Instrumentation and Overall Evaluation of Perpetual and Conventional Flexible Pavement Designs

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

The perpetual structural pavement design is currently being explored for usage in Canada and worldwide. This thick structural design can provide many potential benefits but it also has associated costs. Cold Canadian winters and warm summers impact pavement performance and make pavement design challenging. This is further complicated by a heavy dependence on trucks to transport imports and exports. Consequently, most Canadian roads are subjected to rapid deterioration due to high fatigue stresses and rapid growth of the traffic loads.

The concept of a perpetual pavement design was raised to overcome the limitation of structural capacity of the conventional pavement designs. The concept of perpetual pavement was explained and introduced in this thesis and the benefits behind the perpetual pavement construction were studied.

The Ministry of Transportation of Ontario (MTO) and the Centre for Pavement and Transportation Technology (CPATT) joined their efforts in partnership with Natural Sciences and Engineering Research Council (NSERC), Ontario Hot Mix Producers Association (OHMPA), Stantec Consultant, McAsphalt and others to construct three test sections on the Highway 401. The goal was to monitor and evaluate the performance of three different pavement structural designs. Performance evaluation of test section was performed by evaluating the expected ability of pavement section to withstand the traffic loads and climate impact throughout the design life of that pavement section with minimum damage. The minimum damage is expressed as low vertical pressure on top of subgrade, low shear stresses in the surface course and low tensile strain at the bottom of asphalt layers. Perpetual pavement design with Rich Bottom Mix (RBM) layer, perpetual pavement design without RBM and a conventional pavement design were constructed and instrumented with various types of sensors. These are capable of monitoring the tensile strain in asphalt layers, vertical pressure on the subgrade surface, moisture in the subgrade material and the temperature profile in the pavement sections. The test section construction, sensor installation and preliminary modeling are all part of this thesis.

Preliminary structural evaluation was performed by analyzing the three designs using a Mechanistic Empirical Pavement Design Guide (MEPDG) model representing the three pavement designs constructed on the Highway 401. In addition, the WESLEA for Windows software was used to validate the long life performance of the perpetual pavement design. Life Cycle Cost Analysis (LCCA) was also performed for the perpetual and conventional pavement designs to evaluate the cost benefits associated with pavement designs for 70 year analysis period.

In addition, the perpetual Pavement design philosophy for moderate and low traffic volume roads was also examined in this research. This pavement design involved creating a complete comparison and validation of the benefits of using perpetual asphalt pavements versus the conventional pavements in all road types and traffic categories. Structural evaluation of the pavement sections in moderate and low traffic volume roads was performed. In addition, LCCA was implemented to validate the perpetual and conventional structural pavement designs in moderate and low traffic volume roads.

Acknowledgements

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I would like to thank my supervisor Prof. Susan Tighe from the Civil and Environmental Engineering Department at the University of Waterloo as well as Dr. Khaled Galal from Stantec Consultants for their guidance and encouragement throughout my research.

Several industry professional and colleagues assisted me with this research and I would like to extend my gratitude to them: Ms. Becca Lane, Mr. Joseph Ponniah and Mr. Warren Lee from the MTO, Mr. Mark Latyn from Capital Paving Ltd. for their guidance and assistance, Ms. Jodi Norris, Ms. Vimy Henderson, Mr. Riyadh Ul-Islam, Mrs. Shila Khanal, Ms. Rabiah Rizbi, Mr. Antonin Du Tertre and Mr. Peter Chan for their countless hours of assistance in the sensor installation at the test sections.

Dedication

I would like to dedicate this thesis to my parents and sister for supporting me throughout the years of my academic study and professional career. I would like to thank my adorable wife Manal and dear son Adam for their encouragement and reinforcement throughout my graduate studies.

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Chapter 1
Introduction

1.1 Background

In the modern era, the existence of a reliable well functioning road network is of critical importance. The investment in road construction provides many positive impacts on the social and economic development of the nation. Personnel and freight transportation has major impact on the economic growth of the nations. According to Transport Canada, seventy three percent of the Canadians drive their vehicles to commute to work. In addition to this, another seven percent of the employees use buses as the main public mode of transportation. In addition, forty one percent of the Canadian freight is transported by trucks using the Canadian highway and road network. The Highway and road network pavement condition plays a vital role in the social and economic development of Canada. According to the 2008 survey by Statistics Canada, Ontario has the highest population density with a share of 38.8% of the total Canadian population [Transport Canada 2004]. Due to the high population density in Ontario, the Highways and road networks in Ontario are serving more vehicles than any other province. This high traffic is applying extremely heavy loads over the Ontario highway network which causes tremendous deterioration of the pavement condition in the major highways. Highway 401 is one of the most vital highways in Ontario as it connects to Quebec from the east to Windsor and then to America at the west end of the highway. The 800 km long highway is considered one of the world's busiest highways, with an estimated Annual Average Daily Traffic (AADT) of over 420,000 in 2005 [MTO 2005 (a)]. Due to its critical contribution to the social and economic development of Ontario, the Ministry of Transportation of Ontario (MTO) heavily invests in the construction, maintenance and rehabilitation of this highway to preserve, maintain and enhance the structural condition. One of the largest ongoing projects taking place on Highway 401 is the widening of the highway from four to six lanes, between Woodstock, ON and Cambridge, ON. In addition to widening 15.3 km of highway, improvements include: median barriers; emergency access roads at Oxford County Road 22; interchange reconstruction at Drumbo Road; and rehabilitation and widening of five structures. These improvements will improve traffic operations and enhance safety.

The Ministry of Transportation of Ontario (MTO) in partnership with the Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo planned and designed an ambitious research project that would validate and evaluate the structural performance of different pavement mixes and structural designs by monitoring three test sections. All test sections will be subjected to the same traffic loads as they are located adjacent to each other on this stretch of the highway. In addition, the project design did not allow for changes in traffic and environmental conditions within the three test section to ensure that the traffic load will be exactly the same among the different structural designs. Two of the three test sections were designed as very thick or perpetual asphalt pavement sections with a Rich Bottom Mix (RBM) at the bottom of asphalt layers and a perpetual asphalt pavement section without the RBM layer. A conventional asphalt pavement section was also included to act as a control section.

Perpetual asphalt pavement designs are characterized by having thick asphalt layers installed over sound granular base and subbase. Although the total thickness of perpetual pavement designs can be less than that of the conventional pavement designs, they are expected to withstand the traffic loads and environmental impacts with minimum deterioration and crack propagation. The installation of stiff and thick asphalt layers enhances the ability of pavement section to resist permanent distresses as rutting, high shear stresses in the surface binder course, high tensile strain at the bottom of asphalt layers that would lead to bottom-up fatigue cracking and high vertical stress at the top of subgrade [Huang 2004].

Monitoring the three pavement test sections will be facilitated by using sensors that are capable of capturing the strain, vertical pressure, moisture content and temperature. Installation of these sensors was also part of this research and involved careful selection and design of sensors and gauges so that data collected that would enable the researchers to monitor and structurally evaluate the performance of the three structural test sections.

1.2 Purpose/Motivation

The purpose for this master's research was to perform a preliminary evaluation of the perpetual asphalt pavement designs and the conventional asphalt pavement designs by constructing and instrumenting several test sections that would validate the costs and benefits of perpetual pavement designs in terms of technical, economic and environmental aspects. The need to quantify the perpetual pavement design performance was established by using pavement structural evaluation softwares as MEPDG and WESLEA to predict the pavement performance in the form of rutting depth, bottom-up fatigue cracking propagation, and permanent deformations in pavement sections. The structural performance evaluation of pavement sections depends on its ability to withstand the shear stresses at the surface asphalt course, tensile strain at the bottom of asphalt layers and the vertical pressure at the top of subgrade.

The structural evaluation of perpetual and conventional pavement designs is performed to validate the theories in the literature and to develop a Canadian database that can be used to calibrate and validate the accuracy of the structural models in predicting perpetual pavement field performance. Construction of a real world test section in this research will provide reliable monitoring of the pavement performance and life cycle cost associated with the use of both perpetual and conventional pavement designs.

1.3 Scope and Objectives

The objective of this thesis can be summarized as follows:

1. Provide a literature review on pavement sensor installation and construction of pavement test sections, pavement modeling and computer based pavement structural analysis tools. In addition to perpetual pavements structural design methods.
2. Perform structural and economic evaluation of two perpetual pavement structural designs and the conventional structural pavement designs considering three levels of traffic loading (Heavy, moderate and low traffic volume roads).
3. Evaluate the costs and benefits of perpetual and conventional pavement designs under the three traffic volume levels.
4. Recommend the best pavement design methodology with respect to traffic, environmental and economic conditions for the southern Ontario region.

5. Design and instrument two perpetual pavements and one conventional pavement section in Ontario on Highway 401.

1.4 Methodology

The methodology followed in this thesis to achieve the research objectives can be generally described as follows:

1. Review literature on sensor installation and construction of the pavement test sections. The sensor installation design and data collection frequency is a key factor in performing a precise structural evaluation of the different pavement designs.
2. Review literature on structural evaluation modeling and computer based analysis models that can simulate the pavement performance through its life time.
3. Design and implement a sensor instrumentation plan for evaluating two perpetual and one conventional structural pavement design for Highway 401.
4. Create a numerical simulation models using different pavement computer software packages to evaluate the structural performance of different pavement designs in Ontario, Canada. These computer based models should take into account several mechanical, physical and environmental aspects to enhance the model accuracy. Also, they should consider use of sensor data.
5. Perform LCCA for the different pavement designs based on assuming a maintenance and rehabilitation program that is capable of retaining the pavement section condition. The structural evaluation result of the different pavement designs should be considered while designing the maintenance and rehabilitation programs.
6. Evaluate which pavement design is best suited for heavy, moderate and low traffic volume roads in southern Ontario region.

1.5 Organization of Thesis

Chapter one provides and introduction to the research project. A general overview about the thesis scope and objectives is provided. In addition, the research methodology is explained.

Chapter two provides a literature review about the sensor installation procedures, construction of pavement test sections and test tracks. This chapter includes the literature review performed for the computer based pavement modeling and structural evaluation of pavement designs

Chapter three presents the sensor types, designs and installation procedures on the Highway 401 project.

Chapter four presents structural evaluation and computer modeling for the three pavement designs constructed on the Highway 401. Economic evaluation of the pavement designs is also done in this chapter.

Chapter five introduces the example of moderate and low traffic volume road designs. Perpetual and conventional pavement structures were designed and evaluated in terms of both technical and economic cost benefits.

Chapter Six provides the research conclusion and recommendations for future work.

Chapter 2

Literature Review

2.1 Pavement Categories

Pavements are generally classified as rigid and flexible. The main component of the rigid/concrete pavement is aggregate, cement and various supplementing cementing materials. On the other hand, the flexible pavements are usually called asphalt pavement and they are composed of asphalt cement and aggregate. The common materials in both pavement types are the fine and coarse aggregates. The adhesive material that bonds the fine and coarse aggregates together varies from cement paste in the rigid pavement to the asphalt cement - one of the petroleum byproducts - in the flexible pavements. Both pavement types are classified to several subcategories. Several types of concrete pavements are installed on different highways and roads worldwide and are categorized as the Jointed Plain Concrete Pavement (JPCP), Jointed Reinforced Concrete Pavement (JRCP), Continuous Reinforced Concrete Pavement (CRCP) and Prestressed Concrete Pavement (PCP) [Huang 2004]. The flexible pavement is generally divided to conventional, deep strength, full depth or perpetual asphalt pavements. The main objective of this research is to compare the performance of the conventional and perpetual asphalt pavement designs. Thus, more detailed information about the conventional and perpetual asphalt pavement layers will be presented in the following section.

2.1.1 Conventional Asphalt Pavements

The regular conventional asphalt pavement structure for Highway/Interstate facilities consists of the hot mix asphalt (HMA) layers over Granular Base and subbase layers on top of the naturally compacted or stabilized subgrade soil [Huang 2004]. Each layer in the conventional asphalt pavement design addresses a certain distress type and enhances the pavement section structural strength and drainage system. The HMA layers are characterized by their high stiffness and hardness compared to the Granular Base and subbase layers. The HMA layers are the most expensive layers in the conventional asphalt pavement section. The HMA cost is relatively high due to the addition of asphalt cement (Petroleum byproduct), mixing, manufacturing and transportation. The cost of the granular material is dependent on the crushing operation and availability of material. HMA is very expensive and is used in the upper layers to withstand and distribute the traffic loading. Due to its low stiffness, Granular Base and subbase are showing the ideal properties to withstand the stresses caused by the freeze thaw cycles and improving drainage [Huang 2004]. The main layers forming the conventional asphalt sections are explained in the following sections:

2.1.1.1 Surface Course Layer

The top asphalt layer that is directly subjected to the traffic loads. This layer is usually rehabilitated whenever the top down cracks starts to propagate to the following layers. The well prepared preservation program should be able to eliminate the pavement surface distresses before their propagation to the following layers. This layer is playing a vital role in pavement surface drainage as it should prevent the propagation of excessive surface water to the following HMA, base and subbase layers [Garcia 2001].

2.1.1.2 Intermediate Upper Binder Course Layer

The Intermediate layer is made of HMA and is characterized by its large thickness. The intermediate HMA layer is less rigid than the surface course. The main role of the intermediate layer is to distribute the traffic loading on the base layer so that stresses transmitted to the asphalt layer foundation will not result in permanent deformation of that layer [Garcia 2001].

2.1.1.3 Base Lower Binder Layer

This layer is constructed below the intermediate course. In the base layer is designed to withstand the stresses resulting from the freeze thaw cycles and enhance the drainage in the conventional pavement section. This layer can be constructed from HMA or high quality granular material [Garcia 2001]. Another expression representing the base layer (if constructed using granular material) is widely used in Ontario and Canada which is “Granular A”

2.1.1.4 Subbase Layer

The subbase layer is constructed using granular material and some references and text books refer to it under the name of “Granular B”. This layer is located on top of the subgrade which is the normal soil in the construction site. The construction of subbase layer is not essential in all construction projects. The need for subbase layer depends on the quality of the subgrade layer. A strong and sound subgrade layer can replace the subbase layer and the base can be placed directly over it. The subbase material quality should be better than that of the subgrade. Subbase construction can enhance the drainage in the pavement section, minimize the freeze thaw impact and act as a sound platform for construction of the following layers [WSDOT 2009].

2.1.2 Perpetual Asphalt Pavements

The perpetual asphalt pavement is described as a long life pavement as it is designed to last for fifty years or more with minimum maintenance and rehabilitation activities performed. This target can be achieved by increasing the asphalt layers thickness. Increasing the thickness of asphalt layers would increase the total stiffness of the pavement section and decrease the stresses transferred to the subgrade layer. Due to the large thickness of asphalt layers, higher resistance to bottom-up fatigue cracking, structural rutting and permanent distresses is expected compared to the conventional pavement designs. The total thickness of perpetual pavement designs can be less than that of the conventional pavement designs due to the decrease in base and subbase granular layers [Newcomb 2006] [Brown 2004]. Perpetual pavements should be constructed over a relatively stiff and structurally sound subgrade characterized by high structural capacity. In case the subgrade layer is not as stiff as recommended, a thick subgrade layer is constructed to replace the subgrade layer and distribute the traffic load on the subgrade layer. A thick asphalt layer (over 370 mm) is expected to act as perpetual design even if constructed on a weak subgrade material and subjected to a heavy traffic loading [Nunn 1998]. This pavement design methodology is attempting to minimize the different pavement distresses affecting the lower asphalt layers. Another debate has concluded that pavement sections constructed of 160 mm or thicker asphalt sections are expected to perform as perpetual pavements and would not be affected by any noticeable stresses at the bottom of pavement section [Uhlmeyer 2001]. The perpetual pavement design is expected to be subjected to top down cracks only and this is due to traffic loading, friction in the surface coarse and environmental impacts. This distress can be maintained by applying mill and patch for the surface coarse before the propagation of top down cracks to the following layers. This maintenance activity would retain the pavement section condition and preserve it from deterioration [Newcomb 2001]. The typical perpetual pavement design consists of three main layers. Each layer is designed to overcome a certain distress type. The characteristics of each layer in the perpetual asphalt pavement design are as follows:

2.1.2.1 Wearing/Surface Layer

Resisting top-down cracking, surface initiated distresses and rutting of pavement upper layers (100 mm) to prevent permanent deformation in the pavement surface are the main objectives for the surface layer. The most effective material to be used for this layer is the Stone Mastic Asphalt (SMA). High quality Performance Grade (PG) should also be selected to withstand the traffic and weather conditions. In-place air void content is recommended in the range of four to six percent to ensure proper layer stiffness and durability [Superpave 2001] [Harm 2001].

2.1.2.2 Intermediate/Upper Binder Layers

This layer should be characterized by the stability and durability in order to achieve the rutting and fatigue potential resistance. The rutting resistance is accomplished by the stone-on-stone contact design, while the fatigue resistance is usually accomplished by the appropriate mixtures design, target proper field compaction levels and selecting appropriate asphalt binder grade. Selecting the appropriate asphalt binder with a suitable high-low-temperature grade is an essential requirement for long-term performance of this layer. This layer is primarily responsible for withstanding the repeated traffic loads expected over the pavement's service life. Similarly, depending on the total pavement thickness one or more intermediate layer can be constructed.

2.1.2.3 HMA Base Layers/Lower Binder

Fatigue resistance and bottom-up cracking prevention are the main stresses being addressed by this layer. Increasing the tensile strength of the base layer to overcome the tensile strain generated by the traffic loads can be accomplished by increasing the binder content and decreasing the in-place air void percentage. Thus, creating high density asphalt layer was reported to improve the fatigue cracking resistance and bottom-up cracking [Timm 2004(a)] [Newcomb 2001]. In addition, the total pavement thickness plays a vital role in decreasing the tensile strain generated at the bottom of the asphalt layer by distributing the traffic loads over a larger area. One or more base layers can be used in perpetual pavements based on the total pavement thickness required and the traffic loads.

The behavior of a long life pavement section is based on its ability to resist tensile stresses at the bottom of the asphalt layers. The concept of a perpetual pavement was believed to be accomplished by limiting the tensile strain at the bottom of asphalt layers to 60 μs or less and the vertical compressive strain at the top of subgrade layer to 200 μs [Monismith 1999] [Newcomb 2001]. Later research proved that the tensile strain limit at the bottom of asphalt layers proposed by [Monismith 1999] was more conservative. The perpetual pavement tensile strain limit at the bottom of the asphalt layers was estimated to be a variable number that can be calculated for every pavement section. The idea of having a variable tensile strain limit which was named the Fatigue Endurance Limit (FEL) was introduced in [Thompson 2006]. Through all the case studies that were applied in that research project, none of the pavement sections required strain below 70 μs to perform as a perpetual pavement section [Thompson 2006] [Peterson 2004]. Figure 2-1 show the results obtained for estimating the FEL values for different pavement sections.

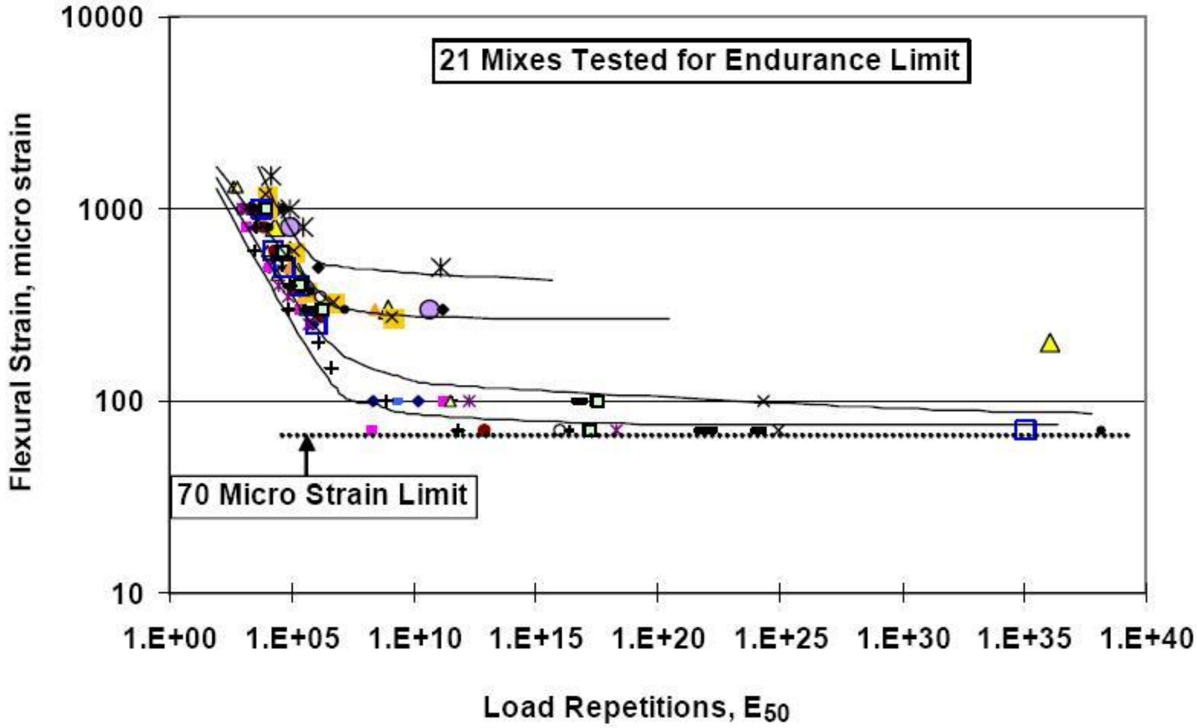


Figure 2-1: Strain-Load Relationship Illustrating the Fatigue Endurance Limit [Thompson 2006]

2.1.2.4 Subgrade Characteristics and Strength

Subgrade soil properties as characterized by soil resilient modulus, bearing capacity, and shrink-swell as well as frost susceptibility potential are strongly affecting the total perpetual pavement thickness and individual layer properties. The minimum soil resilient modulus recommended for perpetual pavement construction is 172,000 Kpa (25,000 psi) [Von Quintus 2001]. However, most subgarde soils have much lower moduli of that recommended in the literature. Both mechanical and chemical soil stabilization techniques can be utilized to accomplish higher subgrade resilient modulus values. Expansive and frost susceptible soils should be replaced by high quality granular materials and/or mechanically or chemically stabilized using lime, cement and/or fly ash before construction. Soil stabilization depth (150 mm to 600 mm) should be function of both existing soil conditions and expected traffic loadings.

2.2 Instrumentation and Test Section Construction

The structural evaluation of conventional and perpetual pavement designs should be performed by monitoring the different test sections that are constructed in this research. This means that each test section on Highway 401 should be instrumented. To perform a precise structural evaluation, several types of sensors are installed in the pavement sections to collect data that enables the researchers from evaluating the pavement section performance through its life time. The construction of the test section and sensor installation procedures has previously been carried out by several educational institutions and ministry of transportations. Construction reports of projects as the Minnesota Road research project (MnRoad), National Center for Asphalt Technology test track (NCAT test track) and the Marquette interchange perpetual pavement test section project, Centre for Pavement and Transportation Technology (CPATT) Test Track, University of Calgary Test Road and Laval University Road Experiment Site served as a sound background for sensor installation and test track construction. Figure 2-2 presents the test tracks constructed in the United States of America. Although most of these test sections were confined and constructed on traffic controlled roads, the basic procedures of sensor installation in these projects were generally followed in the construction of the test sections and sensor installation on the Highway 401 project. Modifications to the sensor installation procedures were made when needed due to the topographic features of the construction site and based on construction constrains that were essential to ensure the project construction quality and meeting the contractor's productivity rate. Technical advice was provided by the sensor supplier and manufacturers to ensure the functionality of the sensors after installation and increase their accuracy and reliability of data collected.

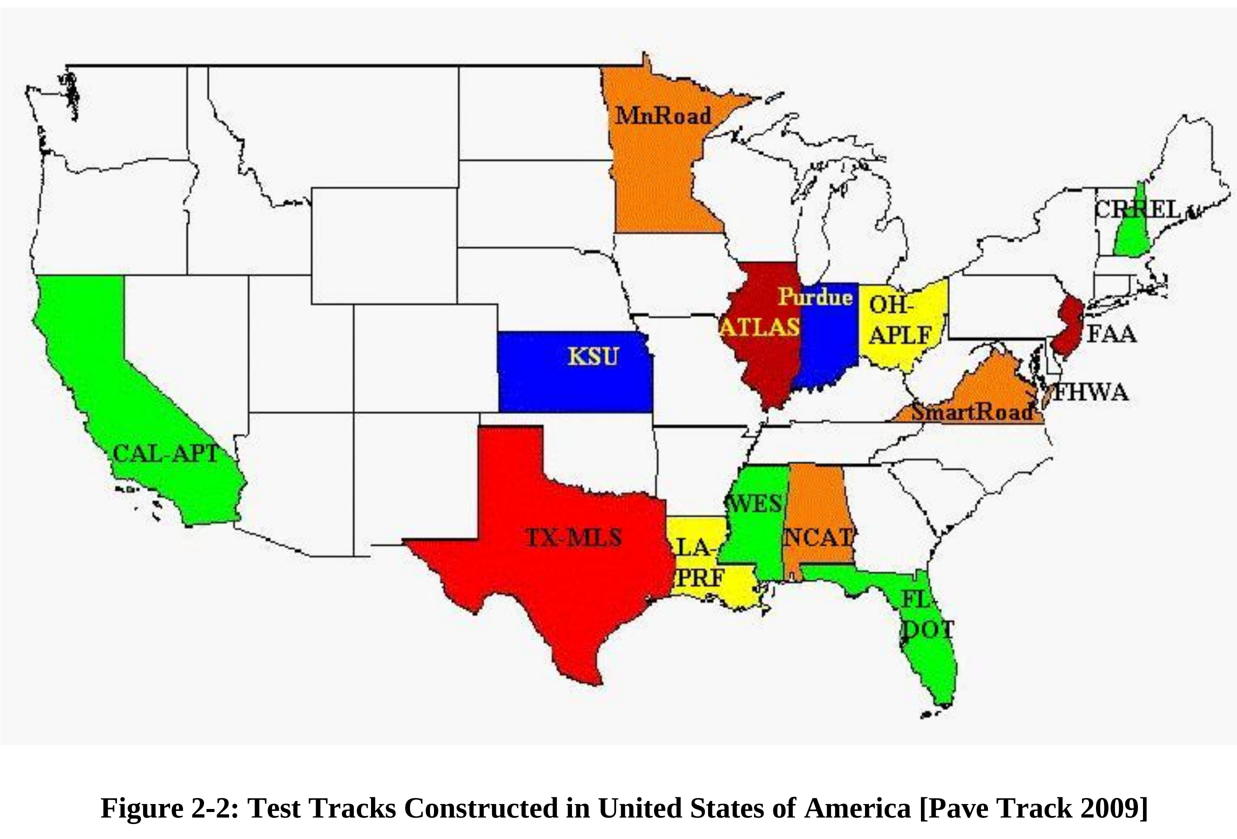


Figure 2-2: Test Tracks Constructed in United States of America [Pave Track 2009]

The test sections construction projects included in the literature review are all equipped with sensors that are similar to those installed in this research project. These sensors are dynamic strain gauges, earth pressure cells, moisture probes and temperature probes. The following section introduces other projects where various sensors have been used.

