University of Alberta

Performance Evaluation of Flexible Pavements in Alberta Using Falling Weight Deflectometer Data

by

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A thesis submitted to the Faculty of Graduate Studies and Research

in partial fulfillment of the requirements for the degree of

Master of Science

in

Transportation Engineering

Department of Civil and Environmental Engineering

Spring 2013

Edmonton, Alberta

To my parents for their full support and love in my whole life.

Abstract

The highway network of Alberta is the major component of its infrastructure system which is difficult to be maintained. The first part of this study focuses on seasonal variation of pavement layers' stiffness and its influence on the pavement life. Pavement structures are supposed to be in weakest condition during thawing period in early spring. The results shows that damage imposed to pavement during thawing period can be approximately 90% more than during the recovering period. The second part of the study centers on comparing the performance of different asphalt overlay strategies in the Province of Alberta. For this purpose, the data collected under Long Term Pavement Performance in Alberta Specific Pavement Studies 5 section was used. In addition, Mechanistic Empirical Pavement Design Guide software was used to simulate different overlay strategies. The results indicate that overlay thickness is the most influencing factor on the performance of the overlaid sections.

Acknowledgment

I would like to express my sincere gratitude to my supervisor Dr. Alireza Bayat for his continuous support during my M.Sc. study and research. I wish to thank Dr. Somayeh Nassiri for her valuable guidance, without which the completion of this thesis would not be possible. My gratitude also extends to Dr. Hamid R. Soleymani for all his help. I would also like to thank Ms. Tejay Gardiner and Mr. Mohammad H. Shafiee for their help on editorial and technical review of this thesis. I would also like to thank my dear friends Mr. Ali Hosseini and Mohammad Sharifi for being there for me all the times. My last but not least thanks goes to my dear parents for their continuous support during my studies.

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List of Symbols

AASHTO:	American Association of State Highways and Transportation Officials
AC:	Asphalt Concrete
ASTM:	American Society for Testing and Materials
AT:	Alberta Transportation
C-LTPP:	Canadian Long Term Pavement Performance
C-SHRP:	Canadian Strategic Highway Research program
DOT:	Department of Transportation
FHWA:	Federal Highway Administration
FWD:	Falling Weight Deflectometer
GBC:	Granular Base Course
GPS:	General Pavement Studies
IRI:	International Roughness Index
LTPP:	Long Term Pavement Performance
MEPDG:	Mechanistic Empirical Pavement Design Guide
NCHRP:	National Cooperative Highway Research Program
NDT:	Non-Destructive Testing
NWP:	Non-Wheel Path
PCC:	Portland Cement Concrete
PMS:	Pavement Management System
RAP:	Reclaimed Asphalt Pavement
RMSE:	Root Mean Square Error

SDDOT:	South Dakota Department of Transportation
SHRP:	Strategic Highway Research program
SLR:	Spring Load Restriction
SPS:	Specific Pavement Studies
SRB:	Spring Road Ban
SS:	Subgrade Soil
TB:	Treated Base
WSDOT:	Washington State Department of Transportation

1. Chapter 1 Introduction

1.1 Background

A large amount of capital is spent each year in Canada to keep the existing transportation infrastructure in acceptable ride conditions. Based on the latest assessment report regarding Canada's infrastructure system, the gross stock of Canada's public infrastructure was estimated to be $286.2 billion in 2007 (Gangon et al. 2011). The average age of these assets was reported 16.3 years in 2007. Highways and roads are the largest component of Canada's infrastructure with a gross value of $170 billion in 2007, which is equal to approximately 60 percent of the value of entire infrastructure system in Canada. The average age of highways and roads in Canada continuously increased from the 1970's up to 1994, during which average age of highways and roads reached its peak of 16.9 years. However, this average age decreased to 14.9 years in 2007 (Gangon et al., 2011)

The Province of Alberta owned 12 percent of the entire national infrastructure with a total value of $35.2 billion in 2007. At the time, with an average age of 15.6 years, it was the third youngest infrastructure system in the country. Highways and roads make up approximately 62 percent of Alberta's total infrastructure, with a value of $21.8 billion.

To maintain this valuable infrastructure, Alberta Transportation dedicates nearly 50 percent of its annual budget to rehabilitate and maintain the existing highway network. (Soleymani et al., 2002). Alberta is known for its cold, dry climate, with Freezing Index as high as 2700˚C.days, based on the historical temperature data

(1997-2008) extracted from Environment Canada (Environment Canada, accessed April 2012). Winter temperatures can drop to -50˚C, while summer temperatures can reach 40˚C (Watson et al., 2000). Pavements located in such climate conditions are prone to excessive damage, since they undergo frost penetration in the winter and severe temperature variations in the summer, as well as numerous cycles of freezing and thawing.

Repetitive cycles of freezing and thawing weaken the pavement structure. In winter, resilient moduli of the pavement's unbound layers increase due to ice-bonding between the soil particles in the base and sub-base layers and the formation of ice lenses in the subgrade (Simonsen et al., 1999). The pavement structure starts to weaken during the thawing season in early spring. As a result, the moisture in the unbound pavement layer increases in this period resulting in drastic decrease in unbound layers' moduli. Pavement structure is in its weakest condition during this period and there is high potential for damage due to the application of heavy truck loads (Watson et al., 2000). This phenomenon raises the necessity to enforce preventive regulations to mitigate excessive damage to the pavement during this critical period. Many highway agencies across the world use preventive methods to minimize the damages imposed on the pavement during thawing season. One preventative measure is to restrict the maximum allowable loads. Such methods, used frequently for secondary roads, are known as the Spring Road Ban (SRB) or Spring Load Restriction (SLR).

The difficulty with imposing SRB's or SLR's is that the trucking industry requires year round access. Therefore, SRB and SLR periods should be optimized to consider highway jurisdictions' concerns, together with the trucking and transporting needs of the community.

One of the methods that can be used to implement SRB is using pre-determined dates to apply and remove SRB based on historical data and local experience. The other methods are engineering judgment and visual observation. "Visual observations can include water seeping through cracks in the pavement from the subsurface layers as traffic loads are applied, rapid deterioration of the surface layer, and soft shoulders" (Ovik et al., 2000).

Recently, mechanistic approaches, which strongly rely on mechanical properties of the pavement structure, such as stiffness of each layer, are implemented in developing SRB instead of empirical and judgment based methods (Watson et al., 2000 and Ovik et al., 2000).

Alberta Transportation implements a numerical heat transfer model to predict the start of the thawing, along with a series of Falling Weight Deflectometer (FWD) tests, which assist with deciding the duration of the SRB for different roadways (C-SHRP Technical brief, 1999).

Another aspect of pavement management concerns maintenance and rehabilitation. Pavement structure loses structural integrity as it ages and hence is more prone to damage. Also, the manifestation of cracks at the surface makes it easier for moisture to infiltrate into the unbound layers. Excessive moisture can result in earlier deterioration of the pavement structure. Based on the latest Alberta Transportation's Pavement Management System (PMS) report in 2011, 56 percent of highways in Alberta are older than 10 years (Alberta Transportation, 2011). Pavement Sections older than 10 years are naturally more susceptible to damage (Al-Suleiman et al. 1991) and need more extensive treatments, such as resurfacing or rehabilitation within a short period of time. A thorough understanding of the most effective rehabilitation strategies in Alberta's climate condition is prominent. The need to investigate various rehabilitation strategies was foreseen in the Long-Term Pavement Performance (LTPP) program Specific Pavement Studies (SPS) 5. A total of 18 SPS 5 sections are scattered in North America (West et al., 2011), of which one is located in Alberta. The data for SPS 5 in Alberta recorded for 16 years between 1990 and 2006. This database is a valuable source of information to investigate effectiveness of different overlay strategies.

1.2 Research Impetus

As mentioned in the previous section, one of the issues that Alberta network is faced with is the effect of severe freeze-thaw on the pavement structure. Accumulated moisture in the unbound layers decreases the overall pavement's moduli resulting in vulnerability to extensive damage to the pavement. AASHTO 1993 Design Guide takes seasonal variation into account in its design procedure. Minimum subgrade modulus is assumed to occur in March (17MPa) and then gradually recovers to 27MPa in April and May. It is assumed to be 48MPa from June to October during the recovering period and the maximum subgrade modulus happens in December, January and February with a modulus of 137MPa (AASHTO, 1993). The drop in subgrade modulus from February to March can cause significant damages to the pavement structure if no preventive strategy is employed.

Therefore establishing preventive strategies to minimize the damage imposed to the pavement during this period seems to be highly required. The first step to establish such strategies is to investigate the magnitude of the problem in case of the province of Alberta. Alberta Transportation performs a series of FWD tests during the thawing and recovering period every year to obtain an understanding of the pavement stiffness during the thawing season which can be a valuable source of information. To date, limited studies focused on using Alberta Transportation's FWD database to investigate seasonal variation of pavement layers' stiffness. McMillan et al. (2010) used the FWD data from only three highway sections in Alberta. Their study focused on seasonal variations of pavement layers' moduli as well as investigation of the FWD test repeatability. In their study Alberta historical FWD database was not used. Instead, they performed special FWD testing which consisted of two tests in specific locations every month during thawing and recovering period. The fact that no comprehensive study has been done to use FWD database in order to investigate seasonal variations in pavement layers' stiffness necessitates performing a study using the FWD database to investigate seasonal variation in pavement layers' characteristics as investigation and understanding of the pavement's seasonal stiffness is a key factor in developing efficient methods to minimize the imposed damage.

Another issue that is currently a major concern regarding the highway network of Alberta is the average age of the network. As mentioned previously, 56 percent of the entire highway network of Alberta is older than 10 years. In addition, 50 percent of the entire network is already overlaid.

Alberta LTPP SPS 5 section's data is available in the LTPP database providing the opportunity to study the performance of different overlay strategies in the province. Carvalho et al. (2011) included Alberta SPS 5 in their study of LTPP sections in North America and compared distresses between different sections; but they did not look into the Alberta SPS 5 sections in depth. The effect of different overlay strategies on the structural response of the pavement still needs to be investigated further for Alberta climate conditions. Combining structural and functional analysis of the Alberta SPS 5 sections can provide guidance for the future rehabilitation projects in Alberta. This makes it crucial and beneficial to study the precious database created over 16 years to investigate the performance of the different overlays

1.3 Research Objectives

The primary objective of this study is to investigate the best strategies to maintain Alberta's highway network. For this purpose, the study focuses on the following primary objectives:

- Investigating the behavior of the pavement structure in the early thawing and comparing it to the recovering performance to assess the difference between thawing and recovering period.

- Evaluating the effect of different overlay strategies on the long-term performance of the pavement.

The findings from the study will provide a better understanding of pavement performance throughout the year (e.g. thawing and recovering period) and also for the long-term. This improves the ability to make more accurate decisions regarding the spring road ban practice and pavement maintenance and rehabilitation.

1.4 Research Overview and Methodology

The first portion of the current study (Chapter 3) is dedicated to investigating the behavior of different Alberta highways during the critical period of thawing. In doing so, the FWD test data collected during the thawing season for 11 years between 1997 and 2008 was used. The FWD data was used to backcalculate the pavement layers' moduli using Evercalc© developed at Washington State Department of Transportation (WSDOT). Backcalculated moduli were then used as the inputs in Everstress© to predict the pavement's structural responses. The tensile strain at the bottom of the Asphalt Concrete (AC) layer and the vertical strain at the top of the subgrade are the most commonly used parameters as the pavement mechanistic responses (Rutherford et al., 1985). These pavement responses can be used as indicators of the damage imposed to the pavement. Several equations are available that can be used to predict the allowable load repetition using the pavement responses. The pavement structural responses predicted using Everstress© were plugged into the equations developed by Asphalt Institute (Huang, 1993) and US Army Corps of Engineers (Wardle et al.,

2003) to predict the remaining allowable load repetition that the pavement can undergo before failure. Comparing the values predicted in recovering period to values in thawing period, demonstrates the effect of the spring thaw weakening on the pavement behavior. In addition, the effect of load reduction on the pavement responses and remaining allowable load repetition on the pavement was investigated by applying different load levels on the simulated pavement in Everstress©.

The other portion of the study (Chapter 4) focuses on evaluating different overlay strategies to assess effectiveness and long term performance. Alberta LTPP SPS 5 section's database, which is publicly available online and in the DVD, was used in the study. First, the structural response of each overlay strategy was compared to each other. A series of FWD tests were performed on this section on a regular basis. The peak deflection underneath the FWD load was used as the pavement's structural response indicator. The average peak deflection for each overlay section was compared to the other sections' peak deflections using a statistical paired t-test to investigate if the difference between the sections was significant. Second, pavement distresses in each overlay section were compared to the other sections to assess the effect of overlay type on the pavement distresses. Based on the available data, alligator, transverse and longitudinal cracks as well as international roughness index (IRI) were used to investigate performance of different overlaid sections. Moreover, the sections were simulated using the Mechanistic Empirical Pavement Design Guide (MEPDG) Version 1.1 to compare the predictions of the MEPDG with the field distress data collected for the SPS 5 during the pavement monitoring period.

1.5 Thesis Structure

This study is presented in the following organization.

- Chapter 1: Introduction- Provides a background on the area of research for the study such as backcalculation of FWD tests, seasonal variation of pavement layers' strength, Spring Road Ban practice and pavement maintenance and rehabilitation, as well as the problem statement and research methodology
- Chapter 2: Literature Review- Provides a brief discussion about the FWD test procedure and the methods used to analyze the FWD data. Next, the backcalculation process and the available software packages frequently used to backcalculate the pavement layers' moduli are described. This chapter also provides a discussion on seasonal variation of pavement strength and SRB practices around the world. Pavement maintenance and rehabilitation and pavement distresses are also discussed in this chapter. Lastly, this chapter focuses on reviewing the findings of other researchers regarding the LTPP SPS 5 experiment.
- Chapter 3: Evaluation of Seasonal Variation in Pavement Mechanistic Responses Using Falling Weight Deflectometer Data. Alberta Transportation's historical FWD database is used to evaluate the structural response of pavements in Alberta in the early spring by backcalculating FWD data. Responses in thawing period are then compared to the

responses in recovering period to investigate the significance of the thaw weakening period on pavement structural responses. The effect of load reduction level on the remaining allowable load repetition on the pavement is also studied in the final part of this chapter.

- Chapter 4: Performance Evaluation of LTPP SPS 5 Sections in Alberta – This chapter focuses on evaluation of performance of different overlay strategies in Alberta's LTPP SPS 5 section. In the first part of this chapter, peak deflections under the FWD are used as the pavement's structural response indicator. Peak deflection in all section is compared to the other sections to investigate the effect of different overlays on the peak deflection. In the second part of the chapter, distresses such as alligator, transverse and longitudinal cracks and also IRI and rutting developed in each overlay section are compared to the other sections to assess the effect of different overlays.

- Chapter 5: Conclusions- this chapter summarizes the findings of the studies in Chapters 3 and 4. Recommendations for future research are also provided in this chapter.

2. Chapter 2: Literature Review

2.1 Falling Weight Deflectometer Equipment

Non-destructive testing (NDT) method, used to evaluate in-service pavement's performance, has long been used to assess pavement conditions (Huang, 1993). The most common NDT methods are deflection testing, wave propagation, impact hammer, ground-penetration radar and impedance devices (Huang, 1993). The Benkelman beam, California travelling deflectometer and LaCroix deflectometer are conventional devices used to measure pavement deflection (Huang, 1993). In FWD testing devices were introduced to the pavement industry in the 1980s to evaluate the structural capacity of the pavement. FWD devices measure the deflections under a dynamic load by simulating the passing of one wheel load. The FWD measurements can be used to evaluate the pavement's structural response for design, rehabilitation, and pavement management purposes (NCHRP synthesis 381, 2004). According to the American Association of State Highway and Transportation Officials (AASHTO) 1993 Design Guide, deflection testing is recommended to evaluate the effective structural capacity and also determining the seasonal variation of pavement layers' strength (Von Quintus et al., 1997).

A typical FWD includes the following components, according to ASTM D4694-96: Standard Guide for General Pavement Deflection Measurements:

- An impulse-generating device with a guide system. This device allows a variable weight to be dropped from a variable height.

- A loading plate, for uniform force distribution on the test layer. When the weight affects this plate, this loading plate ensures that the resulting force is applied perpendicularly to the test layer's surface.
- A load cell for measuring the actual applied impulse.
- One or more deflection sensors.
- A system for collecting, processing, and storing deflection data. (ASTM D 4694, 1996).

As mentioned, FWD devices are capable of generating different load levels by changing the drop height. Typical load levels used in common FWD testing are 27, 40, 53 and 71 kN (NCHRP synthesis 381, 2004). The FWD devices are normally equipped with five, seven or nine deflection sensors at different offsets from the load plate to measure the deflections under the applied load. Terms deflection sensors and geophones are used interchangeably in FWD testing devices (NCHRP Synthesis 381, 2004). Geophones are electronic devices, which can measure the relative vertical movement of the pavement. "Such devices may include seismometers, velocity transducers or accelerometers" (NCHRP Synthesis 381, 2004). Load cell is used to accurately measure the applied load and load plate is used to evenly distribute the applied load to pavement surface (NCHRP Synthesis 381, 2004).

2.2 Backcalculation Process

Backcalculation of the layers' elastic moduli from the deflection basin obtained from the FWD test is a popular approach to evaluate the whole pavement's structural capacity, as well as each individual layer's modulus. This knowledge assists with making better decisions for rehabilitation and design purposes (Von Quintus et al., 1997). There are various backcalculation procedures used to estimate the pavement layers' moduli based on FWD deflection data. While all methods are based on minimizing the error between the predicted and measured deflections, they can be separated into different categories based on the type of load analysis (static or dynamic) and the layers' material type analysis (linear or non-linear) (Uzan, 1994). Accordingly, the backcalculation procedures can be categorized into the following four general groups:

- Static, linear backcalculation
- Static, non-linear backcalculation
- Dynamic, linear backcalculation
- Dynamic, non-linear backcalculation

Of the four categories, the simplest method is the static load applied to a linear material. The challenge is to find a series of moduli which will result in pavement deflections corresponding to the measured deflections. Each load level is analyzed separately and different sets of moduli are obtained for each load level (Uzan, 1994).

