck has not passed over the sensor in that particular month with a zero MAF. As a matter of fact, the summation of the all MAFs for one class in a year is equal to 12 which can be investigated through looking at Equation 5.1.

In the following, MAF variations in 2009 and 2010 for 6 locations are presented in Tables 5.2 to 5.7.

In Table 5.2, which provides information on different MAFs between 2009 and 2010 in Highway 2:24, different months are almost similar to each other with a few exceptions. As it can be seen, January, February, August, November, and December have some recordings which the difference between 2009 and 2010 MAFs are noticeable. In terms of different FHWA classes, Classes 5 and 7 are the two classes having the most frequent big differences in MAFs.

Table 5.2 – MAFs for Highway: Control Section 2:24 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	0.9	0.6	0.9	1.4	1.0	1.0	1.1	0.8	0.8	1.1
	2010	1.1	1.0	0.9	0.9	0.9	0.9	0.9	1.0	1.0	0.9
Feb	2009	0.9	0.7	0.9	1.4	0.9	0.9	1.0	0.9	0.8	0.9
	2010	1.0	1.2	1.0	0.8	0.9	0.9	0.9	0.9	0.9	0.9
Mar	2009	1.0	0.8	0.9	1.1	0.8	1.0	1.1	1.0	1.0	0.9
	2010	1.1	0.9	1.0	0.9	1.0	1.1	1.0	1.2	1.1	1.1
Apr	2009	0.9	0.8	0.8	1.0	1.0	1.0	0.9	1.1	1.1	0.9
	2010	1.0	1.0	1.0	0.9	1.0	1.0	1.0	1.2	1.1	0.9
May	2009	1.1	1.0	1.0	0.8	1.0	1.0	0.9	1.1	1.0	1.0
	2010	1.0	1.1	1.0	0.9	0.9	1.0	1.0	1.1	1.0	1.0
Jun	2009	1.1	1.1	1.1	0.9	1.2	1.0	1.0	1.1	1.0	1.1
	2010	1.1	1.3	1.1	1.0	1.1	1.1	1.1	1.2	1.0	1.0
Jul	2009	1.1	1.4	1.2	1.0	1.2	1.0	1.0	1.0	1.1	1.1
	2010	1.1	1.4	1.0	1.3	1.1	1.1	1.0	1.0	1.1	1.0
Aug	2009	1.1	1.4	1.1	0.9	1.2	1.0	1.0	1.0	1.0	1.1
	2010	0.9	0.9	1.0	1.0	1.1	1.1	1.1	0.9	1.1	1.1
Sep	2009	1.1	1.1	1.2	1.0	1.2	1.1	1.1	1.0	1.2	1.1
	2010	1.1	0.8	1.2	1.2	1.1	1.1	1.1	1.0	1.0	1.2
Oct	2009	1.1	1.0	1.0	0.9	1.0	1.0	1.0	1.2	1.1	1.0
	2010	1.1	0.8	1.2	1.2	1.1	1.1	1.1	1.0	1.0	1.2
Nov	2009	1.0	1.2	1.1	1.0	1.0	1.0	1.1	1.1	1.0	1.0
	2010	0.9	0.5	0.9	1.2	1.0	0.9	1.0	0.7	0.9	1.0
Dec	2009	0.8	0.9	0.8	0.7	0.8	0.9	0.8	0.9	1.1	0.9
	2010	0.9	0.5	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Table 5.3, showing the difference of MAFs for Highway 2:30, have the biggest numbers for MAFs. As a consequence, the differences between MAFs in 2009 and 2010 reach the maximum values. The reason for this is that in 2010, from July to December, the MAFs are calculated to be equal to 0. Due to this malfunction of sensors in those six months, the MAFs are around 2, instead of 1 because half of the MAFs are zero.

Table 5.3 – MAFs for Highway: Control Section 2:30 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	1.0	1.0	1.0	0.8	1.1	1.0	0.9	1.0	0.8	0.9
	2010	2.1	2.2	2.0	1.9	2.1	1.9	1.9	1.9	1.9	1.7
Feb	2009	1.0	1.0	1.0	0.8	0.9	0.9	0.9	0.9	0.8	0.8
	2010	1.9	2.1	2.0	1.7	1.9	1.9	1.8	1.6	1.9	1.7
Mar	2009	1.1	0.9	1.0	1.0	0.9	1.0	1.0	1.0	1.0	0.9
	2010	2.2	1.9	2.2	2.2	2.3	2.2	2.2	1.9	2.2	2.1
Apr	2009	1.0	0.9	0.9	0.9	1.0	1.0	0.9	1.0	1.1	0.8
	2010	2.0	1.9	2.0	1.8	2.1	2.1	2.0	1.9	2.3	2.1
May	2009	1.2	1.2	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.0
	2010	2.2	2.2	2.1	2.2	1.9	2.2	2.2	2.3	2.2	2.3
Jun	2009	1.1	1.1	1.1	1.3	1.1	1.1	1.3	1.1	1.1	1.1
	2010	1.6	1.8	1.8	2.1	1.7	1.7	2.0	2.4	1.6	2.2
Jul	2009	1.0	1.3	1.1	1.1	1.0	1.0	1.1	1.0	1.1	1.2
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Aug	2009	1.0	1.1	1.0	1.1	1.1	1.0	1.1	1.0	0.9	1.2
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sep	2009	1.0	0.9	1.0	1.2	1.1	1.1	1.1	1.1	1.1	1.2
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Oct	2009	1.0	0.9	1.0	1.1	1.0	1.1	1.1	1.1	1.1	1.1
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Nov	2009	1.0	0.8	1.1	1.0	1.0	1.0	1.0	1.1	0.9	1.0
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dec	2009	0.9	1.1	0.9	0.7	0.8	0.9	0.7	0.8	1.0	0.8
	2010	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

MAFs for Highway 2A:26 are shown in Table 5.4. Almost all of the months show some noticeable differences in MAFs between 2009 and 2010. Regarding the FHWA classes 7, 8, 11, 12, and 13 have the biggest MAF differences. For Class 12, in January to March the MAF is calculated to be equal to zero due to no presence of that class during the winter time.

Table 5.4 – MAFs for Highway: Control Section 2A:26 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	1.0	0.7	0.9	0.7	1.1	0.9	0.9	1.3	0.3	1.0
	2010	1.0	0.9	0.6	0.2	1.0	0.8	0.7	0.1	0.0	0.7
Feb	2009	1.0	0.6	0.8	0.4	0.7	0.8	0.9	0.9	1.0	0.9
	2010	0.9	0.8	0.6	0.2	0.8	0.9	0.8	0.2	0.0	0.8
Mar	2009	0.9	0.8	0.8	0.5	0.9	1.0	0.9	0.6	0.6	1.0
	2010	1.1	0.9	0.8	0.6	0.9	1.0	1.0	1.7	0.0	1.0
Apr	2009	1.2	0.9	0.9	0.6	1.0	1.0	0.9	1.1	1.9	0.8
	2010	1.1	1.1	0.8	0.6	0.9	1.0	0.8	0.9	1.9	0.7
May	2009	1.2	1.1	1.3	1.4	1.0	1.0	1.0	0.7	1.0	1.1
	2010	1.0	1.1	1.0	1.0	1.1	1.1	1.1	1.4	0.8	1.0
Jun	2009	1.3	1.2	1.2	0.9	0.9	1.2	1.2	0.4	1.3	1.2
	2010	1.0	1.2	1.2	1.1	1.1	1.1	1.1	1.1	0.4	1.0
Jul	2009	0.9	1.5	1.1	1.6	1.1	1.0	1.1	0.7	1.0	1.1
	2010	1.0	1.3	1.2	1.2	1.2	1.0	1.1	1.4	1.5	0.9
Aug	2009	0.8	1.5	1.1	1.1	1.1	1.1	1.0	0.9	1.0	0.9
	2010	1.1	1.2	1.3	1.5	1.2	1.1	1.2	0.8	2.6	1.3
Sep	2009	1.1	1.2	1.1	1.2	1.0	1.1	1.1	1.3	1.3	1.1
	2010	1.0	1.0	1.3	1.7	1.3	1.1	1.4	0.8	1.5	1.5
Oct	2009	1.0	1.0	1.0	1.6	1.0	1.1	1.4	1.5	1.3	1.3
	2010	1.0	1.0	1.3	1.7	1.3	1.1	1.4	0.8	1.5	1.5
Nov	2009	0.9	0.8	1.0	1.3	0.9	1.0	0.9	1.9	1.0	1.0
	2010	0.9	0.7	0.9	1.3	0.8	0.9	1.1	1.5	1.5	1.1
Dec	2009	0.7	0.6	0.8	0.6	1.2	0.9	0.9	0.7	0.6	0.6
	2010	0.8	0.6	1.0	1.2	0.8	0.8	0.7	1.1	0.4	0.6

Table 5.5 provides information about different MAFs for 2009 and 2010 in Highway 3:08. Different months have different trends. Regarding the diversion in MAFs between these two years, January, March, June, July, September, October, and December show a considerable level of difference. In terms of different FHWA classes, Class 5, 7, 11, and 12 show noticeable differences.