2.2.1 Minnesota Road Research Project (MnRoad)

The Minnesota Road Research Project (MnRoad) construction started in 1990. It is Located in Otsego, Minnesota, Mn/ROAD is about 64.4 km (40 miles) northwest of the Minneapolis-St. Paul metropolitan area on westbound Interstate 94. The test section is divided to two test roads. Part of the MnRoad is constructed on the I-94 highway to provide the researchers with data obtained from a high traffic volume road. The other section is constructed on a low-volume design test road that provides data from known load conditions. The total number of test sections in both roads is forty different sections. The research team working on construction of the MnRoad gained valuable experience in sensor installation in both concrete and asphalt pavements as 4,572 different sensors have been installed in the forty test sections [MNDOT 2001]. The sensors installed are divided to two categories. Some sensors are used to collect dynamic data that is captured from traffic loads passing over the sensors as the strain gauges and earth pressure cells. Other sensors are used to monitor the environmental parameters affecting the pavement performance as the moisture probes and thermocouples. Strain gauges are installed in groups of three spanning under wheel path. Part of the strain gauges is installed to monitor the strain occurring in the longitudinal direction while others are capturing the transverse strain. Data collection from the test track is obtained through downloading the data from the data loggers via internet. The current data can be accessed by researchers in Mn/DOT Materials Research and Engineering Laboratory in Maplewood. The future plan is to extend the accessibility of data obtained from the MnRoad to be available for public on the internet so that all pavement researchers worldwide can benefit from this valuable data.

Several research projects are currently taking place in the MnRoad test section. Research topic performed includes research related to the relation between resilient modulus versus drainage, thaw weakening versus time, thaw weakening versus moisture, frost heave versus drainage, comparison of horizontal strain and deflection in bituminous pavements and other various projects [MNDOT 2001]. Figure 2-3 presents a plan showing the MnRoad site location.

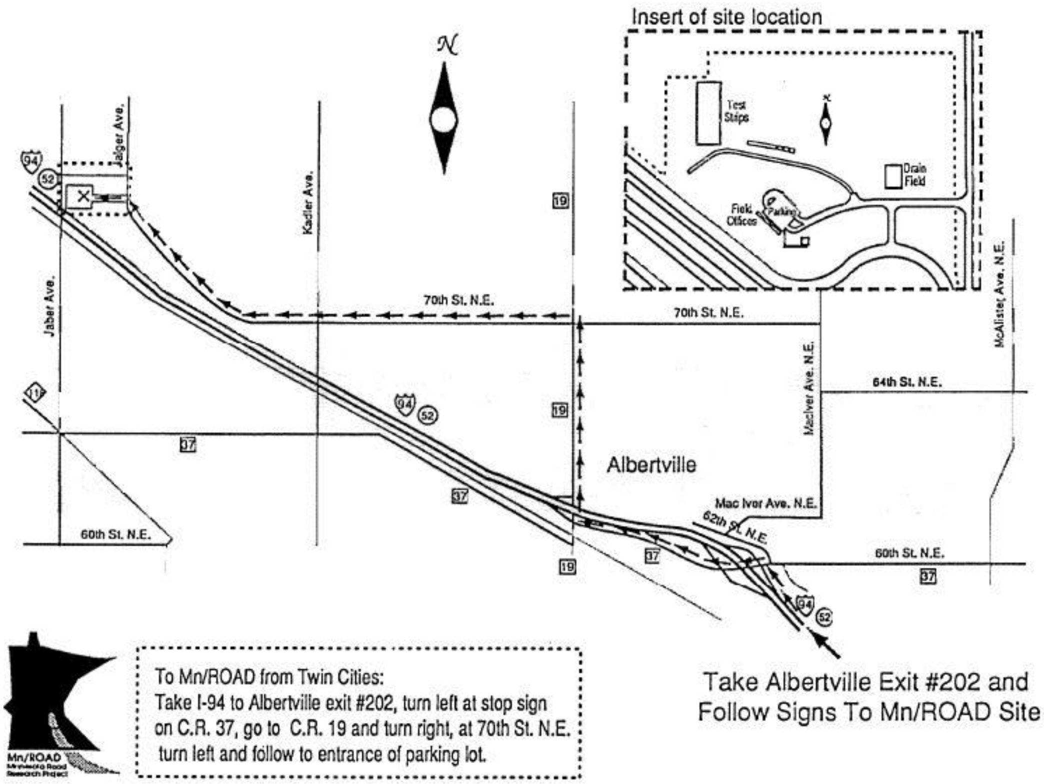


Figure 2-3: MnRoad Site Map [MNDOT 2001]

2.2.2 National Center for Asphalt Technology (NCAT) Test Track

The National Center for Asphalt Technology (NCAT) has constructed its 2.74 km (1.7 miles) test track in Opelika, Alabama. The test track consists of forty five different flexible pavement sections each of 61 m (200 ft) and designed for a ten million Equivalent Single Axle Loading (ESAL) [NCAT 2009]. The loading on pavement sections is applied using heavy trucks passing over the test track different sections. The benefit of applying the same traffic load on all pavement sections is the possibility of performing structural comparison between different sections. The test sections are equipped with different types of sensors that are monitoring the pavement performance and collecting data as strain in asphalt layers, vertical pressure and environmental factors impacting the pavement performance. Thermistor strings are installed in all test sections to monitor the temperature variation in all pavement layers in all seasons. In addition a permanent weather station is installed in the test track. The pavement response to traffic loading is monitored by installing strain gauges and pressure cells. Data is collected in a high speed and transferred using wireless mesh network that is installed along the entire track length [Pave Track 2009]. Figure 2-4 presents the NCAT test track layout.

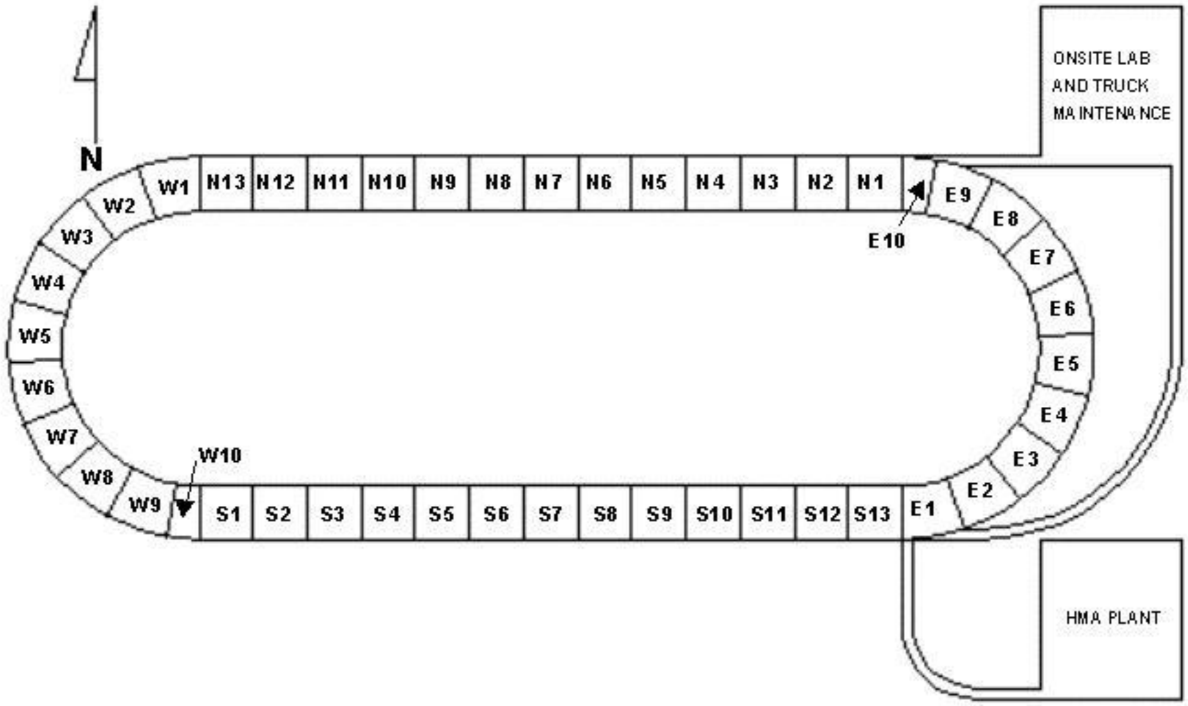


Figure 2-4: NCAT Test Track Layout [Pave Track 2009]

Eight sections of the NCAT test track were installed with sensors to monitor the dynamic pavement responses in 2004. The eight sections varied in the pavement cross section and mix designs. All sections were equipped with asphalt strain gauges of CTL brand which is used in the Highway 401 project. In addition to that, earth pressure cells, soil moisture probes and temperature probes were installed in these pavement sections. The sensor installation was generally successful with few strain gauges that did not survive the installation procedures [Timm 2004(b)].

2.2.3 Marquette Interchange Project

In 2006 the construction work was completed in the Marquette Interchange project. This project was awarded to the Marquette University Transportation Research Center (MU-TRC) by the Wisconsin Highway Research Program in 2005. The test section constructed in Marquette interchange project is located on the Interstate highway forty three (I-43) between stations 385+00 and 385+50 in the rightmost lane of the northbound direction. The instrumentation plan included installing twenty five strain gauges, two earth pressure cells and two temperature probes. Figure 2-5 presents the test section layout and the sensors' location [Hornyak 2007]. The strain gauges were installed to monitor the longitudinal and transverse strain values. Strain gauges used in this project were manufactured and supplied by two different companies in order to compare their performance. Seventeen strain gauges were manufactured by CTL and the remaining eight were manufactured by Dynatest. The CTL strain gauges used in the Marquette interchange test section are identical to those installed in this research project on the Highway 401.

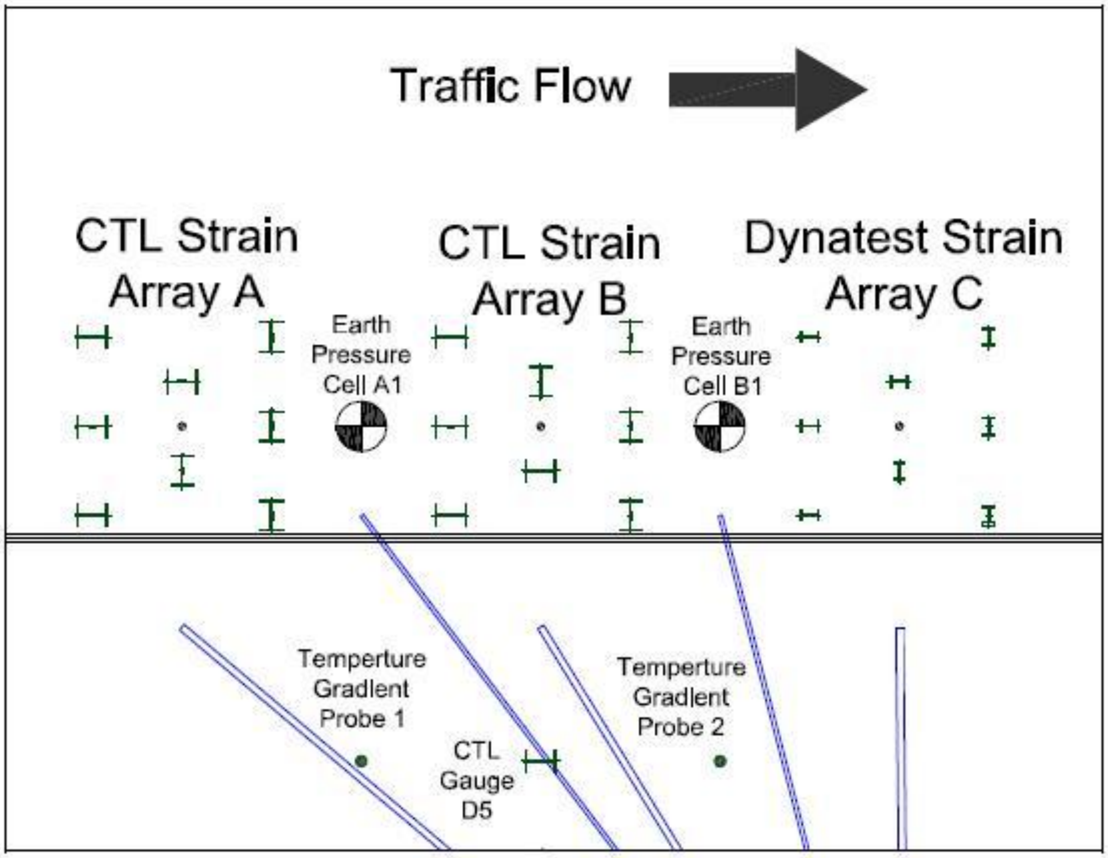


Figure 2-5: Marquette Interchange Instrumentation layout [Hornyak 2007]

2.2.4 Centre for Pavement and Transportation Technology (CPATT) Test Track

This test track is constructed by the University of Waterloo in corporation with Regional Municipality of Waterloo's waste management facility. The 709 meter long contains various pavement designs as standard Hot Laid 3 (HL3), Polymer-Modified Asphalt (PMA), Stone Mastic Asphalt (SMA) and superpave in addition to four Interlocking Concrete Pavement cross walks. The test sections are monitored using asphalt and concrete strain gauges, earth pressure cells, moisture probes and thermocouples [Tighe 2007]. A Weigh In Motion (WIM) was installed in the test track in addition to the static scale located at the beginning of the test section. Construction of the test section in the waste management facility provides unique features to the research projects implemented in this test track. The heavy weighted trucks passing over this test section results in 300,000 ESALs over a 3 week period and the traffic loads passing over the pavement sections can be measured accurately using the station static scale and can be compared to the WIM records. In addition, regular performance testing is applied on the different pavement sections including visual distress surveys, roughness evaluation, Falling Weigh Deflectometer (FWD), skid and rutting evaluations. Figure 2-6 presents the CPATT flexible test track plan view [Tighe 2007]. In addition, CPATT has been involved in instrumenting several satellite test sections with thermistor strings, moisture probes and strain gauges. These are located in City of Toronto, Vancover, Dryden and Chapleau.

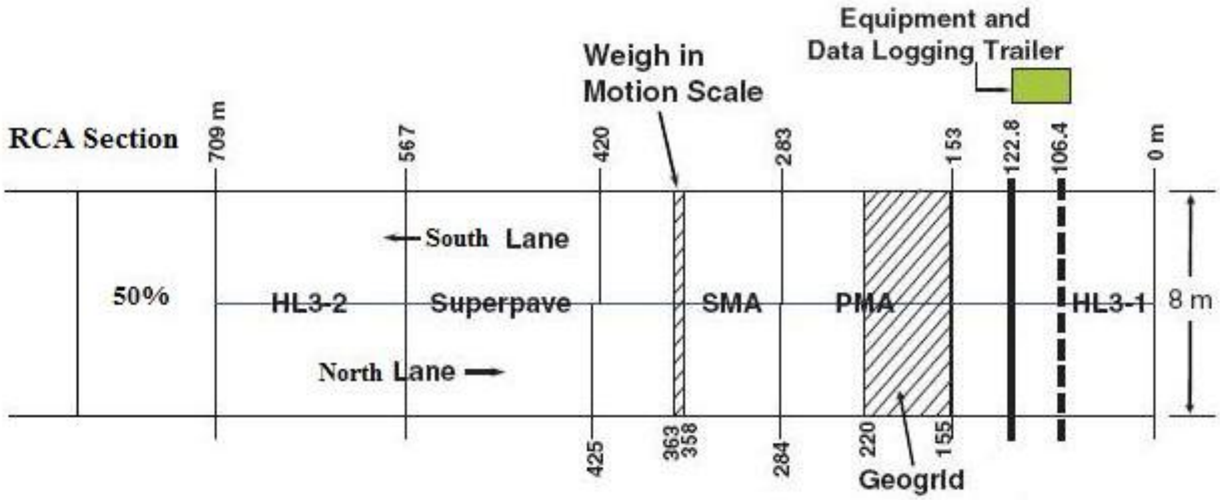


Figure 2-6: Flexible pavement portion of the CPATT Test Track after [Tighe 2007]

2.2.5 University of Calgary Test Road

This test road was constructed in 2005 to investigate the impact of the super single vehicles on typical thin pavements found in the Alberta. The test road was built in an industrial zone located near Edmonton International Airport. Three pavement structures are monitored in this project including Hot Mix Asphalt (HMA), Cold Mix Asphalt (CMA) and Granular Base Course (GBC) [Tighe 2007]. The three test sections are 50 m long each and are instrumented with pressure cells, strain transducers, moisture and temperature gauges. Construction and instrumentation of the test road were funded by a consortium gathering various industrial, academic and governmental agencies. The objectives behind construction of the University of Calgary Test Road includes development of new performance models, update the historical data about dynamic loading and seasonal factors, in addition to updating and validating the existing road-user regulations. The long term plan for the test road is to monitor the different sections every 3 months by recording the pavement deterioration to develop more accurate Load Equivalency Factors (LEFs) that represents the traffic and climate conditions of Alberta [Tighe 2007].

2.2.6 Laval University Road Experiment Site

The Site Expérimental Routier de l’Université Laval (SERUL) was constructed in 1999 at Laval University’s experimental forest, located 60 km north Quebec City. SERUL was constructed in parallel to the Route 33. The unique geometric design of the SERUL enables traffic shifting of the vehicles passing over the test section to the Route 33 whenever surveys, construction or maintenance activities are scheduled. The test road included a one km section of the road, an experimental bridge and a control building. Three test sections were constructed in the SERUL. The first section included a 300 m standard pavement structure where research studies are performed to study different types of low-cost surfacing materials. Maintenance and rehabilitation techniques applied to each surfacing material are being investigated. The materials resulted from the maintenance and rehabilitation activities are recycled and used in other research projects [Tighe 2007]. The second section includes a 150 m long full pavement structure. The pavement is constructed over a 3m deep concrete pit placed over a selected subgrade soil. The third test section is constructed as a standard pavement section over two different types of subgrade.

This test section is used for pavement-vehicle interaction research. The pavement sections were instrumented to monitor their structural performance. The instrumentation project of the SERUL was performed in corporation with the Quebec Ministry of Transportation and the Laboratoire Central des Ponts et Chaussées in France. The instrumentation of SERUL included Thermistor strings, moisture sensors, frost gauges, piezometers and heave gauges.

2.3 Pavement Computer-Based Modeling

Several research projects have been implemented to develop computer software packages that are capable of modeling pavement sections and predicting the pavement performance through its lifetime. These software modeling packages have been enhanced and a variety of pavement modeling programs is now available in the market. The accuracy and preciseness of any of these pavement modeling software packages depends on the research work and data collected to form the scientific background for the program coding. Some of these softwares were noticed to be accurate and efficient when modeling rigid pavements but when they were found to be unable to model the flexible pavements performance accurately. The pavement modeling software programs started to address even smaller branches of the pavement designs. Some pavement modeling softwares were developed to model certain categories of flexible or rigid pavements. The softwares that were of interest to this research project are known to be able to accurately model flexible pavements generally. Other programs as the PerRoad software [Timm 2006] have been eliminated due to its limited accuracy in modeling conventional asphalt pavements as this software have been designed and developed to model perpetual asphalt pavements. Using the same modeling software that is known for its acceptable accuracy in modeling different flexible pavement categories ensures the research consistency and eliminates any errors due to software capabilities when comparing the pavement performance of different pavement flexible designs. The Mechanistic Empirical Pavement Design Guide (MEPDG) has been used to model both the perpetual and conventional pavement designs in this research work. The MEPDG is used by several ministries of transportations in North America and worldwide to perform structural assessment prediction for the pavement sections and have proved to be reliable structural evaluation software for all flexible pavement designs [Schwartz 2007]. In addition to the MEPDG, the WESLEA for Windows software have been used to ensure the Long life pavement behavior of the perpetual pavement designs by predicting the maximum tensile strain values at the bottom of the asphalt layers.

2.3.1 Mechanistic Empirical Pavement Design Guide (MEPDG)

In 1996, the American Association of State Highway and Transportation Officials (AASHTO) raised the issue of the need for developing a mechanistic empirical design guide tool. The AASHTO Joint Task Force on Pavements (JTFP) has sponsored the project and this was the beginning for the National Cooperative Highway Research Program (NCHRP) project 1-37A. The project was successfully completed by Applied Research Associates research team, Arizona State University, Fugro Consultants LP and several other consultants [Schwartz 2007].

The MEPDG has enabled different stakeholders involved in the pavement industry to achieve cost effective pavement designs and rehabilitation strategies. Unlike the 1993 AASHTO design guide, the MEPDG integrates analysis approach for predicting pavement condition taking into account the traffic loading, climate impact and the pavement structure and characteristics. The MEPDG allows the pavement designers to make better informed decisions and take cost effective advantage of new materials and features. The MEPDG can also used to analyze the condition of existing pavements and predict their future performance.

The MEPDG model development is performed by following several steps. The program will be unable to model the pavement section if any of these steps is eliminated. Figure 2-7 represents the main steps that are to be followed to create the MEPDG model [Schwartz 2007].

The MEPDG software adopts the variation of inputs reliability level. The higher the reliability level of inputs, the more accurate the input data should be and thus more engineering effort in determining design inputs is needed. The input level should be should be reasonably considered in accordance to the project's importance, size, and budget. The input levels in the MEPDG are presented as follows:

1. Level 1: The most accurate and precise input level. Usually used for designing the Major highways, Interstates and strategic roads. It requires field and laboratory testing for all materials used in the pavement layers and subgrade soil.
2. Level 2: This is the medium accuracy input level. It is used when limited testing for the material used in pavement layers is performed. Mechanical, physical or chemical properties that were not determined by laboratory or field testing should be assumed based on previous experience.
3. Level 3: This input level represents the least accuracy level. It is used for low volume roads and when field or laboratory testing for the materials is unavailable. Default values recommended by the local agencies for material characterization are used in this input level.

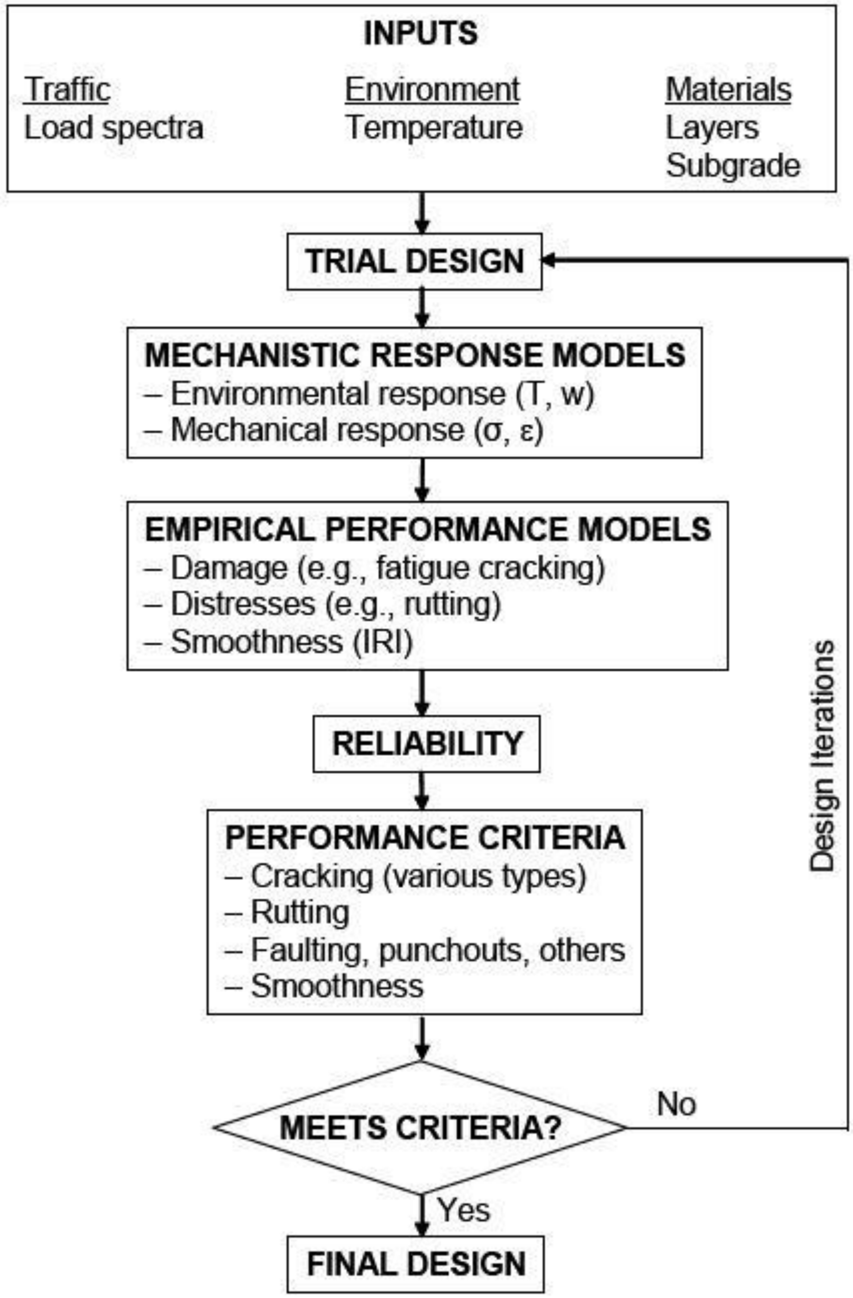


Figure 2-7: MEPDG Flow Chart Diagram [Schwartz 2007]

The mechanistic response models are the analysis procedure or equations that the MEPDG is using to predict the pavement performance based on the inputs previously entered. The environmental impact on pavement performance is simulated using the Enhanced Integrated Climatic Model (EICM). The EICM is a numerical program that simulates changes in the behavior and characteristics of pavement and subgrade materials due to variation in temperature and moisture [Schwartz 2007]. The methodology of mechanical response calculation in the flexible pavement model depends on the input level at which the MEPDG model was created. For models created by input levels one or two, the mechanical response calculation methodology is following the Multi Layer Elastic Theory (MLET) while the MEPDG models created using input level three are simulated using the Finite Element (FE) modeling theory. The main difference between the two modeling theories is that the MLET theory assumes a constant stiffness for the various asphalt layers. However, the asphalt stiffness changes depending on the stresses applied to the asphalt section. The FE modeling theory calculates the asphalt stiffness based on the stress value applied to the section. The FE is more complicated modeling theory as the stress applied on the pavement section is in dynamic rapid variation. Although the FE modeling method is more accurate, it requires long time for simulating the pavement section [Schwartz 2007].