In contrast to static linear backcalculations, static non-linear backcalculations require load levels to be analyzed together. Finding a set of moduli that corresponds to the measured deflections at all load levels can be challenging. "Dynamic backcalculation applies to the NDT equipment that applies either a steady state vibratory load or an impact load" (Uzan, 1994). In the case of a steady-state vibratory load with limited number of frequencies, the challenge is to find a set of complex moduli which will result in pavement deflections corresponding to the measured ones. In the case of applying an impact load, two alternatives can be implemented: 1) using frequency domain fitting, and 2) using time domain fitting. (Further descriptions of these alternatives can be found in the references such as [Uzan, 1994]). Lastly, dynamic non-linear backcalculation is a combination of dynamic linear and static non-linear backcalculation methods. Calculated non-linear complex moduli of the pavement layers should predict time history deflection responses close to the measured one for all load levels simultaneously (Uzan, 1994).

2.2.1. Backcalculation Techniques

The four concepts described in Section 2.2 can be applied using three different backcalculation techniques. One technique is the Iterative approach in which a forward calculation is used in each step of iteration until the error criteria is satisfied. The second technique is Theoretical Equations which uses direct equations to calculate pavement layers' moduli (Lytton, 1989). A third approach is Database Search, which uses forward calculation to build a database, which will either help formulate regression equations to determine the layer moduli or use database to interpolate the deflections in order to avoid forward calculation (Ameri et al., 2009). The Iterative and Equivalent Layers Modulus are the most commonly used techniques and will be expanded upon in the following sections.

2.2.1.1 Iterative Approach

Iterative backcalculation includes a series of steps to estimate the pavement layers' moduli. Figure 2.1 describes the backcalculation process (Lytton, 1989). As seen in Figure 2.1, the first step is to calculate a deflection basin based on the user-supplied seed moduli, load and pavement structure (e.g. layer thickness) and compare the predicted deflections to the measured ones. The goal is to minimize the differences between these two. The objective is achieved in repetitive iterations and changing the moduli values of each layer in each iteration to satisfy a convergence criteria (ASTM D5858, 2008).

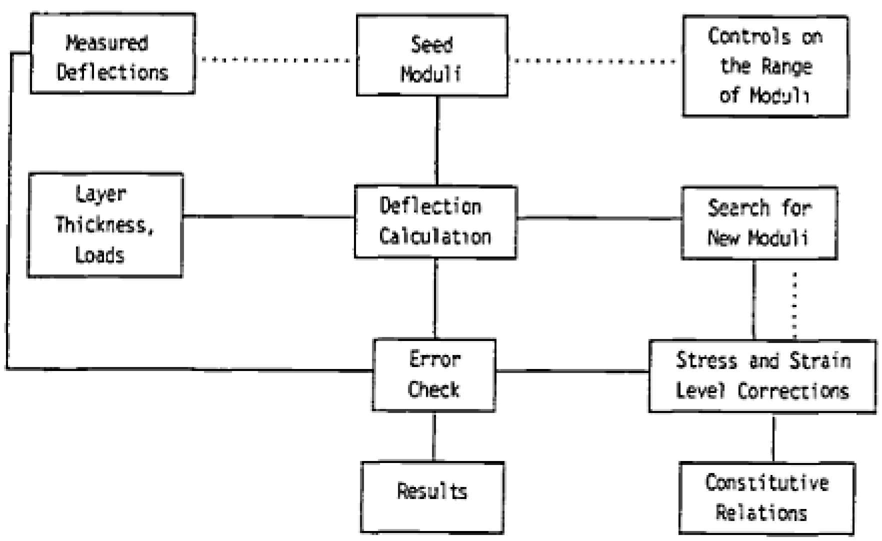


Figure 2.1 Procedure followed in the iterative backcalculation process (Lytton, 1989).

2.2.1.2 Theoretical Equations

One of the most commonly used theoretical approaches is discussed in this section. This procedure, which is used by more than 20 percent of DOTs across the United States (Von Quintus et al., 1997), is introduced in AASHTO 1993

Design Guide to backcalculate subgrade modulus. This procedure is based on the static load on linear materials concept.

As seen in Figure 2.2, in a FWD test, deflections that are far enough from the loading plate correspond to the subgrade layer and the other layers' stiffness do not affect these deflections. As an example, as seen in Figure 2.2 the deflection measured by fourth and fifth deflection sensors are depending on the subgrade stiffness and base and AC layer's moduli do not affect them.

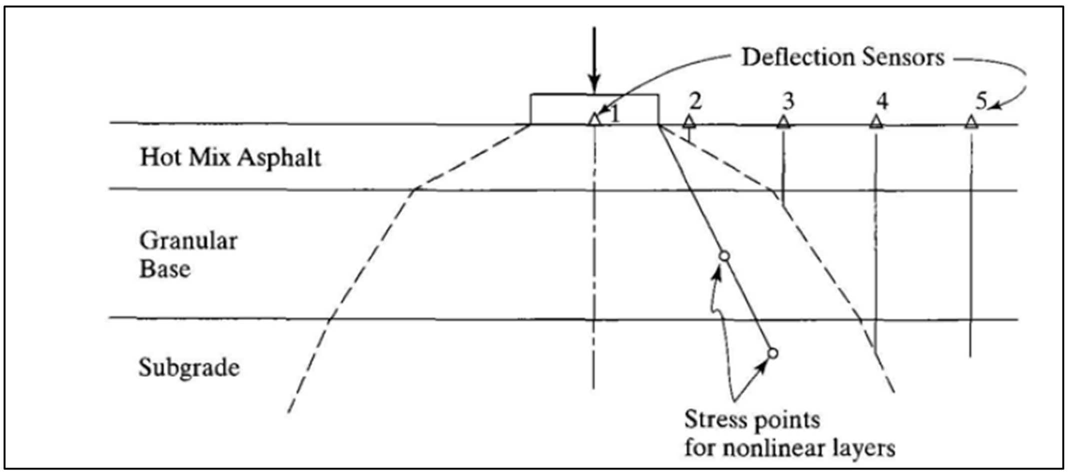


Figure 2.2 Stresszone within pavement structure (Huang, 1993)

In AASHTO 1993 Design Guide, the subgrade modulus is calculated using Equation 1.

$$M_R = \frac{0.24 * P}{d_r * r} \tag{1}$$

Where,

M_R = backcalculated subgrade resilient modulus, psi,

P = applied load, lb,

d_r = deflections at a distance r from the centre of the load, inch,

r = distance from the centre of the load, inch,

Meanwhile, the distance from the centre of the load should be kept as minimum as possible. The minimum value for r, the distance at which the deflections can be used to estimate the subgrade resilient modulus is determined using the following relationship.

$$r \geq 0.7 * \ a_e \tag{2}$$

Where,

$$a_e = \sqrt{\left[a^2 + (D * \sqrt[3]{\frac{E_p}{M_R}})^2\right]} \tag{3}$$

a_e = radius of stress bulb at the subgrade-pavement interface, inches

a = FWD load plate radius, inch,

D = total thickness of pavement layers above the subgrade, inch,

E_p = effective modulus of all pavement layers above the subgrade, psi.

E_p can be calculated using Equation 4, if the subgrade resilient modulus and total thickness of all layers above subgrade are known.

$$d_0 = 1.5 * p * a * \left\{ \frac{1}{M_R * \sqrt{1 + (\frac{D}{a} * \sqrt[3]{\frac{E_p}{M_R}})^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + (\frac{D}{a})^2}}\right]}{E_p} \right\} \tag{4}$$

Where,

d_0 = deflection measured at the center of the load plate (and adjusted to a standard temperature of 68°F), inch,

All the other variables as previously defined. (AASHTO, 1993).

2.2.3 Backcalculation Tools

There are a number of computer tools and software packages that can be used to backcalculate the pavement layers' moduli using the deflection basin from the FWD tests. Most of these tools and software packages use static load, elastic layer theory to calculate each layer's modulus. The most commonly used software can be listed as BISDEF, CHEVDEF, ELMOD, ELSDEF, EVERCALC, ISSEM4, MODCOMP, MODULUS, and WESDE. Results obtained from each of these tools can be different due to different iteration and modulus calculation methods (Von Quintus et al., 1997). Each highway agency or authority uses specific FWD equipment, which generate different FWD data output, therefore, each agency should develop its own software package to ensure compatibility with the specific file format.

ELMOD

More than 20 percent of the DOT in the Unites States use ELMOD developed by Dynatest® (NCHRP Synthesis 381, 2004). ELMOD gives the user the option to choose between finite element method, linear elastic theory or the non-linear subgrade algorithm for the backcalculation process (Dynatest.com, October 2012). ELMOD uses the iterative method to perform backcalculation and can handle any number of existing layers. It can also accommodate up to 15 geophones. ELMOD 6, which is the latest available version of the software, can read Dynatest file format Versions 9, 10, 20, and 25, and also Microsoft Access database files generated by Dynatest WinFWD (Dynatest Website, accessed October 2012).

Evercalc©

Evercalc was developed for the Washington State Department of Transportation (WSDOT). Evercalc© uses WESLEA, which uses multi-layer linear elastic theory as the tool for forward calculation. Up to five pavement layers and up to seven sensors can be defined in the software. Evercalc© uses the iterative method to perform backcalculation. The user can provide the seed layers' moduli as an input to the software and Evercalc© changes these moduli in each set of iteration to minimize the differences between the calculated and measured deflection basin. This difference is identified by calculating RMSE. The software allows for the option of including a stiff layer and will internally calculate the depth of the stiff layer if the modulus is provided by the user. The newest version of Evercalc© reads file formats of Dynatest version 25 and FWD file format. The deflection data in other formats should be entered manually (William, 1999).

AASHTO DARWin

AASHTO DARWin Version 3.1 works based on the procedure introduced in the AASHTO 1993 guide for the design of pavement structures, discussed earlier in the text, and is used by more than 20 percent of US DOTs (Von Quintus et al., 1997). This program can read the Dynatest Version 20 file formats, as well as KUAB and PDDX formats. DARWin is compatible with Microsoft Windows 95, 98, 2000 and NT operating systems. This software allows for filtering deflection data based on the load range, test type, sensor configuration, and test location. (http://darwin.aashtoware.org, Accessed October 2012)

MODULUS

MODULUS was developed by the Texas Transportation Institute (TTI) for Texas DOT. This software uses WESLEA, which works based on the linear elastic theory, as the forward calculation tool, and uses Database Search method in the backcalculation. WESLEA generates several deflection basins using different values for the layers' moduli and then iterates to find the deflection basin that best fits the deflection basin from the FWD test. This software can handle up to seven geophones as well as up to four unknown layers (William, 1999 and Goktepe et al., 2005).

2.2.2 Limitations of Backcalculation Process

When performing a backcalculation analysis the limitations and shortcomings of the backcalculation procedure should be noted. First, the backcalculated moduli are not unique. There can be numerous sets of moduli, which comply with the deflection basin resulting in errors within the acceptable range (Von Quintus et al., 1997). Second, there are limitations to accurate prediction of the layers' moduli in the areas with longitudinal or transverse cracking, since these cracks disturb the assumption of the layers extending to infinity. Similar limitations exist when measuring the deflection basin close to the edge of the pavement (Uzan, 1994). Layer thickness is another important factor, which can affect the backcalculated moduli of pavement layers. Modulus of surface layers less than 75mm are difficult to be accurately backcalculated due to the geometry of the loading and measuring system (Dynatest, November 2012).The other factor that can significantly affect the backcalculation results for flexible pavements is the temperature at the time of testing. AC layer stiffness is strongly sensitive to temperature (Park et al., 2001). Several studies focused on developing equations that can adjust the AC backcalculated modulus or FWD deflections to a reference temperature. Akbarzadeh et al. (2012) in their study have summarized results of a number of these studies. Table 2.1 presents a summary of different models investigated in their study. As seen in Table 2.1, several researchers develop equations to adjust the AC modulus to a reference temperature based on their local climate condition and the purpose of their study. Among them, eight of the developed models use exponential equations in order to adjust the temperature to a reference temperature while three models are logarithmic and one model introduced by Antunes is linear. The models are developed based on various sources of data including four pavements in North Carolina and in Taiwan, AASHO road test, Mobile Load Simulator (MLS) research project and Virginia Smart Road test sections. More details regarding these models can be found in Akbarzadeh et al., (2012) and the references provided in the table..

Table 2.1 Summary of FWD Temperature Correction Models for Asphalt Pavement Modulus (Akbarzadeh et al., 2012)

No.	Author or Computer Program (Reference)	Temperature Correction Model	Explanation of Parameters	Additional Information
1	Stubstad et al. (1994)	$\frac{E_{ref}}{E_{AC}} = \frac{1}{1-2.2\log\left(\frac{T_{AC}}{T_{ref}}\right)}$	E_{ref} and E_{AC} = reference and backcalculated asphalt moduli T_{ref} = reference temperature (°C) T_{AC} = temperature at 1/3 of pavement thickness (°C)	Proposed for BELLS temperature prediction model
2	Baltzer and Jansen (1994)	$\frac{E_{ref}}{E_{AC}} = 10^{-0.018(20-T_{AC})}$	T_{AC}, E_{ref}, E_{AC} = as defined in No. 1 Reference temperature = 20°C	Proposed for BELLS temperature prediction model
3	Lukanen et al. (2000)	$ATAF = 10^{slope.(T_r - T_m)}$	$ATAF$ = asphalt temperature adjustment factor $Slope$ = slope of the log modulus versus temperature curve, recommended as -0.0195 for the wheelpath T_r = reference temperature (°C) T_m = pavement temperature at mid-depth (°C)	Proposed for BELLS2 model
4	Kim et al. (1995)	$\frac{E_{68}}{E_T} = 10^{-0.0153(68-T)}$	E_{68} = asphalt modulus at temperature 68 F° (20°C) E_T = backcalculated asphalt modulus at temperature T T = temperature at mid-depth of asphalt pavement (° F) Reference temperature = 68° F (20 °C)	Based on data from four pavements in North Carolina

Table 2.1Summary of FWD Temperature Correction Models for Asphalt Pavement Modulus (Akbarzadeh et al., 2012)

No.	Author or Computer Program (Reference)	Temperature Correction Model	Explanation of Parameters	Additional Information
5	Johnson and Baus (1992)	$\frac{E_{std}}{E_{field}} = 10^{-0.0002175\ (70^{1.886} - T^{1.886})}$	E_{std} = AC modulus at standard (reference) temperature E_{field} = AC modulus field temperature T = measured temperature (° F) Reference temperature = 70° F (21.1°C)	Based on approximation from the Asphalt Institute.
6	Ullidtz and Peattie (1982)	$\frac{S_T}{S_{15}} = 1 - 1.384 \log(\frac{T}{15})$	S_T , S_{15} = asphalt moduli at temperatures of T (°C) and 15°C Reference temperature = 15°C	Based on deflection data from the AASHO Road Test and SHELL procedure For $T > 1$□
7	Ullidtz (1987)	$\frac{E_{To}}{E_T} = \frac{1}{3.177 - 1.673 \log T}$	E_{To} , E_{T1} = asphalt moduli at temperatures of T_0 and T (°C)	Based on backcalculated moduli from the AASHO Road Test deflection data
8	Antunes (1993)	$\frac{E_{T1}}{E_{T2}} = \frac{1.635 - 0.0317\, T_1}{1.635 - 0.0317\, T_2}$	E_{T1}, E_{T2} = asphalt moduli at temperatures of T_1 and T_2 (°C)	
9	Chen et al. (2000)	$\frac{E_{Tw}}{E_{Tc}} = \frac{(1.8T_c + 32)^{2.4462}}{(1.8T_w + 32)^{2.4462}}$	E_{Tw}, E_{Tc} = asphalt moduli at temperatures of T_w and T_c (°C) (mid-depth temperature)	Based on data from Mobile Load Simulator (MLS) research project

Continues

Table 2.1 Summary of FWD Temperature Correction Models for Asphalt Pavement Modulus (Akbarzadeh et al., 2012)

No.	Author or Computer Program (Reference)	Temperature Correction Model	Explanation of Parameters	Additional Information
10	Chang et al. (2002)	$\frac{E_r}{E_0} = 10^{-0.02822\,(25-T_c)}$	E_r = adjusted modulus to 25°C E_0 = measured modulus at temperature T_C T_C = mid-depth asphalt pavement temperature (°C) Reference temperature = 25°C	Based on data from 1176 FWD tests on two specific sections in Taiwan
11	Appea (2003)	$\frac{E_{25}}{E_T} = e^{-0.031\,(25-T)}$	E_{25}, E_T = moduli at temperatures of 25 and T (°C) T = measured temperature at the bottom of asphalt pavement Reference temperature = 25°C	Based on data from Virginia Smart Road test sections
12	EVERCALC, MICHPAVE	$TAF = 10^{-0.000147362.(77^2 - T_p^2)}$	TAF = temperature adjustment factor T_p = asphalt pavement temperature (° F) Reference temperature = 77° F (25° C)	Based on the relationship between modulus and temperature for WSDOT Class B HMA

2.3 Seasonal Variation in Pavement Moduli

Environmental conditions can have a significant effect on the pavement materials, especially in cold climate conditions (Zaghloul et al., 2003). Stiffness of the pavement layers change due to different environmental conditions (Khogali et al., 1996), this affects the ability of the pavement to carry traffic loads in different seasons (Watson et al., 2000). Seasonal variations in the properties of different pavement layers have been shown to considerably affect the accumulated damage imposed to the pavement structure (Birgisson et al., 2000). "During the winter, the stiffness of the unbound layers generally increases because of the ice bonding between the soil particles in the base and subbase, and ice lens formation in the subgrade " (Simonsen et al., 1999). On the other hand, as the pavement structure becomes saturated from the thawed ice the bearing capacity of pavement is dramatically reduced. Pavement is the most sensitive to heavy loads during this period which can greatly shorten the pavement life. To maintain the pavements in the operational condition during the desired service life, two options are available.

1. Design the pavement structure considering the pavement's conditions during the thaw weakening period,
2. Reduce the load applied to the pavement during the thawing period (Rutherford et al., 1985).