Table 5.5 – MAFs for Highway: Control Section 3:08 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	0.3	0.1	0.2	0.2	0.2	0.3	0.2	0.3	0.2	0.2
	2010	1.0	0.6	0.9	1.6	0.9	0.9	0.8	0.3	0.9	1.0
Feb	2009	1.1	0.5	0.8	0.7	1.0	1.0	0.9	0.7	0.7	1.0
	2010	1.0	0.4	1.1	2.3	0.8	1.0	0.9	0.5	0.6	1.0
Mar	2009	1.0	0.6	1.0	0.6	1.1	1.1	1.1	0.8	1.0	1.0
	2010	1.1	0.7	1.1	1.7	1.0	1.1	1.1	1.3	0.7	1.1
Apr	2009	1.0	0.8	1.1	0.6	1.1	1.1	1.2	1.0	1.1	1.1
	2010	0.9	0.8	1.1	0.7	1.0	1.0	1.0	1.1	0.8	1.2
May	2009	1.1	1.2	1.2	0.8	0.9	1.1	1.1	1.2	1.1	1.0
	2010	1.0	1.1	1.0	0.6	1.0	1.1	0.9	1.2	1.0	1.1
Jun	2009	1.0	1.4	1.1	1.1	1.0	1.2	1.2	1.5	1.6	1.2
	2010	1.0	1.4	1.1	0.7	1.0	1.0	1.0	0.9	1.0	1.0
Jul	2009	1.1	2.1	1.5	0.6	1.1	1.1	0.9	0.8	1.2	1.0
	2010	0.9	1.7	0.9	0.8	1.0	1.1	1.0	1.0	1.0	1.0
Aug	2009	1.0	1.8	1.0	0.7	1.1	1.0	1.0	1.3	1.0	0.9
	2010	1.1	1.4	1.1	0.8	1.1	1.0	1.2	1.3	1.2	0.9
Sep	2009	1.2	1.5	1.1	1.7	1.1	1.0	1.1	0.8	1.4	1.1
	2010	1.2	1.0	1.1	0.8	1.0	0.9	1.2	1.0	1.5	0.9
Oct	2009	1.3	1.0	1.1	1.5	1.4	1.1	1.5	1.0	1.0	1.4
	2010	1.2	1.0	1.1	0.8	1.0	0.9	1.2	1.0	1.5	0.9
Nov	2009	1.0	0.7	1.1	2.2	1.1	1.0	1.2	1.2	0.8	1.2
	2010	1.0	0.6	0.8	0.6	1.1	1.0	1.1	1.2	1.3	1.0
Dec	2009	0.9	0.5	1.0	1.4	1.1	1.0	0.8	1.3	0.9	0.9
	2010	0.8	0.4	0.7	0.6	1.1	0.9	0.7	1.0	1.4	0.9

MAFs in Highway 16:06 are shown in Table 5.6 for 2009 and 2010. Classes 5, 6, 8, and 11 show large diversions. As it can be seen in the table, Class 11 shows an anomaly with two MAFs of 6 in 2009. In 2010 all the MAFs are calculated to be 0 which shows that in that year there is no presence of this class. Regarding the months, only September and November have some considerable differences in MAFs. All other months are categorized as similar to each other.

Table 5.6 – MAFs for Highway: Control Section 16:06 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	1.2	0.7	1.2	1.2	1.0	1.1	1.2	6.0	1.2	1.2
	2010	1.0	0.6	1.0	0.9	0.8	0.8	0.9	0.0	0.8	0.9
Feb	2009	1.2	0.6	1.2	1.1	0.9	0.9	1.2	0.0	0.9	1.1
	2010	1.1	0.7	1.0	1.1	1.0	0.9	1.1	0.0	1.0	1.0
Mar	2009	1.2	0.7	1.1	1.3	0.9	1.0	1.2	0.0	1.1	1.1
	2010	1.3	0.9	1.1	1.4	1.4	1.1	1.2	0.0	0.9	1.2
Apr	2009	0.8	0.7	0.8	0.6	0.8	1.0	0.8	0.0	0.9	0.9
	2010	1.0	0.9	0.7	0.6	0.8	1.0	0.9	0.0	0.9	0.9
May	2009	0.8	1.0	0.8	0.6	0.8	1.1	0.8	0.0	1.0	0.9
	2010	0.9	1.2	0.8	0.7	1.1	1.1	0.9	0.0	1.1	0.9
Jun	2009	1.0	1.5	1.1	1.0	1.1	1.0	1.0	0.0	0.7	0.9
	2010	1.0	1.1	1.0	1.1	1.3	1.1	1.0	0.0	1.1	1.0
Jul	2009	0.9	1.6	1.0	1.0	1.2	1.0	0.9	0.0	1.1	1.0
	2010	1.1	1.6	1.0	1.0	1.1	1.1	1.1	0.0	1.2	1.0
Aug	2009	0.9	1.9	0.9	0.9	1.2	1.0	0.9	0.0	1.0	1.0
	2010	0.7	1.0	0.7	0.8	0.8	0.8	0.6	0.0	0.9	0.7
Sep	2009	0.9	1.1	0.9	0.9	1.2	1.1	1.0	6.0	1.0	1.0
	2010	0.9	1.0	1.4	1.3	0.9	1.2	1.2	0.0	1.2	1.2
Oct	2009	1.0	0.8	1.0	1.1	1.0	1.1	1.1	0.0	1.2	1.0
	2010	0.9	1.0	1.4	1.3	0.9	1.2	1.2	0.0	1.2	1.2
Nov	2009	1.0	0.6	1.0	1.1	1.0	1.0	1.1	0.0	1.0	1.0
	2010	1.0	0.9	1.4	1.2	0.9	1.1	1.2	0.0	1.1	1.2
Dec	2009	1.0	0.8	1.1	1.1	0.9	0.9	1.0	0.0	0.8	1.0
	2010	1.2	0.8	1.1	1.1	1.1	1.0	1.1	0.0	1.1	1.1

Table 5.7 shows different MAFs for Highway 44:00 in 2009 and 2010. January, April, May, June, September, November, and December are the months with big difference between MAFs for these two years. Regarding the FHWA classes, some classes have considerable differences in MAFs. Classes 5, 6, 7, 10, and 12 are put into the category of noticeable MAF variations. Class 11 is an anomaly with all the MAFs in 2009 and 2010 equal to zero. It shows that there was no presence of this truck type in this highway during 2009 and 2010.

Table 5.7 – MAFs for Highway: Control Section 44:00 in 2009 and 2010

Month	Year	FHWA Class									
		4	5	6	7	8	9	10	11	12	13
Jan	2009	1.4	0.6	0.8	1.0	0.4	0.8	0.6	0.0	1.5	0.8
	2010	1.1	0.7	0.7	0.7	0.5	0.8	0.6	0.0	0.6	0.7
Feb	2009	1.2	0.5	0.9	1.2	0.5	0.9	1.0	0.0	1.0	0.9
	2010	1.0	0.9	0.7	0.6	0.3	0.8	0.5	0.0	0.6	0.7
Mar	2009	1.3	0.6	1.0	1.4	0.7	1.1	0.8	0.0	0.8	1.0
	2010	1.0	0.8	0.8	1.3	0.4	1.0	0.7	0.0	0.7	0.8
Apr	2009	0.7	0.6	1.3	0.6	0.8	0.8	0.4	0.0	0.7	0.6
	2010	0.8	1.1	1.0	1.2	0.5	1.0	0.7	0.0	0.5	0.8
May	2009	0.7	0.8	1.7	0.6	0.9	0.8	0.7	0.0	0.4	0.8
	2010	0.7	0.7	0.9	0.8	1.1	1.0	1.0	0.0	1.0	0.9
Jun	2009	0.9	0.8	1.1	1.1	1.4	1.1	1.1	0.0	0.4	1.2
	2010	1.0	1.2	1.3	1.0	1.6	1.2	1.5	0.0	1.4	1.3
Jul	2009	0.9	0.7	1.0	1.1	1.4	1.0	1.1	0.0	1.0	1.1
	2010	0.8	0.8	0.9	0.9	1.2	0.9	1.1	0.0	1.0	0.9
Aug	2009	0.9	0.8	0.8	1.4	1.5	1.2	1.5	0.0	1.3	1.3
	2010	1.0	0.8	1.0	1.1	1.7	1.1	1.4	0.0	1.0	1.4
Sep	2009	0.9	0.7	0.8	1.4	1.7	1.4	1.8	0.0	1.8	1.6
	2010	1.3	1.3	1.4	1.2	1.8	1.2	1.5	0.0	1.2	1.3
Oct	2009	1.0	0.6	0.8	1.2	1.6	1.2	1.7	0.0	1.4	1.4
	2010	1.3	1.3	1.4	1.2	1.8	1.2	1.5	0.0	1.2	1.3
Nov	2009	1.0	2.3	1.0	0.5	0.8	0.8	0.8	0.0	0.8	0.7
	2010	1.3	1.3	1.1	1.1	1.1	0.9	1.0	0.0	1.0	1.0
Dec	2009	1.1	3.0	0.9	0.5	0.4	0.8	0.5	0.0	0.9	0.7
	2010	1.2	1.0	0.8	0.9	0.4	0.8	0.6	0.0	1.2	0.8

Generally, a two year set of data is not large enough to establish a sophisticated comparison. Having more years data could be more helpful to get a reasonable result out of MAFs generated from WIM sensors. It should be noted again that as for making comparison to MEPDG's values, all the MAFs are equal to 1 in the Guide.

In the next step, based on different MAFs in place, a simple statistical analysis is done on these differences to see how big they are or on the opposite side, how small they could be.

With regards to the maximum differences of these MAFs for each class, Table 5.8 is formed. Columns are different truck classes while the rows represent different locations. As it was shown before, the difference of MAFs between 2009 and 2010 could be calculated. Next, the maximum of various "MAF differences" for each class is derived for each class in every location with related month of occurrence. It should be emphasized that Table 5.8 does not provide any information on the MAFs themselves rather it provides information on the maximum of differences between 2009 and 2010.