The MEPDG performance modeling generates the expected stresses that the pavement section will be exposed to and the corresponding pavement response to the stress values. The pavement response is translated to fatigue cracking, rutting, International Roughness Index (IRI) and thermal cracking expected values over the analysis life time.

However, MEPDG includes several transfer functions that are calculating the software outputs using the physical properties of the pavement layers, traffic characteristics and climate data. These transfer functions were obtained by analyzing construction reports from 19 states in the USA and monitoring the pavement deterioration. The MEPDG can be used to predict the pavement deterioration in flexible and rigid pavements. MEPDG limitations are that it is unable to simulate the effect of freeze-thaw cycles accurately; it is incapable of modeling special types of pavements as interlocking concrete pavements, warm mix asphalts. In addition, it is unable to simulate the effect of using additives as Recycled Asphalt Pavement (RAP), fibers and Recycled Asphalt Shingles (RAS).

Several research projects –similar to our Highway 401 project- are monitoring pavement sections in order to enhance the MEPDG models and verify its outputs. MEPDG software was used in this thesis for comparison between the deterioration of different pavement designs. Thus, in case the software is underestimating one of the pavement physical characteristics impact on the pavement deterioration, the underestimation will occur in all pavement designs. This will lead to a relatively consistent comparison between the conventional and perpetual pavement designs even if the MEPDG is still under modification and research [NCHRP 2008] [Zhou 2009].

The field outcomes should be used to calibrate the MEPDG software and enhance its ability to predict various distresses taking into account weather impact in Ontario.

Chapter 3

Construction and Instrumentation of Test Sections

The following chapter describes the work implemented to accomplish the construction and instrumentation of several test sections for monitoring pavement structural performance. Test sections were constructed and fully equipped by various types of sensors and devices that are capable of providing the researchers, contractors, consultants, policy makers and all other parties involved in the pavement field with clear and sound data that properly determines the pavement performance of such designs while subjected to different traffic and climatic characteristics.

3.1 Introduction

The Ministry of Transportation in Ontario (MTO), Ontario Hot Mix Producers Association (OHMPA), the Centre for Pavement and Transportation Technology (CPATT), Natural Science Engineering Research Council (NSERC), Stantec Consultants and McAsphalt Industries Ltd. are partnering to evaluate the pavement performance of three flexible pavement designs. The three pavement designs include two perpetual pavement designs and one conventional flexible pavement design. This research will help designers, researchers, contractors, consultants working in the pavement field to better understand how perpetual pavement designs perform and deteriorate taking into account the environmental and traffic impact specially the effect of freeze-thaw cycles on pavement performance and crack propagation.

Three flexible pavement designs will be monitored by installing sensors in the different pavement layers including asphalt, Granular and subbase layers. The project includes construction of six monitoring stations divided in two construction stages. Stage one of the construction project includes preparation and instrumentation of three monitoring stations on the lane three (left lane) of the Highway 401. The stage two of the project includes construction and instrumentation of another three monitoring stations located in lane one (right lane/the driving lane) of the Highway 401 located near Wookstock, Ontario. Sensors installed are capable of collecting strain, vertical pressure, temperature and moisture content. The asphalt strain gauges (ASG) installation was designed to target the strain in the critical zones where cracks initiation is expected. In order to study the rutting phenomenon, earth pressure cells (EPC) were installed under the wheel path in order to measure the vertical pressure on the top of subgrade. Thermistor strings (TS) are installed to monitor the temperature of the different pavement layers. As the moisture content in the subgrade layer plays great role in the pavement deterioration and performance, moisture probes (MP) were installed to measure the moisture content in the subgrade layer.

In addition to these sensors, weigh-in-motion (WIM) sensors will be installed to capture the axle load of the vehicles. Thus loads, strain, vertical pressure and environmental parameters affecting the pavement performance can all be monitored and used to evaluate the pavement mixes.

Data collected from the previous sensors will be used to create a numerical simulation model to predict the performance of the three pavement mixes in the future. In addition, the maintenance programs for the different mixes will be assumed according to the pavement performance to ensure extending the lifetime of each mix and grantee a minimum acceptable performance and safety level of the road. This numerical model will take into account the environmental and climatic conditions in this part of the world in order to evaluate the benefits of using these mixes in our region.

3.2 Project Location

The project is located on the eastbound lanes of Highway 401 between exits 238 and 250 in southwestern Ontario. This section of the highway is located between Waterloo and Woodstock Ontario. Figures 3-1, 3-2, 3-2 and 3-4 introduces the project location.

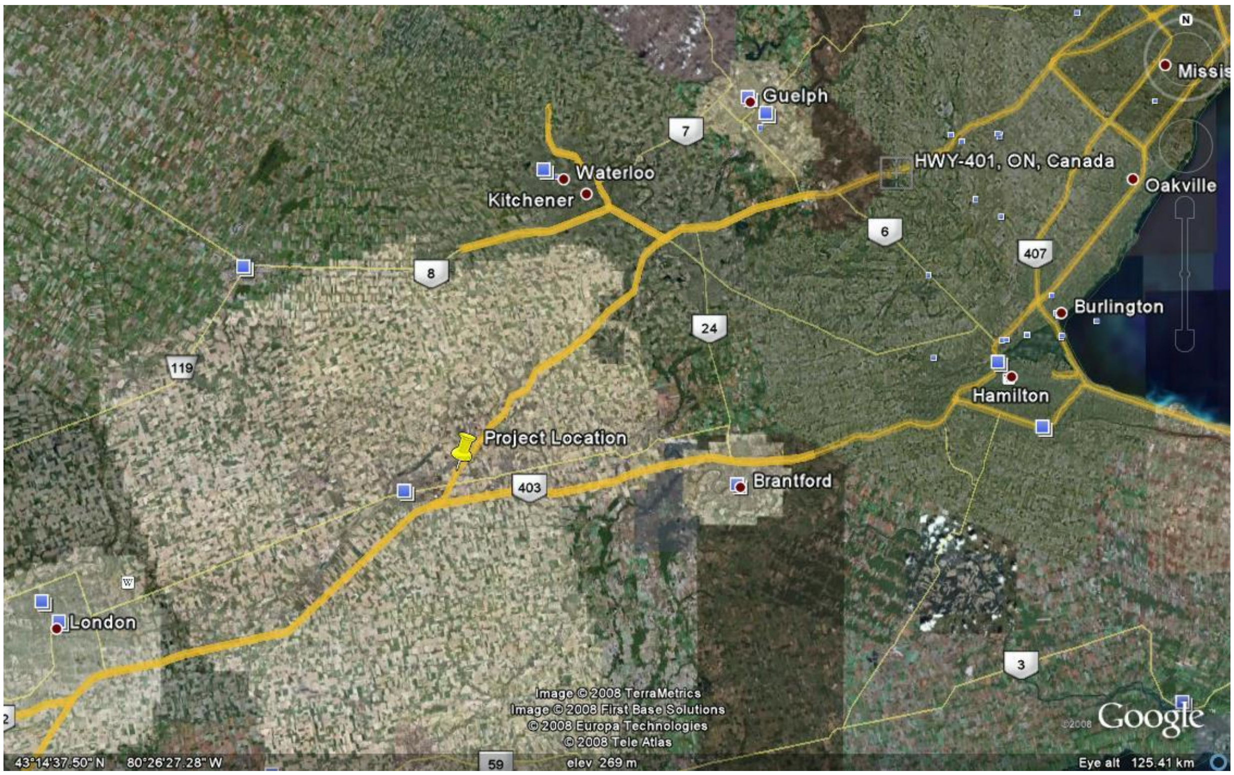


Figure 3-1: Project Location Overview [Google Earth 2008]

Figure 3-2: Project Location [Google Earth 2008]

Figure 3-3: Stations 12+230 and 12+350 [Google Earth 2008]

Figure 3-4: Station 13+067 [Google Earth 2008]

3.3 Pavement Sections

The aim of the project is to compare the performance of three pavement designs and evaluate the benefits of using thicker pavement structures constructed using expensive materials in order to achieve better field performance, longer pavement life and lower overall maintenance cost. To fulfill these goals, three different pavement designs were designed and the same instrumentation will be installed in all of them. The station 12+230 will be used to monitor the performance of the perpetual pavement mix that includes a rich bottom mix layer (RBM). Sensors installed in station 12+350 are used to collect data from the perpetual pavement mix without the rich bottom mix layer (RBM). While the station 13+067 is the monitoring station for the conventional flexible pavement mix. Figures 3-5, 3-6 and 3-7 show the pavement cross sections of the three mix designs.

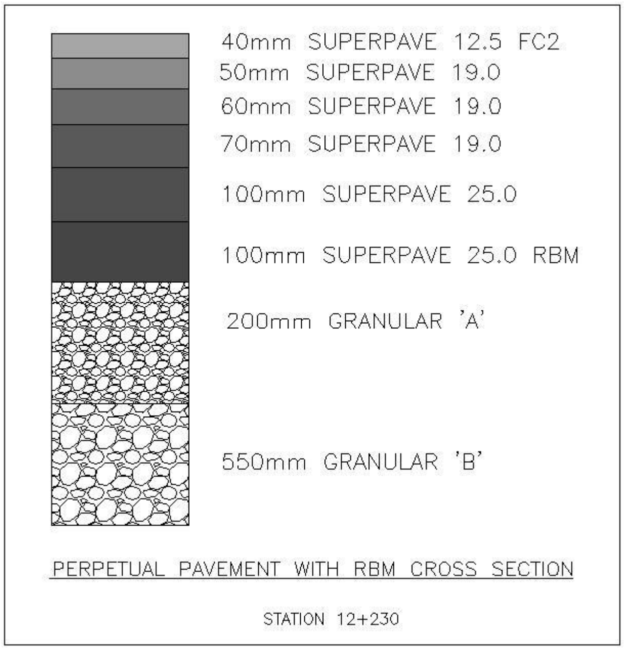


Figure 3-5: Cross Section of Perpetual Pavement with RBM

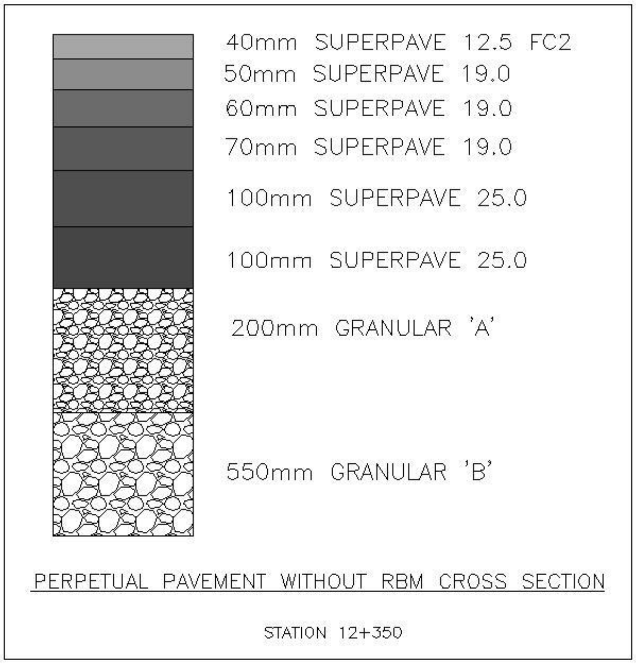


Figure 3-6: Cross Section of Perpetual Pavement Without RBM

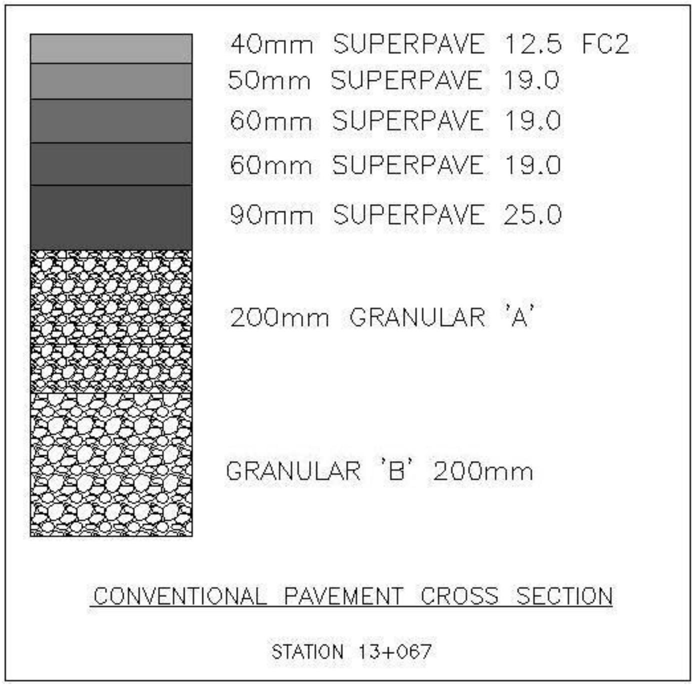


Figure 3-7: Cross Section of Conventional Pavement

3.4 Installation Location

The sensor installation was designed to provide the research team with the most accurate and pertinent data. The sensor locations in the pavement layers play a vital role in validating real time pavement performance. Figure 3-8 shows a schematic drawing for the monitoring stations and the cross sections of the three pavement structures. Detailed drawings showing the plan and the road cross section are presented in Figures 3-9, 3-10 and 3-11. The location of each sensor is designed to provide engineering data that can later be used to model long term performance. The Asphalt Strain Gauges (ASGs) are installed under the left and right wheel paths where the vehicles drive over them. The sensors are installed to measure the strain values in the longitudinal direction and the transverse direction perpendicular to traffic (μ_x and μ_y respectively). The vertical location of the Asphalt Strain Gauges (ASG) is at the top and bottom of the lowest asphalt layer installed on top of the granular layers. This is the location subjected to highest tension and thus crack initiation is expected to take place from the bottom of the asphalt layers under the wheel paths. Therefore, these gauges will provide strain information necessary to determine whether cracking is likely to occur or not. Earth Pressure Cells (EPCs) are installed to determine the vertical strain on top of the subgrade layer to determine the total rutting values over time. To fulfill the installation purpose, Earth Pressure Cells (EPCs) are installed under the right wheel path on the top of subgrade layer. Moisture probes (MPs) are installed to determine the moisture content in the subgrade layer. The Moisture Probes (MPs) are installed 40 centimeters deep in the subgrade layer. The moisture content in the subgrade layer affects the frost-thaw impact cycles thus, affecting the deterioration rate of pavement sections due to fatigue cracking.

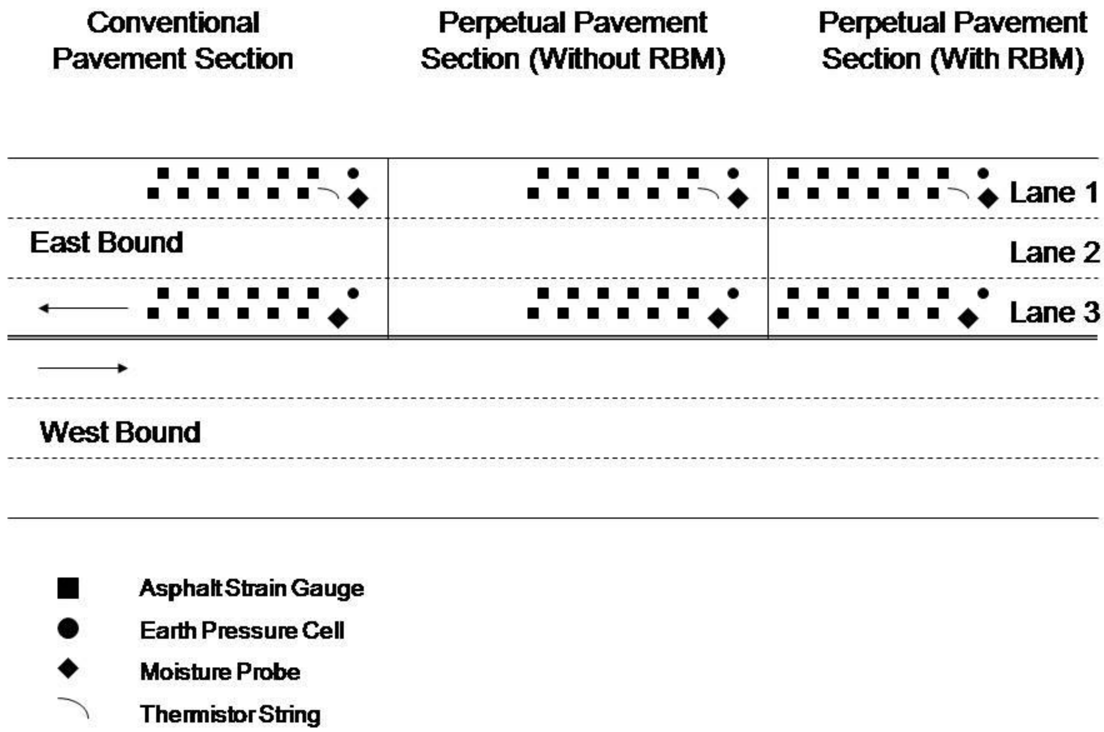
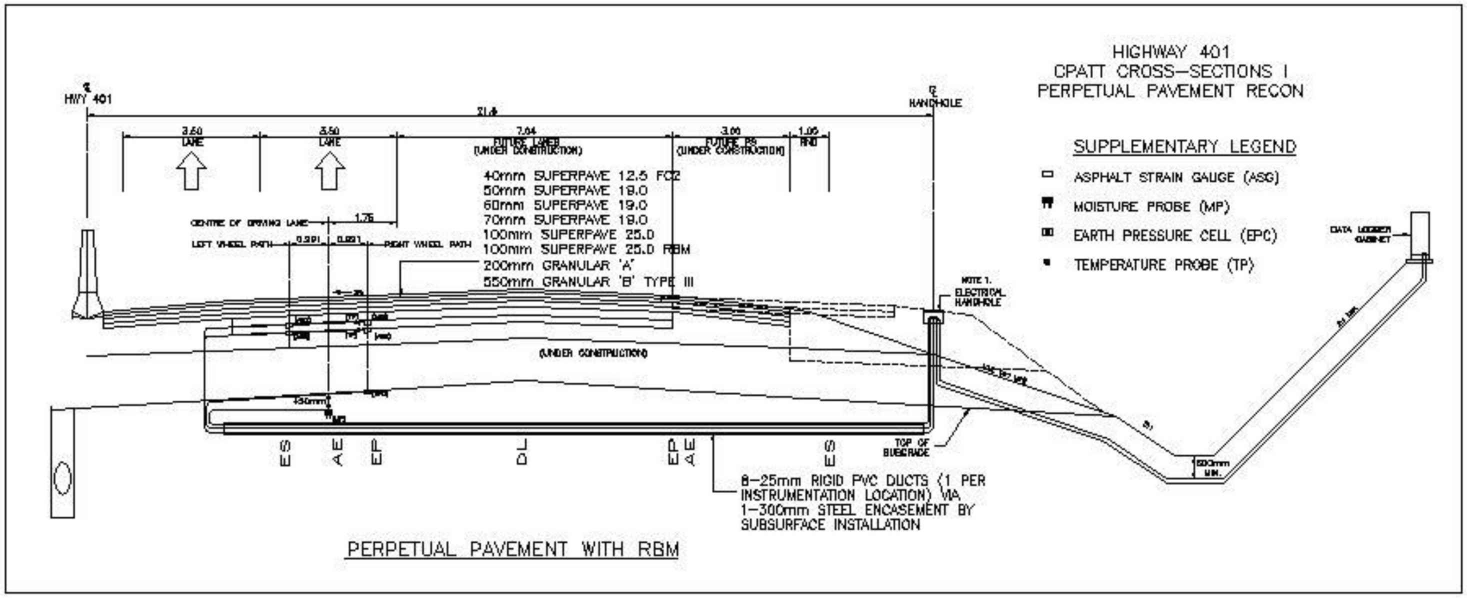


Figure 3-8: Plan View of Sensors at the Three Monitoring Stations

The project instrumentation plan included installation of Thermistor Strings (TSs) in the six monitoring stations. Thermistor Strings (TSs) are used to determine the temperature profile as it captures the temperature every 10 centimeters starting the pavement surface reaching the subgrade layer. Due to construction constrains, Thermistor Strings (TSs) installation in lane three was cancelled. Phase two of the project is currently under construction in lane one and will include installation of three Thermistor Strings (TSs) – one per monitoring station – in the paved shoulder.



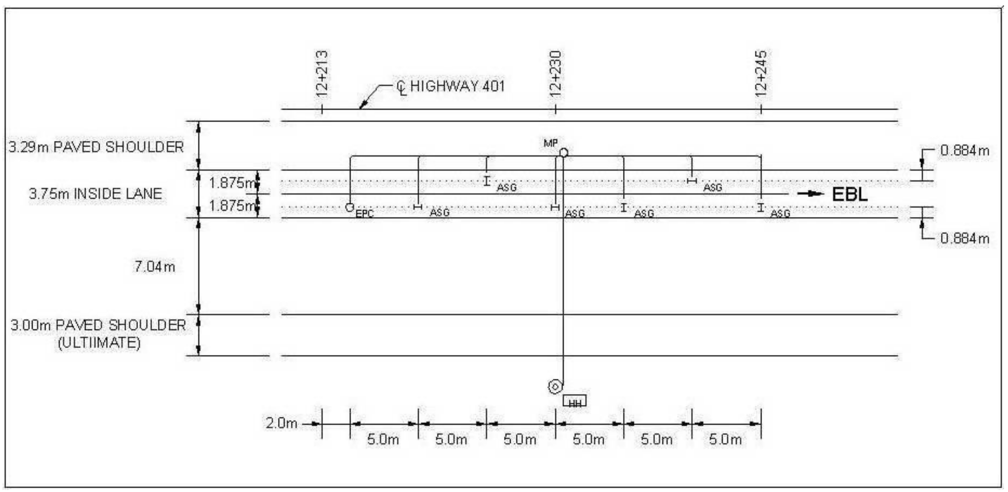
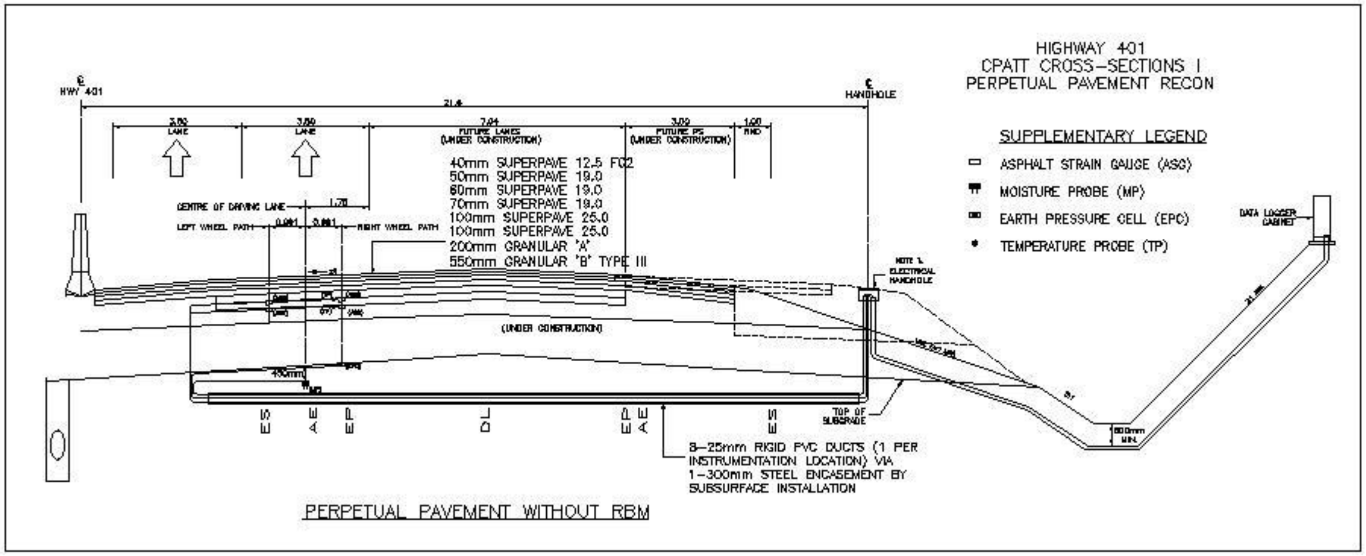


Figure 3-9: Cross Section and Plan of Perpetual Pavement with RBM (Stage One)