The first choice is not feasible for many highway agencies due to financial limitations (Rutherford et al., 1985). Therefore, the only choice is the second option: restricting the load during the critical thawing period, which is known as

SRB or SLR. Applying a proper SRB requires a series of questions to be answered the most important of which are:

- Where is it necessary to place the SRB?
- What time of year should the SRB start?
- How long should the SRB be in place?
- How much the load should be reduced? (Rutherford et al. 1985 and Eusen et al., 1998)

Several efforts have been made to answer these questions (Rutherford et al., 1985 and MnDOT, 1986). It should be noted that the answer to the above mentioned questions depends on many local variables, such as climatic conditions, pavement material type, pavement drainage ability and other variables. (Eusen et al. 1998). Therefore the findings of a study in a specific region cannot be the best solution for another region. A brief review of SRB practices in different parts of the world is presented in next section.

2.4 Spring Road Ban across the World

Based on available equipment and resources, highway agencies across the world have developed their own practice to implement SRB for their local conditions. Current practices in Canada, Europe and the United States are discussed herein.

2.4.1 SRB Practices in Canada

All Canadian provinces have SRB programs (C-SHRP report, 2000). Each province has its own methodology in applying SRB, based on the available data resources and equipment. A summary of SRB practices implemented by different Canadian provinces can be seen in Table 2.2. Canadian provinces do not restrict the load in primary highways and the allowable load for the secondary roads is reduced by 50, 75 or 90 percent of the maximum allowable load (Levinson et al., 2005 and C-SHRP report, 2000). Alberta uses a heat flow model to predict advancement of thawing in the pavement to determine the start of SRB and perform FWD testing (as described previously) during the spring. Load is reduced to 50, 75 or 90 percent of the maximum allowable load. In British Columbia, restrictions are imposed only when engineering judgment necessitates it. Manitoba considers two stages of imposing SRB. in the first stage, the load is reduced to 90 or 95 percent of the basic allowable load while in the second stage which is imposed 14 days after imposing Stage 1 and is removed seven days before removing Stage 1, the load is reduced to 65 percent of the basic load. As seen in Table 2.1 in New Brunswick, load in specific collectors and local roads is reduced to 90 percent of the basic load while load in other secondary highways 80 percent of the basic allowable load is allowed. In Newfoundland, local roads are monitored and restrictions are imposed whenever needed. In Ontario commercial vehicles gross weight should not exceed 5000kg. Also 2-axle tanker trucks should not exceed 7500kg/axle. Load in secondary highways can be also restricted up to 50 percent of the basic allowable load. As Table 2.1 represents, Prince Edward Iceland does not restrict load in Trans-Canada arterials and some collectors while in the other highways load is reduced to 75 percent of the basic load. In Saskatchewan, load per tire is reduced to 6.25kg/mm width and to a maximum of 1650kg/tire. In addition, some primary highways are downgraded to secondary highways during May and June.

Table 2.2 Summary of SRB Practices in Different Provinces Of Canada (Levinson et al., 2005)

Province	Start of SRB	End of SRB	Restriction	Determination of Restriction
Alberta	30 cm of thaw and a heat flow model	from FWD testing	50%, 75%, 90% of max. allowable load	FWD testing
British Columbia	mid-February	mid-June	No overloads, 50% , 70% of max. allowable load	Frost probes, deflections, historical data
Manitoba	Northern. Zone: April 15 Southern Zone: March 23	May-31	65%, 90%, 95% of max. allowable load	Deflections
New Brunswick	Second or third week in March	mid- or end May	80%, 90% of max. allowable load	Dynaflect
Newfoundland	February	April	as needed	n/a
Nova Scotia (Southern Region)	March 2nd	Apr-24	70%. 75% of basic load	Dynaflect
Ontario	Typically First Monday in March	mid-May	5000kg per axle, 50% of max. allowable load	n/a
Prince Edward Island	March first	Apr-30	75% of max. allowable load	Dynaflect
Quebec	30 cm of thaw	90 cm of thaw below road surface	80%, 85% of max. allowable load	frost probes
Saskatchewan	Second or third week in March	Max. six weeks	90% or 80% of max. allowable load	Deflections

2.4.2 SRB Practices in Europe

Among several countries in Europe that apply load restrictions, Norway stands out for extensive efforts in research on implying SRB. The old Norwegian procedure, which was in place since 1979, included imposing SRB when the thaw reached a depth of 5 to 15 cm in the pavement. SRB was removed, when the pavement recovered minimum 90 percent of summer strength (no information on how they measure the strength was provided in the references) . As of 1995 based on the result of a report by Senstad et al. (1995), “Better Utilization of the Bearing Capacity of Roads”, all spring load restrictions on the national roads were removed based on the cost/benefit study presented in the report. In exchange, an extra fund was to be allocated to maintain the roads appropriately (Refsdal et al, 2004) (Levinson et al. 2005). A summary of SRB practices in some European countries, including Norway, is listed in Table 2.3. As seen in Table 2.3, in France, SRB is applied to secondary roads and the policy regarding primary roads is frost prevention. The weight limits are based on total weights of 3.5 and 9 tons corresponding to 2.5, 4, 6 and 8-ton single dual tire axle. Finland applies vehicle total-weight restrictions. Sweden reduces axle load of 10 tons to 4, 6 or 8-tons based on the decision of local road maintenance supervisor. Also total weight of vehicles and trucks might be limited to 4, 7, 9 or 12-tons (C-SHRP Technical Report, 2000).

Table 2.3 Summary of SRB Practices In Different European Countries (C-SHRP report, 2000 and Levinson et al., 2005 and).

Country	Start of SRB	End of SRB	Restriction	Determination of Restriction
Finland	April	May	Gross weights: 4- , 8- . 12- , 18- tonne; complete closure	FWD experience
France	n/a	n/a	2.4 , 4, 6, 8- tonne for single dual tire axles	Frost depth measurements
Iceland	30 cm of thaw	n/a	Depends on vehicle type and axle configuration	Frost depth measurements
Norway (Old Procedure)	5- 15 cm of thaw	Min. 90% of summer pavement strength	Changed yearly	FWD and frost measurements
Sweden	April	May	4- , 6- , 8- tonne per axle	FWD experience and frost depth measurements

2.4.3 SRB Practices in the United States

Several states, especially in northern part of the United States, place load restrictions during the critical thawing period of spring. The methodology to determine when to place SRB and when to remove it varies in each state and even within states (Ovik et al. 2000). One or more of the following methods are used in developing the SRB (Ovik et al. 2000 and C-SHRP report, 2000):

- Engineering judgement
- Pavement history
- Pavement design
- Visual observations, such as water seeping from the pavement

- Daily air and pavement temperature monitoring
- Frost depth measurement using drive rods, frost tubes, and various electrical sensors
- Deflection testing

Several states such as North Dakota, Idaho, Maine, Montana, New Hampshire, Oregon, New York, Iowa, Wisconsin, Michigan and Illinois rely on engineering judgment and visual observation (Ovik et al. 2000). Other states, such as Washington, Alaska, Minnesota and South Dakota add analytical methods to engineering judgment and visual observations (Ovik et al. 2000). The study conducted at WSDOT, which focuses on establishing general guidelines on when and where to place SRB and also a brief description of the method used by South Dakota DOT (SDDOT) is discussed in more detail in the next sections.

WSDOT Guidelines for SRB

In the WSDOT study three different stages are considered during the thawing period. The first stage is the time when thaw reaches the bottom of the base layer; the second stage is the time when thaw penetrates 10cm (four inches) into the subgrade; and the third stage is when thawing is complete. Different pavement layers moduli were assumed for each stage, based on the field and laboratory results. The AC layer modulus was assumed to be 8,273MPa (1,200,000 psi) during the entire thawing period. During the thawing period the base layer modulus was considered 1.5 times the subgrade modulus and the subgrade modulus was considered to be 50MPa (7,500 psi) before thaw reaches the subgrade (Stage 1). After thaw penetrates 10cm (4 inches) into the subgrade in

Stage 2, the subgrade modulus was considered 5 or 15 percent of the summer modulus and after complete thawing (stage 3) modulus would be 15, 20 or 25 percent of the summer value for fine grained soils. For frozen conditions subgrade modulus is considered 344MPa (50,000 psi). The objective of this part of the WSDOT study was to determine the load in each stage, which would result in equal damage to the pavement comparing to the damage caused by the maximum allowable load in the summer. In doing so, ELSYM5 (Ahlborn, 1972) developed at the University of Berkley, California, which is a layer elastic analysis software, was used to predict two main pavement structural responses (tensile strain at the bottom of the AC layer and vertical strain at the top of the subgrade) for a specific pavement structure and a specific loading scenario during each stage of thawing (with appropriate layers' moduli) discussed previously. Results of this part of study indicate how much the load should be restricted during thawing period. The results reveal that only 20 percent decrease in the maximum load can increase pavement life approximately 60 percent while a 50 percent load reduction will result in 95 percent increase in pavement life.

The other issue addressed in the study is the duration of load restriction. Since the equipment to measure deflections, such as FWD devices are not available all the time and to all agencies, the WSDOT study focused on developing a guideline for imposing SRB, based on ambient temperature obtained from local weather stations, which are used to calculate Freezing Index (FI) and Thawing index (TI). FI and TI are calculated using the preceding year's winter and spring season ambient air temperature. TI is calculated using the following equations based on average daily temperature, defined as the average of lowest and highest daily temperature.

$$TI = \sum T - T_0 \tag{6}$$

$$T = (T_l + T_h)/2 \tag{7}$$

Where,

TI = Thawing index, unit

$T_0 = datumn\ temperature, 29\ °F$

T = average daily temperature, $°F$

T_l = lowest daily temperature, $°F$

T_h = highest daily temperature. $°F$

Two levels (Should and Must) are defined for applying the SRB for thin and thick pavements based on TI. The Should Level is considered as the time when thawing reaches the bottom of the base layer (Stage 1) and the "Must Level" is the time when thaw penetrates four inches into the subgrade (Stage 2). The TI values corresponding to the Should and Must Levels were established for Thin and Thick pavements using a thermal analysis performed using temperature data from 60 locations in all states except for Alaska. Table 2.4 shows the TI corresponding to the Should and Must Levels for the Thin and Thick pavements.

Table 2.4 WSDOT Guideline on When to Apply Spring Load Restrictions (Rutherford et al., 1985).

Pavement Structure			Thawing Index (˚F. Days)	
Pavement Structure Thickness	BST*/AC Thickness	Base Course Thickness	Should Level	Must Level
Thin	5cm (2 inches)or less	15cm (6 inches) or less	10	40
Thick	More than 5cm (2 inches)	More than 15 cm (6 inches)	25	50

*Bituminous Surface Treatment

The duration in which the load restriction should be applied is the time needed to reach the complete thawing condition (Stage 3). Two different equations were developed in the study based on regression equations to predict how long the load restriction should be effective. The first equation which was developed for fine-grained subgrade soils is:

$$\text{Duration (Days)} = 22.62 + .011*(FI) \quad (8)$$

Where,

$$FI = \sum (32-T) \quad (9)$$

T as previously defined

The second approach is based on TI and considers the estimated time needed for complete thawing to occur. This approach is indicating that when TI equals the value calculated in Equation 10, SRB should be removed. This approach does not give the duration of SRB from its beginning and is based on ambient air temperature during the thawing season (reflected in TI) as well as temperature during the winter (reflected in FI).

TI=4.154+.259*(FI) (10)

South Dakota DOT Method

SDDOT uses the FI and TI criteria along with the precipitation data in August to November of the previous year to determine when to apply and remove the SRB. The accumulated freezing and thawing indices are calculated using the below equation:

$$FI(today) = FI(Yesterday) + \tag{12}$$

$$[-(average\ daily\ temperature\ yesterday)]$$

$$TI(today) = \ TI(Yesterday) + \tag{13}$$

$$[(average\ daily\ temperature\ yesterday) + 1.6$$

The day after which TI remains greater than zero for the remainder of the spring is designated as the beginning of the thaw season. FI can increase and decrease based on the temperature during the winter season; however, cannot be less than zero and also the largest accumulated value that FI reaches to is used to determine the duration of spring road ban period. TI can also grow and drop based on daily temperature, but cannot be less than zero.

To consider the effect of soil moisture content in implementation of SRB, fall (Aug-Nov) precipitation is related to the calculated FI and TI. The critical TI for implementing SRB varies based upon the precipitation in August to November of the previous year. Restrictions should be applied after TI reaches the critical TI corresponding to its fall precipitation, as seen in Table 2.5.

To determine the duration of restriction, the critical TI values are calculated as a percentage of FI. These values are again related to the precipitation in the fall and are shown in Table 2.5. For instance, in an area with precipitation of more than 20 cm (7.75 inches), by the time the accumulated TI is 35 °F days restriction should be implemented and by the time thaw index is 40 percent of freezing index, restrictions can be removed (SDDOT website, accessed Nov. 2012).

Table 2.5 Relationship between Fall Precipitation and Critical and Removal Thaw Index

Aug-Nov Precipitation (inch)	Critical TI (°F. Days)	Removal TI (percentage of FI)
7.75	35	40
6.25	40	35
5.50	45	30
4.75	50	25

2.5 Pavement Distresses

Despite all the maintenance efforts, every pavement structure undergoes various damages during its life service. The damages can have different sources such as traffic loads, environmental loads or poor construction and material. These damages can cause different distresses propagate in the pavement. Understanding the mechanism and possible causes of various pavement distresses can help to make more accurate and appropriate decisions regarding pavement maintenance and rehabilitation.

One of the most common distresses observed in flexible pavements is alligator cracking, which occurs due to repeated traffic loading (LTPP distress identification manual, 2003). Alligator cracking generally develops in the wheel path and is a series of interconnected cracks resembling chicken wire or alligator pattern, which create small cracks from 25 to 150 mm in size (Lavin, 2003 and LTPP distress identification manual, 2003). Bottom up cracks initiate from high tensile strain areas at bottom of the AC layer and propagate upward (Huang, 1993). Figure 2.3 shows the pattern of alligator cracks (Huang, 1993).

Figure 2.3 Fatigue cracking in flexible pavement (Pavementinteractive.org, accessed October 2012)

In some cases, cracks initiate from the top of the AC layer in the wheel path and propagate downward in the AC layer known as top-down cracking. These cracks are an indicator of inadequate structural strength of the pavement, weak subgrade or overloading of the pavement (Lavin, 2003).

Longitudinal cracking if not in the wheel path, is not load associated and can be the result of surface shrinkage due to low temperature, reflective cracks or poor joint construction (Huang 1993 and Lavin, 2003). Longitudinal cracks in the pavement increases pavement roughness and also allows for moisture infiltration into the pavement. Crack sealing can be effective to repair low-severity longitudinal cracks while for high severity cracks an overlay should be constructed (Pavementinteractive.org, October 2012). Figure 2.4 shows a typical longitudinal crack.

Figure 2.4 Wheelpath (load induced) longitudinal cracking (Pavementinteractive.org, October 2012)

Transverse cracks usually extend across the pavement centerline perpendicular to traffic flow (Huang, 1993). Transverse cracks can be divided into two categories of temperature-induced and load-induced transverse cracks. “Pavement movements due to temperature changes and aging related shrinkage of the asphalt binder cause temperature-induced transverse cracks”. Load-induced transverse cracks can propagate on pavement overlays that have unfilled joints or gaps (Lavin, 2003). These cracks allow moisture into the pavement, which can cause greater damages. Also, the rideability of the pavement decreases as a result of the increase in roughness. Similar to longitudinal cracks, transverse cracks are measured in linear feet or linear meters in Standard units (Huang, 1993). Figure 2.5 shows a severe transverse crack developed in the pavement.

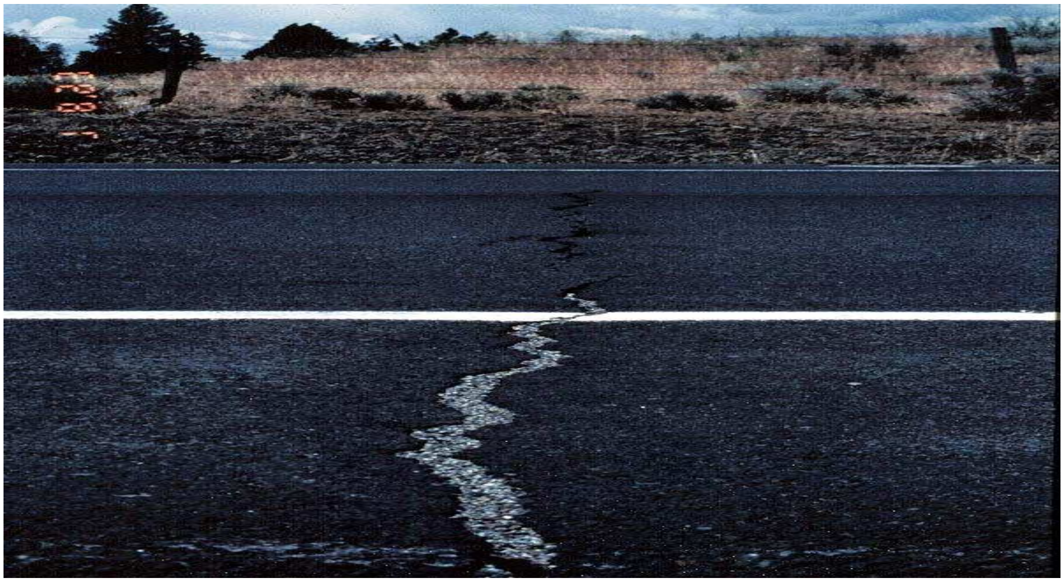


Figure 2.5 Picture of a high-severity transverse crack (Pavementinteractive.org, October 2012)

“A rut is a surface depression in the wheel paths” (Huang, 1993). Rutting is also referred to as the permanent deformation in all or a part of the pavement layers (Frazer et al., 1990). Rutting can occur due to lateral movement of the pavement layer or subgrade as a result of repetitive traffic load (Huang, 1993 and Frazier et al. 1990). Rutting can ultimately cause major structural failure and also increases the possibility of hydroplaning and spray (Huang, 1993 and Frazier et al. 1990). Possible reasons for rutting are: insufficient AC layer compaction, improper mix design or improper structural design (Pavementinteractive.org, 2012). Figure **2.6** shows a case of sever rutting in the pavement. Figure 2.6 points out importance of rutting in safe driving as rutting can compromise safety because it forces the driver into the rut path which is a big safety issue.

Figure 2.6 Sever rutting in the pavement (Pavementinteractive.org, October 2012).