Table 5.8 represents the absolute maximum differences of MAFs for two consecutive years of 2009 and 2010. Highway 2:24 shows the least variation between other locations. On the other hand, Highway 2:30 has the biggest numbers for all classes. Talking about Highway 2A:26 and 3:08 the absolute maximum differences ranges from 0.3 - 1.7 and 0.4 – 1.6, respectively. The highest number for this defined value occurs in Highway 16:06 at 6.0 for Class 10 and 11 trucks. Highway 44:00 keeps the highest number at 2.0 for Class 5 and the lowest of 0.3 for Class 8, 9 and 13.

Table 5.8 – Absolute maximum difference of MAFs for 2009 and 2010 data

Highway	FHWA Class									
	4	5	6	7	8	9	10	11	12	13
2:24	0.2 (Jan, Aug, Nov)	0.7 (Nov)	0.2 (Apr, Jul, Oct, Nov)	0.6 (Jan, Feb)	0.3 (Dec)	0.1 (Jan, Mar, Oct, Nov, Dec)	0.2 (Jan, Dec)	0.4 (Nov)	0.2 (Jan)	0.2 (Oct)
2:30	1.2 (Mar)	1.3 (Jul)	1.2 (Mar)	1.2 (Mar, May, Sep)	1.4 (Mar)	1.2 (Mar, May)	1.2 (Mar, May)	1.3 (Jun)	1.2 (Mar, Apr, May)	1.3 (Apr, May)
2A:26	0.3 (Aug)	0.3 (Aug)	0.3 (Oct)	0.5 (Jan, Dec)	0.4 (Dec)	0.1 (Feb, May, Jul, Oct, Dec)	0.2 (Jan, Aug, Nov)	1.2 (Jan)	1.7 (Aug)	0.4 (Aug)
3:08	0.7 (Jan)	0.4 (Jan, Jul, Aug)	0.8 (Jan)	1.6 Feb, Nov)	0.7 (Jan)	0.7 (Jan)	0.6 (Jan)	0.6 (Jun)	0.7 (Jan)	0.8 (Jan)
16:06	0.3 (Jan)	0.8 (Aug)	0.4 (Nov)	0.4 (Jan)	0.5 (Mar)	0.3 (Jan)	0.4 (Sep)	6 (Jan)	0.4 (Jan)	0.3 (Jan, Aug, Sep)
44:00	0.3 (Jan, Mar, Oct)	2 (Dec)	0.7 (May)	0.6 (Feb, Nov)	0.3 (Mar, Apr)	0.3 (Sep)	0.5 (Feb, Sep)	- -	1 (Jun)	0.3 (Nov)

The maximum values of MAFs differences occur in different months for various highways and locations. In the following, the numbers of maximum occurrences based on the months are listed. In other words, the frequencies of each month with the maximum MAF difference are represented in the followings:

- January – 23
- February – 5
- March – 11
- April – 4
- May – 7
- June – 3
- July – 4
- August – 9
- September – 5
- October – 6
- November – 10
- December – 7

January, with the highest number of maximum difference occurrence stands at the top among other months for MAF comparison. It shows that in January 2009 and January 2010, the MAF differences are maximized at a frequency of 23 times. March, November, and August are the next months with highest occurrences.

5.2.4 Axle Load Distribution Factors

Currently the ESAL concept is used for pavement design according to AASHTO 1993. All the traffic loads from passenger car weights to a Class 14 heavy truck weights are converted to a single 8.1 ton standard axle load (ESAL). The ESAL is calculated based on the proportion of every axle passing over the section to the standard axle load of 8.1 ton. This proportion is then powered to a specific number based on the axle type (Single, Tandem, or Tridem). At the end, all the traffic for the design life is reflected in one single number for the design ESAL.

The new approach is different as all the axle loads are accepted as inputs which form an axle load distribution (spectrum) for each axle type. The key difference between these two approaches is that in the former approach, only one number represents the traffic effect while in the latter, the traffic is represented by a load spectrum rather than a single value. The axle load distribution for the different six highways is conducted in this section. The axle load distribution represents the percentage of the total axle applications within each load interval for a specific axle type (8). The MEPDG includes four types of axles: 1) Single axles, 2) Tandem axles, 3) Tridem axles, and 4) Quad axles.

Regarding load intervals, based on the axle type, the following load intervals are considered. For single axles: 3,000 to 41,000 lbs. at 1,000-lb intervals, tandem axles: 6,000 to 82,000 lbs. at 2,000-lb intervals, and for tridem and quad axles: 12,000 to 102,000 lbs. at 3,000-lb intervals.

Based on the WIM traffic data for six different locations in Alberta, the axle load distribution for different axle types and classes of trucks are evaluated. For this purpose, two years of data, 2009 and 2010, are available for 20 different lanes. Furthermore, there are four different types of axles to be studied and also 10 different FHWA classes of trucks. Additionally, in each month of the year, the axle load distribution is from the WIM measurement is different. The axle load distribution from the MEPDG is annually normalized.

As it was shown previously, 19200 different trends are available to do the analysis. This large number of trends could be decreased in a way that fewer numbers of them are filtered and after that, with a smaller size of data, the analysis is performed.

TTC groups in 5.2.1 show that the majority of truck classes are Class 5, 9, 10, and 13 as their percentage of presence is very large in comparison to other FHWA classes for both AT's data and MEPDG default values. Consequently, in order to compare axle load distribution factors, these four classes are selected and discussed in the following (deduction from 10 to 4 FHWA classes). Regarding lanes, similar to previous sections, only the truck lanes (right lanes in each direction) are considered (deduction from 4 to 2 lanes). The third deduction occurs for axle types. MEPDG has 4 types of axle which are single, tandem, tridem and quad axles. In Alberta, by reviewing the recorded data, it is found that there is no presence of quad axles in none of the locations within 2 years of data collection by WIM sensors (deduction from 4 axle types to 3). As a consequence, of doing these modifications in the database, the number of trends for WIM is decreased to 3456. On the other side, the MEPDG's number of trends decreases from 40 to 12. The reason is that the number of axle types to be compared goes down from 4 to 3 as there is no need to compare them with WIM data. Number of truck classes which should be evaluated decreases from 10 to 4 as it was discussed before. The new magnitude of database is as follows:

- WIM:

$$2(year) * 12(month) * 12(\text{lane}) * 3(axle\ type) * 4(FHWA\ class) = 3456\ trends$$

- MEPDG:

$$4\ (FHWA\ class) * 3\ (axle\ type) = 12\ trends$$

3456 different trends are finally selected for the comparison purpose. Each axle type (single, tandem and tridem) have a specific axle load distribution for FHWA Classes 5, 9, 10, and 13 trucks. Furthermore, each location has a total of 144 trends (12 month, 2 year, 2 lane, and 3 axle type) to be compared to the MEPDG default values of which for a specific highway and axle type (for instance, single axle for Highway 2:24) 48 trends exist (12 month, 2 year, 2 lane).

The procedure for evaluating axle load distributions consists of the following steps: 1) Having the database clustered for four FHWA classes, for each location and axle type, a table is created showing the distribution for different load groups for 24 months and 2 lanes and an additional MEPDG distribution. For each specific FHWA class, 18 tables are created for 6 highway locations and 3 axle types. It should be noted that only the axle type and truck class influence the MEPDG default distribution. 2) For every single distribution (trend) in each location, summation of the product of load group and its linked frequency is calculated. This summation provides a very good indication of the distribution effect. As this number gets larger, the influence of that specific distribution increases. 3) Maximum, minimum and median of these summations are marked as well as the MEPDG default value.

This procedure is repeated for other classes, axle types, and locations. Maximum, minimum and median values are for three different distributions with the highest, lowest, and average influence on the pavement. In fact, as the frequencies of larger load groups increase, the summation increases consequently resulting in a larger value for the summation number. Maximum and minimum values are chosen to compare the extreme values and their related effects. Median value is chosen, additionally, to provide more statistical information about axle load distributions. These values are then compared to MEPDG summation number.

Table 5.9 to 5.12 summarize above procedure for different FHWA classes. For each axle type and location, summation of product of each load group (different for various axle types) and its related frequency for MEPDG is presented in third column. Similar calculations are done for AT’s data and the maximum, minimum, and median numbers are derived. In the three last columns, the deviation percentage from MEPDG is calculated to better present the diversity of in place data.

Table 5.9 provides information on the FHWA Class 5 axle load distributions. The MEPDG sum of product numbers are 759,180 lbs, 1,449,040 lbs, and 2,889,740 lbs for single, tandem, and tridem axles, respectively. For single axle, the percent differences range from a minimum -35.74% to a maximum of 11.40%, for highway 44:00. Regarding tandem and tridem axle types, the deviations are larger in comparison to single steering axle and are in negative side. The WIM data have smaller sum of products in comparison to the default MEPDG calculated numbers.