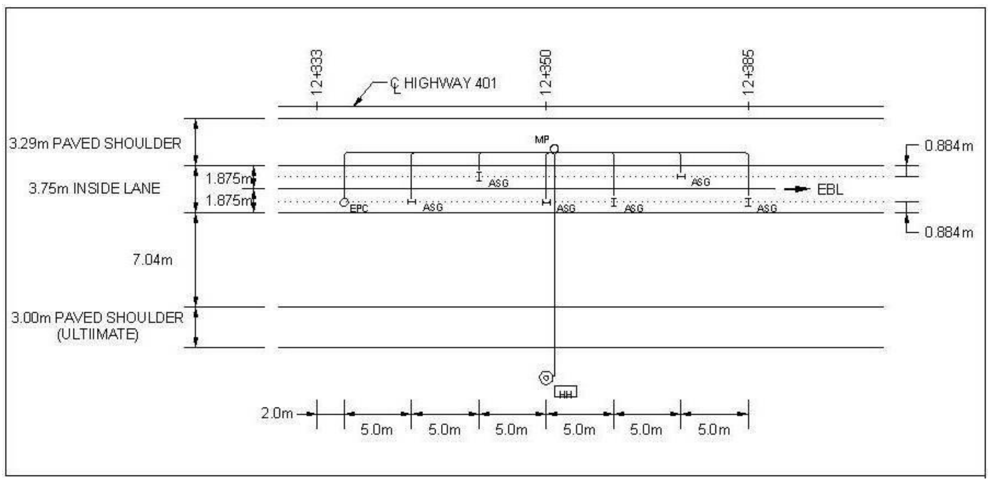
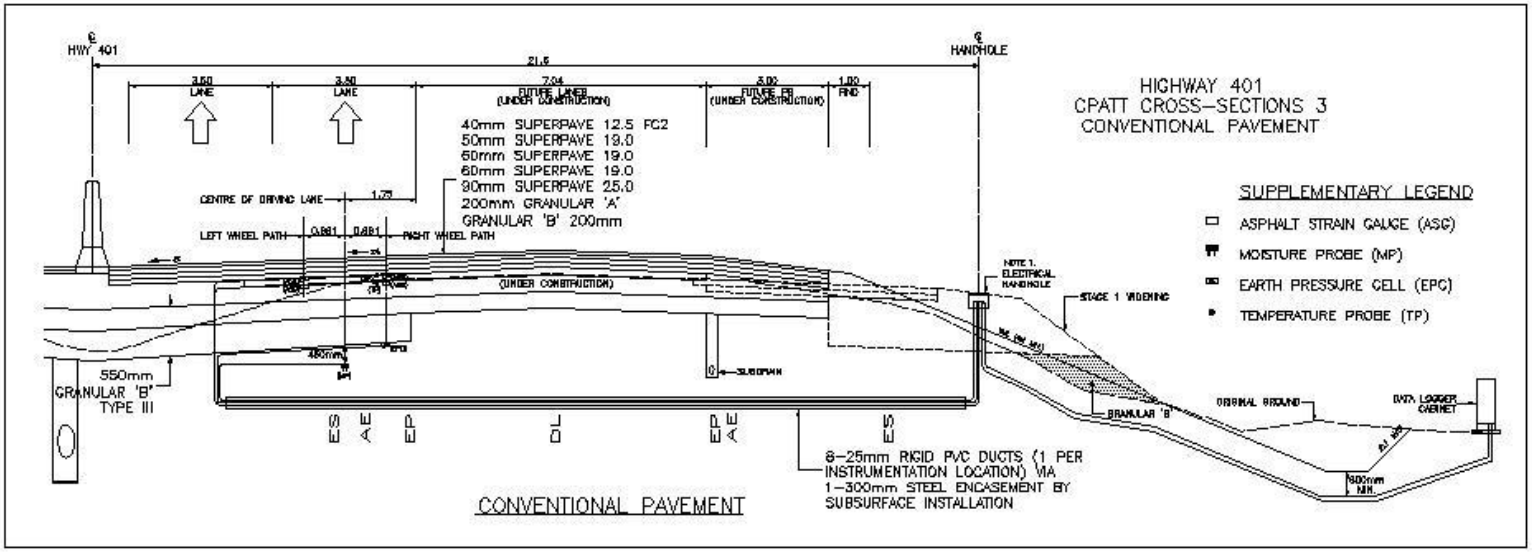


Figure 3-10: Cross Section and Plan of Perpetual Pavement without RBM



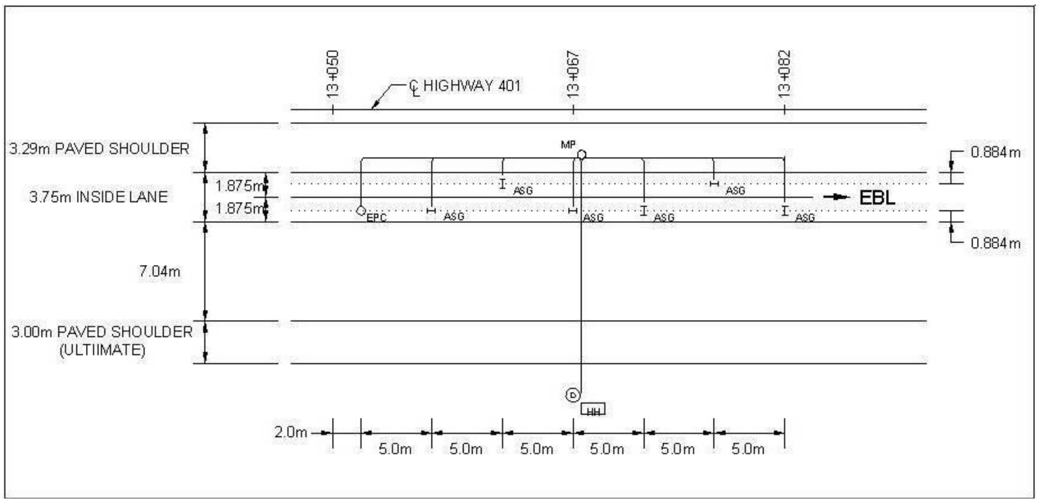


Figure 3-11: Cross Section and Plan of Conventional Pavement

3.5 Sensor Selection

Based on an extensive literature review, it is noticeable that most of the structural pavement distresses start due to weakness in two locations in the asphalt pavement structure. The most critical location is under the wheel path. This area is subjected to the most severe loads. Fatigue Cracks are initiated at the bottom of asphalt layers or the top of Granular A as this is the part subjected to the highest tension level. On the other hand, structural rutting is expected to be predicted through monitoring the vertical pressure at the top of subgrade material [Timm 2003].

Instrumentation plans in similar projects were reviewed prior to the sensor selection and installation. Projects as the Minnesota Road Research Project [Baker 1994], the Virginia SmartRoad [Louliz 2007] and the National Center for Asphalt Technology (NCAT) test track [Timm 2004(b)] had valuable reports and detailed information regarding their experiences in construction and installation of various sensors in their pavement test sections.

3.5.1 Asphalt Strain Gauges (ASG)

Asphalt strain gauges are used to capture the dynamic strain at the critical locations where crack initiation is expected. The most critical location is expected to be at the bottom of the asphalt concrete layer specifically under the wheel path. This area is subjected to the maximum tension. Thus, crack initiation takes place in this area in most cases. Previous experience gained from the sensor installation projects developed a sound background for the selection of the proper asphalt strain gauge. Quotations from Hoskin Scientific and Construction Technologies Laboratories (CTL) were received and evaluated. The asphalt strain gauges manufactured by CTL have been installed successfully in several test sections in North America as the NCAT pavement test track [NCAT 2009], Minnesota road test section (MnRoad) [MNDOT 2001] and the Marquette interchange test section in Wisconsin [Hornyak 2007]. Therefore, ASGs manufactured by Construction Technologies Laboratories (CTL) were selected to be installed in this research project. The sensor specifications, dimensions and picture are as shown in the product brochure.

Figure 3-12: Asphalt Strain Gauges Brochure [CTL 2008]

3.5.2 Earth Pressure Cells (EPC)

Earth Pressure Cells are used to measure the vertical stresses. They consist of two circular steel plates, welded together around their periphery. Between the two plates, an annulus filled with de-aired glycol exists. The cell is connected via a stainless tube to a transducer forming a closed hydraulic system. EPC is designed to be installed on the top of subgrade material and under the wheel path. This location enables the research team to study the subgrade rutting phenomenon. Total Earth Pressure Cell (EPC) manufactured by RST instruments were selected based on their technical capabilities and low cost compared to other gauges. The model selected is the vibrating wire LPTPC-V. The sensor specifications are presented in the product brochure.

DESCRIPTION	LPTPC-V
Transducer Type	Vibrating Wire
Range	Up to 34,500 kPa (5,000 psi)
Overrange	200% F.S.
Accuracy	0.1% F.S.
Resolution	0.1% F.S. minimum
Excitation Voltage	5 V sq. Wave
Signal Output	1200 - 2000 Hz
Conductor	2 X #20
Operating Temperature	-29° to +65°C -20° to +150°F

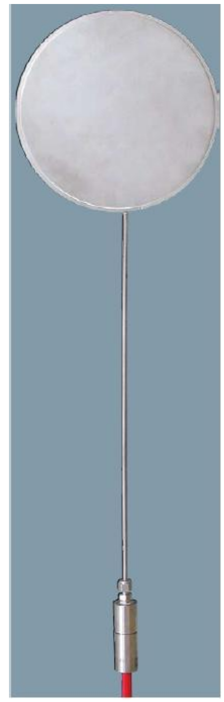


Figure 3-13: Earth Pressure Cell Brochure [RST 2008]

3.5.3 Thermistor String

Thermistor String (TS) is used to collect temperature of all pavement layers. Data obtained from the Thermistor String will enable the research group to obtain the temperature profile through the different pavement layers. The Thermistor string was manufactured as custom product by Campbell Scientific. The Thermistor string (TS) was provided with 15 temperature sensors spaced at 10 cm. Furthermore, CPATT has successfully used these Thermistor strings at other locations [Baiz 2008] [Mabood 2008].

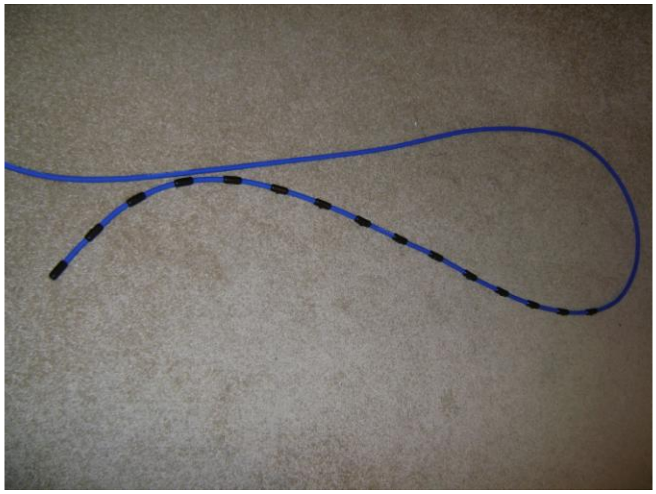


Figure 3-14: Thermistor String

3.5.4 Moisture Probe (MP)

Moisture Probe (MP) is used to determine the moisture content of the subgrade soil. The moisture probe CS616 that is manufactured by Campbell scientific was selected due to its sound reputation and it has been used in other University of Waterloo projects [Baiz 2008] [Mabood 2008].

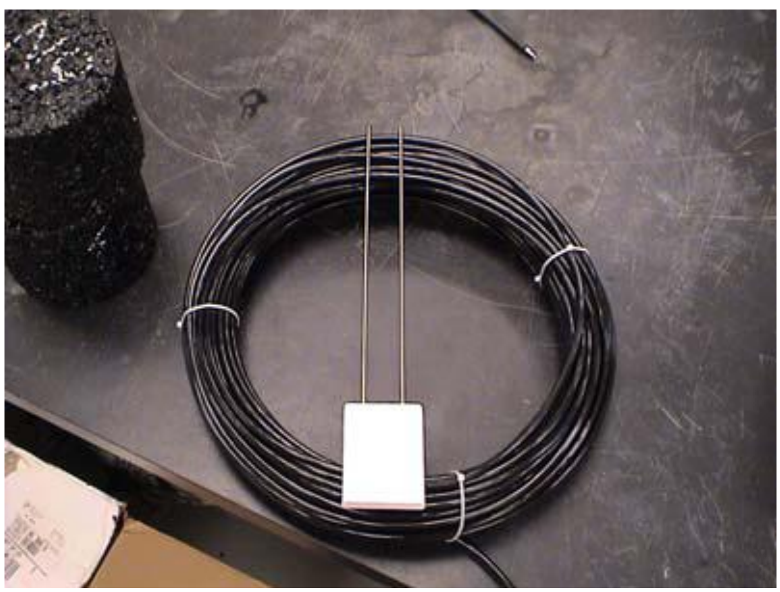


Figure 3-15: Moisture Probe

3.5.5 Data Logger

The data logger used in this research is the CR-1000 manufactured by Campbell Scientific. In addition, 1 GB compact flash card is used to store the data collected. The CR-1000 is capable of scanning all sensors attached at a rate of 1 Hz. The CR-1000 includes 8 channels in case of double wired sensors or 16 single wired sensor channels. All the sensors which are to be installed are double wired. Thus, several multiplexers will have to be used due to the large number of channels needed which exceeds the available number of channels in the CR-1000. Solar panel will be used to provide the sufficient electrical power for the operation of the data logger and the sensors.

Figure 3-16: CR-1000 Data Logger

3.6 Installation Procedure

According to the sensor installation plan, all stations were to be fully equipped by Asphalt Strain Gauges, Earth Pressure Cells, Thermistor Strings and Moisture Probe. Installation per station should have been as follows:

- Twelve Asphalt Strain Gauges (ASGs).
- One Earth Pressure Cell (EPC).
- One Moisture Probe (MP).
- One Thermistor String (TS).

During stage one of the project construction, three monitoring stations were constructed and equipped with all the sensors according to the previous plan. Due to construction constraints, Thermistor strings were excluded from the instrumentation plan in the three monitoring stations of stage one. This is due to the inability to make any cores on the left lane of the Highway 401 as a result of the staging. In addition, there was a highly likelihood that the thermistor string cable may be damaged as it will be unprotected on the left lane of the highway. The distance between the concrete barriers and the marking line identifying the boundaries of the lane is also unsafe and insufficient to protect the Thermistor String cable. Based on the previous construction and technical reasons, instrumentation plan was modified by installing the moisture probes in stage two of construction in the paved shoulder next to the right lane (lane one).

3.6.1 Installation Infrastructure

Site preparation, opening trenches, conduit installing and construction of concrete foundations for the three monitoring station cabinets were the main activities accomplished in the pre-installation phase.

Stage one of the instrumentation plan was designed to install different sensors on the lane three (left lane) of Highway 401. The data logger and solar panels location is designed to be located on the right side of the highway shoulder. This design provides a safe access to the data logger and the solar panel. The research team at the CPATT needed a safe access to the data logger through all seasons to facilitate the data collection procedure. Based on the previous instrumentation plan, the installation of a conduit crossing in the subgrade material of the Highway 401 was essential to allow cable passage from lane three (left lane) to the highway shoulder and thus to the location of the data logger cabinet. The open trench technique was used in the conduit installation process. A steel pipe of 31 cm diameter was installed 1.5 m deep in the subgrade. Eight PVC tubes of five cm diameters were installed inside the steel pipe. Figures 3-17, 3-18, 3-19, 3-20 and 3-21 show the conduit dimensions and conduit installation process.

Figure 3-17: Steel Conduit Used for Cable Crossing

Figure 3-18: Steel Conduit Before Installation

Figure 3-19: Pavement Marking Prior to Open Trench Excavation

Figure 3-20: Conduit Installation

Figure 3-21: PVC Tubes Inside the Steel Conduit

3.6.2 Asphalt Strain Gauge (ASG) Installation

Construction and instrumentation reports of several test sections served as a sound background and provided the CPATT research group with a wide prospective and expectation for the ideal methodology that was followed in other projects as the NCAT, Virginia Smart Road, MnRoad and the Marquette Interchange test section [Timm 2004 (b)] [Baker 1994] [Hornyak 2007]. In addition, the Asphalt Strain Gauge manufacturer and supplier – Construction Technology Laboratories Inc. (CTL) – provided the CPATT research team with technical advice and installation recommendations. The installation procedure provided by CTL was modified due to the construction and topographic constrains characterizing this particular research project. However, all modifications and the installation procedure followed by CPATT team were approved to be efficient by the technical support group of CTL.

The installation procedures followed for installing Asphalt Strain Gauges (ASGs) are as follows:

1. Prior to installation, the exact location of each strain gauge was determined accurately and marked.
2. A trench was excavated in parallel to the highway longitudinal direction, leading to the conduit edge. This trench was used to protect the strain gauge cable by burying them in the longitudinal trench and then inserting the cables in the crossing conduit. During this process, enough cable was left unburied in the longitudinal trench to ensure the Strain Gauge can be installed at the appropriate location.
3. Using the same wire, left over Strain Gauge cables were pulled through the conduit from the highway shoulder and protected in waterproof bags and installed in a box under the shoulder surface level.
4. Strain Gauges and the remaining cable was protected in waterproof bags and placed under safety cones to ensure their safety and to protect them from being damaged by heavy equipment during the construction activities.
5. On the installation day, the Strain Gauge location was re-determined and marked before the paving equipment approached the installation zone. A ROADTEC RP190 asphalt paver was used in the asphalt paving operation in addition to the shuttle buggy ROADTEC SB2500D. The CPATT team then installed sensors after the asphalt paver placed the asphalt layer. Using shovels, the CPATT team and the paving crew started to remove the asphalt material creating a space for the Strain Gauge placement at the bottom of the asphalt layer. A trench was excavated so the Strain Gauge wires were pulled to the pavement edge to protect the wires from being damaged by the vibratory compactors.
6. After the Strain Gauge was placed at the bottom of the asphalt layer, the strain gauge was recovered by hot asphalt material obtained from the same mix. The trench was excavated in the asphalt layer so the gauge wire could be covered up. During the asphalt placement over the Strain Gauges, the CPATT group removed any sharp edged coarse aggregate particles from the asphalt mix in direct contact to the Strain Gauge to avoid damaging the gauge due to the passage of the vibratory compactors. The pressure composed by the compactors on the gauges could damage the gauges so the vibratory mode was turned off when the compactors ran over the gauges..

7. Based on the Ministry of Transportation of Ontario (MTO) and the Centre for Pavement and Transportation Technology (CPATT), vibration mode of the vibratory compactor was disabled during the passage over the sensor location and within a ten meter boundary zones upstream and downstream the installation location.

Figures 3-22, 3-23, 3-24, 3-25, 3-26 are showing the steps and installation procedure followed for installing the Asphalt Strain Gauges (ASGs).

Figure 3-22: Pre-Installation Site Preparations

Figure 3-23: Asphalt Paver and Shuttle Buggy

Figure 3-24: Removing Asphalt Material to Place the Strain Gauge at Bottom of the Layer

Figure 3-25: Placement of Strain Gauge and Covering the Wire Trench

Figure 3-26: Monitoring Station after Strain Gauge Installation and Before Compaction

3.6.3 Earth Pressure Cell (EPC) Installation

Earth Pressure Cells (EPCs) were installed at the top of subgrade under the right wheel path to capture data that will enable researchers to predict subgrade rutting phenomenon and the associated structural distresses.

The steps followed for the EPC installation were based on the previous projects experience and installation reports. Background for EPC installation was gained through the same projects where ASG were also installed.

The following steps were followed during the installation of EPC:

1. After the installation of the Granular A and Granular B layers, the two layers were excavated to place the EPC directly on top of the subgrade layer.
2. A trench is excavated for the EPC cable leading to the crossing conduit entrance.
3. Using the gauge wire, the EPC cable is pulled across the highway in the conduit. The cable is then protected in a waterproof bag and placed in a box on the highway shoulder.
4. The trenches that were excavated are all covered using the same material (Granular A and Granular B) and well compacted to be prepared for the paving process.
5. The EPC installation was planned as close as possible to the paving date in order to avoid the damage of the EPC due to loads cause by heavy equipment used in construction site.

Figure 3-27 shows the EPC placed on the subgrade layer.

Figure 3-27: Installation of Earth Pressure Cell

3.6.4 Moisture Probe (MP) Installation

Moisture Probes (MPs) were installed 40 centimeters deep in the subgrade layer to monitor the moisture content in the subgrade layer. The purpose behind this is to predict the freeze-thaw cycle impact over the pavement structural performance.

The following procedures were followed when installing the Moisture Probes (MP):

1. Before installing the Granular B layer, a trench of 40 centimeters deep is constructed in the subgrade leading to the location of the MP.
2. Manual compaction is carried out to level the soil below the MP is placed. After the manual compaction is performed, the moisture probe is placed and covered with the same subgrade material. The subgrade layer should be re-compacted after the MP installation.
3. A trench is excavated for the moisture probe cable leading to the crossing conduit entrance. The trench should be of the same depth where the moisture probe was installed.
4. The MP cable is pulled through the crossing conduit using the wire. The cable is protected in a waterproof bag and placed in a box buried in the highway shoulder.

Figures 3-28, 3-29 and 3-30 show the procedures followed for installing the MP.

Figure 3-28: Compaction of Subgrade Surface Prior to Moisture Probe Installation

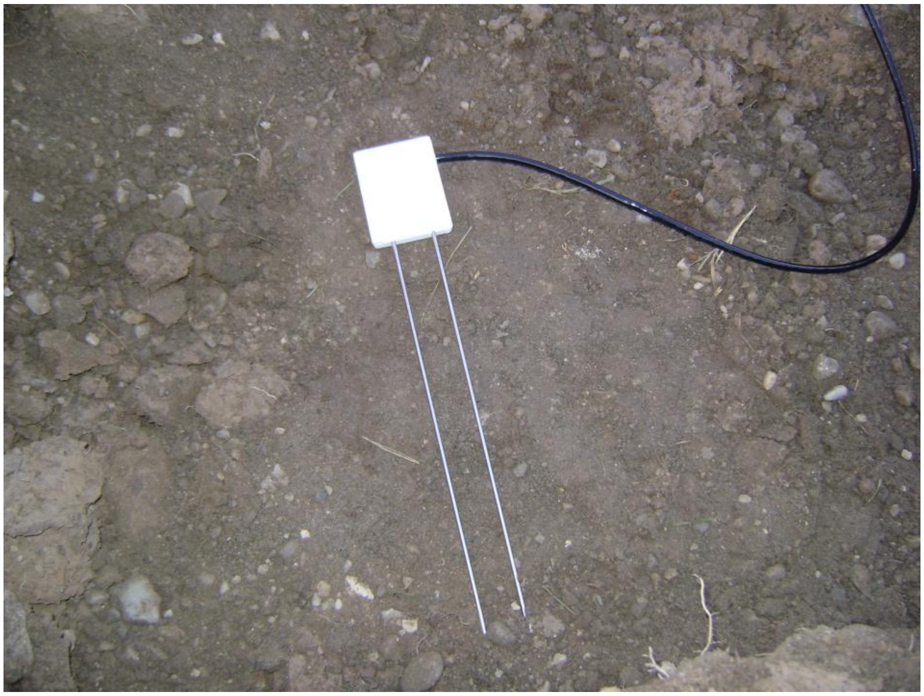


Figure 3-29: Moisture Probe Installation

Figure 3-30: Conduit end on the Highway Shoulder and the Box where Cables are protected

3.6.5 Thermistor String (TS) Installation

Thermistor String (TS) are the last sensor to be installed. Stage two is taking place in summer 2009 and the installation of the Thermistor Strings (TS) should be at the end of the construction stage.

The following procedures have been adopted from previous CPATT research [Baiz 2008] [Mabood 2008] and are as follows:

1. Once all the asphalt layers are placed and compacted, a vertical core should be cored in the paved shoulder to a depth that is slightly longer than the Thermistor String length.
2. The Thermistor String is then placed in a PVC tube to protect it and then installed vertically in the vertical core.
3. The space around the PVC tube should be filled by the granular material that was cored and the asphalt layers are replaced by cold asphalt mix to ensure stability of the Thermistor String.

Figure 3-31 shows the cross section in the core made in the paved shoulder [Esch 2004].

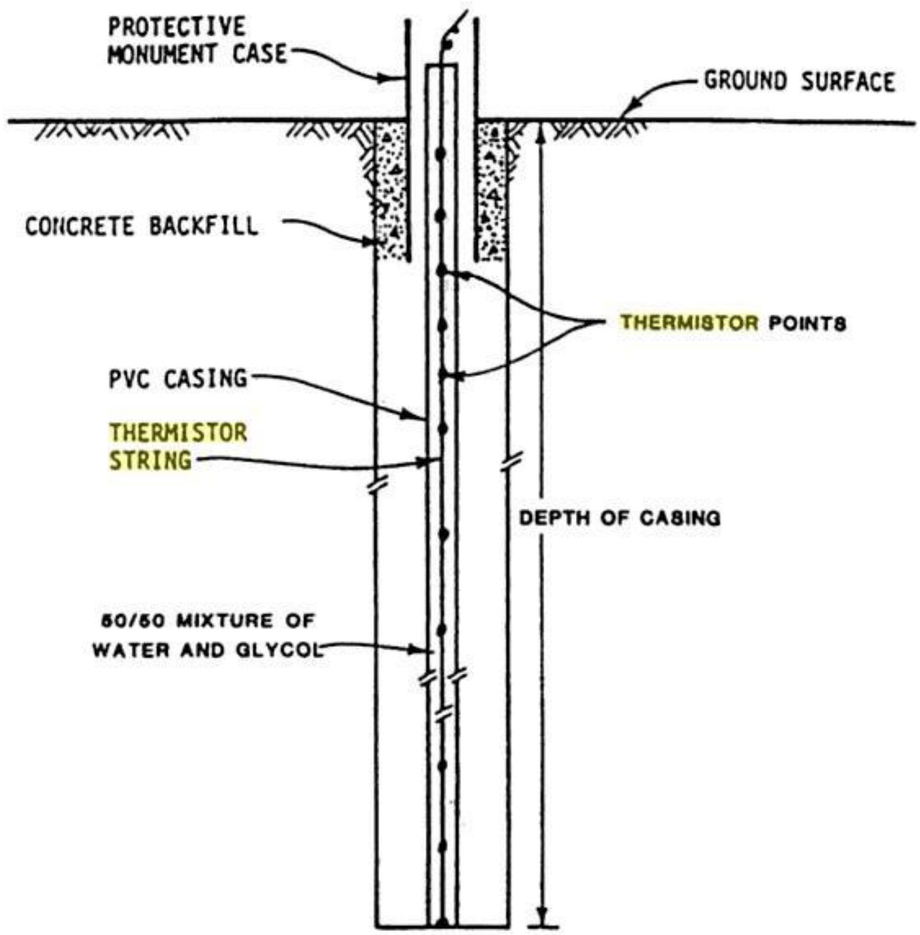


Figure 3-31: Cross Section Showing the Thermistor String Installation Procedure [Esch 2004]

Chapter 4
Structural and Economic Evaluation of Pavement Designs

This chapter provides a full preliminary evaluation of the three different pavement designs. The three different pavement designs were assessed based on structural, technical and economic evaluation.

4.1 Introduction

The pavement structural and economic evaluation provides necessary benchmarks for performance. In short, prior to the construction and instrumentation phase, the research team needs to do a pre-engineering evaluation whereby a general performance model is assumed. The three pavement designs that were designed by the Ministry of Transportation of Ontario (MTO) for the Highway 401 instrumentation project were analyzed structurally using the newly engineered Mechanistic Empirical Pavement Design Guide (MEPDG). The mechanical and physical properties needed for the model creation were determined and calculated during this research. In order to ensure the behavior of the perpetual pavement design is acting as a Long life pavement, a numerical model representing the two perpetual and one conventional pavement designs were created and analyzed using the WESLEA for Windows software. This model was used to predict the normal strain values at the bottom of the asphalt layers. In addition to the structural evaluation, a Life Cycle Cost Analysis (LCCA) was performed for evaluating the three pavement designs over the entire life cycle. The maintenance and rehabilitation activities were based on state of the practice of the Ministry of Transportation of Ontario (MTO) recommendations. The maintenance and rehabilitation reports and experience of the research team provided a reliable data source for different maintenance activities and its expected cost.

4.2 Mechanistic Empirical Pavement Design Guide (MEPDG) Model

An evaluation model was created using Mechanistic Empirical Pavement Design Guide (MEPDG) software version 1.003 to mechanistically evaluate the three pavement structures, including asphalt mixtures that is used for construction of these pavements [Schwartz, 2007]. The MEPDG software is unique as it predicts the pavement performance with regard to several distress types in addition to providing roughness measurements. The MEPDG software outputs include pavement performance predications to surface down cracking, bottom up damage for fatigue (alligator) cracking, thermal cracking, rutting and International Roughness Index (IRI) values expected through the analysis time. The evaluation of both pavement structures assumed an analysis period of 50 years.