2.6 Pavement Maintenance and Rehabilitation

Distresses in the pavement can accumulate if roads are not maintained after a period of time. Maintenance can be described as timely activities used to safeguard pavement during its service life, until the time that deterioration of the pavement layer materials and subgrade is such that a minimum acceptable level of serviceability is reached (Haas et al., 1997). Factors that contribute to the severity of pavement distress include pavement structure, construction quality, and environmental condition. Rehabilitation serves to increase the structural capacity of pavement and serviceability.

Common routine maintenance methods for flexible pavements are crack seal, fog seal, scrub seal, flush seal, micro-surfacing, chip seal, pothole repair, spray patching, patching and drainage improvement(Haas et al., 1997 and Jusang et al., 2010). Rehabilitation techniques are usually overlay construction, overlay plus milling prior to overlay construction, hot in-place recycling, cold in-place recycling and full depth reclamation.

2.6.1 Asphalt Concrete Overlay

Constructing an extra AC layer on top of the existing pavement surface (AC or Portland Cement Concrete [PCC]) is AC overlay construction (Shahin, 2005). AC overlay construction is used to increase structural capacity or functional properties of a pavement. Structural defects “arise from any conditions that adversely affect the load-carrying capacity of pavement” (AASHTO, 1993) such as insufficient thickness, cracking, distortion and disintegration. Functional defects can be listed as poor surface friction, hydroplaning, splash from wheel path rutting and extensive surface distortion (AASHTO, 1993). When constructing AC overlay, the issue of reflection cracks should be taken into account. However, overlay construction can increase structural capacity of the pavement; reflection cracks can influence the riding quality and structural reliability of the rehabilitated pavement. Reflection cracks can be defined as transferring of the existing surface crack patterns to the overlay (ITS, University of California, 1984).

Preparation prior to overlay construction can be significantly effective. There are a number of actions that can help success of an overlay project, such as:

- Repairing weak areas by deep patching
- Construction of a leveling course when the surface is distorted
- Using crack sealing mixtures to fill the wide surface cracks
- Cutting a certain thickness of existing surface (e.g. milling)

For any maintenance or rehabilitation project, timeliness is of the essence and can be achieved by applying the appropriate method at the right time. A life cycle cost analysis is required to determine when and how an extensive rehabilitation should be applied (ITS, University of California, 1984).

Milling and recycling are the most commonly used techniques in overlay projects. The techniques introduced here can be combined for more efficiency (Shahin, 2005).

Cold Milling

Cold milling is defined as removal of a certain thickness of surface layer with several purposes such as:

1. Increase the pavement level of service
2. Remove a deteriorated layer
3. Provide good bonding with overlay

Cold milling can be most effective when combined with recycling process, (Shahin, 2005).

Cold Recycling

In cold recycling the paving mixture is produced from reclaimed asphalt pavement from the milling process and additional water, without using heat. This technique is usually applied to the pavements that are significantly deteriorated. The pavement is removed, and the removed pavement would be used as the reclaimed asphalt in the recycled cold mix (Shahin, 2005).

Hot Recycling

Hot recycling is defined as “using reclaimed asphalt pavement obtained from a cold milling and mixing it with new aggregate, new asphalt cement and a recycling agent to produce recycled hot mix” (Shahin, 2005).

2.7 Long-Term Pavement Performance Program

The LTPP program was initiated in 1987 to collect pavement performance data in the long-term for research conducted under the Strategic Highway Research Program (Smith et al., 2004) LTPP consists of two different studies, General Pavement Studies (GPS) and Specific Pavement Studies (SPS). More than 2,500 test sections within North America are included in the LTPP program (FHWA website, Oct. 2012). Data in different categories are collected from every active site and stored in an online publicly accessible database. The categories in which data are collected are “Inventory, Maintenance, Monitoring (Deflection, Distress, and Profile), Rehabilitation, Materials Testing, Traffic, and Climatic” (FHWA.dot.gov, Oct. 2012).

Availability of a large amount of pavement long-term performance data from the LTPP program along with advancements in computer modeling capacity, has led to development of more accurate tools for pavement design and rehabilitation. The Mechanistic Empirical Pavement Design Guide (MEPDG) is the newest tool available to pavement designers. The guide uses a series of predictive models to predict development of each distress in the pavement. Data obtained from LTPP program had a vital role in calibration of these models.

The Canadian long-term pavement performance (C-LTPP) was initiated in 1989 (Smith et al., 2004) to consider factors that are of particular interest in Canada.

The overall goal of C-LTPP is to “increase pavement life through the development of cost-effective pavement rehabilitation procedures, based upon a systematic observation of in-service pavement performance” (Smith et al. 2004). When defining the overall goal of C-LTPP four individual objectives were taken into account.

- “to evaluate Canadian practice in the rehabilitation of flexible pavements, and to subsequently develop improved methodologies and strategies;
- to develop pavement performance prediction models and validate other models or calibrate them to suit Canadian conditions;
- to establish common methodologies for long-term pavement evaluation, and to provide a national framework for continued pavement research initiatives;
- to establish a national pavement database to support the preceding C-LTPP objectives as well as future needs.” (Smith et al. 2004)

C-LTPP consists of 24 highway test site across Canada. Figure 2.7 shows the distribution of these sections across Canada. Unlike the practice in US-LTPP which studies various pavement structures, C-LTPP aims to study one specific case; construction of an AC overlay over an existing AC pavement with a granular base (CLTPP Database user’s guide, 1997). Every test site in C-LTPP consists of at least two adjacent test sections resulting in a total number of 65 pavement sections (CLTPP Database user’s guide, 1997), which will create the opportunity to compare different rehabilitation strategies while the other conditions such as traffic, climate, soil type, etc. are similar.

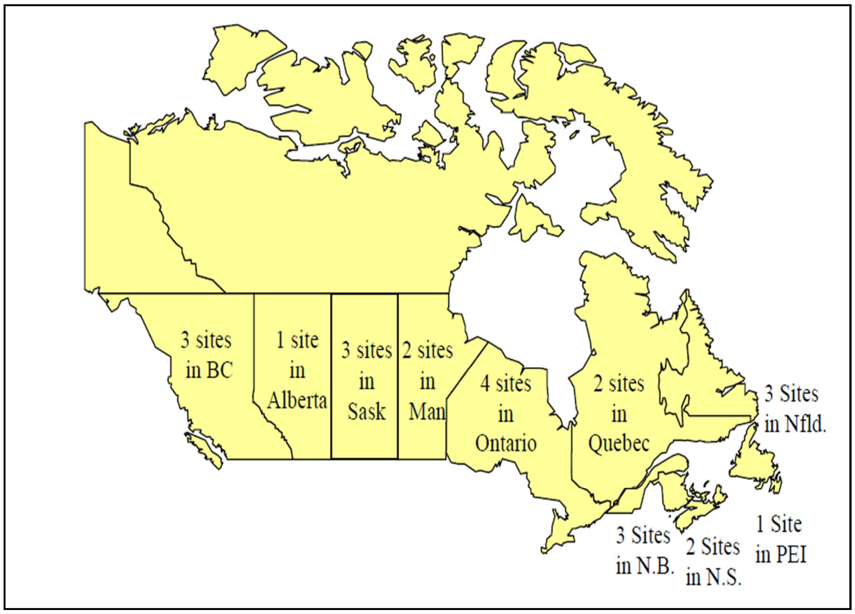


Figure 2.7 Distribution of C-LTPP sections across Canada (CLTPP Database user's guide, 1997).

Among different experiments designed in the LTPP program one of them is SPS 5, which intends to investigate effects of different overlay strategies such as overlay thickness, asphalt overlay material and pre-overlay preparation on the long-term performance of the pavement. Each SPS 5 section in the LTPP program consists of nine 150-meter sections (Sections 501 to 509). The first section (501) is the control section, which receives no treatments during the monitoring period, while the other eight sections are overlaid with two different asphalt mixtures, at two different thicknesses and under two different pre-overlay conditions. Sections 502-505 were not milled prior to overlay construction. Sections 502, 503, 508 and 509 were overlaid with Reclaimed Asphalt Pavement (RAP) materials while the other sections were overlaid with virgin materials. And Sections 502, 505, 506 and 509 received a thin (50mm) overlay while the other sections have a thick (125mm) overlay.

2.7.1 Alberta SPS 5

The LTPP SPS 5 sections in Alberta are located in the westbound truck lane of Highway 16, Control Section 6, at approximately 3.5 km west of junction with Highway 32 and 36 km east of Edson (Kwan & Stoski, 2007). The road section is considered a rural principal arterial intestate according to the LTPP database DVD Version 26. As mentioned previously, each SPS 5 section consists of nine sections. It should be noted that the sections' order of appearance does not follow the sections' numbering; some variations exist in the structural design of the sections. Sections 503 to 507 include a 50- to 80-mm asphalt Treated Base (TB) layer, while Sections 502, 508 and 509 do not include this layer. Existing AC layer also varies between 125- and 165 mm among the nine sections.

The LTPP SPS 5 sections of Alberta joined the LTPP program in May 1990 and the construction of the new overlay was completed in September 1990 (again, the control section did not receive an overlay). Since the completion of the overlay construction, each of the eight sections received different maintenance depending on their conditions. The most common treatments applied to the sections were crack sealing and pothole patching. Two of the sections, 502 and 509, reached the end of their service life in 2006, 16 years after the overlay construction and were overlaid in fall 2006. This study focuses on the evaluation of the test sections during the period between 1990 and 2006.

2.7.2 Findings from other SPS 5 Studies

Since this study will focus on evaluation of pavement performance in the Alberta SPS 5 section, it is worthwhile to review the findings from other SPS 5 studies.

To date, several studies have been conducted on the SPS 5 sections across North America to investigate the effect of different overlay strategies on pavement performance West et al., (2011) used the latest recorded data for all the SPS 5 sections (total of 16 states in the United Sates and two provinces in Canada) to statistically compare the performance of the nine sections. It was concluded in their study that mixture type and milling prior to overlay construction can greatly affect the pavement performance in terms of fatigue, transverse and longitudinal cracking. It was also concluded that overlay thickness does not have a significant effect on longitudinal cracks. In another study, Hall et al. (2003) used the most recently updated measurements of the International Roughness Index (IRI) for all the SPS 5 sections across North America. Their study revealed that no significant differences exist between long-term IRI for the RAP versus virgin as well as milled versus not-milled overlay sections. The effect of pre-overlay IRI, overlay age and average annual temperature was found to be considerable on long-term IRI. They also found a correlation between the annual precipitation and the increase in long-term IRI for the virgin overlays. According to Hall et al. (2003), the factors affecting long-term cracking are pre-overlay cracking, age and traffic loads during the service life, while mixture type or milling proved to have no effect on long-term cracking. In another effort to investigate the SPS 5 sections' performance, Carvalho et al. (2011) used the distress data available over the pavement's life from Arizona's SPS 5 sections. They reported that in the long-run, sections overlaid with the virgin mixture are smoother than those overlaid with the RAP mixture. The latter sections also demonstrate higher rutting and longitudinal cracking. They also concluded that milling prior to overlay construction improves fatigue cracking performance and found that thin overlays (50 mm) demonstrate better short-term rutting; while thick overlays (150 mm) outperform the thin overlays in terms of long-term rutting and all other performance indicators.

In another study conducted by Smith et al. (2003) on Canadian LTPP sections, IRI measurements under LTPP program until 2003 was used as the roughness indicator and effect of different factors such as overlay thickness, climatic zone and subgrade soil type was investigated on roughness development in CLTPP sections. They also developed a series of equations to predict roughness in different climatic zones within the Canada and concluded overlay thickness and climatic zones can have a considerable effect on roughness while subgrade soil type cannot greatly affect it.

3. Chapter 3: Evaluation of Seasonal Variation in Pavement Mechanistic Responses Using Falling Weight Deflectometer Data[1]

[1] *A version of this chapter has been submitted for publication. Norouzi, Nassiri and Bayat 2012. Transportation Research Record, Journal of Transportation Research Board of National Academies.*

3.1 Introduction

Pavement structures experience stiffening and weakening cycles as the moisture in the unbound layers freezes in the winter and thaws in the spring. The effect of freezing and thawing cycles on the pavement are more severe in regions with cold-climate conditions, since the frost penetrates deeper into the pavement structure. In Canada's cold climate, frost can penetrate as deep as two meters in the pavement and pavement temperature can vary from -40 to 40 °C throughout the year (Watson et al., 2000). Under these conditions, the pavement experiences its weakest state at the start of thawing in the spring of each year. This period is followed by the recovery period during which, excessive moisture in the unbound layers dissipates/drains and the pavement starts to gradually regain its stiffness. The seasonal behaviour of the unbound layers' material properties is captured in the newly developed Mechanistic Empirical Pavement Design Guide (MEPDG) (Liang et al., 2008). When using the MEPDG for pavement design, seasonal adjustment factors are applied to the unbound layers' moduli in different times of the year, depending on the moisture content of the layer and ground water table at that time (Larson et al., 1997). Accurate prediction of seasonal changes in the pavement structure's load bearing capacity can help adjust legal load limits for different roadways to prevent possible damage during this critical period. Many highway agencies and jurisdictions in North America and other places in the world have implemented regulations to reduce the maximum allowable loads during the critical thawing period to prevent excessive damage to the pavement (Van Deusen et al., 1998 and Ovik et al., 2000). Several concerns regarding

Spring Road Ban (SRB), such as the time of the year to restrict the load and the extent of load restriction need to be investigated and established. Highway agencies have adopted different methods for establishing their SRB, based on their available equipment and resources (Kestler et al., 2000).

Some highway authorities enforce the load restrictions when distresses manifest in the pavement during the spring *(*Kestler et al., 2007*)*. Clearly, that is not the best practice for pavement preservation, since the pavement has already experienced some damage when the load restriction is applied. Many agencies across Canada use predetermined dates and duration to apply SRB every year, based on past experience *(*Kestler et al., 2007*)*. Other highway jurisdictions including Alberta Transportation use a series of Falling Weight Deflectometer (FWD) tests during the spring and summer/fall period to capture seasonal changes in the pavement layers' moduli. The backcalculated pavement moduli from the FWD tests can help agencies achieve a better understanding of seasonal variation in the pavement stiffness and make appropriate decisions regarding SRB.

To date, several research studies have focused on analyzing seasonal FWD test data for the purpose of developing appropriate SRB *(*Rutherford et al., 1985 and Levinson et al., 2005*)*. Major findings from some of the respective local studies are discussed herein. Watson and Rajapakes *(2000)* conducted a study on seasonal changes in flexible pavements in the Province of Manitoba. The pavement on one section of Highway 1 was instrumented at different depths and tested with the FWD device from February 1994 to May 1995. The backcalculated moduli were used to develop polynomial models to show the variation of the subgrade and base layers' moduli as a function of thawing index. A study of seasonal variations of the subgrade stiffness was conducted by Alberta Transportation, based on the FWD test data from 1989 to 1994 *(Kurlanda et al., 1994)*. This study focused on backcalculation of the pavement moduli for a limited number of test sections, using ELMOD *(ELMOD quick start manual)*. A general conclusion drawn in the study was that on average, the subgrade for conventional pavements was the weakest right after the frost left the ground. At that time, for many subgrade types, modulus reduction was up to 50 percent.

Khogali and Anderson in 1996 conducted a study on Highway 16 (west of Edmonton, Alberta) to investigate seasonal variations in the pavement subgrade *(*Khogali et al., 1996*)*. FWD tests were conducted during two years at regular time intervals at various locations of the road section. It was concluded in the study that following a significant reduction in the subgrade stiffness upon thawing, a long period of relatively constant strength prevailed. Later, a period of gradual recovery and subsequent increase in the stiffness was experienced as freezing approached. They also concluded that the temperature of the Asphalt Concrete (AC) layer at the time of FWD testing does not affect the unbound layers' moduli (Khogali et al., 1996). In another study by Alberta Transportation, the results of the FWD tests conducted approximately twice a month in 2008 and 2009 on three road sections in Alberta was analyzed. It was concluded in the study that the accuracy and repeatability of the FWD equipment does not have a significant impact on seasonal variation of the pavement layers' backcalculated moduli. Also, monthly variation of the subgrade modulus was found to be significant enough to influence the pavement design *(*McMillan et al., 2009).

This study provides a widespread evaluation of the FWD tests conducted at a network level in Alberta. The deflection data from the FWD tests performed every year on a regular basis during the critical thawing period is used to evaluate the effect of seasonal variation in the pavement stiffness. Further, the predicted pavement critical responses: compressive strain at the top of the subgrade and tensile strain at the bottom of the Asphalt Concrete (AC) layer are used to estimate the changes in the pavement's load bearing capacity in terms of rutting and fatigue cracking, as a function of seasonal changes in the moduli. An approach is also suggested for establishing the appropriate load level reductions in the spring using the Asphalt Institute and US Army Corps of Engineer's allowable load limit models. Lastly, the effect of load reduction during the spring on the MEPDG-predicted pavement performance life is investigated.

3.2 Alberta Transportation's FWD program

Alberta Transportation acquired its first FWD test equipment in 1989, and started using the FWD deflection data to evaluate the pavement structure for final stage paving and overlay design in 1992 *(AT pavement design manual, 1997).* Alberta Transportation follows two FWD test programs every year: the inventory and the SRB programs. The inventory is the routine FWD testing program used every year, mainly to make decisions regarding the rehabilitation strategies for pavement sections across the province. The program is conducted when the subgrade is expected to have stabilized after the thawing season. This program covers a portion of Alberta's primary highways and secondary roads each year. The SRB program, on the other hand, has replaced the traditional Benkelman Beam test for removing the SRB on secondary highways and is conducted on pre-defined 2-km stretched of road sections on target highways. A series of tests are conducted starting typically in March and ending in June every year (Terms of reference for FWD testing of Alberta). The historical network FWD test data from the SRB program since 2000 was used in this study to investigate the seasonal behaviour of pavement sections in the province.

3.3 Analysis of NETWORK FWD Deflection Data

The entire SRB database was screened for data availability in consecutive years for each highway section. Of the large historical FWD database available, ten highway sections were selected for this study based on the availability of the FWD data for several successive years. The FWD data was available for these ten sections for seven years, from 2000 to 2006. The ten highway sections tested in seven years with an average of three FWD tests at different times in the thawing and recovery period of each year resulted in the analysis of more than 250 FWD data files, each including five tests along the 2-km test section in each traffic direction. In this study, the test locations in only one traffic direction were used in the analysis. The geographical distribution of the ten sections across the province is shown in Figure 3.1 The structural design, construction history, start and end km and subgrade type for the selected highway sections is provided in Table 3.1 It should be noted that all the ten sections include AC as the wearing course placed on granular base courses (GBC).