Table 5.9 – FHWA Class 5 axle load distribution effect

Axle Type	Highway	Load (lb): Σ(Frequncy * Load Group)	Deviation from MEPDG (%)		
		MEPDG	Min	Median	Max
Single	2:24	759,180	-28.86	-19.00	-12.66
	2:30	759,180	-30.14	-24.44	-14.91
	2A:26	759,180	-17.34	-9.81	-3.36
	3:08	759,180	-24.55	-15.38	-9.26
	16:06	759,180	-29.95	-18.94	-10.23
	44:00	759,180	-35.74	-15.13	11.40
Tandem	2:24	1,449,040	-44.27	-40.89	-35.33
	2:30	1,449,040	-45.95	-44.01	-38.07
	2A:26	1,449,040	-46.23	-38.42	-25.42
	3:08	1,449,040	-49.80	-41.78	-34.72
	16:06	1,449,040	-46.47	-41.90	-33.46
	44:00	1,449,040	-44.67	-35.38	-19.80
Tridem	2:24	2,899,740	-53.95	-49.31	-45.27
	2:30	2,899,740	-55.04	-52.42	-43.91
	2A:26	2,899,740	-57.32	-51.06	-37.93
	3:08	2,899,740	-58.62	-51.47	-36.71
	16:06	2,899,740	-54.08	-49.21	-39.59
	44:00	2,899,740	-55.73	-50.41	-37.11

In Table 5.10, the summation of products of different axle type distributions are presented for FHWA Class 9. MEPDG sum of product numbers are 1,039,950 lbs., 2,302,260 lbs., and 1,673,100 lbs. for single, tandem, and tridem axle types, respectively. Regarding single axle type, deviated sum of products ranges from a minimum of -40.59% to a maximum of 32.31% for Highway 44:00 and 2A:26, respectively. The minimum value for tandem axle type happens in Highway 44:00 at -38.64%. Tridem axle type has more largely scaled deviations in maximum side. For Highways 3:08 and 2A:26 deviations of 133.10% and 124.13% are, respectively.

Table 5.10 – FHWA Class 9 axle load distribution effect

Axle Type	Highway	Load (lb): Σ(Frequncy * Load Group)	Deviation from MEPDG (%)		
		MEPDG	Min	Median	Max
Single	**2:24**	1,039,950	-21.04	-4.03	1.41
	2:30	1,039,950	-28.55	-13.35	11.19
	2A:26	1,039,950	-0.64	20.83	32.31
	3:08	1,039,950	-9.99	-5.38	20.28
	16:06	1,039,950	-22.44	-6.71	4.46
	44:00	1,039,950	-40.59	-15.79	20.84
Tandem	**2:24**	2,302,260	-19.93	-12.31	-3.50
	2:30	2,302,260	-35.22	-5.26	13.99
	2A:26	2,302,260	-35.41	-19.61	-9.48
	3:08	2,302,260	-8.77	-2.54	11.76
	16:06	2,302,260	-11.66	1.69	15.15
	44:00	2,302,260	-38.64	-16.91	10.84
Tridem	**2:24**	1,673,100	-12.03	4.26	32.37
	2:30	1,673,100	-16.32	-1.38	24.37
	2A:26	1,673,100	-28.28	7.59	124.13
	3:08	1,673,100	-23.79	13.56	133.10
	16:06	1,673,100	-28.28	5.79	30.00
	44:00	1,673,100	-12.27	12.48	77.93

Axle load distribution of FHWA Class 10 for different highways and axle types are presented in Table 5.11. MEPDG summation of product number ranges from 1,027,300 lbs. to 3,006,780 lbs. for single and tridem axle types. For all axle types, the WIM data have maximum and minimum values in positive and negative sides. A maximum of 41.33% exists for single axle in Highway 2A:26. The minimum value of sum of product occurs in Highway 44:00 for tandem axle at -47.16%. In some cases, the median value is almost the same as MEPDG value which shows how different distributions are centered near the default value.

Table 5.11 – FHWA Class 10 axle load distribution effect

Axle Type	Highway	Load (lb): Σ(Frequncy * Load Group)	Deviation from MEPDG (%)		
		MEPDG	Min	Median	Max
Single	2:24	1,027,300	-16.97	0.65	5.21
	2:30	1,027,300	-30.07	-10.17	15.26
	2A:26	1,027,300	11.19	28.22	41.33
	3:08	1,027,300	-8.60	0.57	26.32
	16:06	1,027,300	-48.24	-40.49	-33.66
	44:00	1,027,300	-35.89	-8.30	25.21
Tandem	2:24	2,469,900	-10.89	1.35	12.18
	2:30	2,469,900	-32.73	-20.40	23.98
	2A:26	2,469,900	-23.15	-11.92	-1.67
	3:08	2,469,900	-18.32	-2.78	15.15
	16:06	2,469,900	-5.01	12.36	24.22
	44:00	2,469,900	-47.16	-8.37	38.73
Tridem	2:24	3,006,780	-13.17	-2.06	17.67
	2:30	3,006,780	-27.50	-16.14	35.92
	2A:26	3,006,780	-20.55	1.36	20.63
	3:08	3,006,780	-15.36	5.89	27.32
	16:06	3,006,780	-11.01	14.26	25.27
	44:00	3,006,780	-44.62	-6.92	44.04

Table 5.12 provides information on axle load distribution effect for FHWA Class 13. MEPDG sum of product numbers are 1,022,640 lbs., 2,439,860 lbs., and 3,924,630 lbs. for single, tandem, and tridem axle types, respectively. Median values for all axle types are well centered on zero percentage of difference from MEPDG sum of product numbers. However, the minimum and maximum values are -59.50% and 39.73% for tandem axle in Highway 44:00 and single axle in Highway 2A:26, respectively. This shows the magnitude of diversity in calculated sum of products of different distributions in this FHWA class.

Table 5.12 – FHWA Class 13 axle load distribution effect

Axle Type	Highway	Load (lb): Σ(Frequncy * Load Group)	Deviation from MEPDG (%)		
		MEPDG	Min	Median	Max
Single	2:24	1,022,640	-14.03	0.82	8.02
	2:30	1,022,640	-28.99	-3.93	17.25
	2A:26	1,022,640	14.73	29.16	39.73
	3:08	1,022,640	-14.21	-3.30	19.52
	16:06	1,022,640	-18.67	-0.44	8.87
	44:00	1,022,640	-31.31	-4.74	24.09
Tandem	2:24	2,439,860	-19.79	0.85	14.79
	2:30	2,439,860	-23.61	-3.36	10.83
	2A:26	2,439,860	-31.33	-10.26	6.08
	3:08	2,439,860	-30.99	-0.79	21.84
	16:06	2,439,860	-43.58	-4.12	33.59
	44:00	2,439,860	-59.50	-16.09	32.64
Tridem	2:24	3,924,630	-24.91	-9.53	8.26
	2:30	3,924,630	-22.70	-15.46	-0.73
	2A:26	3,924,630	-19.96	3.20	18.48
	3:08	3,924,630	-37.66	-10.56	13.93
	16:06	3,924,630	-46.97	-11.52	8.52
	44:00	3,924,630	-50.51	-19.63	12.82

5.3 Designing the Inputs Matrix for the Sensitivity Study

After performing the comparative study on the traffic characteristics from the WIM data and the default values from the MEPDG, the effects of these differences on the pavement performance are discussed in this section through a sensitivity analysis. For this purpose, the MEPDG Software Version 1.100 is used. The extreme deviations from the MEPDG for each traffic input are used in the sensitivity analysis.

5.3.1 TTC

In Section 5.2.1, TTC graphs were plotted to investigate their differences from each other and the MEPDG default value. TTC 11 was selected as the best match with the AT's WIM data for all the six locations. Based on the visual comparison between the TTC graphs, a representative of each location's TTC is shown in Figure 5.14 having the most deviated values from MEPDG default TTC 11.

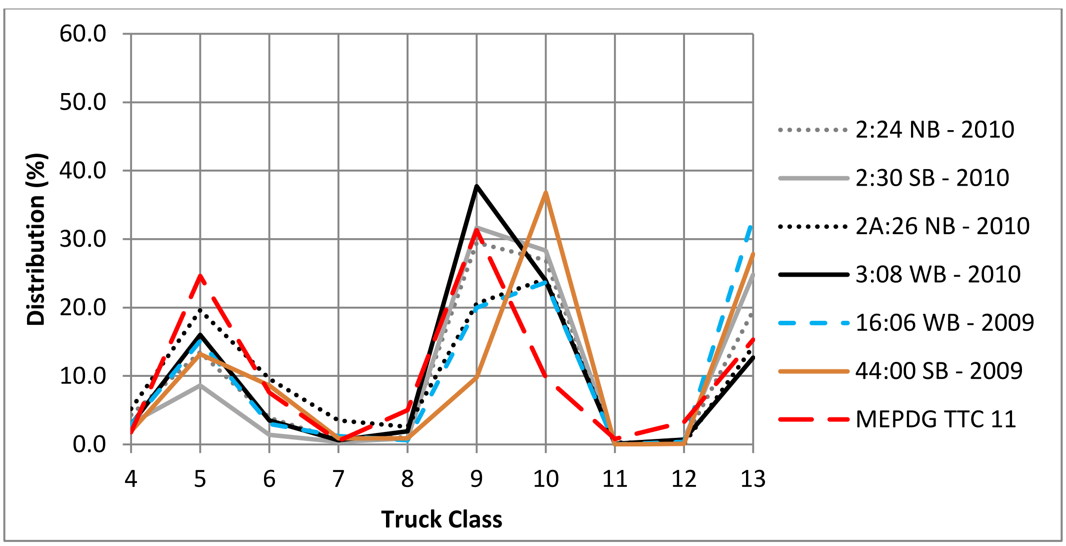

Figure 5.14 – TTC extremes as representatives of each WIM site location

As Figure 5.14 shows, Highways 16:06 and 2A:26 are similar to each other with differences seen in Class 13 and 5. A similar approach can be performed for Highways 3:08, 44:00, 2:24, and 16:06 with similar portion in Class 5 with distinct differences around Class 9, 10 and 13.