4.2.1 MEPDG Model Inputs

A summary of the MEPDG model inputs for the conventional pavement section, perpetual pavement section with Rich Bottom Mix (RBM) layer and perpetual pavement section without Rich Bottom Mix (RBM) are presented in Tables 4-1, 4-2 and 4-3 respectively. The MEPDG model inputs were provided by one of our research partners, Capital Paving Limited. Capital Paving Limited, the subcontractor was responsible for: providing all paving materials to the Highway 401 site, installation and compaction of the layers. The values used in the research came from several standard pavement material tests. The inputs that were not available from the laboratory testing, were based on state of the art practice [D'Angelo 1998][Kandhal 1998]. Further research will be carried out in the future on these materials used in the experiment.

Table 4-1: MEPDG Inputs for Conventional Pavement Section

Layer 1		**Layer 3**	
Thickness (mm)	40	Thickness (mm)	90
PG	PG 70-28	PG	PG 58-22
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 4**	
Thickness (mm)	170	Thickness (mm)	200
PG	PG 64-28	Aggregate base layer (Granular A) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2

Layer 5	
Thickness (mm)	200
Aggregate subbase layer (Granular B) Permeable layer	
Maximum dry unit weight (pcf)	127.2
Specific gravity of solids, Gs	2.7
Saturated hydraulic conductivity (ft/hr)	0.05054
Optimum gravimetric water content (%)	7.4
Calculated degree of saturation (%)	61.2

Table 4-2: MEPDG Inputs for Perpetual Pavement with RBM Section

Layer 1		**Layer 4**	
Thickness (mm)	40	Thickness (mm)	100
PG	PG 70-28	PG	PG 58-28
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	2.4
Volumetric binder content (%):	13	Volumetric binder content (%):	14
Total unit weight (pcf):	153	Total unit weight (pcf):	152
Layer 2		**Layer 5**	
Thickness (mm)	180	Thickness (mm)	200
PG	PG 64-28	Aggregate base layer (Granular A) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2
Layer 3		**Layer 6**	
Thickness (mm)	100	Thickness (mm)	550
PG	PG 58-28	Aggregate base layer (Granular B) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	12	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	151	Calculated degree of saturation (%)	61.2

Table 4-3: MEPDG Inputs for Perpetual Pavement without RBM Section

Layer 1		**Layer 4**	
Thickness (mm)	40	Thickness (mm)	100
PG	PG 70-28	PG	PG 58-28
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 5**	
Thickness (mm)	180	Thickness (mm)	200
PG	PG 64-28	Aggregate base layer (Granular A) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2
Layer 3		**Layer 6**	
Thickness (mm)	100	Thickness (mm)	550
PG	PG 58-28	Aggregate base layer (Granular B) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	12	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	151	Calculated degree of saturation (%)	61.2

The MEPDG model was created with Level Three inputs. The climate data file used in the model implementation was created from downloading the data monitored in the Niagara Falls, New York. This weather station is the closest weather station to the project location. The distance between the project location and the Niagara Falls, New York weather station is 160 km. This approximation in weather conditions is believed to be acceptable due to similarity of most climatic characteristics of the two areas.

4.2.2 MEPDG Model Results

The MEPDG model outputs provide a reliable overview about the pavement performance during the analysis period. The analysis period in the model created was 50 years. Another model was created to analyze the pavement performance for 70 years but unfortunately that model caused computer crash and hibernation due to hardware/software limited capabilities. No outputs were obtained from the 70 year analysis period model.

The 50 year analysis period model produced the expected pavement structural performance expressed by the surface down cracking damage percentage, bottom up fatigue cracking and the associated damage percentage, International Rutting Index (IRI), rutting depth in asphalt layers, base layer and the total rutting.

In general, all the MEPDG results show a superior performance prediction for the perpetual pavement structure over the more traditional conventional pavement structure.

The MEPDG analysis predicted that surface down cracking for the perpetual pavement design will be minimal to that of the conventional design. This shows that the actual surface down crack propagation is less likely to occur in the perpetual pavement structures compared to conventional asphalt pavement structure. In addition the perpetual design with Rich Bottom Mix (RBM) layer have shown better surface down cracking performance in comparison with the perpetual design without Rich Bottom Mix (RBM) layer. The results of the surface down cracking model are not accurate for either the conventional or perpetual designs. The MEPDG runs were repeated with slightly different inputs for several times and the data inputs were double checked but the results were almost identical. This may be due to some form of error in the MEPDG surface down software model. Overall, this result reflects only the performance trend of the two pavement designs. Figure 4-1 and 4-2 represents the surface down cracking damage percentage for both conventional pavement design and the two perpetual pavement designs respectively.

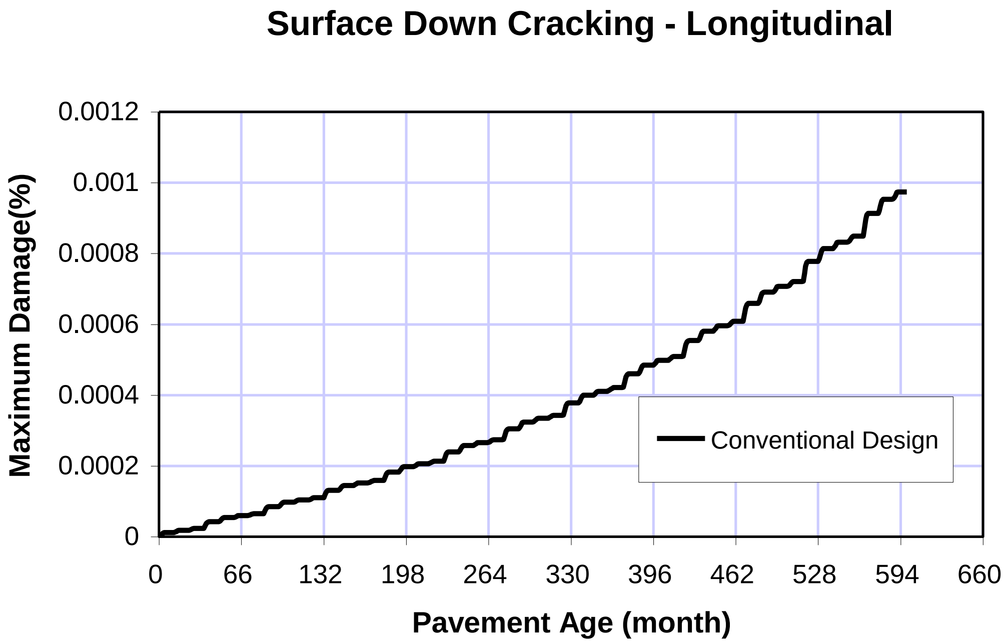


Figure 4-1: Surface Down Cracking Damage Percentage for Conventional Design

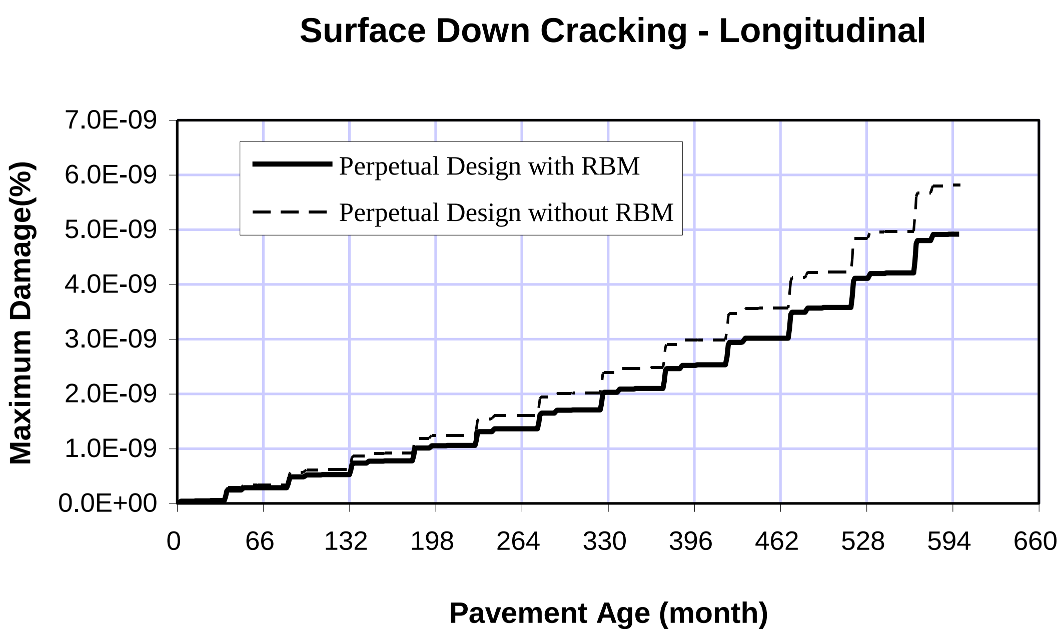


Figure 4-2: Surface Down Cracking Damage Percentage for Perpetual Designs

It is noted that the perpetual pavement design with Rich Bottom Mix (RBM) layer have deteriorated in a slower rate compared to that of the perpetual design without the Rich Bottom Mix (RBM) layer.

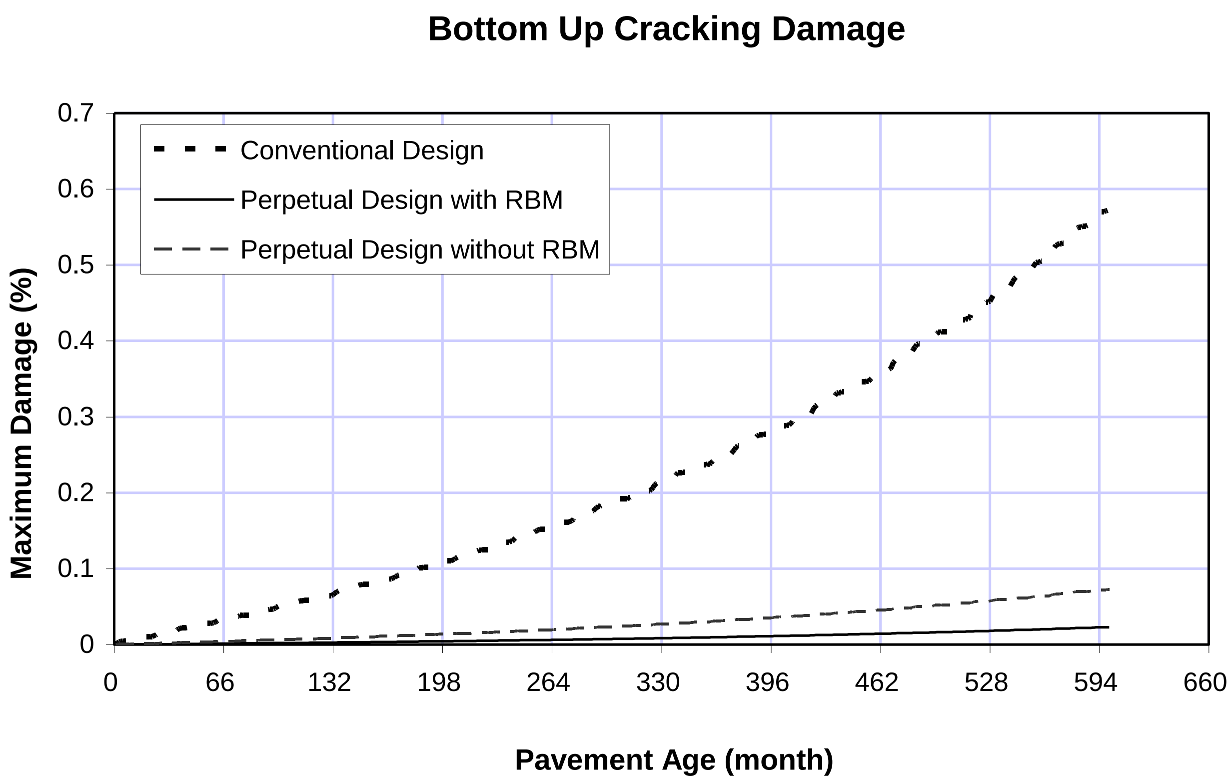


Figure 4-3: Bottom Up Cracking Damage

Figure 4-3 shows the benefits of perpetual pavement construction specially when associated with a Rich Bottom Mix (RBM) layer. The Rich Bottom Mix (RBM) layer proved to be the most optimum solution for fatigue bottom up cracking compared to the perpetual design without Rich Bottom Mix (RBM) layer and the conventional design. The deterioration rate of the conventional design proves that this pavement design will suffer structural damage and bottom up cracks in the short term and will require a more intensive and expensive maintenance and rehabilitation program compared to the perpetual designs.

Figure 4-4 represents the expected structural resistance of the alligator fatigue cracking in the conventional and perpetual pavement designs with and without the Rich Bottom Mix (RBM) layer. The MEPDG model results shows that the perpetual pavement designs are expected to have higher fatigue cracking resistance compared to the conventional design. In addition, the Rich Bottom Mix (RBM) layer proved to increase the pavement section resistance against the alligator fatigue cracking.

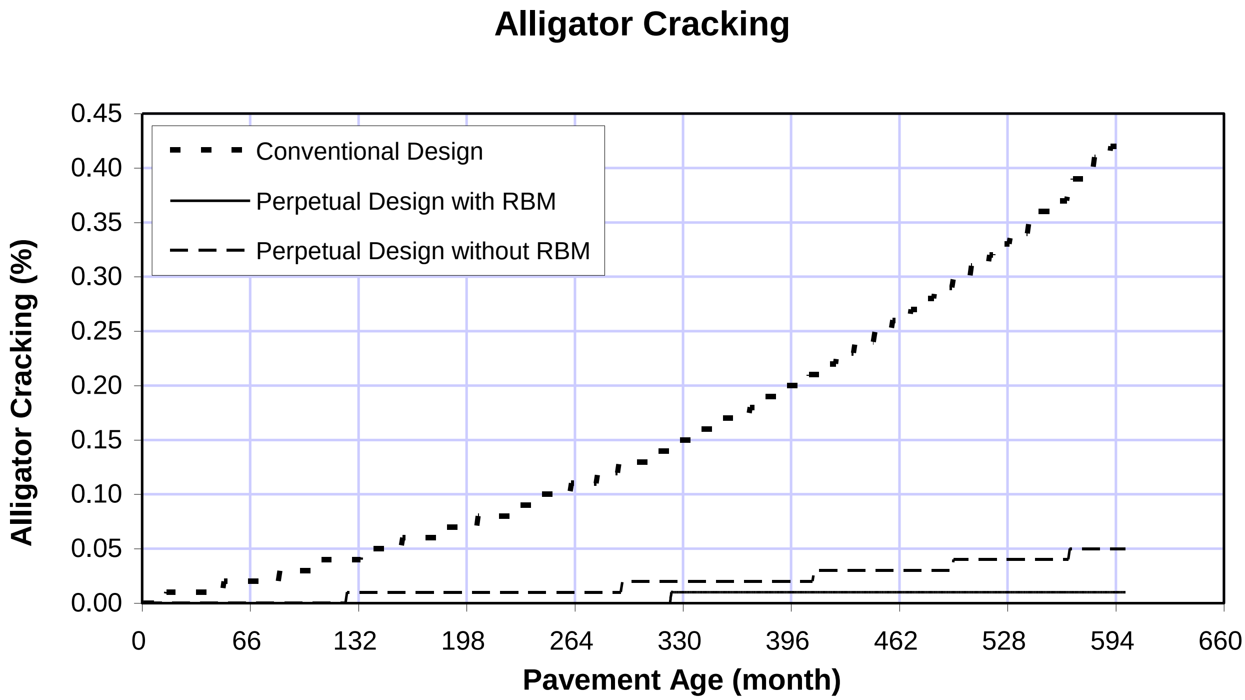


Figure 4-4: Bottom Up Alligator Cracking

Figures 4-5 and 4-6 shows the Mechanistic Empirical Pavement Design Guide (MEPDG) rutting model results for the three pavement designs. Both perpetual pavement designs have shown better performance in rutting throughout the analysis period. Analyzing Figure 4-5, the base rutting in both conventional and perpetual with Rich Bottom Mix (RBM) designs is almost the same. The main factor behind the difference in the total rutting in both pavement designs is the total asphalt layers. The thick asphalt layers are showing enormous rutting resistance to the rutting phenomenon. It is also noticed that in the first five years, the base rutting is primer type of rutting and having the highest contribution to the total rutting. After the fifth year, the rate of increase in the base rutting decreases tremendously while that of the asphalt layers rutting remains in a linear trend and becomes the primer factor behind the total overall rutting. Figure 4-6 shows the rutting model results for the conventional pavement design and the perpetual pavement design without Rich Bottom Mix (RBM) layer. The results shows that the performance of the perpetual design without Rich Bottom Mix (RBM) layer is subjected to less rutting values compared to that in the conventional design. When comparing the two perpetual asphalt pavement designs, the rutting models created for both pavement types gave almost the same results. The charts for both pavement designs were identical but by reviewing the data a slight difference was deduced that's giving advantage to the perpetual design with Rich Bottom Mix (RBM) layer. This slight difference is showing the impact of adding 0.5% of asphalt content to the bottom asphalt layer of the design as both perpetual designs are identical with the exception of the increase in the asphalt content at the bottom asphalt layer.

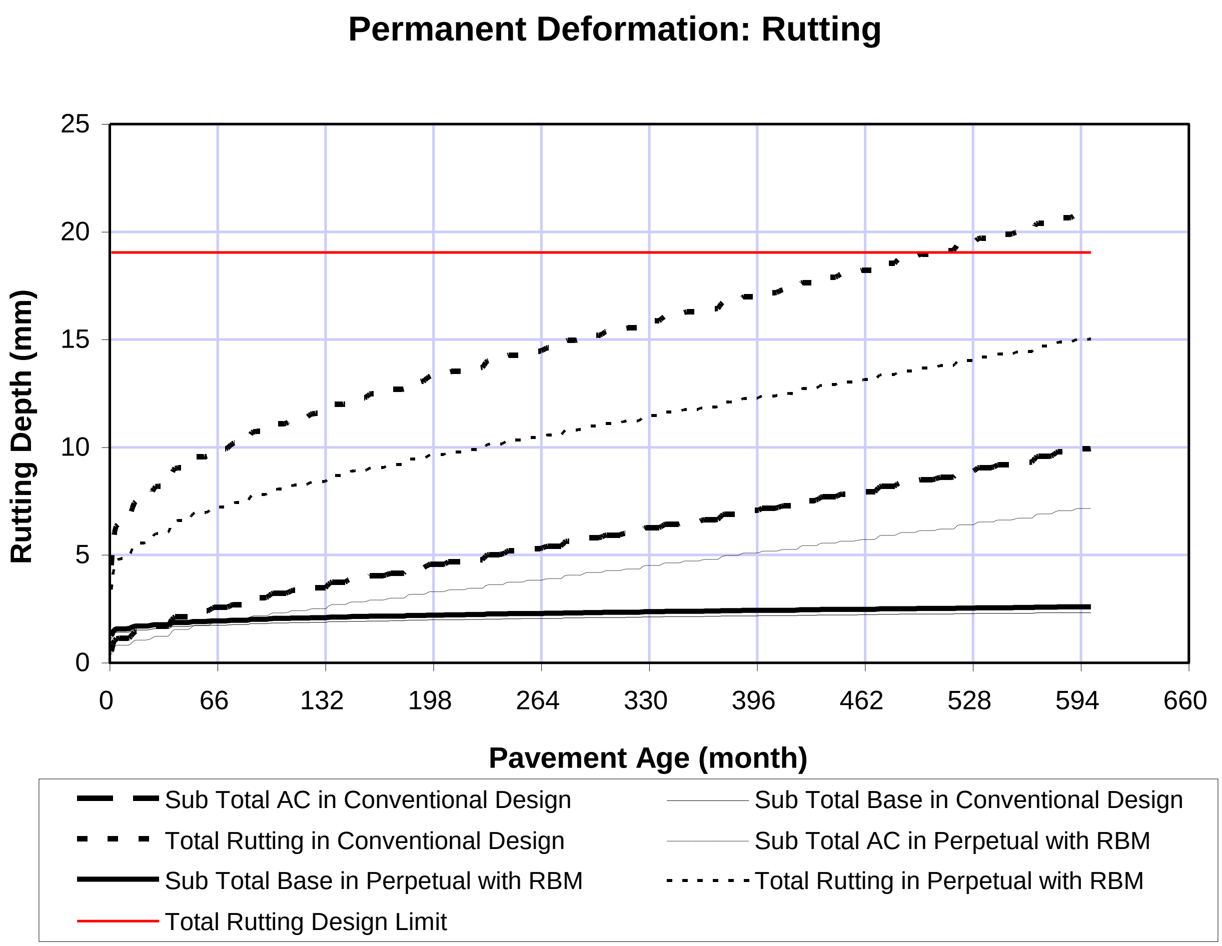


Figure 4-5: Rutting in Conventional and Perpetual design with RBM

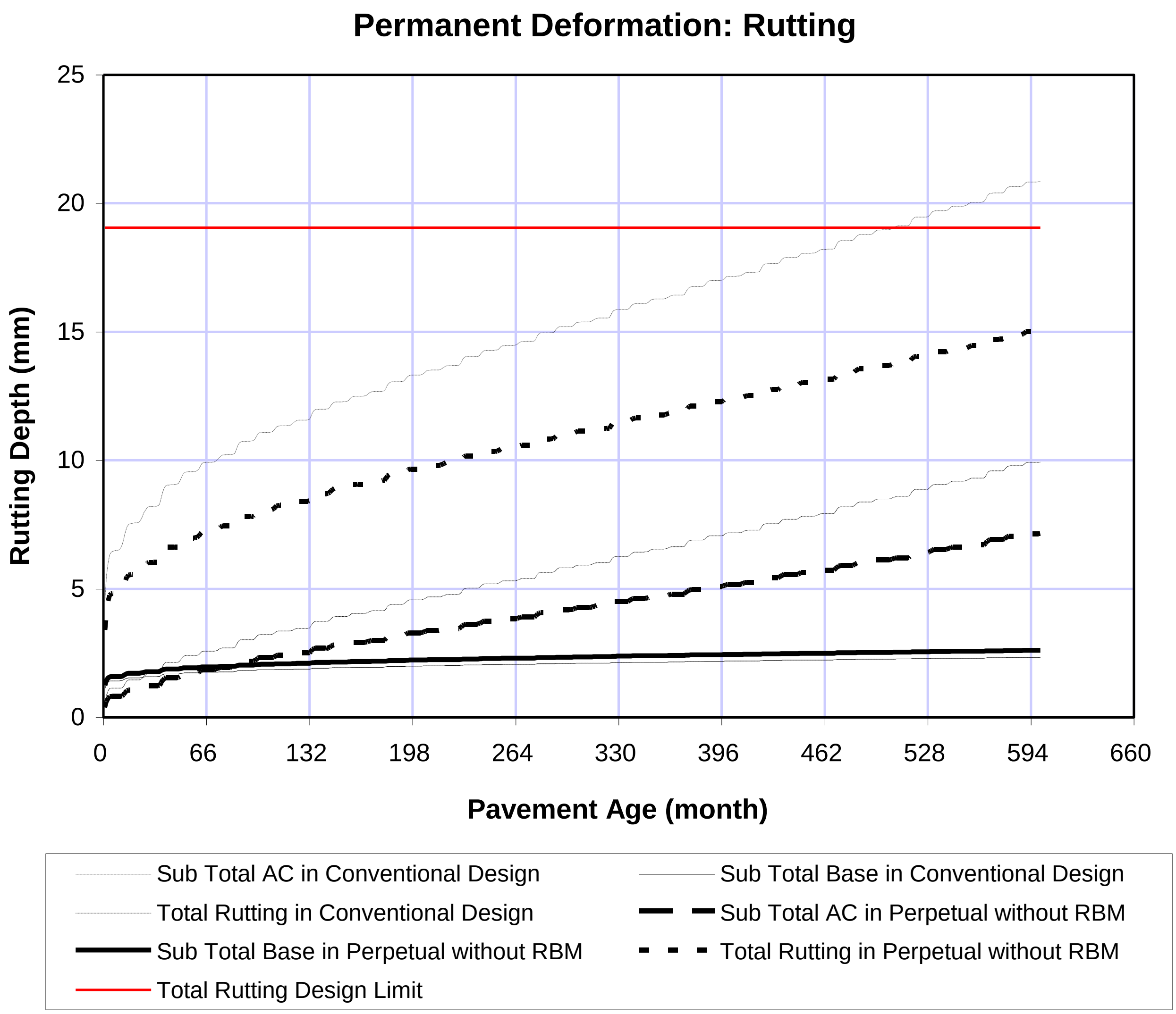


Figure 4-6: Rutting in Conventional and Perpetual design without RBM

Figure 4-7 presents the International Roughness Index (IRI) model results for all three pavement designs. It is obvious that all three pavement mixes IRI values were equal immediately after construction. The rate of deterioration of the International Roughness Index (IRI) of the conventional asphalt pavement design is higher than that of the perpetual designs. In addition, the Rich Bottom Mix (RBM) layer provided extra resistance for the perpetual asphalt pavement design including the Rich Bottom Mix (RBM) layer compared to that without that layer. The deterioration of International Roughness Index (IRI) can be resolved by replacing the surface course every ten years approximately to maintain the IRI values.