Figure 3.1 Geographical distribution of ten select test sections.

Table 3.1 Structural Design and Construction History for Ten Select Highway Sections.

Case No.	Highway : Section	Start km: End km	Layer Thickness (mm)		Subgrade Type
			AC	GBC	
1	520:2	35:37	150	100	CI-CH
2	530:2	16:18	50	250	Unknown
3	570:1	3:5	60	230	Unknown
4	608:2	5:7	60	230	Unknown
5	744:4	35:37	180	200	CI-CH
6	753:4	26:28	60	250	Unknown
7	817:4	22:24	210	200	CL-CI
8	827:4	17:19	60	250	CL-CI
9	840:2	22:24	150	150	CH
10	842:8	0:2	50	100	CI

3.3.1Seasonal Trends in FWD Deflection Data

The pavement structure is the stiffest in the freezing season, weakens when the unbound layers start to thaw and recovers once the thawing season is over. To capture the complete weakening and recovering behaviour of the pavement, a complete FWD test data over the thawing period and recovery period of every year is required. The FWD peak deflections (the deflection underneath the center of the load plate) were used to investigate the seasonal behaviour of each of the ten sections in different years. Figure 3.2 is an example graph presenting the peak deflection trends for Highway Section 520:2 from 2000 to 2006. As seen in Figure 3.2, for most years, except for 2001 and 2002, just a portion of the pavement behaviour is captured by the FWD tests. In years 2000, 2003 and 2004 just the ascending trend in peak deflection (weakening period) is captured; while in years 2005 and 2006 deflections decrease to a point and then start to increase. For years 2001 and 2002, the weakening behaviour followed by the recovering period can be noted. The same type of analysis was performed for all other nine test sections. The analysis revealed that the FWD data available for each test section is limited to only a few months and in most cases does not fully capture the behaviour of the road sections over the thawing and recovery period. Only a limited number of years for each test section include both the weakening and recovering periods in the pavement stiffness. For each test section, only the test years during which the complete behaviour of the pavement structure was captured by the FWD tests were selected for further analysis, resulting in a total of 13 cases. Table 3.2 presents a list of the select 13 cases, together with the minimum and maximum peak deflections observed for each case in the recovery and thawing period, respectively. The weakest and the recovered condition of the pavement were used to establish the maximum possible variation in the pavement performance.

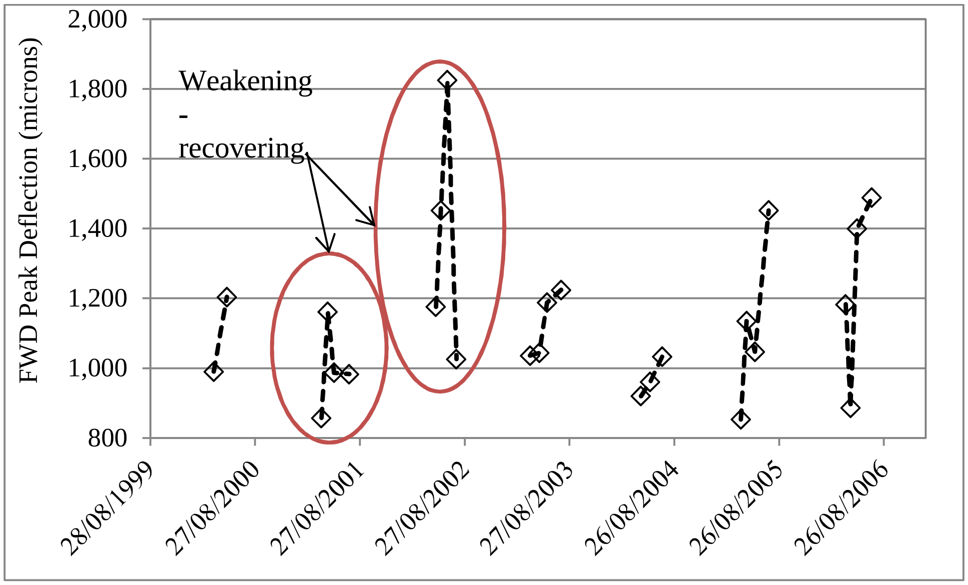


Figure 3.2 Seasonal trends in the FWD peak deflections in seven years for Highway Section 520:02.

Table 3.2 Peak Deflections in Thawing and Recovering Seasons for 13 Select Cases.

Case No.	Highway : Section	Test Year	FWD Peak Deflection (microns)	
			Thawing Period	Recovery Period
1	520:02	2001	1217	998
2		2002	1863	1034
3	530: 02	2001	1960	1888
4		2002	1867	1760
5		2003	2117	1887
6	570:01	2001	1898	1628
7	753:04	2000	1525	1109
8		2002	1431	1397
9		2004	1644	1476
10		2005	1814	1614
11	842:08	2001	2600	1271
12		2002	1715	1372
13		2004	1991	1883

Twenty-six backcalculation analyses were conducted for the 13 cases in Table 3.2 for the thawing and recovering periods. Evercalc© 5.0 (Everseries users' guide, 2005) developed by the Washington Department of Transportation (WSDOT) was used to conduct the backcalculation analysis in accordance with the American Society for Testing and Materials (ASTM) D5858: Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using

Layered Elastic Theory (ASTM D 5858). Currently the FWD testing devices in Alberta are equipped with nine geophones at offsets of 0, 200, 300, 450, 600, 900, 1200, 1500 and 1800 mm from the centre of the 30-cm diameter load plate. Also, each FWD test consists of one seating drop and another three drops at target loads 26.7, 40 and 53 kN. Prior to backcalculation, the deflections were normalized to the target load, whenever the applied load deviated from the corresponding target load by more than five percent, according to ASTM D 5858. The seed moduli required for the backcalculation analysis were defined as the typical values of 3500, 200 and 50 MPa for the AC, GBC and subgrade layers respectively, based on the ASTM D 5858 recommendations. Each backcalculation analysis was conducted considering 1) a stiff layer with a modulus of 345 MPa, 2) a stiff layer with a modulus of 4500 MPa, and 3) no stiff layer. The stiff layer with a modulus of 4500 MPa at apparent depths estimated internally by Evercalc resulted in the best backcalculated moduli with regards to the Root Mean Square Error (RMSE) between the calculated and measured deflection basins. Table 3.3 presents the backcalculated modulus for each layer together with the corresponding RMSE for each backcalculation, as well as the depth to the stiff layer and the AC layer's mid-depth temperature at the time of testing for all 13 cases. As seen in Table 3.3 all cases show values between approximately 1 and 4 percent for the RMSE. The depth to the stiff layer varies between approximately 2 to 15 m as seen in Table 3.3. The backcalculated AC layer modulus for most cases agree with the changes observed in the AC temperature (higher temperature resulting in softer AC), except for Cases 3, 4, 12 and 13. One possible reason for the unreasonable trend in the AC modulus with respect to AC temperature for the four cases can be errors in measuring or recording the AC temperature during testing. The four cases were not included in the analysis in future sections of the paper. According to Table 3.3 the average backcalculated modulus for the base layer for the 13 cases is 122 MPa in during thawing period. This value increases by 30 percent and reaches to an average value of 175 MPa in the recovery period. The backcalculated subgrade modulus increases by 22 percent on average form thawing to recovering period for the 13 cases. The maximum increase in the subgrade modulus was observed for Case 2 and is 79 percent. For Cases 1 and 6, the subgrade modulus in the thawing period is slightly higher than the modulus in the recovery period, however an approximately 20 percent increase is observed in the base layer for the cases. This shows that while the base recovered, thawing did not reach the subgrade at the time of FWD testing.

Table 3.3 Backcalculation Results for all 13 Cases in Different Times of the Year.

Case No.	Hwy	Test Year	Period of Testing	Backcalculated Modulus (MPa)			RMSE (%)	Depth to Rigid Layer (m)	T_{AC} (°C)	Adjusted AC Modulus
				AC Layer	GBC	Subgrade Layer				
1	520	2001	Thawing	770	139	50	0.98	1.77	19	532
			Recovering	1592	191	48	0.5	1.86	14	769
2	520	2002	Thawing	255	134	34	3.26	1.85	22	213
			Recovering	1003	150	61	0.99	1.79	16	563
3*	530	2001	Thawing	5565	69	29	0.74	1.88	18	3594
			Recovering	4152	79	34	1.06	1.97	12	1712
4*	530	2002	Thawing	6172	65	35	0.46	2.15	15	3218
			Recovering	9500	50	36	1.51	1.98	15	4953
5	530	2003	Thawing	3007	43	32	1.52	1.81	21	2365
			Recovering	4433	74	40	1.47	2.01	23	3943
6	570	2001	Thawing	3628	85	38	1.53	9.90	21	2853
			Recovering	4157	97	35	1.27	9.76	24	3923

Continues

Table 3.3 Backcalculation Results for all 13 Cases in Different Times of the Year.

Case No.	Hwy	Test Year	Period of Testing	Backcalculated Modulus (MPa)			RMSE (%)	Depth to Rigid Layer (m)	T_{AC} (°C)	Adjusted AC Modulus
				AC Layer	GBC	Subgrade Layer				
7	753	2000	Thawing	3687	87	48	0.79	2.24	13	1648
			Recovering	5728	119	61	0.67	2.32	12	2361
8	753	2002	Thawing	4149	87	44	1.74	2.56	13	1854
			Recovering	4302	100	45	0.73	2.22	14	2079
9	753	2004	Thawing	4189	74	39	1.21	2.12	15	2184
			Recovering	4339	80	50	1.54	2.41	20	3201
10	753	2005	Thawing	4561	61	33	0.91	2.48	12	1880
			Recovering	4378	71	42	1.54	2.37	14	2116
11	842	2001	Thawing	1084	108	42	1.96	15.24	22	907
			Recovering	1071	175	53	2.55	15.24	25	1071
12*	842	2002	Thawing	59	1202	56	3.8	15.24	21	79
			Recovering	2305	258	75	1.57	15.24	12	950
13*	842	2004	Thawing	2340	117	53	2.98	15.24	32	3390
			Recovering	2360	145	55	1.89	15.24	13	1055

*These cases were excluded from future analysis

As seen in Table 3.3 the AC temperature varied during the testing in the thawing period in comparison to the recovery period due to different times of testing during the day and changes in ambient temperature. In an attempt to isolate the effect of ambient temperature on the backcalculated AC modulus and focus on the recovery of the unbound layers moduli, the backcalculated AC moduli were adjusted to a reference temperature of 25° C. In doing so, several adjustment models available in the literature were evaluated *(, Antunes, 1993, Appea, 2003, Baltzer, 1994, Chang, 2002, Chen, 2000, Kim, 1995, Lukanen, 2000, Stubstad, 1994and Ullidtz, 1987)*. The relation developed by Chen et al., presented in Equation 1, was found to best adjust the backcalculated moduli for different temperatures in the study. The adjusted AC modulus is provided for each case in Table 3.3

$$E_{Tw} = E_{Tc} [(1\ 8T_w + 32)^{2.4462}\ (1\ 8T_c + 32)^{2.4462}] \qquad (1)$$

Where,

E_{Tw} is the adjusted modulus of elasticity at T_w (MPa),

E_{Tc} is the measured modulus of elasticity at T_c (MPa),

T_w is the temperature to which the modulus of elasticity is adjusted (°C),

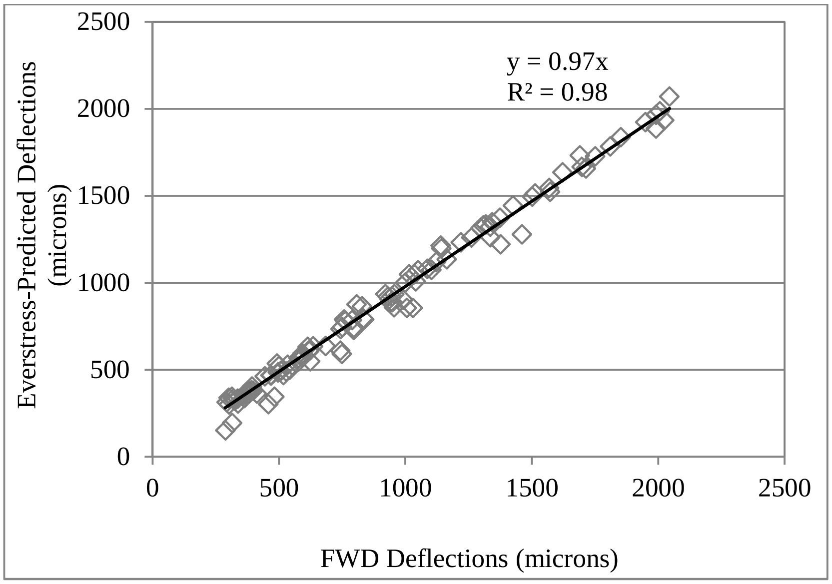
and

T_c is the mid-depth temperature at the time of FWD data collection (°C).

3.4 Seasonal Structural Response

To investigate the effect of thaw weakening on the pavements critical structural responses, multi-linear elastic models were developed for Highway Sections

520:2, 530:2, 570:1, 753:4 and 842:8, using Everstress© 5.0 developed by the WSDOT. The structure for each section was defined using the information presented previously in Table 3.1. The FWD backcalculated modulus for each layer (presented in Table 3.3) was used to define the stiffness of the AC, base and subgrade layers. Poisson's ratio was defined as 0.35, 0.4 and 0.45 for the AC, base and subgrade layers, respectively, according to the recommendations found in the ASTM D 5858. A total of 18 numerical models (one for the thawing and one for the recovering periods for each case) were developed to simulate the nine accepted cases from Table 3.3. In doing so, the load was defined as 53 kN (equal to the FWD load) and was applied on a 30-cm diameter load plate. To assure that the Everstress© model simulates the FWD tests realistically, each case was validated through comparing the deflection basins measured by the geophones during the FWD testing with the deflection basin predicted using Everstress. The unadjusted AC layer's moduli were used for validation for each case. Figure 3.3 shows the comparison of the predicted and measured deflection basins. As seen in 3Figure 3.3a very strong agreement with a coefficient of determination (R-squared) of 98 percent is achieved, implying that Everstress© can simulate the FWD test for the pavement sections.



3Figure 3.3Comparison of the Everstress-predicted and FWD measured deflection basins.

Two major failure modes for flexible pavements are fatigue cracking and rutting. The two pavement responses of tensile strain at the bottom of the AC layer (ε_t) and compressive strain at the top of the subgrade (ε_c), which correlate to bottom-up fatigue cracking and rutting, respectively were extracted from the Everstress models for each case. The adjusted AC moduli were used in these models to eliminate the effect of temperature on the predicted strains as much as possible. The percent difference between the strains in the thawing and recovering periods for both responses is presented in Figure 3.4. According to Figure3.4, the decrease in ε_c from thawing to recovering period varies between a minimum of six percent for Highway 753 in 2002 and maximum of 45 percent for Highway 520 in 2002. The compressive strain at the subgrade level drops between approximately 10 to 40 percent from the thawing to the recovering periods for different cases.

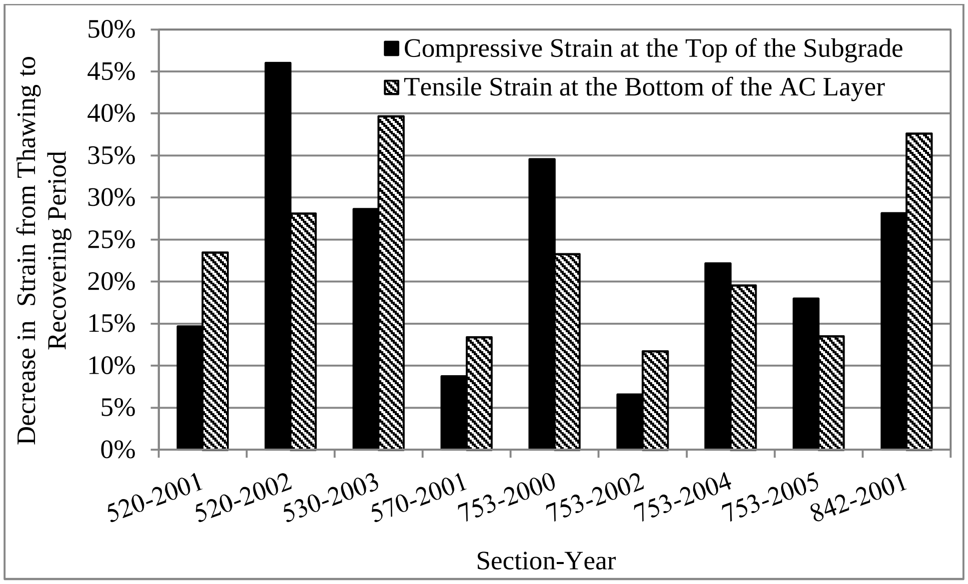


Figure3.4 Percent decrease in ε_c and ε_t from the thawing to recovering for nine cases.

During the thawing season, the pavement structure is weak; thereby any applied load can cause a greater damage to the pavement. Hence, the allowable loads on the pavement are restricted during this period. The two main questions that might rise are: 1) in the case of zero load restrictions, how much decrease is expected in the pavement's load bearing capacity; 2) what is an appropriate load reduction during the critical period of thawing. It will be attempted in this section to address these questions based on the available data.

Pavement critical responses (ε_t and ε_c) from the previous section were used in this section to establish the allowable number of load repetitions to produce 20 percent fatigue cracking (N_f) and rutting failure in the pavement (N_d). In doing so, the Asphalt Institute's fatigue cracking model and the model for rutting failure developed by the U.S. Army Corps of Engineers as presented in Equations 2 and 3, respectively were used *(Huang, 1993 and Wardle et al., 2003)*. When employing the rutting failure model (Equation 3), one should note that only the subgrade modulus is incorporated in the model. Hence, the recovery in the base layer's modulus or potential for rutting in the base layer when thawing occurs in this layer and the subgrade remains frozen although reflected in the predicted ε_c, are not directly incorporated in the model.

$$N_f = 0.0796 * (\epsilon_t)^{-3.291} * E^{-0.854} \qquad (2)$$

Where,

N_f is the maximum number of allowable load repetitions to produce 20 percent fatigue cracking,

ϵ_t is the tensile strain at the bottom of the AC layer,

E is the adjusted AC layer's backcalculated modulus.

$$N_d = [\frac{k}{\epsilon_c}]^b \qquad (3)$$

Where,

N_d is the maximum number of allowable load repetitions,

ϵ_c is the compressive strain at the top of the subgrade layer,

k and b are constants determined using the following equations:

$$k = 1.64 * 10^{-9} * E^3 - 4.31 * 10^{-7} * E^2 + 2.18 * 10^{-5} * E + .00289 \qquad (4)$$

$$b = -2.12 * 10^{-7} * E^3 + 8.38 * 10^{-4} * E^2 - 0.0274 * E + 9.57 \quad (5)$$

Where,

E= the backcalculated subgrade Modulus (MPa).