Finally, Highway 44:00 with the most deviation from MEPDG TTC is selected for the sensitivity analysis. Table 5.13 presents the selected values for the sensitivity analysis. The MEPDG default values for TTC 11 is also presented in Table 5.14

Table 5.13 – Selected TTC as the extreme case for the performance sensitivity analysis

Location	Direction	Year	Vehicle/Truck Class Distribution (percent)									
			4	5	6	7	8	9	10	11	12	13
44:00	SB	2009	1.9	13.2	8.6	0.9	0.9	9.8	36.8	0.0	0.1	27.8

Table 5.14 – TTC 11 as MEPDG default for the performance sensitivity analysis

MEPDG	Vehicle/Truck Class Distribution (percent)									
	4	5	6	7	8	9	10	11	12	13
	1.8	24.6	7.6	0.5	5.0	31.3	9.8	0.8	3.3	15.3

5.3.2 HDF

Similar to TTC selection, the extreme value for HDF is chosen between representatives of extreme HDFs from each site location. In Section 5.2.2 the comparison was made between different HDFs in 2009 and 2010 and for each direction based on six WIM site locations. The representative HDF is selected for each location for the next filtration of HDF. Figure 5.15 shows these six representative HDFs along with the MEPDG default value.

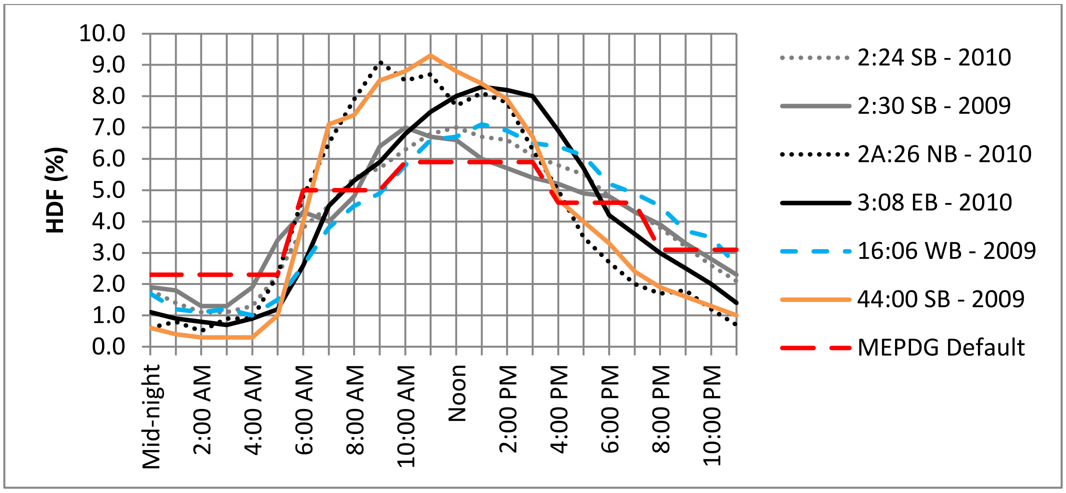

Figure 5.15 – HDF extremes as representatives of each WIM site location

As the figure shows, between 6 different locations, Highway 2A:26 and 44:00 have similar HDF trends in almost all the hours of the day. Similarly, Highways, 16:06 and 2:24 have identical trends in terms of HDF. Visually, it can be seen that Highway 44:00 and 2A:26 have the most deviations from the MEPDG default value (dashed red line) for HDF. Both in peak times and off peak times, the difference is visible from MEPDG. Between these two, Highway 44:00 is selected for the design analysis because of the reason that it has a slightly greater value for 11:00 AM at 9.2%. Table 5.15 provides information regarding the selected HDF for the sensitivity analysis based on AT's data. Moreover, Table 5.16 presents the default values of MEPDG for HDF which is used for sensitivity analysis.

Table 5.15 – Selected HDF as the extreme case for the performance sensitivity analysis

Location	Direction	Year	Time Interval							
			Mid-night	1:00 AM	2:00 AM	3:00 AM	4:00 AM	5:00 AM	6:00 AM	7:00 AM
44:00	SB	2009	0.6	0.4	0.3	0.3	0.3	1	4	7.1
			8:00 AM	9:00 AM	10:00 AM	11:00 AM	Noon	1:00 PM	2:00 PM	3:00 PM
			7.4	8.5	8.8	9.3	8.8	8.4	7.9	6.7
			4:00 PM	5:00 PM	6:00 PM	7:00 PM	8:00 PM	9:00 PM	10:00 PM	11:00 PM
			4.7	4	3.3	2.4	1.9	1.6	1.3	1

Table 5.16 – HDF as MEPDG default for the performance sensitivity analysis

	Time Interval											
	Mid-night	1:00 AM	2:00 AM	3:00 AM	4:00 AM	5:00 AM	6:00 AM	7:00 AM	8:00 AM	9:00 AM	10:00 AM	11:00 AM
MEPDG	2.3	2.3	2.3	2.3	2.3	2.3	5.0	5.0	5.0	5.0	5.9	5.9
	Noon	1:00 PM	2:00 PM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	7:00 PM	8:00 PM	9:00 PM	10:00 PM	11:00 PM
	5.9	5.9	5.9	5.9	4.6	4.6	4.6	4.6	3.1	3.1	3.1	3.1

5.3.3 MAF

Monthly adjustment factors were compared for six different locations and 10 FHWA classes in Section 5.2.3. In order to narrow down the number of selections, only Class 9 truck is considered for choosing the extreme values for MAF. For this purpose, similar to what was done for TTC and HDF in preceding section, representative trends for MAF are chosen for each site location. It means that for each site location, the most deviated MAF from MEPDG default value of 1 for all classes and all months is selected. In the next step, between these six representatives, the farthest MAF from MEPDG is selected for the performance analysis.

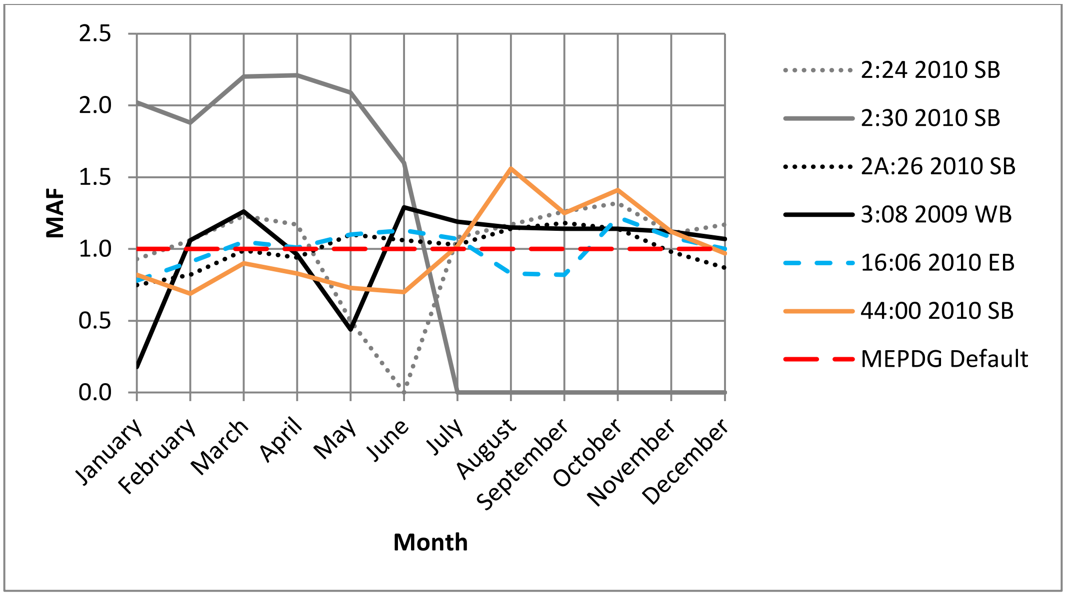

Figure 5.16 – MAF extremes as representatives of each WIM site location

Figure 5.16 provides information on the extreme MAF representatives of WIM site locations. For MAFs in six locations, Highways 2:24 and 2A:26, have obvious differences from MEPDG default value of 1 for all of the months Regarding the remaining trends. Highway 3:08 has also some interesting variations from MEPDG in January and May. Highway 2:30 with five months of MAF around 2 (January to May) and six months at 0 (July to December) is clearly a missing measurement. Finally the best choice for the extreme value of MAF is selected to be Highway 2:24. Table 5.17 presents the selected values of MAF for the sensitivity analysis. Table 5.18 presents MAF values for MEPDG defaults.

Table 5.17 – Selected MAF as the extreme case for the performance sensitivity analysis

Location	Direction	Year	Month (Class 9)					
2:24	SB	2010	Jan	Feb	Mar	Apr	May	Jun
			0.9	1.1	1.2	1.2	0.5	0
			Jul	Aug	Sep	Oct	Nov	Dec
			1.1	1.2	1.3	1.3	1.1	1.2

Table 5.18 – MAF as MEPDG default for the performance sensitivity analysis

MEPDG	Month (Class 9)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

5.3.4 Axle load distribution factor

Axle load distribution extreme trend is selected in this part. Similar to MAF, as the Class 9 truck is the most used truck in Alberta Highways and is considered to be the major means of transportation, this FHWA class is considered for the performance analysis. Additionally, Figures 5.2 to 5.7 proves that this class of truck has the highest proportion with respect to other classes.

In Section 5.2.4, the minimum and maximum deviations of axle load distribution factors from MEPDG default values were evaluated through calculating the production of different axle load group values and their linked frequencies. Table 5.9 to 5.12 in Section 5.2.4 gave above information for four FHWA classes of 5, 9, 10, and 13, respectively. Each table consisted of deviation levels for each location and also each axle type of single, tandem, and tridem.