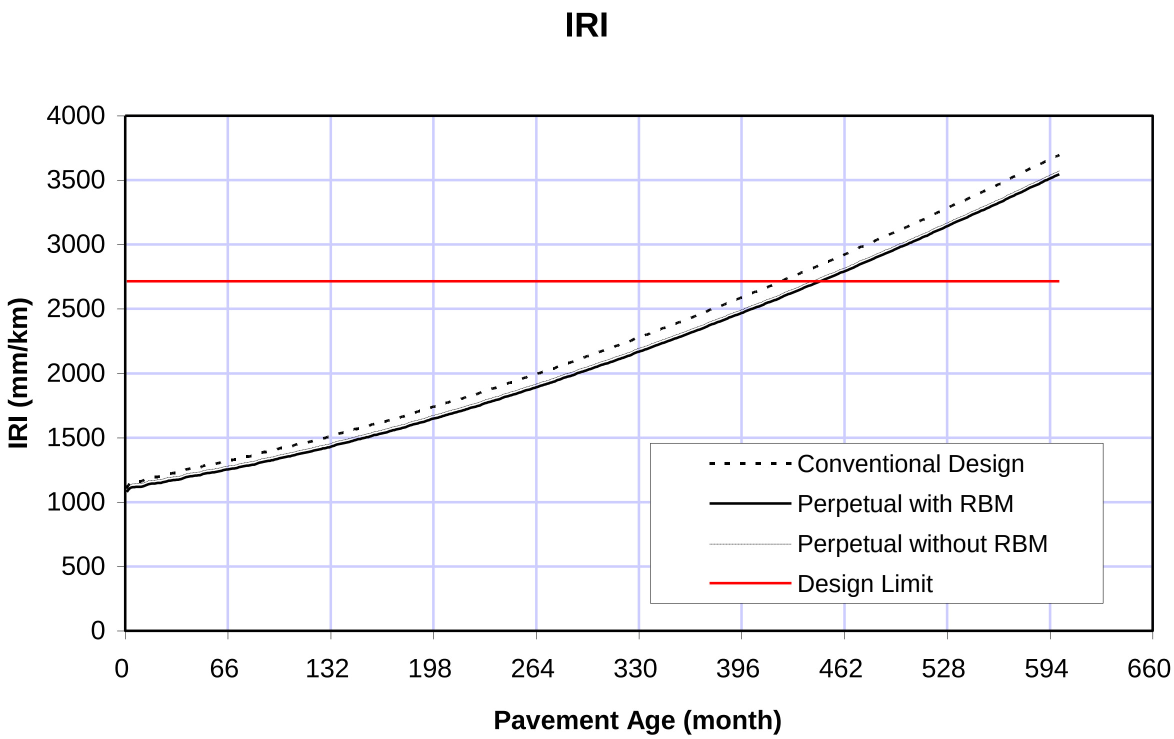


Figure 4-7: International Roughness Index (IRI) Model Results

4.3 WESLEA for Windows Model

The WESLEA for windows version 3.0 was also used to evaluate the long term pavement performance of the perpetual design in comparison to that of the conventional design. The WESLEA software is a modeling computer program that can predict the normal strain values, displacement, normal stress and shear stress in the critical coordinates and depths. The analysis of WESLEA model is focused on the normal strain predicted in both the conventional and perpetual designs. The main factor characterizing the perpetual design is that its normal strain value at the bottom of the asphalt layers should not exceed 70 microstrains (μs). The conventional design is generally expected to be subjected to a higher strain value at the bottom of the asphalt layers. Comparisons between different strain values were performed for the coordinates representing the strain values under the wheel path. The strain values mostly affecting the pavement performance is the strain in the longitudinal direction (Y direction).

4.3.1.1 Model Inputs

The WESLEA model is created based on various data that enables the program to calculate the strain, displacement, normal and shear stresses at any point. The data required for the model implementation included the number of layers, thickness of each layer, the material of the layer, dynamic modulus and poisson's ratio for the layers. In addition the loading configuration (single axle, tandem axle, and others), load magnitude and tire pressure are the data required to calculate the load applied on the pavement section defined in the model. Figures 4-8 and 4-9 are showing windows from the program that is used to define the pavement section and load applied.

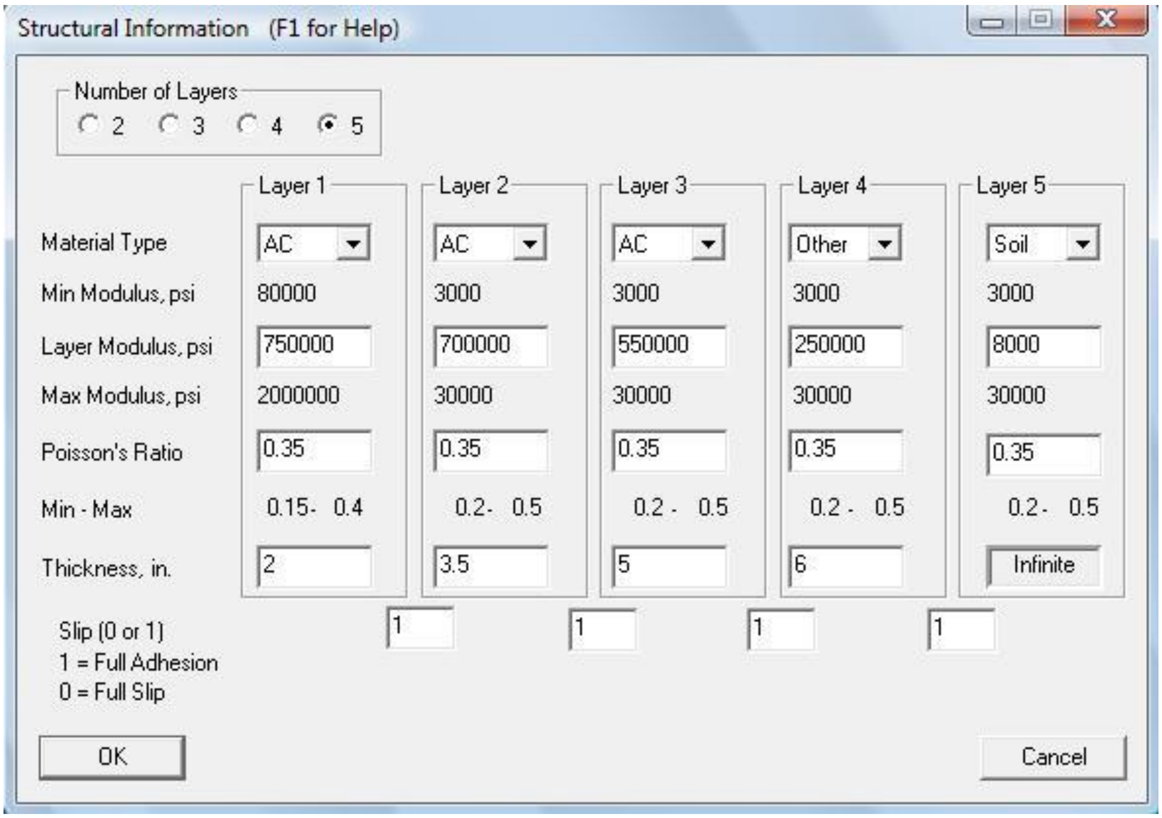


Figure 4-8: WESLEA Pavement Section Input Window

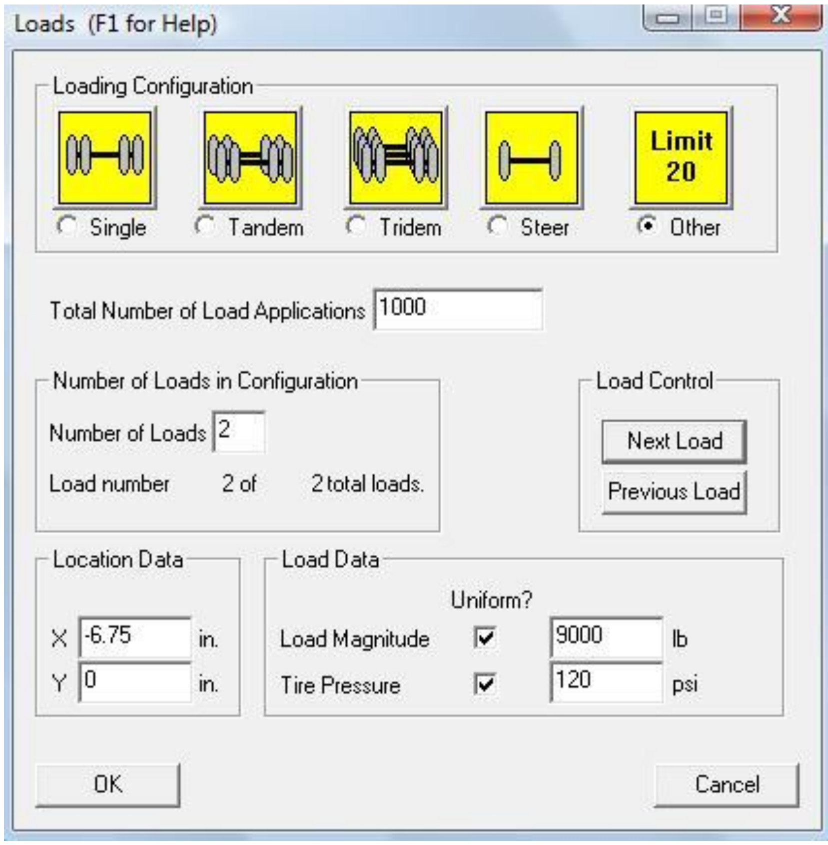


Figure 4-9: WESLEA Loading Input Window

It is important to emphasize that the properties of all pavement layers (modulus of elasticity and Poisson's ratio) were assumed based on a comprehensive literature review of Hot Mix Asphalt (HMA) dynamic modulus data obtained from the FHWA Design Guide Implementation Team (DGIT). Load location, magnitude and pressure tire were assumed to be the default values in the software as shown in figure 4-9.

4.3.1.2 Model Results

The model results are showing that normal strain value obtained at the bottom of the perpetual pavement design have not reached the 70 µs limit and thus is expected to perform as long life pavement. The normal strain value at depth of 420 mm (bottom of asphalt layers) was predicted to be 26.4 µs. This strain value reflects the high fatigue resistance that is characterizing the perpetual asphalt layers. The analysis of the conventional design have also showed that the normal strain value at the bottom of the asphalt layers under the wheel path have not exceeded the 70 µs. The normal strain value for the conventional design at depth of 300 mm (bottom of asphalt layers) is expected to be 45 µs. Although the normal strain values in both pavement designs have not exceeded the perpetual design limit, there is a significant difference between the normal strain value in the perpetual and conventional designs. This significant difference shows the impact of the thick asphalt section and the RBM layer characterizing the perpetual pavement design. The reason behind the low normal strain values in both pavement designs is believed to be the conservative design factors used by the designer of the Highway 401 project. Figure 4-10 shows the results of the two WESLEA models created for the perpetual and conventional models.

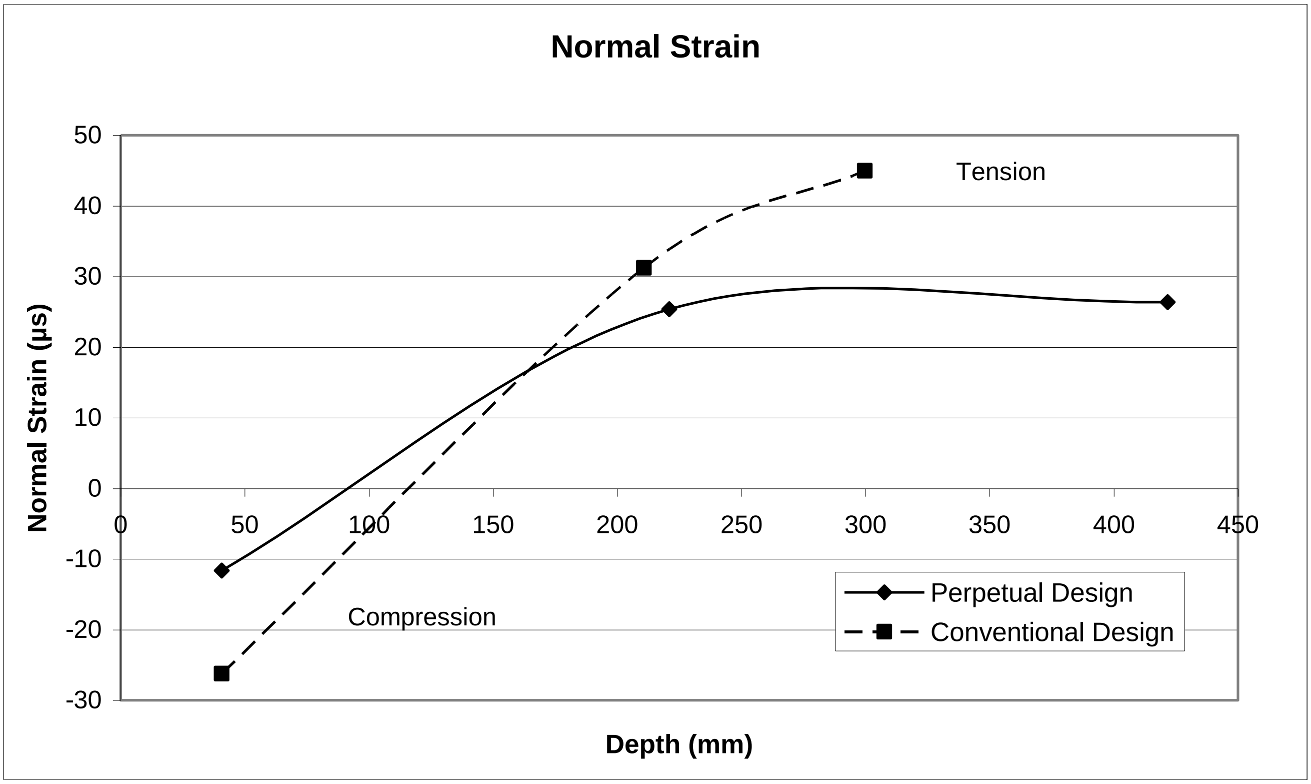


Figure 4-10: WESLEA Results Showing Normal Strain of Conventional and Perpetual Designs

4.4 Life Cycle Cost Analysis (LCCA)

Life Cycle Cost Analysis (LCCA) was also performed to evaluate the conventional and perpetual pavement design methods. The construction difference between the two perpetual pavement designs takes place in the asphalt content of the bottom asphalt layer. The addition of 0.5% of binder content in the bottom asphalt layer is expected to have minor effect on the LCCA. To simplify the LCCA calculations, comparison and evaluation was performed on the conventional pavement design and the perpetual pavement design with RBM as a representative for the perpetual designs. Superior pavement performance predictions supported by reasonable economic analysis is essential to justify any capital investment.

It is important to highlight that the life cycle cost analysis (LCCA) procedure will be performed under the following assumptions:

1. Best possible unit cost estimates for pavement material, maintenance and rehabilitation, and labor in Ontario are obtained through the Ministry of Transportation of Ontario (MTO). The

final life cycle cost analysis reports submitted to MTO in 1998 and 2006 were used for estimating the material, maintenance and rehabilitation costs [Hein 2007] [Smith 1998]. However if necessary, some unit costs were assumed based on national averages.

2. The LCCA evaluation period is proposed to be 70 years for the two pavement design alternatives.
3. Preventative maintenance, scheduled maintenance, and/or rehabilitation treatments were assumed based on the recommendations of the MTO reports.
4. Inflation costs per treatment and/or maintenance activities are not used and are assumed constant between different rehabilitation options. This is a common practice that is mostly used in LCCA.
5. LCCA was conducted at three percent discount rate.
6. Initial construction costs will include labor and materials' costs associated with the constructions of the pavement structure.
7. User delay costs during different maintenance and rehabilitation activities were not taken into account in this LCCA due to the lack of sufficient data and to simplify the LCCA calculation.

It is also important to emphasize the following additional LCCA assumptions:

1. The cost of construction at year zero was based on the unit cost item per ton. The material weight was determined by assuming a unit length of 1 km and road width of 45 m (3 lanes per direction, each of 3.75m width), the thickness of each layer was previously presented in Figures 3-5 and 3-7. Multiplying every layer's volume in cubic meters by the material's density in ton/m3, we can determine the weight needed to pave 1 km of the road.
2. Material costs assumed based on the 1998 LCCA Ontario report, were modified as a result of the inflation rate and price changes: equation 1 was used to modify the material costs: inflation rate was assumed to be 3%.

$$\text{Present cost} = \text{Cost at } 1998 \times (1+r)^n \qquad \text{(Eq. 1)}$$

Where

r = Inflation rate (3%)

n = Number of years between the year at which data was collected and the present (ten years)

3. Longitudinal cracks were assumed to have developed along the joints and/or at both lane edges. While; transverse cracks were assumed to have developed at each 90 m edge to edge.
4. The cost of mill and overlay was based on the unit cost per square meter and the area expected to be overlaid.
5. The salvage value was assumed to be twenty percent of the total pavement cost for conventional asphalt concrete and for the full depth perpetual pavement.

The construction costs of the perpetual asphalt pavement and conventional asphalt pavement are presented in tables 4-4 and 4-5 respectively.

Table 4-4: Initial Construction Cost of Perpetual Pavement Design

Length (m)	(40 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(180 mm) Superpave 19 Density = 2.41 t/m^3	(100 mm) Superpave 25 Density = 2.34 t/m^3	(100 mm) RBM Layer Density = 2.44 t/m^3	(200 mm) Granular A Density = 3.12 t/m^3	(550 mm) Granular B Density = 2.05 t/m^3	SUM
1,000	$238,797	$744,401	$303,732	$348,920	$238,680	$253,688	$2,128,217.10
1,000	$238,797	$744,401	$303,732	$348,920	$238,680	$253,688	$2,128,217.10
						Avg	$2,128,217.10
						Sum	$4,256,434.20

Table 4-5: Initial Construction Cost of Conventional Pavement Design

Length (m)	(40 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(180 mm) Superpave 19 Density = 2.41 t/m^3	(90 mm) Superpave 25 Density = 2.34 t/m^3	(200 mm) Granular A Density = 3.12 t/m^3	(200 mm) Granular B Density = 2.05 t/m^3	SUM
1,000	$170,100	$515,970	$238,140	$238,680	$92,250.0	$1,255,140.0
1,000	$170,100	$515,970	$238,140	$238,680	$92,250.0	$1,255,140.0
					Avg	$1,255,140.0
					SUM	$2,510,280.0

The initial construction cost of the perpetual pavement design is expected to be 70% more than that of the conventional pavement. The main reason behind that significant construction cost difference is the thick asphalt pavement layers that are characterizing the perpetual pavement designs. Both pavement designs are having common thickness and physical and mechanical characteristics in the first two asphalt layers. The difference in construction cost mainly occurs due to the addition of the Rich Bottom Mix layer in the perpetual pavement design and the difference in Granular B thickness in the two pavement designs. Although the additional asphalt layer known as Rich Bottom Mix layer will increase the capital cost of the perpetual pavement construction, it will add structural strength to the pavement design which will enhance the pavement performance and increase its structural resistance to different pavement distresses. The expected maintenance and rehabilitation program to preserve and maintain both pavement sections will reflect the benefits behind construction of the Rich Bottom Mix layer and the increase in the Granular B layer's thickness.

Table 4-6: Maintenance and Rehabilitation Program for Perpetual Pavement Design

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (280m/km)	4
Rout and Crack Sealing (280m/km)	8
3% Mill and Patch 40 mm	10
Rout and Crack Sealing (560m/km)	12
15% Mill and patch 40 mm	15
Mill 50mm Asphalt pavement	21
SMA- 50 mm	21
Tack coat	21
Rout and Crack Sealing (280m/km)	24
Rout and Crack Sealing (280m/km)	28
15% Mill and patch 40 mm	32
Rout and Crack Sealing (560m/km)	36
Mill 50mm Asphalt pavement	38
SMA- 50 mm	38
Tack coat	38
Rout and Crack Sealing (280m/km)	42
Rout and Crack Sealing (280m/km)	46
15% Mill and patch 40 mm	50
Rout and Crack Sealing (560m/km)	54
Mill 50mm Asphalt pavement	58
SMA- 50 mm	58
Tack coat	58

Maintenance and Rehabilitation Activity (Continue)	Year
Partial Reconstruction of Pavement	62
Rout and Crack Sealing (280m/km)	66
Rout and Crack Sealing (280m/km)	70

The maintenance and rehabilitation program for the perpetual pavement design is designed to treat the deformations and distresses occurring due to traffic loads, environmental impacts and physical and mechanical deterioration of material through time. Based on the MEPDG model results, the perpetual pavement design is not expected to suffer severe distresses or rapid structural deterioration. The most common distress that can be observed in perpetual pavement designs is the top down cracks. The appropriate maintenance treatment for this structural distress is by milling and patching the surface layer once this phenomenon is noticed to spread in order to avoid the propagation of cracks to the following asphalt layers.

Table 4-7: Maintenance and Rehabilitation program of Conventional Pavement Design

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (352 m/km)	3
Rout and Crack Sealing (352 m/km)	6
Rout and Crack Sealing (352 m/km)	9
5% Mill and patch 50 mm	9
Rout and Crack Sealing (704 m/km)	12
20% Mill and patch 50 mm	15
Rout and Crack Sealing (704 m/km)	18
Tack Coat	19
Mill 50 mm Asphalt Pavement	20
Superpave 12.5 FC2 - 50 mm	20
Rout and Crack Sealing (352 m/km)	21
Rout and Crack Sealing (352 m/km)	24
Rout and Crack Sealing (352 m/km)	28
20% Mill and patch 50 mm	28
Partial Reconstruction of Pavement	30
Rout and Crack Sealing (352 m/km)	33
Rout and Crack Sealing (352 m/km)	36
Rout and Crack Sealing (352 m/km)	39
5% Mill and patch 50 mm	39

Maintenance and Rehabilitation Activity (Continue)	Year
Rout and Crack Sealing (704 m/km)	42
20% Mill and patch 50 mm	45
Rout and Crack Sealing (704 m/km)	48
Tack Coat	49
Mill 50 mm Asphalt Pavement	50
Superpave 12.5 FC2 - 50 mm	50
Rout and Crack Sealing (352 m/km)	51
Rout and Crack Sealing (352 m/km)	54
Rout and Crack Sealing (352 m/km)	58
20% Mill and patch 50 mm	58
Partial Reconstruction of Pavement	60
Rout and Crack Sealing (352 m/km)	63
Rout and Crack Sealing (352 m/km)	66
Rout and Crack Sealing (352 m/km)	69
5% Mill and patch 50 mm	69

The maintenance and rehabilitation program prepared for the conventional pavement design was prepared based on the MEPDG model results of the conventional design. As the MEPDG model have predicted faster structural deterioration rate for conventional design compared to the perpetual pavement design, the maintenance and rehabilitation program for the conventional pavement design was prepared to treat various distresses and structural deterioration of the pavement section. The maintenance treatments designed for the conventional pavement design are to be performed more frequently than for the perpetual pavement design.

In addition to the more intensive maintenance treatments, the conventional pavement design is scheduled to be partially reconstructed after 30 and 60 years from construction. This partial reconstruction activity projects a reconstruction of the asphalt layers (surface HMA, intermediate HMA and HMA base layers). The alternative to this partial reconstruction activity is usually a thick asphalt overlay to increase the pavement thickness. Based on the structural and economic evaluation of both alternatives, the overlay solution will overcome some structural deformations but it will not be able to address the more serious bottom up cracks. These cracks will continue to propagate due to load repetitions and freeze thaw cycles and the pavement deterioration after the overlay is expected to be faster than the partial reconstruction alternative. Thus the partial reconstruction rehabilitation treatment alternative is expected to be more cost effectively in the long term.

The LCCA total Net Present Value (NPV) of the perpetual and conventional pavement designs is calculated using three percent discount rate for an analysis period of 70 years. The deterministic Net Present Value results at the end of the analysis period were $5,649,711 and $5,437,145 for perpetual and conventional pavement designs respectively. The LCCA results show the two pavement designs are almost having equal Net Present Values. The difference between the NPV of the two pavement designs is almost 4% which provides a slight economic advantage to the conventional pavement design. This Life Cycle Cost Analysis does not take into account the user delay cost due to the construction, maintenance and rehabilitation activities and the lane closures. These factors would to complicate the Life Cycle Cost Analysis calculation and more user delay cost data would be required to obtain a more accurate LCCA. However, the LCCA results are expected to give more advantage to the perpetual pavement design if the user delay costs were to be included in the LCCA as more frequent maintenance activities and treatments are scheduled for the conventional pavement design compared to the perpetual pavement design.

Although the construction costs of the perpetual pavement design is expected to be 70 percent more expensive compared to the conventional design, the overall LCCA NPV costs of the perpetual pavement is higher than that of the conventional design by four percent. The LCCA analysis shows the perpetual pavement design can provide several advantages over the entire life cycle of the asset. Furthermore, if user delay costs are incorporated, the LCCA will further result in long life pavement design such as perpetual pavements so that future maintenance and rehabilitation and the associated user delays are limited. Extension of analysis period to exceed the 70 year limit requires detailed maintenance and rehabilitation records. Unfortunately the current documented maintenance and rehabilitation data is insufficient to create a longer analysis period. In addition, the current MEPDG available version is incapable of performing extended analysis periods as it generates several errors that results in computer freeze and program crash. However, this research does provide new knowledge and provides a basis for a subsequent analysis as more performance data through the sensors on Highway 401 are analyzed.

Chapter 5

Evaluation of Perpetual and Conventional Pavement Designs in Moderate and Low Volume Roads

5.1 Introduction

Construction of perpetual asphalt pavement has showed various structural benefits by increasing the pavement structural capacity especially on high volume roads and increasing its ability to overcome the different traffic and environmental impacts. The corresponding disadvantage for the perpetual pavement designs is the high initial construction cost associated with implementation of perpetual designs. The conventional asphalt pavement design offers a cheaper alternative to the perpetual pavement designs. The conventional asphalt pavement designs are usually associated with lower initial construction cost compared to the perpetual asphalt pavement designs. The disadvantage of the conventional asphalt pavement designs is the limitation of structural capacity or the limited ability to carry traffic loads. The perpetual pavement designs have thick structural layers that enable the pavement section to resist different traffic and environmental impacts. On the other hand, the conventional pavement designs are known with their thick base and subbase granular materials.

Comparison between the perpetual and conventional pavement designs should consider traffic, environmental and economic aspects. The comparison between perpetual and conventional pavement designs represents the usual tradeoff between the cost and quality of structural design. Each of the two pavement designs is suitable and convenient under certain traffic and environmental conditions.

In order to perform a sound comparison between the different pavement designs, all the factors affecting the pavement design procedure were fixed with the exception of the traffic loading. Pavement design is then performed for an average traffic loading representing moderate and low traffic volume roads. The pavement design process was performed by the AASHTO-DARWin 3.1 software that performs the pavement design procedures according to the AASHHTO 1993 guide. Structural evaluation of the conventional and perpetual pavement designs performed for the moderate and low traffic volume roads was performed using Mechanistic Empirical Pavement Design Guide (MEPDG) models and Weslea for Windows software. A Life Cycle Cost Analysis (LCCA) was performed to evaluate the two design approaches economically. The structural and economic evaluation of both pavement design approaches has shown a noticeable advantage for the perpetual asphalt pavement designs over the conventional designs. The expected structural performance of the perpetual pavement designs is overcoming several pavement distresses that the conventional designs are unable to resist. The perpetual designs are characterized by a fatigue resistance asphalt bottom layer with rich Asphalt Content (AC). This layer enables the perpetual sections to resist the bottom up fatigue cracking, rutting and alligator cracking. The Open Graded Drainage Layer (OGDL) placed as a base layer for the perpetual design enhanced its drainage capacity and increased the perpetual design resistance to freeze thaw cycles. The economic analysis of both pavement designs proved that the Net Present Value (NPV) resulted from a 70 LCCA is almost equal for both pavement designs. Although the construction cost of the perpetual design is higher than that of the conventional design, the overall NPV for both design approaches was almost the same due to the intensive maintenance and rehabilitation program needed to maintain the conventional design. The perpetual design is expected to require a more relaxed maintenance program due to its high structural capacity.