The increase in the pavement's load bearing capacity from thawing to recovery period is investigated in this section. In doing so, the N_f and N_d during the thawing period are compared to the N_f and N_d during the recovering period. The respective results in terms of percent increase for both N_f and N_d from thawing to recovering period is presented in Figure 3.4. Figure 3.4 indicates that N_f increases by a minimum of 33 percent and a maximum of approximately 80 percent due to seasonal variation in the pavement stiffness. As seen in Figure 3.4 the highest increase in N_d from the thawing to the recovering period happens for Highway 520 in 2002. According to Table 3.3 the backcalculated modulus for the subgrade for this case shows the highest increase (79 percent) from the thawing to the recovery period. Further, Highways 530 in 2003, 753 in 2003 and 2000 show high increases in N_d, which is attributed to the large recovery observed for their base and subgrade moduli as presented in Table 3.3. Further, according to Figure 3.4, N_d increases by a minimum of approximately 45 percent for Highway 753 in 2002 and a maximum of approximately 95 percent for Highway 520 in 2002. Highway 753 in 2002 showed only a 13 percent recovery in the base and 3 percent in the subgrade backcalculated moduli in Table 3.3, while Highway 520 showed 10 percent increase in base and 79 percent increase in the subgrade backcalculated moduli.

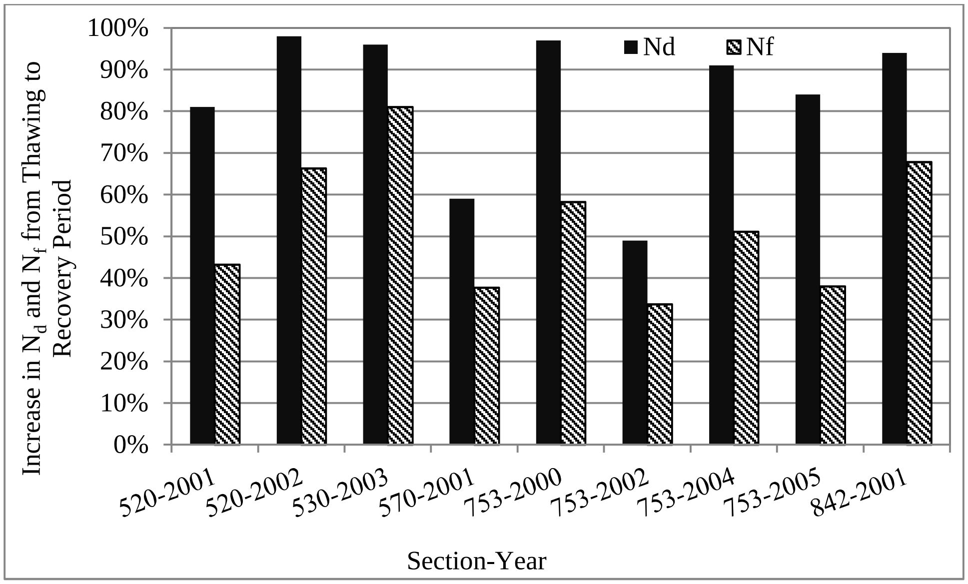


Figure 3.5 Percent increase in N_d and N_f from thawing to recovering period for the nine cases.

3.5 Load Restriction

It was shown in the previous section that N_d for the pavement can decrease as much as 95 percent during the critical period if the maximum allowable load is not limited. Therefore, it seems necessary to take actions toward preventing the excessive damage in the pavement structure. One approach that is practiced by many highway agencies and administrations is to restrict the load level during the critical (thawing) period. As discussed in the previous section, the amount of load restriction is a critical concern, which needs to be addressed. It is attempted in this section to put forward an approach for addressing this concern. In doing so, using the linear elastic models for all the nine cases during the weakening period, the load level was reduced at 10-percent intervals from the typical FWD load of 53 kN. The predicted ε_c from the models were used to establish the increase in N_d. The effect of load reduction on the allowable number of load repetitions in the thawing period is presented in Table3.4 for the fatigue and rutting criteria. As seen in Table3.4 only a 10-percent reduction in the applied load resulted in a 60-percent increase in N_d and 30 percent increase in N_f. Also, a 20-percent reduction in the applied load resulted in 85 percent increase in N_d and 50 percent increase in N_f. Lastly, reducing the load by 50 percent resulted in 95 and 80 percent increase in N_d and N_f, respectively, which is a considerable improvement in pavement life.

Table3.4 Percent Increase in N_d and N_f Due to Different Levels of Load Reduction.

Load Reduction from 53 kN (%)	(%) Increase in Allowable Number of Loads	
	N_d	N_f
10	60	29
20	85	52
30	90	69
40	95	79
50	98	91

3.6 MEPDG Analysis

The effect of load restriction in the thawing period on pavement performance was investigated using the MEPDG Version 1.1. In doing so, one flexible pavement section with a 6-cm AC layer and a 25-cm GBC representative of the five sections modeled using Everstress in the previous section was simulated using the MEPDG. Truck traffic was limited to 70 000/day passes of a two-axle single-unit truck (Class 5 according to the Federal Highway Administration [FHWA] classification) with a 9000 kg (20klbs) single-axle load. This load was selected to represent the legal load limit for single axles in Alberta. Since the five sections are scattered in the central and southern areas of the province, Edmonton

International Airport weather station available in the MEPDG was used to generate the climatic file. The design criteria and all other design input parameters were kept as the default values in the MEPDG.

The applied load was reduced by 25 and 50 percent during four months of April, May, June and July. These four months were selected, since one preliminary run of the MEPDG for Edmonton weather condition showed that the pavement weakening occurs during this period. The MEPDG-predicted pavement life to reach 25 percent alligator cracking, 2000 ft/mile longitudinal cracking, and 0.75 inch rutting at 90 percent reliability was used for the three loading scenarios to investigate the effect of spring load restriction. The effect of load reduction on the three distresses is shown in Figure 3.6. As seen in Figure 3.5, reducing the applied load by 25 percent can increase pavement life 52, 36 and 32 percent based on rutting, longitudinal cracks and alligator cracks criterion, respectively. Also, as seen in Figure 3.6, reducing the load by 50 percent during the thawing period doubles the pavement life based on all three distresses' limit.

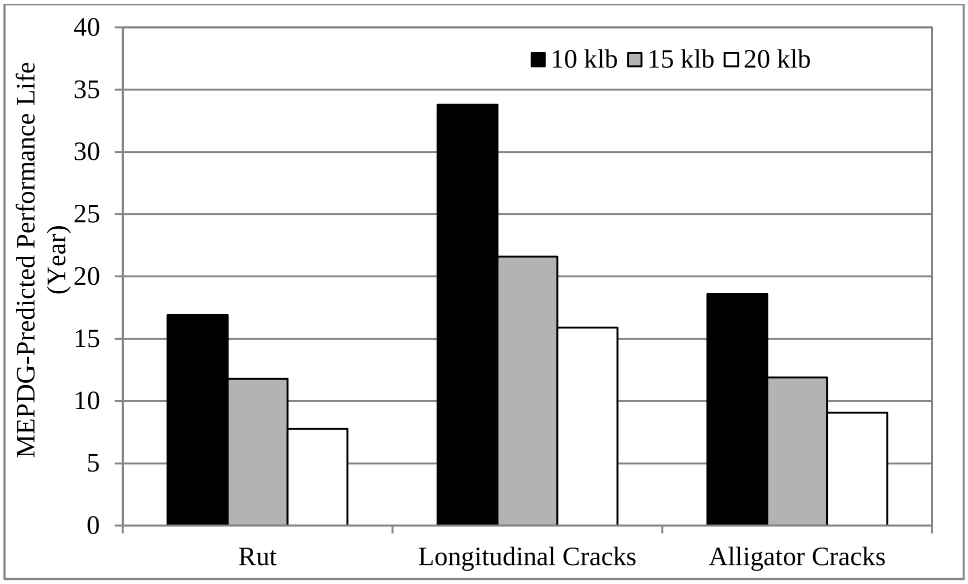


Figure 3.6 Effect of load reduction in thawing period on the MEPDG-predicted performance life.

3.7 Conclusion

The FWD data provided by Alberta Transportation as part of the Spring Road Ban program was screened, resulting in the selection of ten highway sections with the most successive series of tests. The deflection data was used to backcalculate the pavements layers moduli in all years. Multi-linear elastic layer models were developed and validated for the sections that showed a complete weakening and recovering behaviour in different years. A maximum increase of 40 percent was seen in tensile strain at the bottom of the AC layer for the thawing period in comparison to the recovering period. This increase in the compressive strain at the top of the subgrade was 45 percent for highway 520:02 in 2002. The critical predicted strains were used to investigate the level of increase in the allowable number of load repetition for fatigue cracking (N_f) and rutting (N_d) criteria in the thawing period in comparison to the recovering period. A maximum increase of 98 and 88 percent was seen in N_d and N_f, respectively. An approach was proposed to establish the required level of load reduction in the thawing period by decreasing the applied load at 10-percent intervals. It was found that 10-percent load reduction in the thawing period can result in 13 percent increase in N_f and 49 percent increase in N_d. Reducing the load by 50 percent results in more than 80 percent increase in N_f and N_d. Further, based on the MEPDG predictions, applying a maximum load of 10 klb in comparison to 20 klb during the thawing period increases the pavement life approximately 50 percent.

4. Chapter 4: Performance Evaluation of Asphalt overlays in Alberta using Long Term Pavement Performance Specific Pavement Study Sections[1]

[1] *A version of this chapter has been submitted for publication Norouzi, Nassir and Bayat 2012. Canadian Journal of Civil Engineering.*

4.1 Introduction

According to the latest statistics in 2007, the Province of Alberta owned 12 percent of Canada's public infrastructure with a total value of $35.2 billion. Highways and roads made up approximately 62 percent of Alberta's total infrastructure with a value of $21.82 billion (Gangon et al. 2007). The valuable network of highways and roads in Alberta needs to be maintained at an operational condition and acceptable ride quality. In doing so, Alberta Transportation invests nearly 50 percent of its annual budget to rehabilitate and maintain Alberta's highway network (Soleymani et al. 2002). Based on the latest data available in Alberta Transportation's Pavement Management System (PMS) in 2010 approximately 50 percent of Alberta's highway network length was overlaid between year 1985 and 2010, implying that overlay construction is commonly practiced in Alberta as a rehabilitation strategy. Further, the average age of the network is 15.6 years (Gangon et al. 2007), with 56 percent of the entire network length being 10 years or older, requiring rehabilitation in the future. Investigation and evaluation of in-service overlay performance over the years can greatly benefit the province's future decisions concerning several design factors, such as asphalt mixture, pavement structure and construction practices. A valuable source of information for such study is the Long-Term Pavement Performance (LTPP) program's (SPS) 5 in Alberta. The SPS 5 sections in Alberta joined the LTPP program in 1990 and has been regularly monitored and tested since.

Each SPS 5 section in the LTPP program consists of nine 150-meter sections (Sections 501 to 509). The first section (501) is the control section, which receives no treatments during the monitoring period, while the other eight sections are overlaid with two different asphalt mixtures, at two different thicknesses and under two different pre-overlay conditions (milled and not milled). Table 4.1 presents a summary of the overlay characteristics for the nine sections. In Table 4.1, the Reclaimed Asphalt Pavement (RAP) is an asphalt mixture which includes 30 percent reclaimed asphalt material.

Table 4.1 Overlay Type and Thickness for Each SPS 5 Section In The LTPP Program.

Section No.	Overlay Asphalt Mixture Type	Overlay Thickness (mm)	Milled prior to Overlay
501	Control Section- No overlay		
502	RAP	50	No
503	RAP	125	
504	Virgin	125	
505	Virgin	50	
506	Virgin	50	Yes
507	Virgin	125	
508	RAP	125	
509	RAP	50	

To date, several studies have been conducted on the SPS 5 sections across North America to investigate the effect of different overlay strategies on pavement performance. West et al. (2011) used the latest recorded data for all the SPS 5 sections (total of 16 states in the United Sates and two provinces in Canada) to statistically compare the distresses observed for the nine sections. It was concluded in their study that mixture type and milling prior to overlay construction can greatly affect the pavement performance in terms of fatigue, transverse and longitudinal cracking. Their study also showed that overlay thickness does not have a significant effect on longitudinal cracking. In another study, Hall et al. (2003) used the most recently updated measurements of the International Roughness Index (IRI) for all the SPS 5 sections across North America. Their study revealed that no significant difference exist between long-term IRI for the RAP versus virgin, as well as milled versus not-milled overlay sections. The effect of pre-overlay IRI, overlay age and average annual temperature was found to be considerable on long-term IRI. They also identified a correlation between the annual precipitation and the increase in long-term IRI for the virgin overlays. According to Hall et al. (2003) the factors affecting long-term cracking are pre-overlay cracking, age and traffic loads during the service life; while mixture type or milling proved to have no effect on long-term cracking. In another effort to investigate the SPS 5 sections' performance, Carvalho et al. (2011) used the distress data available over the pavement's life from Arizona's SPS 5 sections. They reported that in the long-run, sections overlaid with the virgin mixture are smoother than those overlaid with the RAP mixture. The latter sections also demonstrate higher rutting and longitudinal cracking. They also concluded that milling prior to overlay construction improves fatigue cracking performance and found that thin overlays (50 mm) demonstrate better short-term rutting; while thick overlays (150 mm) outperform the thin overlays in terms of long-term rutting and all other performance indicators.

The present study provides an in-depth evaluation of the performance of the SPS 5 sections in Alberta using the Falling Weight Deflectometer (FWD) test data and distress records available for the sections between 1990 and 2006. The available data is implemented to first, investigate the statistical difference between the structural responses of different overlays to FWD loads in different years. Second, briefly compare long-term pavement performance in terms of alligator, transverse and longitudinal cracking and indicators, such as IRI, for the nine sections. Lastly, the SPS 5 sections are simulated using the MEPDG to compare the MEPDG predictions to the distresses collected in the field for the SPS 5 sections in Alberta.

4.2 Alberta's SPS 5 Section

The LTPP SPS 5 sections in Alberta are located in the westbound truck lane of Highway 16, Control Section 6, at approximately 3.5 km west of junction with Highway 32 and 36 km east of Edson (Kwan & Stoski, 2007). The road section is considered a rural principal arterial intestate according to the LTPP database DVD Version 26. As mentioned previously each SPS 5 section consists of nine sections. It should be noted that the sections' order of appearance does not follow the sections' numbering. Table 4.2 presents the coordinates of the starting point for each section. Also, some variations exist in the structural design of the sections. As presented in Table 4.2 Sections 503 to 507 include a 50- to 80-mm asphalt Treated Base (TB) layer, while Sections 502, 508 and 509 do not include this layer. Existing AC layer also varies between 125- and 165 mm among the nine sections.

Table 4.2 Coordinates And Pavement Structure For Each SPS 5 Sections In Alberta.

Section	Latitude	Longitude	Pavement Structure
501	53.58721	-116.01959	135 mm AC*/70 m TB***/380 mm GBC**
502	53.57894	-116.04516	50 mm AC overlay/135 mm existing AC/380 mm GBC
503	53.58179	-116.03395	125 mm AC overlay/155 mm existing AC /70 mm TB/325 mm GBC
504	53.58277	-116.03169	125 mm AC overlay/155 mm existing AC/71 mm TB/290 mm GBC
505	53.58657	-116.02227	50 mm AC overlay/155 mm existing AC/70 mm TB/290 mm GBC
506	53.58586	-116.02464	50 mm AC overlay//125 mm existing AC/80 mm TB/325 mm GBC
507	53.58504	-116.02697	125 mm AC overlay/145 mm existing AC/50 mm TB/325 mm GBC
508	53.5808	-116.03686	125 mm AC overlay/165 mm existing AC/390 mm GBC
509	53.57963	-116.04187	50 mm AC overlay/165 mm existing AC/390 mm GBC

*Asphalt Concrete **Granular Base Course ***Treated Base

As part of the LTPP program, traffic data in terms of the Equivalent Single Axle Load (ESAL) was available for the test sections on the LTPP DVD Version 26 for the period between 1999 and 2008. As seen in Figure 4.1, the kESAL/year is consistent for all the nine sections in each year. As shown in Figure 4.1, kESAL/year for all the sections varies between 233 and 461 over the 10 years, with a one-percent average growth rate.

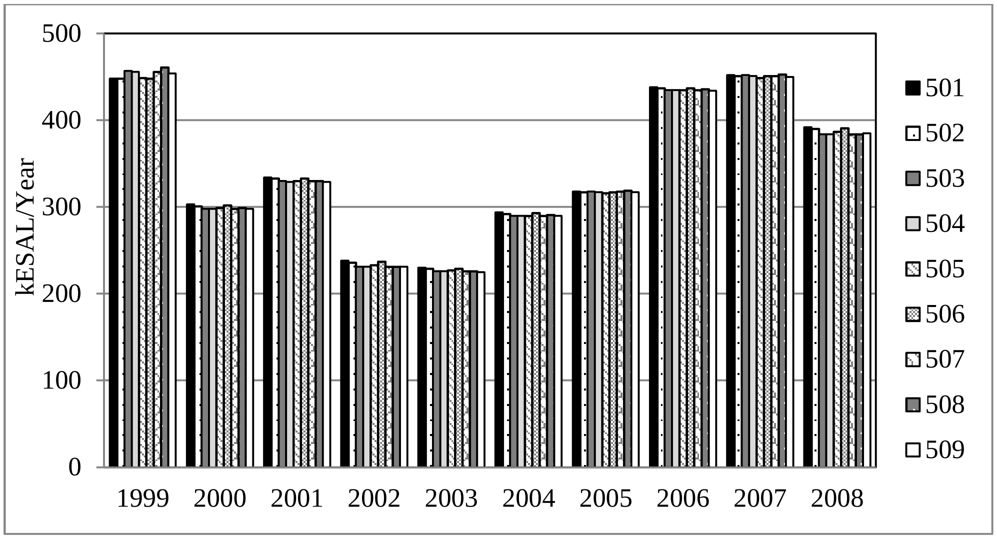


Figure 4.1 kESAL/year from 1999 to 2008 for each SPS 5 in Alberta extracted from LTPP database.