Using Table 5.10, minimum and maximum deviations for each axle type can be derived between six locations. Tridem axle is not included in the analysis because of the fact that most of the distributions for this axle type did not have accepted values, based on the AT's data. Because of the nature of tridem axles, the frequency of these axle types is not high and they are not as common as single and tandem axles. This was concluded after looking at AT's WIM data for tridem axle distribution factors. The AT's data also proves this fact where a portion of AT's data recorded zero presence of this axle type in some months and in some locations such as Highway 2A:26 and 44:00. Additionally, for those with non-zero tridem axle types, include only one load group with a frequency of 100%. For single axle, Highway 44:00 shows a -40.6% deviation from the MEPDG default loads. Thus, this axle load distribution is selected to be used in the sensitivity analysis. For tandem axles, again, Highway 44:00 is selected as it shows a maximum absolute value at -38.64%.

Based on preceding discussions, the selected axle load distributions are selected and shown in the following. Table 5.19 and 5.20 present selected single axle and tandem axle load distribution – FHWA Class 9 – for the performance sensitivity analysis, respectively. Additionally, the linked MEPDG default values for FHWA Class 9 single and tandem axle load distribution values are presented in Tables 5.21 and 5.22, respectively.

Table 5.19 – Selected single axle load distribution for FHWA Class 9 as the extreme case for the sensitivity analysis

Location	Direction	Year	Load Group (lb)												
			3,000	**4,000**	**5,000**	**6,000**	**7,000**	**8,000**	**9,000**	**10,000**	**11,000**	**12,000**	**13,000**	**14,000**	**15,000**
44:00	**SB**	**2009**	1.77	3.13	22.60	34.60	27.51	7.52	2.35	0.37	0.05	0.05	0.05	0.00	0.00
			16,000	**17,000**	**18,000**	**19,000**	**20,000**	**21,000**	**22,000**	**23,000**	**24,000**	**25,000**	**26,000**	**27,000**	**28,000**
			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
			29,000	**30,000**	**31,000**	**32,000**	**33,000**	**34,000**	**35,000**	**36,000**	**37,000**	**38,000**	**39,000**	**40,000**	**41,000**
			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.20 – Selected tandem axle load distribution for FHWA Class 9 as the extreme case for the performance sensitivity analysis

Location	Direction	Year	Load Group (lb)												
			6,000	**8,000**	**10,000**	**12,000**	**14,000**	**16,000**	**18,000**	**20,000**	**22,000**	**24,000**	**26,000**	**28,000**	**30,000**
44:00	**SB**	**2009**	11.10	15.70	16.70	10.21	5.72	5.38	7.54	9.10	7.71	5.23	3.70	1.14	0.48
			32,000	**34,000**	**36,000**	**38,000**	**40,000**	**42,000**	**44,000**	**46,000**	**48,000**	**50,000**	**52,000**	**54,000**	**56,000**
			0.20	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
			58,000	**60,000**	**62,000**	**64,000**	**66,000**	**68,000**	**70,000**	**72,000**	**74,000**	**76,000**	**78,000**	**80,000**	**82,000**
			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.21 – Single axle load distribution for FHWA Class 9 as MEPDG default for the performance sensitivity analysis

	Load Group (lb)												
MEPDG	**3,000**	**4,000**	**5,000**	**6,000**	**7,000**	**8,000**	**9,000**	**10,000**	**11,000**	**12,000**	**13,000**	**14,000**	**15,000**
	1.74	1.37	2.84	3.53	4.93	8.43	13.67	17.68	16.71	11.57	6.09	3.52	1.91
	16,000	**17,000**	**18,000**	**19,000**	**20,000**	**21,000**	**22,000**	**23,000**	**24,000**	**25,000**	**26,000**	**27,000**	**28,000**
	1.55	1.10	0.88	0.73	0.53	0.38	0.25	0.17	0.13	0.08	0.06	0.04	0.03
	29,000	**30,000**	**31,000**	**32,000**	**33,000**	**34,000**	**35,000**	**36,000**	**37,000**	**38,000**	**39,000**	**40,000**	**41,000**
	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00

Table 5.22 – Tandem axle load distribution for FHWA Class 9 as MEPDG default for the performance sensitivity analysis

	Load Group (lb)												
MEPDG	**6,000**	**8,000**	**10,000**	**12,000**	**14,000**	**16,000**	**18,000**	**20,000**	**22,000**	**24,000**	**26,000**	**28,000**	**30,000**
	2.78	3.92	6.52	7.62	7.75	7.01	5.83	5.60	5.17	5.05	5.28	5.53	6.13
	32,000	**34,000**	**36,000**	**38,000**	**40,000**	**42,000**	**44,000**	**46,000**	**48,000**	**50,000**	**52,000**	**54,000**	**56,000**
	6.28	5.67	4.46	3.16	2.13	1.41	0.91	0.59	0.39	0.26	0.17	0.11	0.08
	58,000	**60,000**	**62,000**	**64,000**	**66,000**	**68,000**	**70,000**	**72,000**	**74,000**	**76,000**	**78,000**	**80,000**	**82,000**
	0.05	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.00	0.00	0.00	0.00

5.3.5 Sensitivity Analysis

Different traffic variables, based on AT's provided WIM data, are available to do the sensitivity analysis. The performance of the pavement is evaluated through checking major distress indicators predicted by the software: 1) Total rutting (cm), 2) IRI (m/km), and 3) Alligator cracking (%).

5.3.6 Design Inputs

As the majority of the WIM site locations are located in central Alberta, the weather station in Edmonton shown in Table 5.23 is selected for the analysis:

Table 5.23 – Weather station selected for the analysis

Name	Latitude	Longitude
EDMONTON, AB	53.317	-113.583

A three layer pavement with asphalt concrete (AC) as the top layer is selected based on the current practice of AT in Alberta. In the following section the structural characteristics of the chosen section is summarized:

- Top layer: Hot Mixed Asphalt (HMA)
  - Thickness = 150 mm
  - Aggregate gradation:
    - The MEPDG defaults.
  - Asphalt binder:
    - Conventional penetration grade = 120 -150
- Base Layer: Unbound material A-1-a
  - Thickness = 250 mm
  - Gradation and other properties: MEPDG defaults

- Sub grade Unbound material A-6
    - Thickness = infinite
    - Poisson ratio = 0.35
    - Gradation and other properties: MEPDG defaults

The MEPDG requires some general traffic information for the design. For this purpose, based on the AT's PMS report (4) and considering Highway 2:24, having the largest AADTs based on what was shown in Table 1.1, as the location of the analysis, the following inputs are used in the MEPDG:

- Initial two way AADTT = 4712
- Number of lanes per direction = 2
- Percent of trucks in design lane = 85%
- Percent of trucks in design direction = 50%
- Design year = 20
- Traffic growth -= compound (5%)

5.3.7 MEPDG Performance Predictions

The sensitivity analysis is performed by running the MEPDG software, based on above characteristics. The variables were discussed before with respect to what was done in Sections 5.3.1 to 5.3.4. A total of six runs of the software and their linked outputs form the final part of this chapter

For the first run, all of the four traffic variables are set as the MEPDG default. The outputs (predicted performances) for this run are used as the base for the sensitivity analysis. For the next five runs of the software, each time, one of the four traffic variables is changed from the MEPDG default to the extreme cases from the six WIM stations discussed in the previous sections. Each run of the MEPDG takes approximately 20 minutes on a typical desktop computer (4.00 GB installed memory). In the following sections, three predicted distresses for all five runs are presented in three separate figures for further discussions. The effect of each of the four traffic variables (TTC, HDF, MAF, single and tandem axle load distribution) on the predicted distresses are discussed.

Alligator cracking is the first predicted distress analyzed herein. The five extreme values are entered into the MEPDG separately and the predicted alligator cracking was derived for each run. Figure 5.17 shows that between the five different runs, only the run with the extreme TTC is deviated from other variables. It means that the influence of TTC extreme on the pavement performance is markedly higher than other variables. The maximum alligator cracking happens at the end of pavement's life at 22% for TTC. On the other hand, all other variables except for the tandem axle load distribution factor have the same maximum at 17.5%. This means that a difference of 25.7% between TTC and MEPDG exists. Changing the Tandem axle load to the extreme case will result in lower alligator cracking at a maximum of 13.5% at the end of pavement life. This is equivalent to -22.8% change from MEPDG default. The graph also shows that for a 20 year design, none of the trends reached the limit of 25% cracking at 90% reliability. Two other predicted performance indicators are also presented in Figures 5.18 and 5.19 to look at the level of influence of in interest variables.