5.2 Pavement Design

The pavement design was performed using the AASHTO-DARWin 3.1 version for both perpetual and conventional design approaches and in accordance with the AASHTO 1993 design guide [Hajek 2008]. The main framework for the perpetual pavement design was to design at least three Hot Mix Asphalt (HMA) layers having a minimum total asphalt thickness of 200 mm. These HMA layers are placed on an Open Graded Drainage Layer (OGDL) to improve the pavement drainage and overcome the stresses caused by the freeze thaw cycles. The conventional asphalt pavement design included three HMA layers above a high quality crushed stone Granular A and Granular B layers.

The traffic data used in the pavement design of both perpetual and conventional designs was assumed based on traffic report prepared and posted by the Ministry of Transportation of Ontario (MTO) [MTO 2005(a)] [MTO 2005(b)] and the design procedures recommended by the Applied Research Associates (ARA) which adapts with the Ontario climatic, tepographic, material and soil phisical properties [Hajek 2008].

5.2.1 Moderate traffic Volume Road

Traffic characteristics in this section were assumed to represent a moderate traffic volume road. Reviewing the MTO traffic reports stating the traffic counts in the King's Highways, Secondary Highways and Tertiary Roads in Ontario, the average traffic count that would represent most of the King's highways in Ontario was found to be 30,000 Annual Average Daily Traffic (AADT). The pavement design was performed assuming a new road construction of a two-lane rural highway. The heavy truck percentage was assumed to be 8% to consider the urbanization of the surrounding area in the future. A thirty year design life period was considered for perpetual and conventional asphalt pavement designs. Figures 5-1 and 5-2 are showing all traffic characteristics assumed in the moderate traffic volume road design for perpetual and conventional pavement designs.

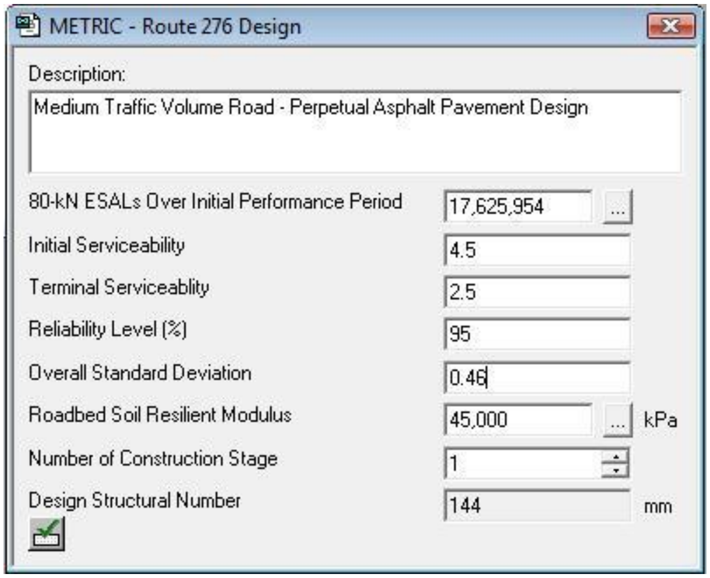


Figure 5-1: Traffic Data Input for Moderate traffic Volume Road (A)

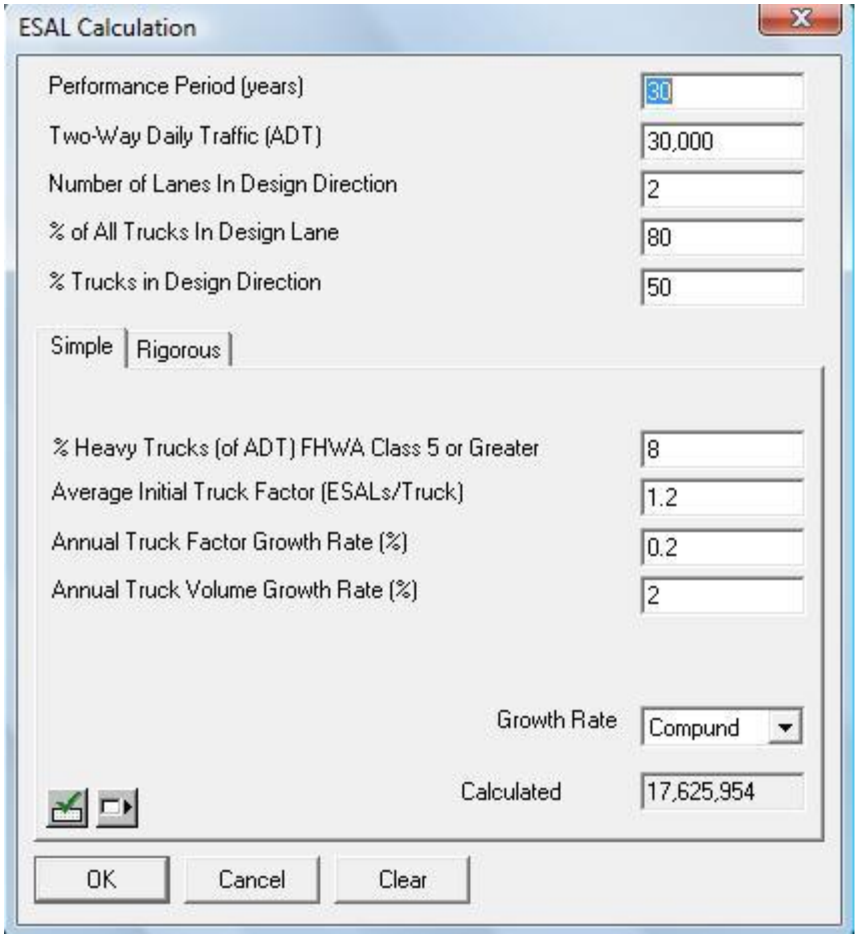
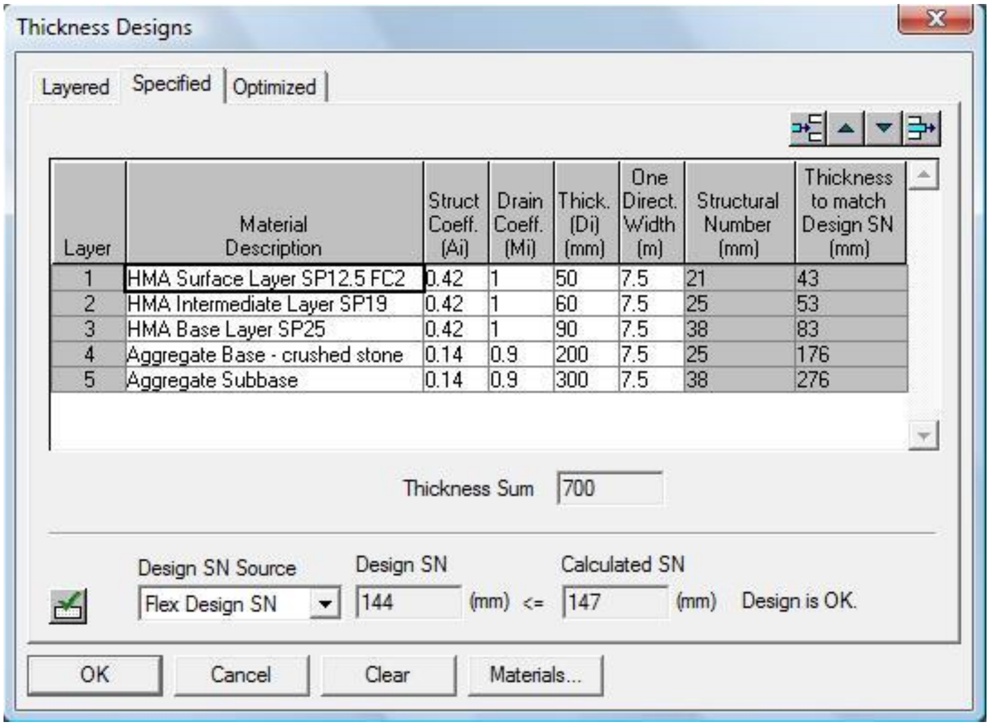


Figure 5-2: Traffic Data Input for Moderate traffic Volume Road (B)

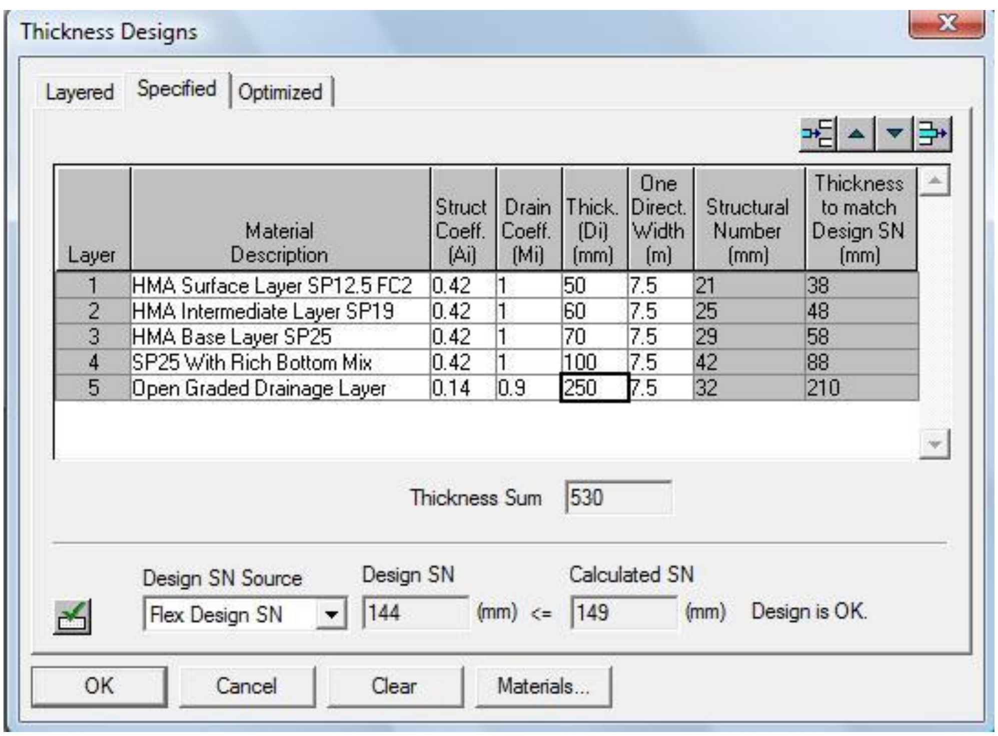
The Pavement design results are showing the need for three asphalt layers in the conventional asphalt pavement design of thickness 50, 60 and 90 millimeters followed by a 200 millimeters of Granular A and 300 millimeters of Granular B. while the Perpetual asphalt pavement design resulted in the need of four asphalt layers of thickness 50, 60, 70 and 100 millimeters followed by a 250 millimeter Open Graded Drainage Layer (OGDL). To enhance the perpetual pavement performance, the bottom asphalt layer is characterized by its rich binder content mix. The Rich Bottom Mix (RBM) layer increases the pavement structural resistance and improves its performance which has significant impact over the maintenance and rehabilitation programs predicted for the pavement design. In addition, the Open Graded Drainage Layer improves the drainage system and this has a major impact on the pavement resistance to freeze thaw cycles and thus on bottom up cracking.

The Figures 5-3 and 5-4 are showing the results of the pavement design for conventional and perpetual pavement designs respectively.



Layer	Material Description	Struct Coeff. (Ai)	Drain Coeff. (Mi)	Thick. (Di) (mm)	One Direct. Width (m)	Structural Number (mm)	Thickness to match Design SN (mm)
1	HMA Surface Layer SP12.5 FC2	0.42	1	50	7.5	21	43
2	HMA Intermediate Layer SP19	0.42	1	60	7.5	25	53
3	HMA Base Layer SP25	0.42	1	90	7.5	38	83
4	Aggregate Base - crushed stone	0.14	0.9	200	7.5	25	176
5	Aggregate Subbase	0.14	0.9	300	7.5	38	276

Figure 5-3: Pavement Design Results for Conventional Design in Moderate traffic Volume



Layer	Material Description	Struct Coeff. (Ai)	Drain Coeff. (Mi)	Thick. (Di) (mm)	One Direct. Width (m)	Structural Number (mm)	Thickness to match Design SN (mm)
1	HMA Surface Layer SP12.5 FC2	0.42	1	50	7.5	21	38
2	HMA Intermediate Layer SP19	0.42	1	60	7.5	25	48
3	HMA Base Layer SP25	0.42	1	70	7.5	29	58
4	SP25 With Rich Bottom Mix	0.42	1	100	7.5	42	88
5	Open Graded Drainage Layer	0.14	0.9	250	7.5	32	210

Figure 5-4: Pavement Design Results for Perpetual Design in Moderate traffic Volume

5.2.2 Low Traffic Volume Road

The low traffic volume road design for both perpetual and conventional pavement designs was performed based on the same assumptions as the moderate traffic volume roads. Fixing all the factors affecting pavement design with exception of the Average Annual Daily Traffic (AADT) on the road ensures the consistency of the comparison between the two pavement designs under fixed environmental and structural features. The Average Annual Daily Traffic (AADT) assumed is an average value representing the secondary rural highways in Ontario [MTO 2005(a)] [MTO 2005(b)]. The AADT representing the low traffic volume roads is assumed as 15,000 and the percentage of heavy vehicles is assumed to be 8% as in the moderate traffic volume road designs. A thirty year design life for the perpetual and conventional pavement designs is implemented.

Figures 5-5 and 5-6 presents the traffic and environmental input data used for pavement design of perpetual and conventional asphalt sections in low traffic volume road by the AASHTO-DARWin 3.1.

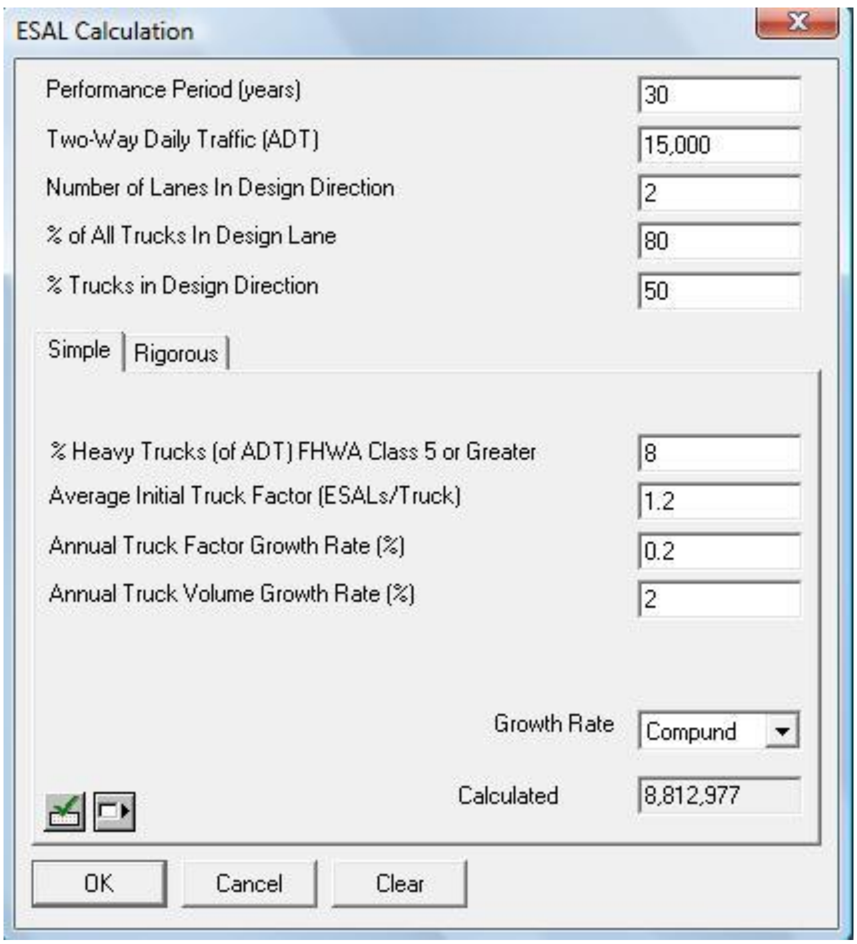


Figure 5-5: Traffic Data Input for Low Traffic Volume Road (A)

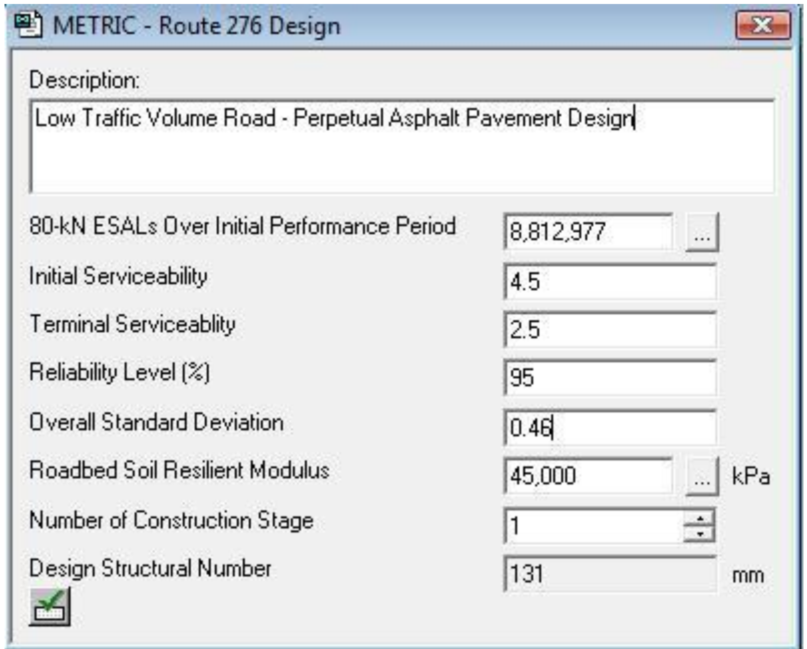
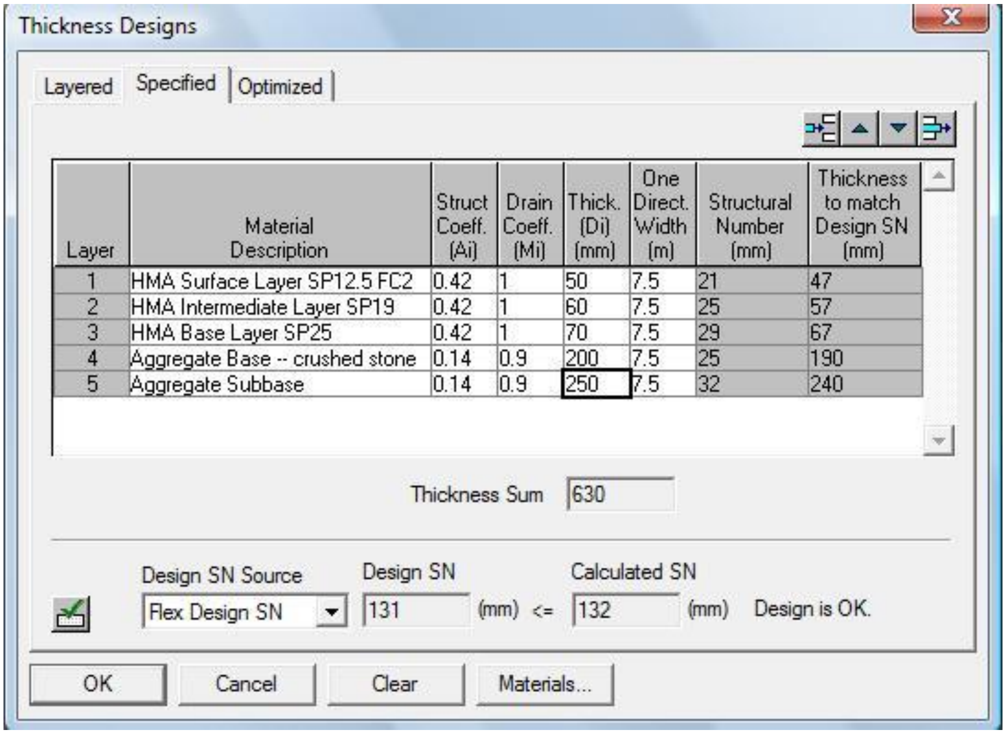


Figure 5-6: Traffic Data Input for Low Traffic Volume Road (B)

The pavement design for the conventional pavement design is the construction of three asphalt layers of thickness 50, 60 and 70 mm followed by a 200 mm of Granular A and 250 mm of Granular B. The asphalt layers of the conventional design of the low traffic volume road are having the same physical and mechanical properties as those of the moderate traffic volume road design. A reduction of twenty millimeters of asphalt layers and fifty millimeters in Granular B are noticed in accordance to the reduction of the traffic volume from the 30,000 AADT representing the moderate traffic volume roads to the 15,000 AADT representing the low traffic volume roads.

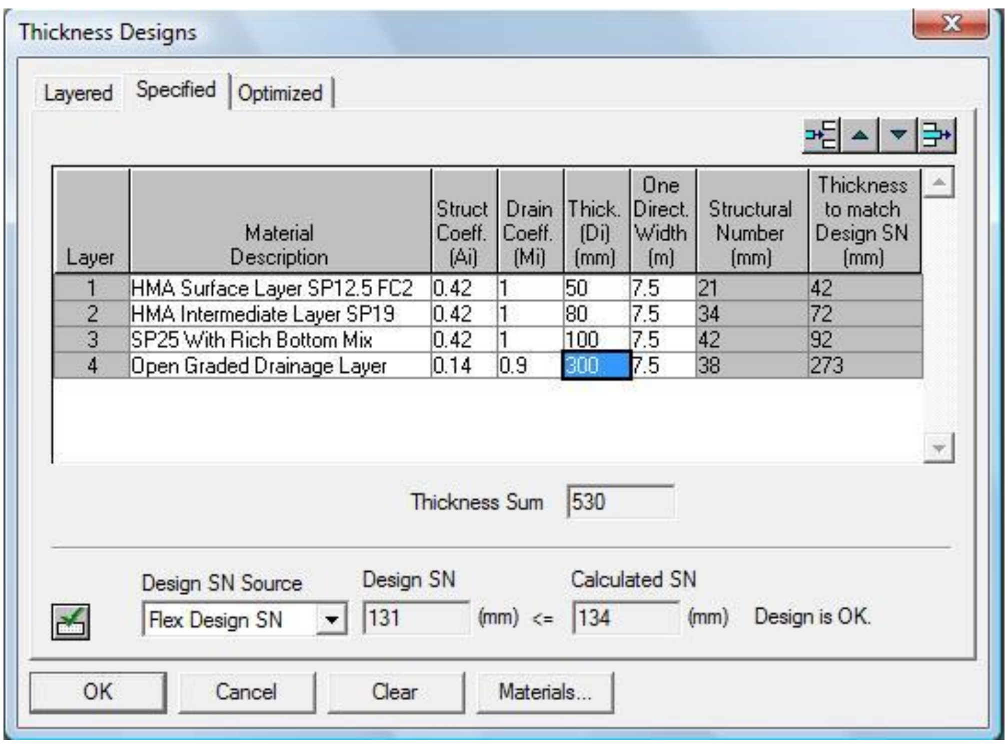
The perpetual pavement design resulted in three asphalt layers of 50, 80 and 100 millimeter layers followed by 300 mm Open Graded Drainage Layer (OGDL). A 50 mm reduction in the asphalt thickness has taken place due to reducing the AADT from the moderate to low traffic volume category and the number of asphalt layers was also reduced from four asphalt layers to three taking into account that the bottom asphalt layer is also a Rich Bottom Mix (RBM) layer as in the moderate traffic volume road design. In addition the thickness of the OGDL was also reduced by 50 mm due to the traffic load reduction.

Figures 5-7 and 5-8 are showing the AASHTO-DARWin 3.1 results for the conventional and perpetual asphalt pavement designs respectively in low traffic volume roads.



Layer	Material Description	Struct Coeff. (Ai)	Drain Coeff. (Mi)	Thick. (Di) (mm)	One Direct. Width (m)	Structural Number (mm)	Thickness to match Design SN (mm)
1	HMA Surface Layer SP12.5 FC2	0.42	1	50	7.5	21	47
2	HMA Intermediate Layer SP19	0.42	1	60	7.5	25	57
3	HMA Base Layer SP25	0.42	1	70	7.5	29	67
4	Aggregate Base -- crushed stone	0.14	0.9	200	7.5	25	190
5	Aggregate Subbase	0.14	0.9	250	7.5	32	240

Figure 5-7: Pavement Design Results for Conventional Design in Low Traffic Volume



Layer	Material Description	Struct Coeff. (Ai)	Drain Coeff. (Mi)	Thick. (Di) (mm)	One Direct. Width (m)	Structural Number (mm)	Thickness to match Design SN (mm)
1	HMA Surface Layer SP12.5 FC2	0.42	1	50	7.5	21	42
2	HMA Intermediate Layer SP19	0.42	1	80	7.5	34	72
3	SP25 With Rich Bottom Mix	0.42	1	100	7.5	42	92
4	Open Graded Drainage Layer	0.14	0.9	300	7.5	38	273

Figure 5-8: Pavement Design Results for Perpetual Design in Low Traffic Volume

5.3 Structural Evaluation of pavement Designs

The structural evaluation of perpetual and conventional pavement designs is essential to generate an overall comparison between the different pavement designs. The structural evaluation is implemented and presented in this session by utilizing the MEPDG and the WESLEA for Windows analysis programs.

5.3.1 Moderate traffic Volume Designs

5.3.1.1 Mechanistic Empirical Pavement Design Guide (MEPDG) Model

5.3.1.1.1 MEPDG Inputs

The moderate traffic volume pavement design MEPDG inputs for both conventional and perpetual designs are presented in Tables 5-1 and 5-2 respectively. The properties of the asphalt layers used in the perpetual and conventional designs are similar to these used in the Highway 401 project and presented in chapter four. The Open Graded Drainage Layer (OGDL) was assumed to be used in the perpetual pavement design to replace the use of Granular A and Granular B in the perpetual design on the Highway 401 project. The OGDL can replace the base and subbase layers due to the magnificent difference in traffic loads on the Highway 401 and the assumed moderate traffic load. The MEPDG inputs for the OGDL were assumed based on previous research work performed at the CPATT test track constructed by University of Waterloo (CPATT) [Smith 2009].