The LTPP SPS 5 sections of Alberta joined the LTPP program in May 1990 and the construction of the new overlay was completed in September 1990 (again, the control section did not receive an overlay). Since the completion of the overlay construction, each of the eight sections received different maintenance depending on their conditions. The most common treatments applied to the sections were crack sealing and pothole patching. Two of the sections, 502 and 509, reached the end of their service life in 2006, 16 years after the overlay construction and were overlaid in fall 2006. This study focuses on the evaluation of the test sections during the period between 1990 and 2006.

4.3 Effect of overlay on Structural Response

Every two years, FWD tests were performed on Alberta's LTPP SPS 5 sections, since the completion of the overlay construction in 1990. The deflection measurements during the FWD tests are used herein to investigate the effects of different overlays on pavement structural responses. For this purpose, peak FWD deflection (measured underneath the plate load) is used as the structural response indicator for the pavement. The FWD tests on SPS 5 sections were performed at four different drop heights, resulting in four impact loads. A similar trend was observed in the peak deflections along each test section under the four load levels. Therefore, deflections corresponding to only 40 kN load, equal to one ESAL, were used for the analysis in this section. The deflections were normalized prior to analysis whenever the applied load deviated from the target load by more than five percent, according to the American Standard for Testing and Materials (ASTM) D5858, Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory (ASTM 2008).

For each test section the FWD tests were performed at approximately 15-metre intervals in each year. First, the peak deflection profile for each individual section at each 15-meter offset was plotted for each year. Figure 4.2, corresponding to Section 503, presents an example of these plots. As seen in Figure 4.2, the deflections show minimal variations along the test section. The slight variations noted along the test section repeat consistently in all years. To further investigate the consistency of the pavement response along the test sections, the possible outliers were visually identified for each test section. The significance of the difference between the two sets of deflection data, one including the outliers and the other excluding the outliers, was established using a paired t-test for each test year according to ASTM D5858. The results of the t-tests revealed that the visually-determined outliers do not have a significant effect on the average deflection data along each test section.

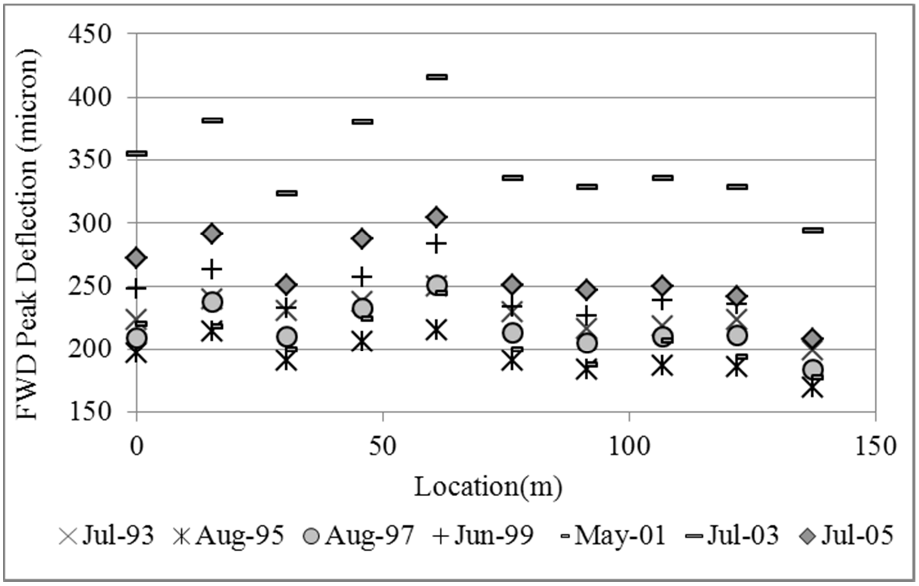


Figure 4.2 FWD peak deflections during each test year along Section 503.

Table 4.3 presents the average peak deflection for each section for each test year. As seen in Table 4.3 the seven-year average peak deflection for the control section (Section 501) is 441 micron, which is higher than all other sections, showing the effectiveness of all overlay strategies on pavement structural response. Based on the seven-year average values, the overlay for Section 508 (thick, milled, RAP) is the most effective strategy with a 54 percent decrease in deflection in comparison to the control section. The minimum effect is observed for Section 502 (thin, not-milled, RAP) with just a seven percent improvement in deflection with respect to the control section.

Table 4.3 Average Peak Deflections for Each Section in Each Year.

	Average Peak Deflections(microns)								
Section / Year	501	502	503	504	505	506	507	508	509
July-1993	486	462	224	344	283	379	324	209	321
August-1995	398	401	192	165	233	314	251	181	295
August-1997	480	324	213	353	301	420	330	190	277
June-1999	413	320	240	209	263	376	315	213	226
May-2001	441	375	205	183	250	368	295	195	264
July-2003	470	520	345	297	275	394	240	237	380
July-2005	398	471	257	230	259	363	193	170	313
Average	441	410	239	254	266	373	278	199	297

It should be noted that the reason why peak deflections vary from one year to another at the same location could be the difference in the climatic conditions during testing, which results in varying pavement temperature and modulus. To eliminate the effect of environmental conditions on the FWD deflections from year to year at one location, the percent differences between the peak deflections for each section with respect to the control section were used: $100 \times (\frac{PeakDef_{Control} - PeakDef_{Overlay}}{PeakDef_{Control}})$. Figure 4.3 presents the percent differences for all seven years. As seen in Figure 4.3, the positive differences for all the sections in all years (except for Section 502 in 2003 and 2005) imply that the overlay has effectively improved the structural capacity of the pavement. The percent difference between the control section and the overlaid sections is decreasing over the years for six sections (at a rate of approximately one percent per year on average), implying that the effectiveness of the overlay on the pavement's structural capacity is gradually decreasing. The exceptional case is

Section 502, which shows higher peak deflections in 2003 and 2005 in comparison to the control section. This behaviour can be an indicator of a complete structural failure for Section 502, which resulted in another overlay construction in 2006 as mentioned in the previous section. Overlay construction has been the most effective for the thick and milled sections (507 and 508). These sections show a percent difference of approximately 55 percent during the entire monitoring period.

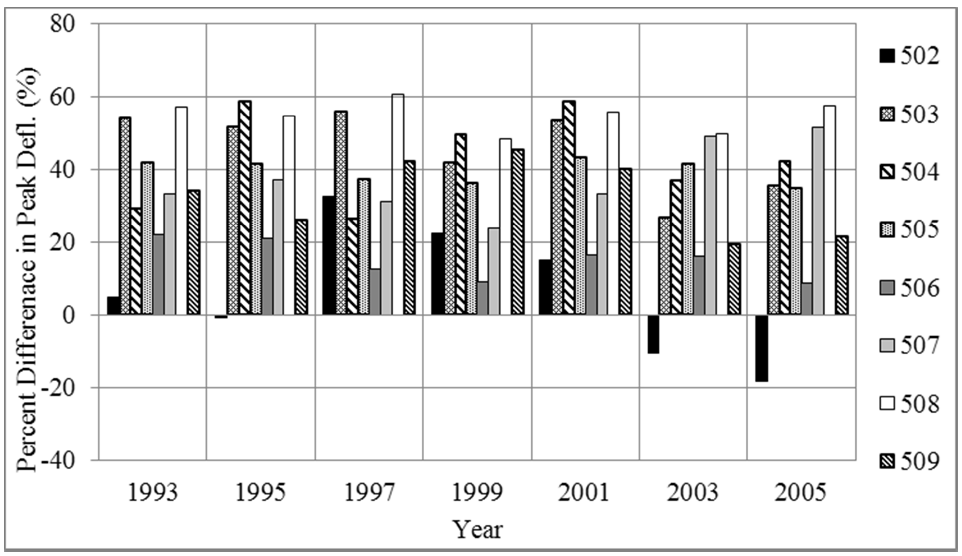


Figure 4.3 Percent difference in peak FWD deflections for each SPS 5 section with respect to the control section.

A series of paired t-tests were conducted on each year's peak deflections for each section together with a paired t-test on the seven-year average peak deflections to investigate the effect of each overlay strategy on the pavement structural response during the monitoring period. The sections were divided into three sets of pairs to investigate the effect of: asphalt mixture type (virgin versus RAP), thickness (50 versus 125 mm) and conditions prior to overlay construction (milling versus no milling) on the pavement's structural response. A significance level (α) of 0.05 and a critical t-stat value of 2.17 was used to identify the pairs that were significantly different from one another. The results of the t-tests for each pair are provided in Table 4.4 through Table 4.6.

Table 4.4 presents the results of the t-tests for the pairs with the virgin and RAP asphalt mixtures for the two overlay thicknesses (thin: 50 mm and thick: 125 mm) and two pre-overlay conditions (not-milled and milled). It must be noted that the AC grade for the RAP and the virgin mixture was reported as AC 5 and AC 10, respectively in West et al. (2011). This implies that the AC mixture including RAP is softer compared to the virgin mixture, hence should show higher deflections. As seen in Table 4.4, for the not-milled thin overlays, the virgin overlay (Section 505) in comparison to the RAP overlay (Section 502) shows significantly lower peak deflections (t-stat positive and greater than t-critical) in all years except for 1997. The t-test results between the seven-year average peak deflections for the not-milled thin sections shows that the RAP section had significantly higher deflections than the virgin section. It must be noted that in addition to mixture type the existence of a 70-mm TB layer for Section 505 contributes to the smaller deflections seen for this section compared to Section 502. For the not-milled thick overlays the virgin overlay (Section 504) in comparison to the RAP overlay (Section 503) shows two exceptional negative t-stat values in 1993 and 1997 (with absolute values greater than t-critical). The result of the t-test on the seven-year average deflections shows that no significant difference exists between the RAP and virgin for the not-milled thick sections.

For the milled sections the RAP sections are outperforming the milled virgin overlays for both thick and thin sections. Also, the results of the t-test between the averages of all years' peak deflections, reveals that for all the milled sections the RAP mixture performs better than the virgin mixture.

Table 4.4 Effect of Asphalt Mixture Type (RAP versus Virgin) On Pavement Structural Response.

Overlay Thickness	FWD Test Year	T-Test Results for RAP versus Virgin			
		Not Milled		Milled	
		T-Stat	P-Value	T-Stat	P-Value
Thin	1993	10.55	0.0000	-4	0.0019
	1995	12.78	0.0000	-1.31	0.1248
	1997	1.85	0.0598	-8.1	0.0000
	1999	4.05	0.0016	-8.1	0.0000
	2001	9.22	0.0000	-6.2	0.0000
	2003	11.69	0.0000	-0.7	0.2687
	2005	12.54	0.0000	-3.3	0.0025
	Average of all Years	4.7	0.0003	-3.4	0.0024
Thick	1993	-9.77	0.0000	-14.5	0.0000
	1995	4.53	0.0008	-12.3	0.0000
	1997	-10.6	0.0000	-16.9	0.0000
	1999	3.44	0.0009	-11.5	0.0000
	2001	2.05	0.0055	-14.2	0.0000
	2003	3.23	0.0029	-5.5	0.0002
	2005	2.34	0.0049	-4.1	0.0015
	Average of all Years	-0.4	0.3568	-3.7	0.0027

When comparing two sections with similar asphalt mixtures and different overlay thicknesses the thicker overlay is expected to perform better under any applied load.

Table 4.5 shows the result of the comparison between the thick and thin overlay sections. The difference between the thin and thick overlays is very well observed for the RAP overlay (both milled and not milled), since the t-stats are all positive and greater than t-critical. For the virgin, not-milled overlay there are three years during which the t-stats are negative, implying that the thin overlay showed smaller deflections. For the milled, virgin overlay, however, the thicker section performs better than the thin section, as expected. Overall, the conclusion from Table 4.5 agrees with the expectations of thick overlay showing lower deflections in comparison to thin sections.

Table 4.5 Effect of Overlay Thickness (50 Mm versus 125 Mm) On Pavement Structural Response.

Mixture Type	FWD Test Year	T-Test Results for Thin versus Thick Overlays			
		Not Milled		Milled	
		T-stat	P-value	T-stat	P-value
RAP	1993	14.06	0.0000	16.2	0.0000
	1995	15.86	0.0000	19.5	0.0000
	1997	9.94	0.0000	11.9	0.0000
	1999	5.75	0.0000	2.6	0.0169
	2001	12.35	0.0000	11.7	0.0000
	2003	8.09	0.0000	13.6	0.0000
	2005	12.15	0.0000	20.1	0.0000
	Average of all Years	4.8	0.0003	4.8	0.0003
Virgin	1993	-4.94	0.0002	3.6	0.0061
	1995	11.66	0.0000	4.3	0.0009
	1997	-3.65	0.0059	5	0.0002
	1999	5.81	0.0000	2.6	0.0065
	2001	6.41	0.0000	4.2	0.0009
	2003	-1.62	0.0069	8.4	0.0000
	2005	2.85	0.0049	11.9	0.0000
	Average of all Years	0.4	0.357	4.1	0.0012

Table 4.6 present the effects of milling on the pavement structural response for the sections overlaid with the virgin and RAP mixtures and at the two thicknesses of 50 and 125 mm. As seen in Table 4.6, the t-stats are all noticeably higher than t-critical and have negative values proving that the thin virgin sections, which were milled prior to overlay construction, have higher peak deflection in comparison to sections which were not milled. For the thick virgin sections for

the years 1993 and 1997 no significant difference is observed between the milled and not milled sections. However, for the years 1995, 1999 and 2001 high negative t-stat values show that not-milled sections showed less deflections than the milled sections. For years 2003 and 2005, on the other hand, the positive t-stat values show that the milled section outperformed the not-milled section. The t-test on the seven-year average deflection for the thick, virgin sections shows no significant difference between the milled and not milled sections. No overall conclusion can be drawn regarding the effect of milling on the peak deflections for thick virgin sections. For the RAP mixture the positive t-stat values in Table 4.6 show that the milled sections outperform the not-milled sections for both thin and thick overlays in contrast with the virgin mixture. The results of the t-tests performed on the seven-year average peak deflection show that for the thick sections there is no significant difference between milled and not milled sections, while for the thin sections, milling has different effects depending on the mixture type. For the virgin sections milling is not effective, while for the RAP section milling has a positive effect.

Table 4.6 Effect of Milling Prior To Overlay Construction on Pavement Structural Response.

Overlay Thickness	FWD Test Year	T-Test Results for Not-Milled versus Milled Sections			
		Virgin Mixture		RAP Mixture	
		T-Stat	P-Value	T-Stat	P-Value
Thin	1993	-6.84	0.0000	8.2	0.0000
	1995	-5.64	0.0000	7.92	0.0000
	1997	-6.47	0.0000	4.16	0.0012
	1999	-5.82	0.0002	7.3	0.0000
	2001	-6.03	0.0000	8.37	0.0000
	2003	-5.95	0.0000	6.66	0.0000
	2005	-6.68	0.0000	9.67	0.0000
	Average of all Years	-7.1	0.0000	3.3	0.0025
Thick	1993	1.45	0.0875	2.62	0.0047
	1995	-13.42	0.0000	2.07	0.0058
	1997	1.68	0.0689	3.31	0.0025
	1999	-10.17	0.0000	3.86	0.0023
	2001	-10.43	0.0000	1.3	0.1248
	2003	5.27	0.0002	9.26	0.0000
	2005	4.68	0.0001	8.91	0.0000
	Average of all Years	-0.6	0.2897	1.8	0.0079

4.4 Effect of Overlay on Pavement Distresses

Based on the availability of data in the LTPP database, alligator, transverse and Non-Wheel Path (NWP) longitudinal cracking, as well as IRI for the pavement sections were used in the study to compare the performance of different overlaid sections. Distress records were not available for the control section, therefore, this section was only included in the IRI analysis. The effect of mixture type, overlay thickness and milling prior to overlay construction on long-term alligator, transverse and NWP longitudinal cracking is presented in Table 4.7. The average

2006-alligator cracking for all four sections with the virgin mixture are compared to the four RAP sections I Table 4.7. As seen in Table 4.7, the sections with the RAP mixture show approximately nine percent more alligator cracking in comparison to the sections with the virgin mixture. The RAP is softer compared to the virgin mixture, resulting in more long-term fatigue cracking. The effect of milling on alligator cracking development is more pronounced in comparison to the effect of asphalt mixture. As seen in Table 4.7 the not-milled sections show approximately 17 percent more alligator cracking with respect to the milled sections. Lastly, the thinner sections show approximately 30 percent more alligator cracking compared to the thick sections, implying that the overlay thickness is the most influential factor for alligator cracking. The results comply with the findings of West et al. (year) and Carvalho et al. (year) who concluded that mixture type, milling, as well as overlay thickness can greatly affect alligator cracking.

The effect of mixture type, overlay thickness and milling prior to overlay construction on transverse cracking in 2006 was also investigated in Table 4.7. Average transverse cracking in 2006 for all the four sections with the RAP mixture, which have a softer AC grade (AC 5), were proven to be half of that for the sections with the virgin mixture with a stiffer AC grade. Also, the sections which were not milled prior to the overlay construction, demonstrated 20 percent more transverse cracking in comparison to the sections which were milled. The effect of overlay thickness on transverse cracking is not as pronounced as its effect on alligator cracking. The thinner sections show 18 percent more transverse cracking with respect to the thick sections. These observations agree with West et al.'s findings that mixture type and milling can affect transverse cracking.

The effect of asphalt mixture type, milling prior to overlay and overlay thickness on NWP longitudinal cracking in 2006 was investigated in Table 4.7. The sections made with the RAP mixture demonstrate approximately 15 percent more longitudinal cracking compared to the sections with the virgin mixture. Milling shows no effect on longitudinal cracking. Carvalho and Hall et al. also concluded there is no difference in longitudinal cracking for the milled and not-milled sections. Finally, thick sections show 15 percent more longitudinal cracking with respect to the thin sections.