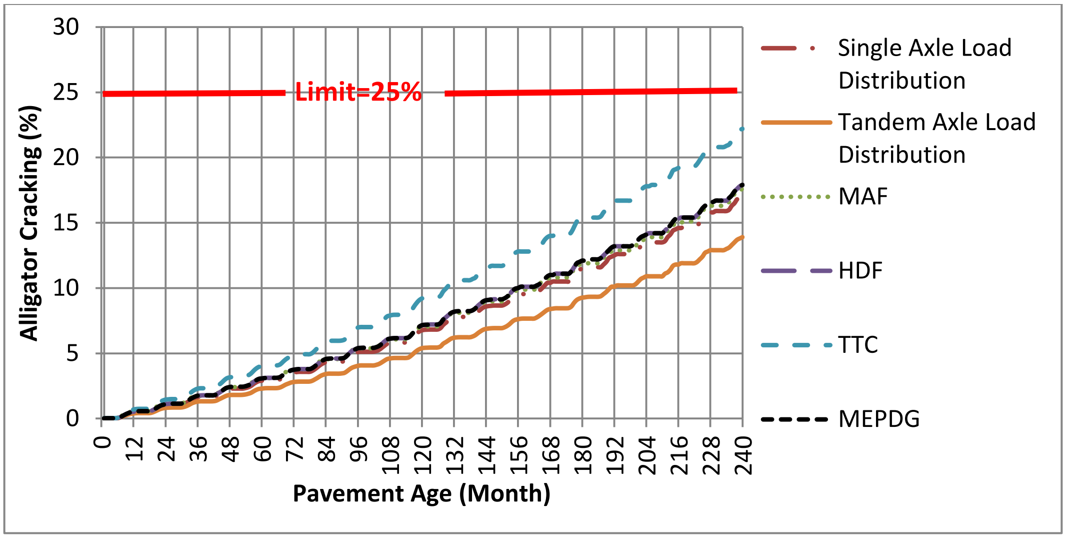

Figure 5.17 – Predicted performance by MEPDG – alligator cracking

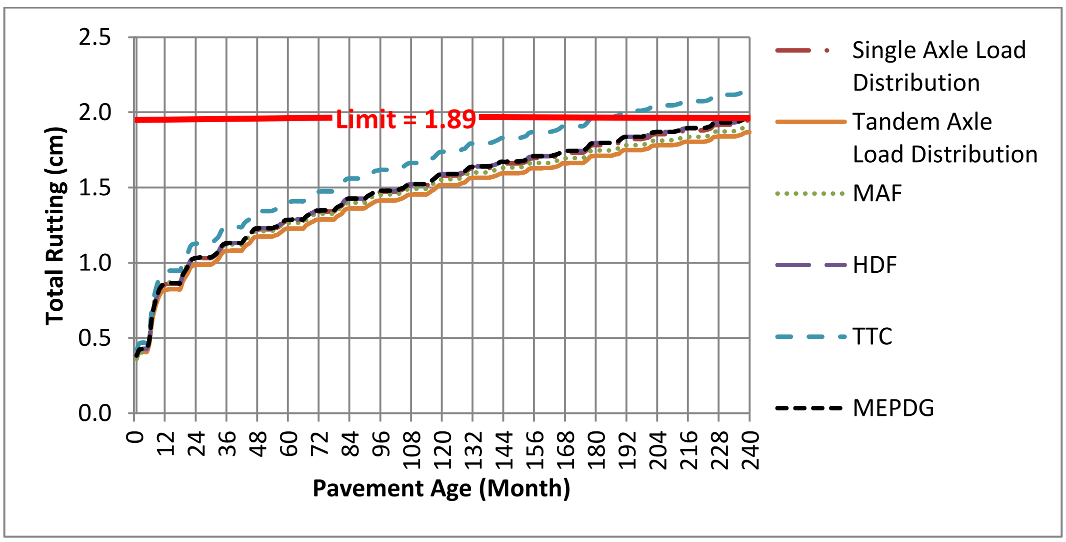

Figure 5.18 – Predicted performance by MEPDG – total rutting

Total rutting is the second output of the analysis which is predicted by MEPDG software and shown in Figure 5.18. It can be seen that over the 20-year pavement life, it is again the TTC that deviates markedly in terms of the influence of studied variables. Total rutting reaches a maximum of 2.2 cm for TTC at the end of the pavement's life where other variables, including MEPDG default values reach a maximum of 2.0 cm. The difference of TTC and MEPDG default values is calculated to be 10.3% at the end of design life.

The limit for the accepted total rutting is 1.89 cm. This limit is marginalized by all of the variables except for the TTC. TTC passes the limit at the age of 15 years. Other variables reach the limit at the last year of the design which is negligible.

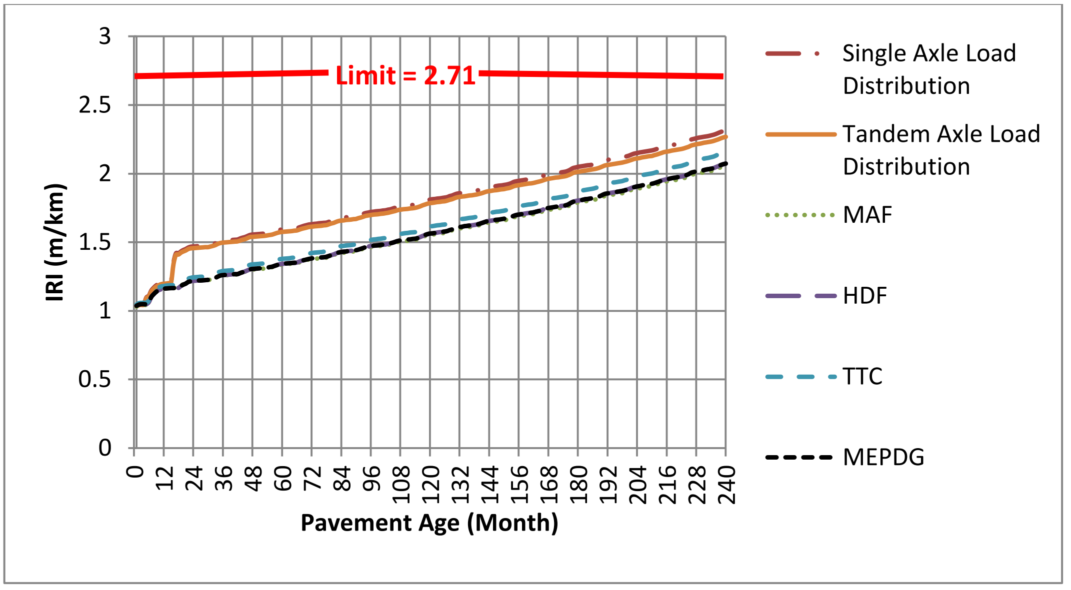

Figure 5.19 – Predicted performance by MEPDG - IRI

Figure 5.19 provides information on the predicted performance of the pavement predicted by the MEPDG software for IRI. Contradictory to what was derived for the preceding distresses, single axle load distribution and tandem axle load distribution present larger IRI values in comparison to other variables. These two axle load distribution factors play the role of an outlier where it deviates from the other variables at a maximum 2.25 m/km. The other runs with those variables reach a maximum of 2.17 m/km. The difference between axle load distributions and MEPDG default is 3.7%. However, decreasing the impact of axle load distribution factors (single and tandem) is supposed to decrease the distress level (similar to alligator cracking and total rutting), the IRI values show an increase in comparison to MEPDG default. The reason for this is the existence of transverse cracking in the predicted performance for these two runs. Transverse cracking influence the IRI number. It should be noted that none of the runs reach the limit of 2.71 m/km for IRI.

In summary, TTC is concluded to make the most influence on the pavement's distress levels for total rutting and alligator cracking. The extreme values for TTC, derived from AT's traffic data in 2009 and 2010 made the outputs to deviate from MEPDG default values as software's traffic inputs. Moreover, the axle load distribution is also concluded to have significant influence on the predicted performance level of the pavement where for IRI, due to transverse cracking, this parameter is showing the highest values at the end of pavement design life.

CHAPTER 6

6.0 Summary, Conclusions, and Future Work

6.1 Summary

Alberta Transportation (AT) installed WIM systems in six major highways in 2004. Piezoelectric WIM sensors were installed in a total of 20 lanes and started collecting data at the time. In order to verify the accuracy of the WIM sensors in Alberta, AT conducted a verification testing program. It includes passing a standard truck over the WIM sensors on a monthly basis for a period of 5 years (2005 to 2010). Conducting static weight measurements for this FHWA Class 9 truck, it is possible to verify the accuracy of the WIM records. A total of 39,126 dynamic weight data points with their respective static weight measurements are available for analysis. Statistical analyses were performed on the errors in the weight records. Additionally, the effect of these errors on the pavement thickness design was investigated.

In chapter 5, the WIM systems in Alberta were evaluated by analyzing 2 years of real-life traffic data (2009 and 2010). As the WIM measurements is the only source for defining the traffic inputs in the Mechanistic Empirical Pavement Design Guide (MEPDG), the data is compared to their default counterparts in the software. A sensitivity analysis was performed to establish the effects of different characteristics of the AT's WIM-based data on the MEPDG-predicted pavement performances. Distress indicators such as IRI, alligator cracking, and rutting were used to in the sensitivity analysis. In the following the conclusions of this study is discussed in detail:

6.2 Conclusions

From the first part of the study (Chapter 3 and 4) which was about the WIM verification testing program, the followings are concluded:

- A total maximum of 173% and minimum of -75% was calculated for WIM weight errors. The average of all errors was 0.5%.
- In terms of the 95.0% compliancy requirement of ASTM E1318 for WIM errors, 77% of single steering, 77% of tandem drive, and 80.2% of tandem load axles fell into the required level. Generally, none of the weight parameters satisfied the code's requirement.
- Three different statistical distributions for the WIM weight errors were derived by fitting the errors into quantile-quantile curves. For errors less than +50%, the normal distribution was fitted. For errors between +50% and +120%, the lognormal distribution was the best curve while a Laplace distribution was formed for the errors more than 120%.
- In order to evaluate the effects of WIM weight errors on the pavement thickness design, four scenarios were considered. For all errors accepted scenario, an extra asphalt layer of 12-21 mm was calculated to be in place because of WIM inaccuracies in collecting traffic data. For the last one, where ASTM E1318 requirements are met, no significant extra asphalt layer is introduced.

For the second part of this thesis (Chapter 5), the following results were found:

- HDF comparisons revealed that for almost all of the WIM site locations, the HDFs from WIM matches with its counterpart in MEPDG. MAF comparisons showed that for six WIM site locations, deviations can be seen between 2009 and 2010 data. January showed the highest frequency of maximum difference occurrence between 2009 and 2010 MAFs. The differences of axle load distribution factors for four FHWA Classes of 5,9,10, and 13 between MEPDG and WIM were calculated to be significant.
- The effects of these deviations, for four parameters of TTC, HDF, MAF, and axle load distribution factor from MEPDG default counterparts were analyzed using the predicted performance indicators of MEPDG software's outputs. The sensitivity analysis were done on the extreme values of above parameters from AT's WIM data. The results were presented for three distress indicators of alligator cracking, total rutting and IRI.
- After running the MEPDG software and getting the results, sensitivity analysis of the WIM-based traffic data revealed that it was the TTC and axle load distribution factor that made the results to deviate markedly. Other parameters such as HDF and MAF did not have significant influence on the distress analysis.