Table 4.7 Effect of Different Overlay Strategies on Alligator, Transverse and NWP Longitudinal Cracking in 2006.

Distress Type	Mixture Effect		Milling Effect		Thickness Effect	
	RAP	Virgin	Not-Milled	Milled	Thin	Thick
Average Alligator Cracking (% area)	25	22	29	24	22	31
Average Trans. Cracking (No. of Cracks/Section)	31	69	60	48	50	59
Average NWP Long. Cracking (m)	135	115	124	127	114	135

Figure 4.4 shows the IRI development in the right wheel path for the SPS 5 sections. Based on the Annual Average Daily Traffic (AADT) for the road section, Alberta Transportation uses an IRI trigger value of 1.9 m/km to identify

the pavements that can be considered for rehabilitation (Alberta Transportation, 2006). As seen in Figure 4.4, of the nine sections, four sections (including the control section) demonstrate IRI values above 1.9 m/km in 2006 and 2005. For the rest of the sections and years IRI is well within the acceptable range. The control section shows the highest IRI of all the section over the monitoring period, since it never received an overlay. As seen in Figure 4.4 the post-overlay IRI values in 1990 are not the same for all the sections. In order to eliminate the effect of initial IRI on long-term IRI, IRI_{change} which is the difference between IRI in 2006 and the post-overlay IRI in 1990 was used.

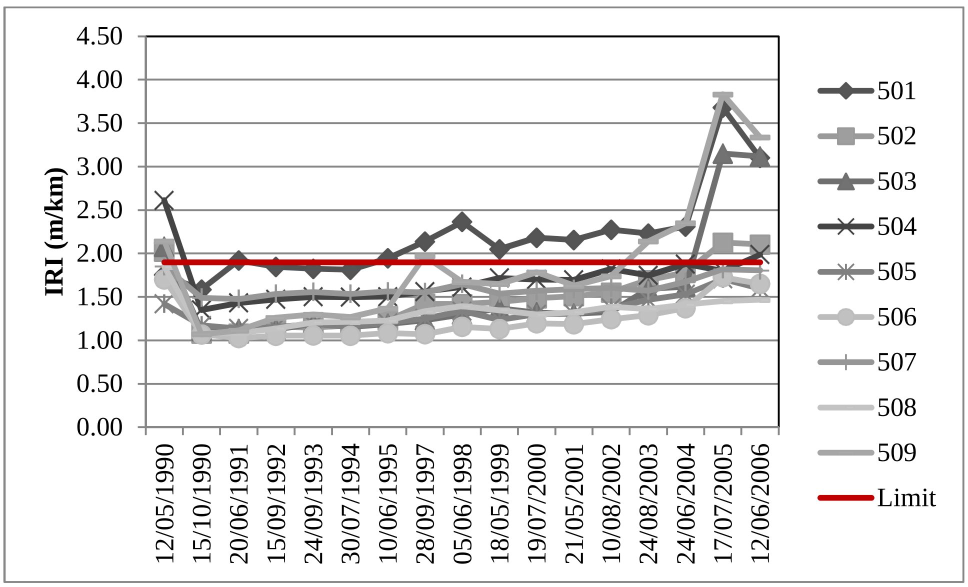


Figure 4.4 IRI measurements over time for the nine SPS 5 sections in Alberta from LTPP database.

Figure 4.5 shows IRI_{change} for the nine sections. Those three sections which received overlays with the RAP mixture (502, 503 and 509) demonstrate the highest IRI_{change} of 1, 2 and 2.27 m/km, respectively. Section 508, which was overlaid with the RAP mixture, is an exception from this trend and does not show a high IRI_{change} relative to the other sections. On the other hand, those sections that received the virgin mixture demonstrate considerably lower IRI_{change} of approximately 0.5 m/km. This agrees with Carvalho et al.'s findings that the sections with the virgin mixture are smoother in the long-term in comparison to the RAP sections.

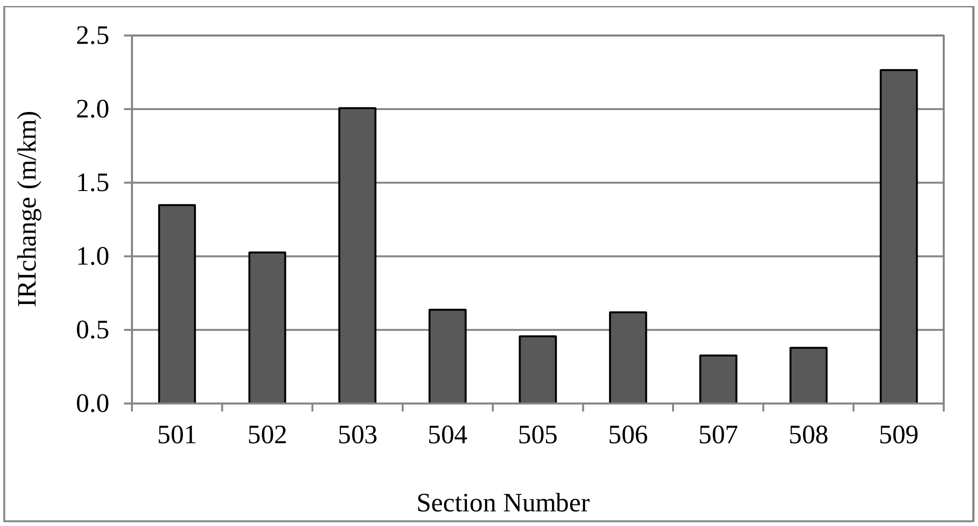


Figure 4.5 IRI_{change} for the nine SPS 5 sections between 1990 and 2006.

4.5 Comparison with MEPDG predictions

4.5.1 MEPDG Simulations

The test sections on the LTPP program played a key role in the development of MEPDG performance models and are the best source for local calibration of the models. In this section the MEPDG Version 1.1 was used to predict the pavement performance for the Alberta's SPS 5 sections. The correspondence between the MEPDG-predicted performance indicators and those observed in the real world for the nine sections was investigated. The nine different sections were simulated using the MEPDG, based on the information presented in Table 4.2 for their structure. The RAP and virgin mixtures were differentiated using AC grade AC 5 for RAP and AC 10 for the virgin sections. The default values available in the MEPDG were used to define all the required design inputs except for those listed in Table 4.8 .

Table 4.8 Design Inputs and Their Values Used In the MEPDG.

Design Input	Value	Notes
Construction year	Control section:1976 Overlaid sections:1990:	From the LTPP DVD Version 26 and Alberta Transportation PMS
Asphalt grade	AC-5 to represent RAP mixture; AC-10 to represent virgin mixture	From west et al. (2011)
Initial AADT	8,000	AADT is reported as 9,980 in the AT's PMS in 2010, AADT in 1990 was backcalculated based on the average growth rate from Figure 4.1.
Condition before overlay	Fair or poor	Defined based on the pre-overlay IRI value for each section
Milling thickness	50 mm (2 inches)	From the LTPP DVD Version 26
Climate data	Edson weather station available in the MEPDG	The climate data covers the period between 1960 and 1969. The weather station is located 36 km away from the test section.
Initial IRI	1.07-1.59 m/km	IRI was measured after overlay construction for each section, from LTPP database.

4.5.2 MEPDG Predictions

Predicted alligator cracking was found to be sensitive only to the pavement structure and overlay thickness. The predicted cracking was the same for all the thick sections at 35 percent and for all the thin sections at a lower level of 29 percent. The MEPDG predictions for alligator cracking do not comply with the measured cracking, which showed sensitivity to both mixture type and milling, as discussed in the previous section. The predicted-transverse and longitudinal cracking were zero for all the sections. Figure 4.6 shows the observed and predicted IRI for each year for all the sections. As seen in Figure 4.6, the MEPDG-predicted and observed IRI agree with a R-squared of approximately 40 percent.

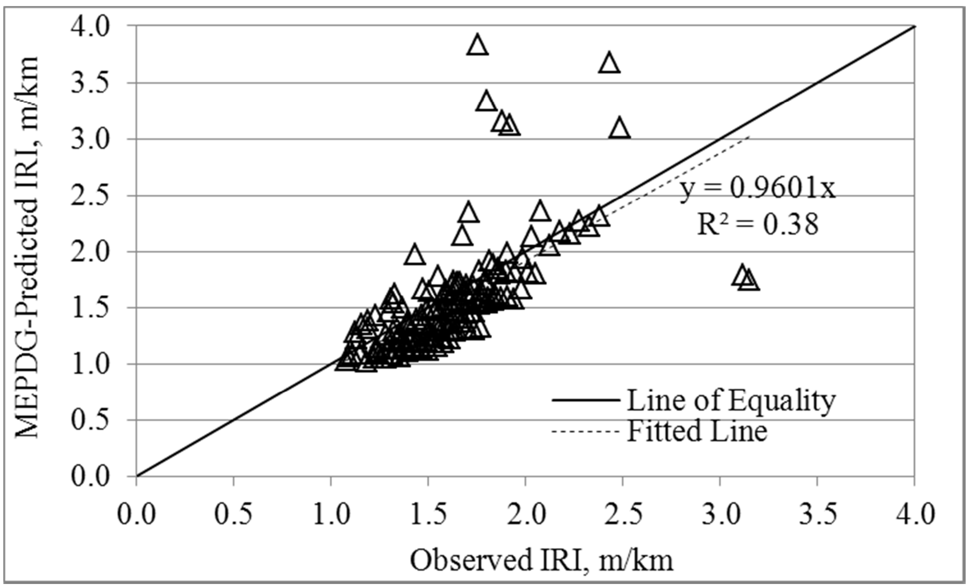


Figure 4.6 Comparison of the MEPDG-predicted and observed IRI_{change} for each year.

The MEPDG-predicted total pavement rut depth progression over the design life is presented in Figure 4.7 for all the nine sections. Figure 4.7 shows that none of the sections reached the MEPDG rut failure criterion of 19 mm. The thin sections with the RAP mixture (milled and not-milled) show the highest predicted rut values, while the thick not-milled sections (RAP and virgin mixture) show the lowest predicted rutting. It is worth noting that the predicted rutting is more sensitive to the pavement structure than the overlay mixture or pre-overlay conditions. Sections 502 and 509, which are thin overlay and do not have a TB layer in the pavement structure, show similar trends and Sections 505 and 506 with thin overlay and a TB layer demonstrate similar trends. Thick overlay sections exhibit similar behavior except for Section 508, which does not have a TB in the structure, showing the effect of pavement structure on the MEPDG predictions for rutting. The Control section has been in place since 1976. In 1990 the predicted rutting for the control section is approximately nine millimetres, these values increase only one millimetre during the next 16 years as seen in Figure 4.7.

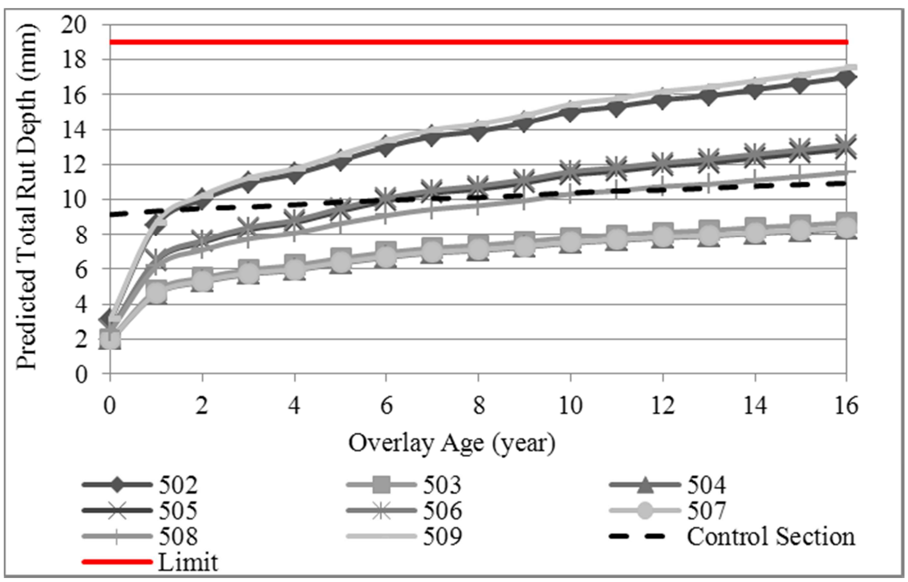


Figure 4.7 MEPDG-predicted total pavement rutting development over the monitoring period.

4.6 Conclusions

The data available for the LTPP SPS 5 sections were used to investigate the performance of asphalt overlays in Alberta's climate conditions. The FWD deflection data was used to compare the effect of each overlay strategy on the structural response of the pavement. The long-term performance of each overlay strategy was investigated using the distress data obtained from the LTPP database.

Alberta's SPS 5 sections were simulated using the MEPDG to investigate the accuracy of the performance predicted by MEPDG. The following conclusions were drawn in the study.

1- The control section showed higher FWD peak deflections for seven years of testing in comparison to the overlaid sections, which shows the effectiveness of all overlays. The difference between the peak deflection for the overlaid sections and the control section decreased during the sixteen year monitoring period, implying that the effectiveness of overlay decreases over time.

2- T-tests between the paired overlay sections showed that thicker overlays show significantly smaller deflections under the FWD load. Milling was the most effective for the thin overlays with the virgin mixture. A firm conclusion could not be drawn on the effect of RAP in the AC mixture on the FWD deflections.

3- Overlay thickness is the most effective factor in the progression of alligator cracking. Seventy-five millimetres of asphalt overlay decreased long-term alligator cracking by 30 percent. Including the RAP in the AC mixture resulted in 11 percent more alligator cracking, also milling prior to overlay reduced alligator cracking by 22 percent.

4- The sections with the RAP mixture showed half long-term transverse cracking than those with the virgin mixture. Milling proved to decrease transverse cracking by 25 percent. Also, the thin sections showed 18 percent more transverse cracking with respect to the thick sections.

5- Higher IRI increase was observed for the sections with the RAP mixture in comparison to the sections with the virgin mixture, with the exception of one thick, milled section with RAP (508), which showed the same behaviour as the corresponding section with the virgin mixture.

6- The MEPDG-predicted IRI and observed IRI showed an agreement at a coefficient of correlation of approximately 40 percent. Also, total pavement rutting predicted by the MEPDG were sensitive the pavement structure and overlay thickness rather than overlay AC mixture or pre-overlay conditions.

5. Chapter 5: Conclusions

5.1 Overall Conclusion

Highway network of the Province of Alberta is the major component of the Province's infrastructure system. This study tries to find solutions to preserve the valuable highway network of the Province of Alberta, maintain it and keep it in operational condition.

The first part of the study in chapter three, investigates the effect of thaw weakening period in early spring on pavement strength and consequently pavement life. For this purpose the FWD data provided by Alberta Transportation which is a part of Spring Road Ban program was screened, resulting in the selection of ten highway sections with the most successive series of tests.

The deflection data was used to backcalculate pavement layers' moduli once in thawing period and once in recovering period for the sections that showed a complete weakening and recovering behaviour in different years. A maximum decrease of 40 percent was seen in tensile strain at the bottom of the AC layer for the thawing period in comparison to the recovering period. This drop in the compressive strain at the top of the subgrade was 45 percent for highway 520:02 in 2002. The critical predicted strains were used to investigate the level of increase in the allowable number of load repetition for fatigue cracking (N_f) and rutting (N_d) criteria in the thawing period in comparison to the recovering period. A maximum increase of 98 and 88 percent was seen in N_d and N_f, respectively.

Also, an approach was proposed to establish the required level of load reduction in the thawing period by decreasing the applied load at 10-percent intervals. It was found that 10-percent load reduction in the thawing period can result in 13 percent increase in N_f and 49 percent increase in N_d. Reducing the load by 50 percent results in more than 80 percent increase in N_f and N_d.

Further, based on the MEPDG predictions, applying a maximum load of 10 klb in comparison to 20 klb during the thawing period increases the pavement life approximately 50 percent.

The second part of study in chapter four focuses on evaluating different asphalt overlay construction strategies in the Province of Alberta. For this purpose, the data collected under Long Term Pavement Performance program in the Alberta SPS 5 section was used.

First, The FWD deflection data was used to compare the effect of each overlay strategy on the structural response of the pavement. The control section showed higher FWD peak deflections for seven years of testing in comparison to the overlaid sections, which shows the effectiveness of all overlays. The difference between the peak deflection for the overlaid sections and the control section decreased during the sixteen year monitoring period, implying that the effectiveness of overlay decreases over time. Also, T-tests between the paired overlay sections showed that thicker overlays show significantly smaller deflections under the FWD load. Milling was the most effective for the thin overlays with the virgin mixture. A firm conclusion could not be drawn on the effect of RAP in the AC mixture on the FWD deflections.

Second, long term performance of each overlay strategy was investigated using the distress data collected during 16 years of monitoring period from 1990 to 2006. It was found that Overlay thickness is the most effective factor in the progression of alligator cracking. Seventy-five millimetres of asphalt overlay decreased long-term alligator cracking by 30 percent. Including the RAP in the AC mixture resulted in 11 percent more alligator cracking, also milling prior to overlay reduced alligator cracking by 22 percent. In addition, the sections with the RAP mixture showed half long-term transverse cracking than those with the virgin mixture. Milling proved to decrease transverse cracking by 25 percent. Also, the thin sections showed 18 percent more transverse cracking with respect to the thick sections. Lastly, Higher IRI increase was observed for the sections with the RAP mixture in comparison to the sections with the virgin mixture, with the exception of one thick, milled section with RAP (508), which showed the same behaviour as the corresponding section with the virgin mixture.

Finally, the long-term performance of each overlay strategy was investigated using the distress data obtained from the LTPP database. Alberta's SPS 5 sections were simulated using the MEPDG to investigate the accuracy of the performance predicted by MEPDG. The MEPDG-predicted IRI and observed IRI showed an agreement at a coefficient of correlation of approximately 40 percent. Also, total pavement rutting predicted by the MEPDG were sensitive the pavement structure and overlay thickness rather than overlay AC mixture or pre-overlay conditions.

5.2 Future Recommendations:

For pursuing the findings of this study, author recommends following:

- Performing frequent FWD tests on limited locations every year to have a better understanding of pavement seasonal behaviour in different years.
- Conducting material characterization tests on the base and subgrade material to verify the backcalcuation at every FWD test.
- Collecting more information such as moisture content of the unbound materials at least from a limited highways

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