6.3 Future Works

For future works it is recommended to perform a study on the calibration procedure of WIM systems in Alberta in order to look at the influence of calibration process on the generated weight errors.

Different variables affect the accuracy of WIM systems. Performing a study on the variables that influence the WIM systems and applicable solutions to remove the sources of these errors is also recommended.

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Appendix A

Example of AT WIM verification results:

Weigh in Motion Calibration Verification Field Sheet

Location	Highway	CS	km	Dir	Lane	Data Recorded by CornerStone Solutions Inc.

				Date	17-Jan-2006					Date	17-Jan-2006		
Pass	**1**			**Time**	16:09:03					**Time**	16:09:14		
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**			**Accept = 1 Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9			1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	108	km/h		1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5100	kg		1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	14700	kg		1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15700	kg		1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	35500	kg		1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.03	m		1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.36	m		1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.04	m		1

10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.80	m	1

			Date	17-Jan-2006					**Date**	17-Jan-2006		
Pass	**2**		**Time**	16:22:30					**Time**	16:22:41		
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1** **Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5300	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	14000	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	14700	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34000	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.04	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.38	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.09	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1

Pass **3** | **Date** 17-Jan-2006 | **Time** 18:01:50 | **Date** 17-Jan-2006 | **Time** 18:02:01

	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1 Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	108	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	4900	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	12700	kg	0
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15100	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	32700	kg	0
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	3.98	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.35	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	8.96	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.78	m	1

Pass **4** | **Date** 17-Jan-2006 | **Time** 19:09:49 | **Date** 17-Jan-2006 | **Time** 19:10:00

Accept = 1

	Measure	Standard		CSI		Min		Max		WIM		Reject = 0
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5300	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	14000	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15300	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34600	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.06	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.38	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.08	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1
			Date	17-Jan-2006					**Date**	17-Jan-2006		
Pass	**5**		**Time**	19:23:31					**Time**	19:23:41		
												Accept = 1
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1

2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5300	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	13800	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15300	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34400	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.06	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.37	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.09	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1

		Date 17-Jan-2006	**Date** 17-Jan-2006
Pass	**6**	**Time** 19:39:46	**Time** 19:39:56

	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1** **Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	108	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5200	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	14000	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15100	kg	1

6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34300	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	3.98	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.36	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	8.94	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.80	m	1
			Date	17-Jan-2006					**Date**	17-Jan-2006		
Pass	**7**		**Time**	19:52:19					**Time**	19:52:30		
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1** **Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5300	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	14200	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15400	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34900	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.06	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.37	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.09	m	1

10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1

			Date	17-Jan-2006					**Date**	17-Jan-2006		
Pass	**8**		**Time**	20:05:21					**Time**	20:05:32		
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1 Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5700	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	15000	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	16300	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	37000	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.06	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.38	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.09	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1

			Date	17-Jan-2006					**Date**	17-Jan-2006		

Pass	9			Time	20:18:18					Time	20:18:28		
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1 Reject = 0**
1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5500	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	13800	kg	1
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15300	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34600	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.06	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.37	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.09	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.81	m	1

				Date	17-Jan-2006					**Date**	17-Jan-2006	
Pass	**10**			**Time**	20:31:14					**Time**	20:31:24	
	Measure	**Standard**		**CSI**		**Min**		**Max**		**WIM**		**Accept = 1 Reject = 0**

1	Configuration	FHWA 9		FHWA 9		FHWA 9		FHWA 9		FHWA 9		1
2	Speed	+/- 5 %	km/h	109	km/h	103.55	km/h	114.45	km/h	109	km/h	1
3	Single Steer Axle Weight	+/- 20 %	kg	5260	kg	4208	kg	6312	kg	5200	kg	1
4	Tandem Drive Axles Weight	+/- 15 %	kg	16150	kg	13728	kg	18573	kg	13700	kg	0
5	Tandem Load Axles Weight	+/- 15 %	kg	16070	kg	13660	kg	18481	kg	15200	kg	1
6	Gross Vehicle Weight	+/- 10 %	kg	37480	kg	33732	kg	41228	kg	34100	kg	1
7	Distance Axle 1 to Axle 2	+/- 10 %	m	4.00	m	3.60	m	4.40	m	4.04	m	1
8	Distance Axle 2 to Axle 3	+/- 10 %	m	1.40	m	1.26	m	1.54	m	1.38	m	1
9	Distance Axle 3 to Axle 4	+/- 10 %	m	9.10	m	8.19	m	10.01	m	9.08	m	1
10	Distance Axle 4 to Axle 5	+/- 10 %	m	1.80	m	1.62	m	1.98	m	1.83	m	1

Appendix B

- Example of AT WIM real-life traffic data - Highway 16:06 Eastbound Right (2009)
    - Single Axle Load Distribution Factor:

ID#	Season	Class	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
6	**January**	**9**	3.08	4.01	3.27	1.56	3.97	7.33	22.70	22.48	20.40	6.98	2.75	0.82	0.35	0.13	0.02
16	**February**	**9**	2.32	2.34	1.72	1.15	3.61	7.05	22.84	25.30	21.00	7.92	3.09	0.90	0.40	0.20	0.05
26	**March**	**9**	2.59	2.31	1.62	1.30	4.21	6.80	23.50	25.07	21.62	6.33	2.87	0.89	0.38	0.35	0.07
36	**April**	**9**	1.76	1.35	1.27	1.25	3.90	7.63	21.27	24.09	23.21	8.44	4.19	1.11	0.26	0.17	0.05
46	**May**	**9**	2.18	2.05	1.06	0.90	3.55	9.30	22.52	23.71	21.74	7.05	3.46	1.26	0.71	0.29	0.07
56	**June**	**9**	1.53	1.32	1.46	1.25	3.89	8.43	20.80	20.07	16.49	8.74	6.55	3.89	2.71	2.12	0.52
66	**July**	**9**	1.83	1.78	1.19	1.39	3.89	9.83	24.35	21.01	19.41	8.12	4.34	1.83	0.46	0.39	0.09
76	**August**	**9**	2.04	1.72	1.29	1.81	5.49	10.41	26.28	23.76	17.51	5.19	2.59	0.93	0.29	0.41	0.23
86	**September**	**9**	2.22	1.72	1.37	1.98	5.23	10.93	25.17	23.73	17.56	6.24	2.51	0.63	0.28	0.39	0.00
96	**October**	**9**	2.66	1.63	1.65	1.52	5.58	10.70	24.32	23.72	18.45	6.50	2.22	0.70	0.15	0.11	0.00
106	**November**	**9**	2.99	1.72	1.36	1.26	5.24	10.83	25.41	22.29	17.94	6.88	2.81	0.83	0.16	0.16	0.02
116	**December**	**9**	4.75	3.87	2.84	1.73	4.57	9.22	24.69	22.16	16.46	5.86	2.45	0.83	0.23	0.18	0.08

16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
0.09	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.02	0.05	0.00	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.02	0.05	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.07	0.04	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.12	0.07	0.00	0.02	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.02	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.07	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.05	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.03	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

- Hourly Distribution Factor (HDF):

Midnight	2.1	**Noon**	6.5
1:00 AM	1.6	**1:00 PM**	6.4
2:00 AM	1.3	**2:00 PM**	6.4
3:00 AM	1.4	**3:00 PM**	6.2
4:00 AM	1.4	**4:00 PM**	6.0
5:00 AM	1.8	**5:00 PM**	5.5
6:00 AM	2.9	**6:00 PM**	5.1
7:00 AM	4.3	**7:00 PM**	4.7
8:00 AM	5.1	**8:00 PM**	4.2
9:00 AM	5.8	**9:00 PM**	3.6
10:00 AM	6.0	**10:00 PM**	3.0
11:00 AM	6.1	**11:00 PM**	2.6

- Truck Traffic Classification (TTC):

Vehicle Class	Distribution
4	2.6
5	20.6
6	2.9
7	1.2
8	0.6
9	18.8
10	22.6
11	0.0
12	0.3
13	30.4

- Monthly Adjustment Factor (MAF):

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.2	0.7	1.2	1.2	1.0	1.1	1.2	6.0	1.2	1.2
February	1.2	0.6	1.2	1.1	0.9	0.9	1.2	0.0	0.9	1.1
March	1.2	0.7	1.1	1.3	0.9	1.0	1.2	0.0	1.1	1.1
April	0.8	0.7	0.8	0.6	0.8	1.0	0.8	0.0	0.9	0.9
May	0.8	1.0	0.8	0.6	0.8	1.1	0.8	0.0	1.0	0.9
June	1.0	1.5	1.1	1.0	1.1	1.0	1.0	0.0	0.7	0.9
July	0.9	1.6	1.0	1.0	1.2	1.0	0.9	0.0	1.1	1.0
August	0.9	1.9	0.9	0.9	1.2	1.0	0.9	0.0	1.0	1.0
September	0.9	1.1	0.9	0.9	1.2	1.1	1.0	6.0	1.0	1.0
October	1.0	0.8	1.0	1.1	1.0	1.1	1.1	0.0	1.2	1.0
November	1.0	0.6	1.0	1.1	1.0	1.0	1.1	0.0	1.0	1.0
December	1.0	0.8	1.1	1.1	0.9	0.9	1.0	0.0	0.8	1.0