The traffic data used in the MEPDG analysis model is based on the traffic data collection performed by the Ministry of Transportation of Ontario [MTO 2005(a)] [MTO 2005(b)] to represent an average AADT value for the King's highway series. The vehicle class distribution factor was assumed based on a research project performed to determine a default vehicle class distribution factor for highways located at southern Ontario reigon [Swan 2008].

The MEPDG model was created in Level Three as all the physical and mechanical data required for all pavement layers were assumed based on other research projects using similar material. The default MEPDG input values were used in case no certain estimation was stated in the literature. The climate condition file needed for the MEPDG runs was created to simulate the weather condition in Niagra Falls, New York state as this was found to be the closest area that represents the southern Ontario reigon climate condtions.

Table 5-1: MEPDG Inputs for Conventional Pavement Design in Moderate traffic Volume Highway

Layer 1		**Layer 3**	
Thickness (mm)	50	Thickness (mm)	90
PG	PG 70-28	PG	PG 58-22
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 4**	
Thickness (mm)	60	Thickness (mm)	200
PG	PG 64-28	Aggregate base layer (Granular A) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2

Layer 5	
Thickness (mm)	300
Aggregate subbase layer (Granular B) Permeable layer	
Maximum dry unit weight (pcf)	127.2
Specific gravity of solids, Gs	2.7
Saturated hydraulic conductivity (ft/hr)	0.05054
Optimum gravimetric water content (%)	7.4
Calculated degree of saturation (%)	61.2

Table 5-2: MEPDG Inputs for Perpetual Pavement Design in Moderate traffic Volume Highway

Layer 1		**Layer 3**	
Thickness (mm)	50	Thickness (mm)	70
PG	PG 70-28	PG	PG 58-28
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 4**	
Thickness (mm)	60	Thickness (mm)	100
PG	PG 64-28	PG	PG 58-28
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	16	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	2.4
Volumetric binder content (%):	13	Volumetric binder content (%):	14
Total unit weight (pcf):	152	Total unit weight (pcf):	152

Layer 5	
Thickness (mm)	250
Open Graded Drainage Layer (OGDL)	
Maximum dry unit weight (pcf)	125.8
Specific gravity of solids, Gs	2.7
Saturated hydraulic conductivity (ft/hr)	0.05054
Optimum gravimetric water content (%)	7.4
Calculated degree of saturation (%)	61.2

5.3.1.1.2 MEPDG model results

Two MEPDG models were created to predict the two pavement designs performance through a 50 year analysis period. The MEPDG results are providing the researchers with a reasonable structural evaluation for the two pavement designs. Several structural distresses are addressed by this MEPDG model as the surface down cracking length per kilometer, damage percentage caused by the surface down cracking, bottom up cracking damage, alligator cracking damage, rutting in asphalt layers, rutting in granular material, total rutting and International Roughness Index (IRI).

Figure 5-9 presents a comparison between the damage caused by the surface down cracking in both perpetual and conventional designs. It is obvious that the perpetual design has a lower deterioration rate In addition it is noticed that the deterioration rate of the conventional design followings an exponential trend. This deterioration rate trend shows that more severe deterioration is expected for the conventional design beyond the 50 year analysis period.

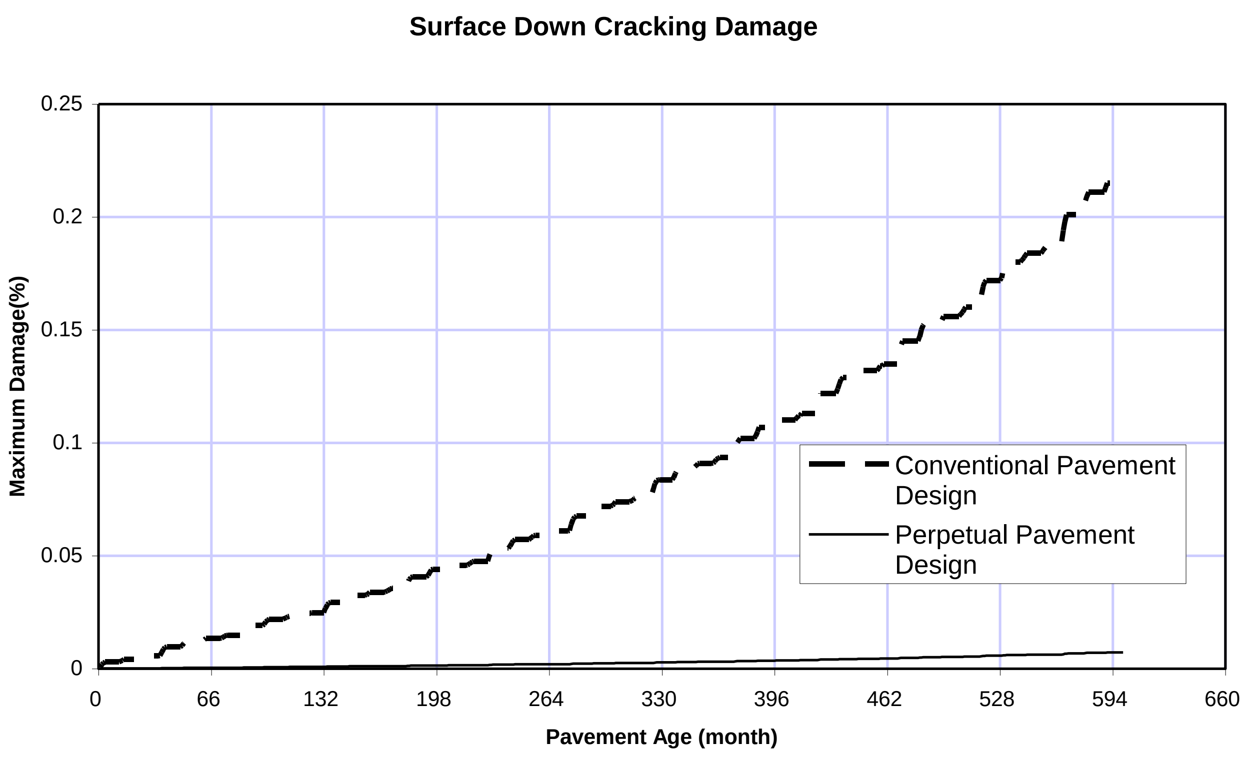


Figure 5-9: Surface Down Cracking Damage for Moderate traffic Volume Designs

The surface down cracking rate model result is presented in Figure 5-10. The results are corresponding with the damage percentage caused by the cracks. Thus, the perpetual design is expected to have higher surface down cracking resistance than the conventional design. The conventional pavement design deterioration trend follows the exponential shape. This model results gives more confidence in the damage percentage model caused by the surface down cracking.

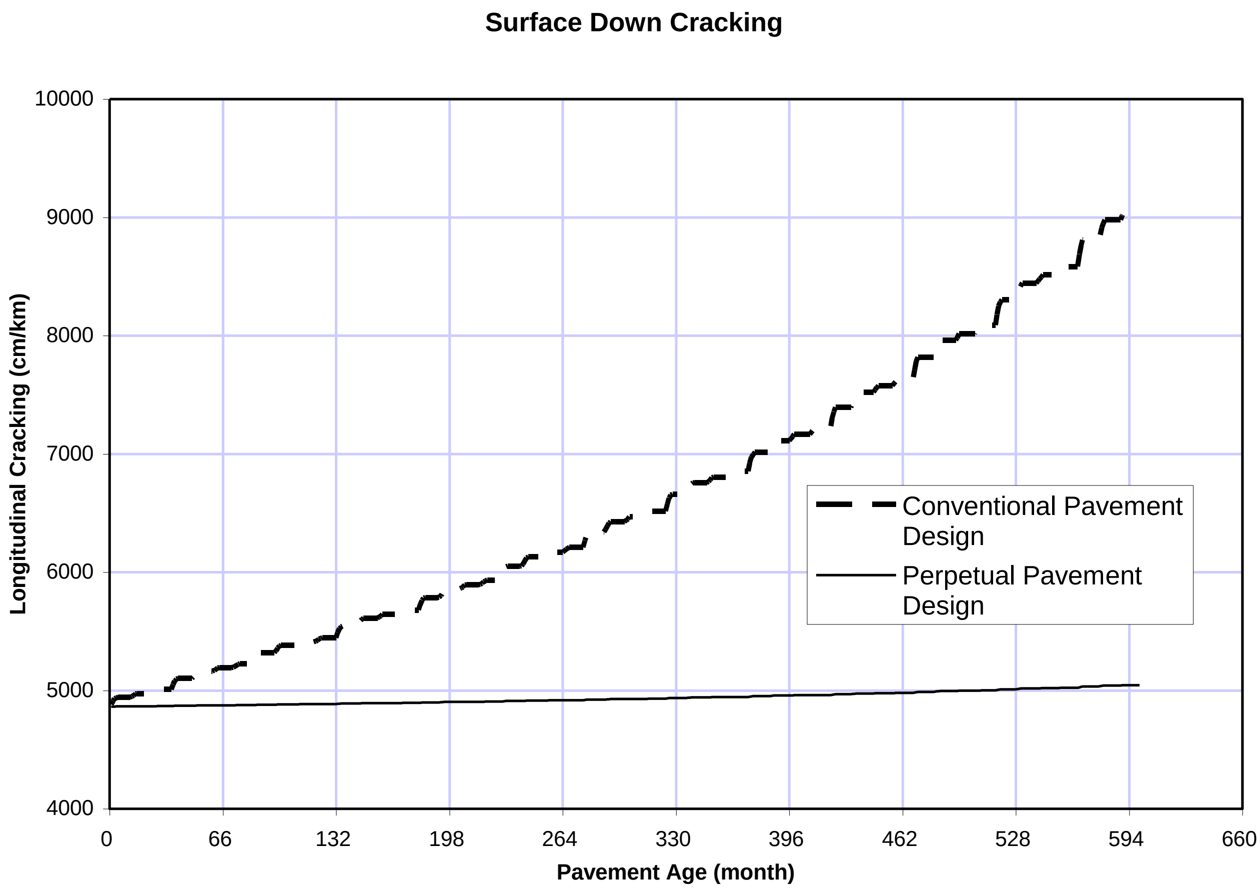


Figure 5-10: Surface Down Cracking Rate for Moderate traffic Volume Designs

The damage caused by bottom up cracking is modeled by the MEPDG and the model result is presented in Figure 5-11. The perpetual pavement design provides an advantage over the conventional design by resisting the bottom up cracking. The reason for this superior performance of perpetual design is the high structural capacity of the Rich Bottom Mix (RBM) layer placed at the bottom of asphalt layers in the perpetual design and the higher total asphalt thickness in the perpetual design which increases its ability to resist the bottom up cracking using the asphalt material own weight. Although the bottom up cracking is increasing exponentially in both the conventional and perpetual designs, the perpetual design is expected to be subjected to 40% less damage as compared to the conventional design.

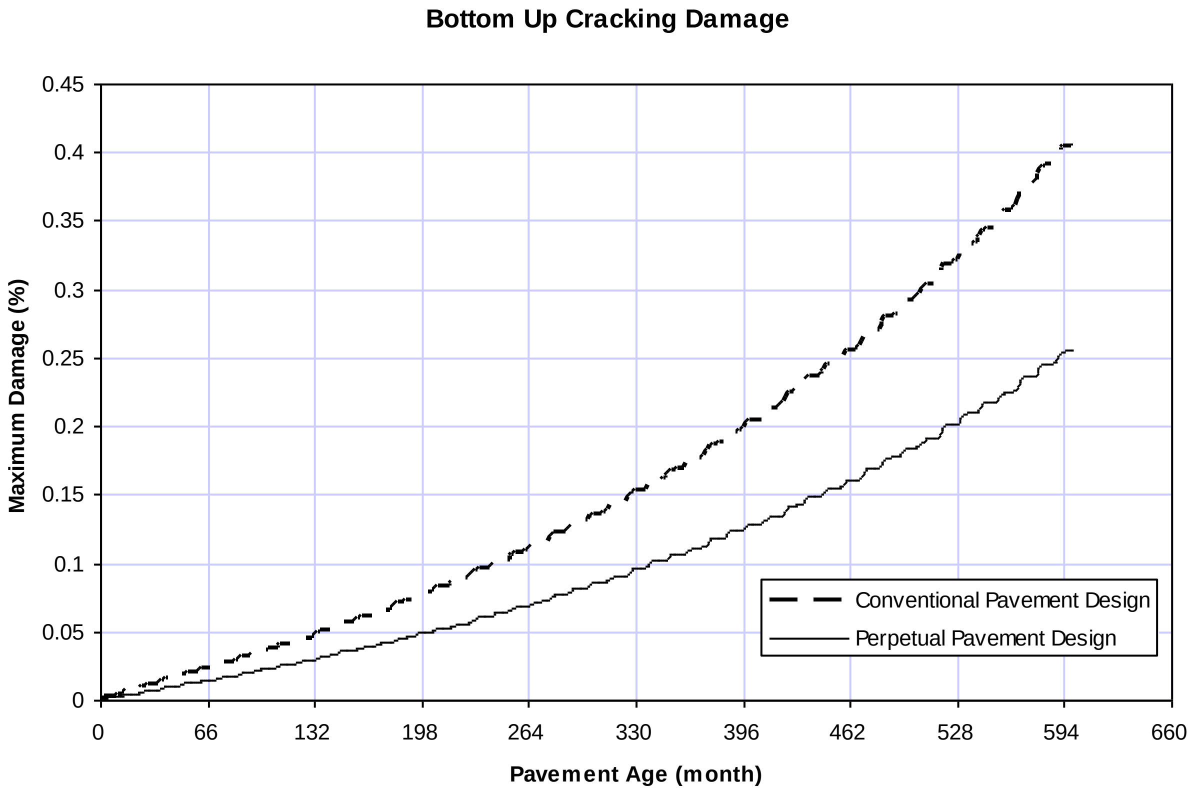


Figure 5-11: Bottom Up Cracking Damage for Moderate traffic Volume Designs

The alligator cracking model for conventional and perpetual pavement designs is presented in Figure 5-12. The conventional pavement design is expected to suffer more severe alligator cracking compared to the perpetual design. The Rich Bottom Mix layer has a significant impact on the superior structural resistance capacity of the perpetual pavement design. The high alligator cracking resistance in the perpetual design can be predicted from the high surface down cracking resistance and the bottom up cracking resistance. High pavement structural capacity is essential to resist the stresses caused by freeze thaw cycles especially when associated with high percentage of heavy vehicles.

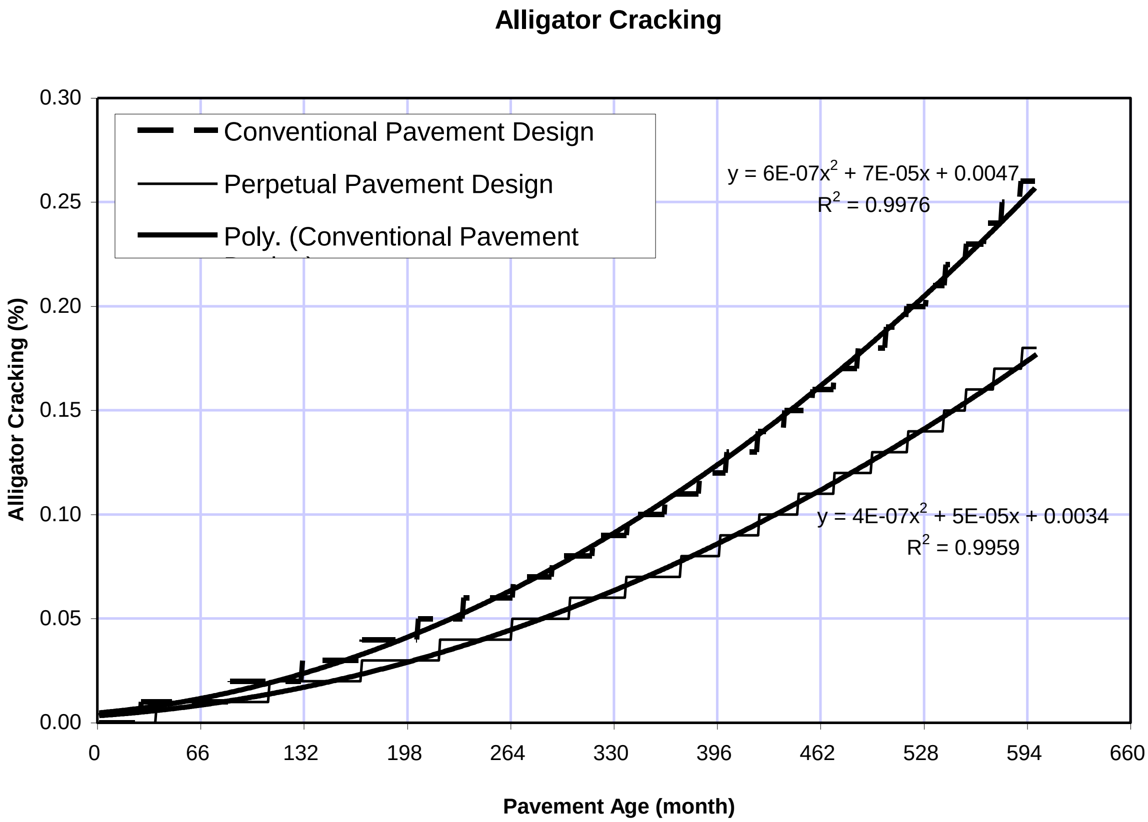


Figure 5-12: Alligator Cracking for Moderate traffic Volume Designs

Figure 5-13 presents the predicted rutting values during the MEPDG model analysis period. It is obvious that the perpetual pavement design is expected to exhibit less total rutting throughout the 50 year analysis period compared to that expected for the conventional design. There is a significant difference between the rutting occurring in the asphalt layers of the perpetual asphalt design and that in the conventional design as the asphalt rutting in perpetual design is very small compared to that in the conventional design. It is noticed that the rutting occurring in the granular material has more severe values in the perpetual pavement design compared to that in the conventional design. The reason behind this is the high voids ratio that characterizes the Open Graded Drainage Layer (OGDL) which allows for settlement of that layer as compared with the conventional Granular A and Granular B layers. It is noticed that the rutting curve representing the asphalt rutting of the conventional pavement design is almost identical to that representing the granular material rutting of the perpetual pavement. The two curves are showing identical rutting values predictions for the same period.

The rutting occurring in granular material in the perpetual design showed lower rutting values compared to rutting in granular material in the conventional design over the initial fifteen years in-service. The deterioration rate of the granular material rutting in conventional design slowed down after the initial fifteen year period while the rutting in granular material of the perpetual design resumed its rapid deterioration rate. However, the overall total rutting prediction of the two pavement designs shows that the entire perpetual pavement design is expected to be subjected to less total rutting values during the analysis period.

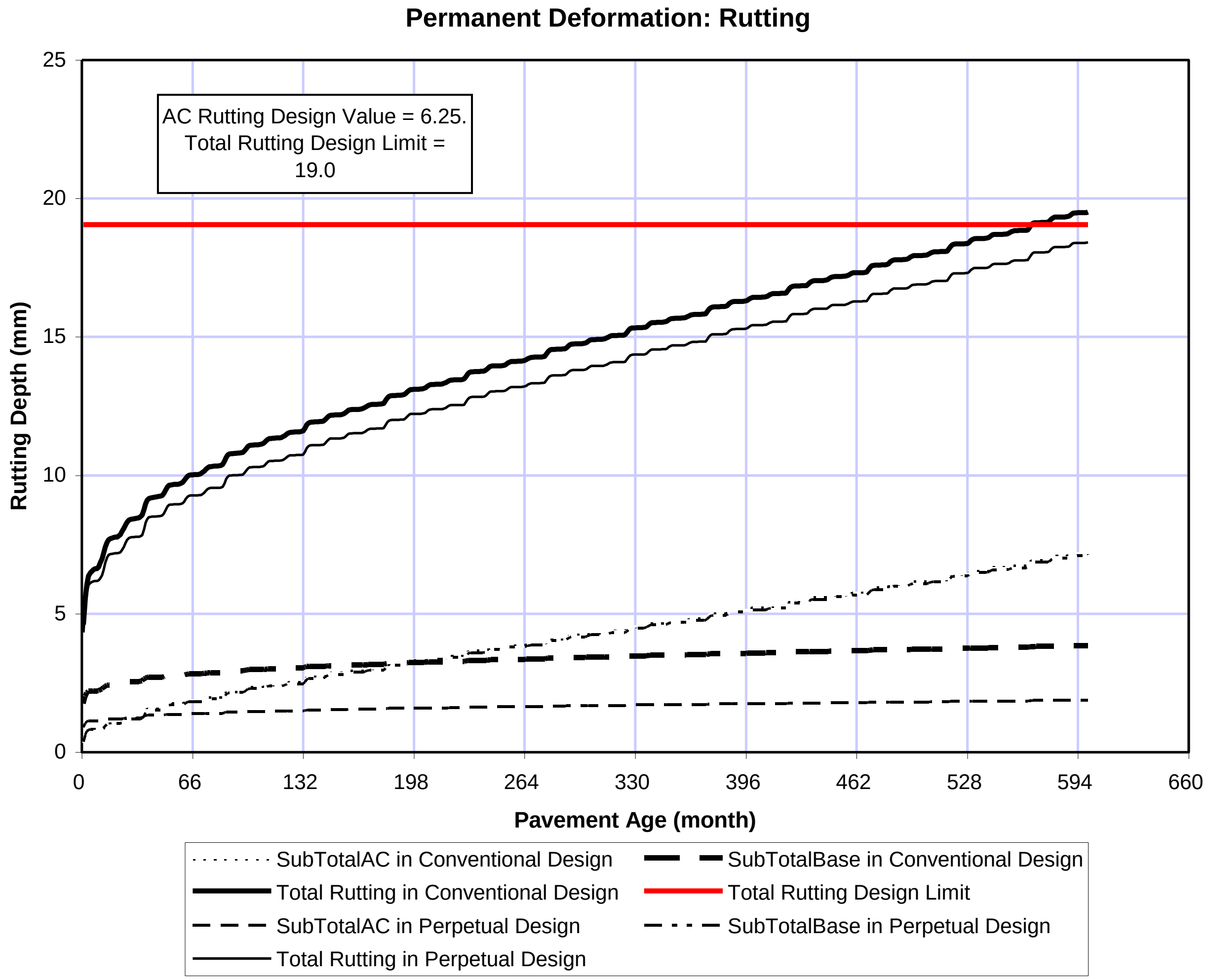


Figure 5-13: Rutting Results for Moderate traffic Volume Designs

Figure 5-14 presents the Results of the MEPDG model for the International Roughness Index (IRI). The model result shows little difference between the IRI deterioration in the perpetual and conventional designs. There is no significant impact on the IRI values when using the perpetual asphalt design compared to the conventional design.

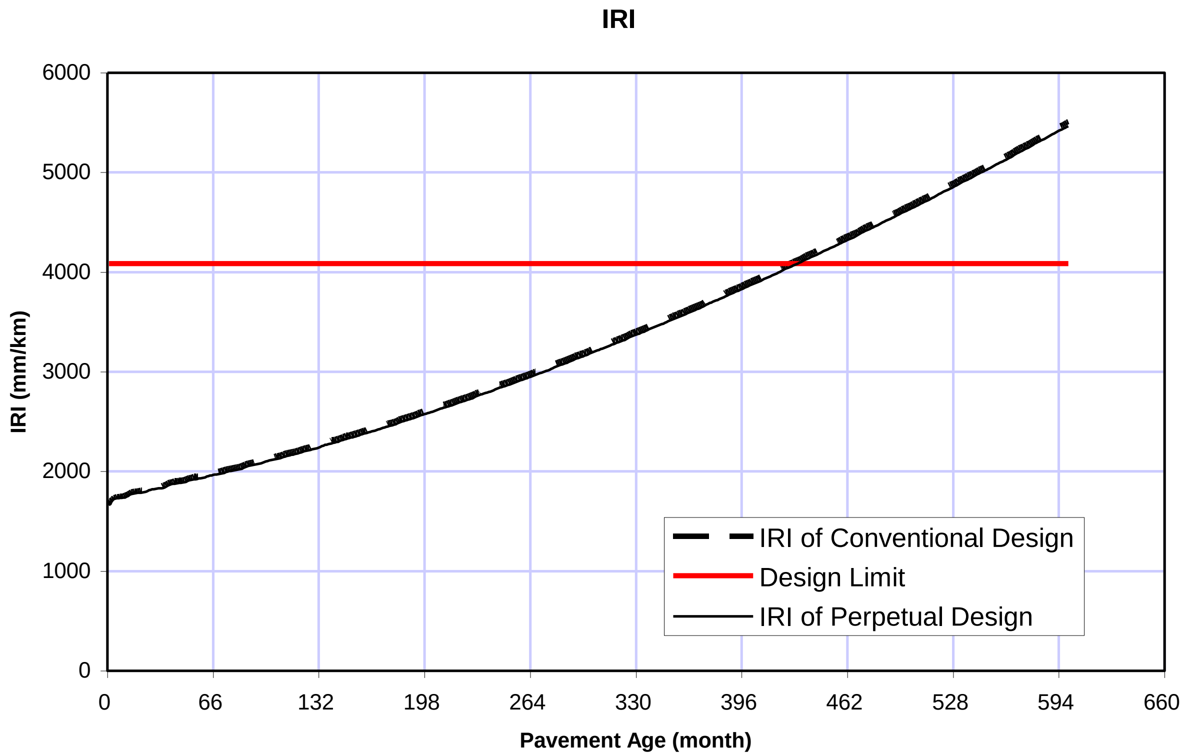


Figure 5-14: International Roughness Index Results for Moderate traffic Volume Designs

5.3.2 Low Traffic Volume Design

5.3.2.1 Mechanistic Empirical Pavement Design Guide (MEPDG) Model

5.3.2.1.1 MEPDG Inputs

The low traffic volume pavement design MEPDG inputs for both conventional and perpetual designs are presented in Tables 5-3 and 5-4 respectively. The properties of the asphalt layers used in the perpetual and conventional designs are similar to these used in the Highway 401 project and presented in chapter four. The climate conditions assumed for the MEPDG model is generated by the weather station located in the Niagara Falls, New York.

An Open Graded Drainage Layer (OGDL) is used in the perpetual asphalt design to replace the Granular A and Granular B materials. The reason for using the OGDL in perpetual pavement design is that it acts as a strong base for the thick asphalt layers in the perpetual design. The perpetual designs require to be constructed on a hard and sound base. In addition the OGDL assists in resisting the fatigue cracking due to the freeze thaw cycles taking place in southern Ontario.

Table 5-3: MEPDG Inputs for Conventional Design in Low Volume Traffic Roads

Layer 1		**Layer 3**	
Thickness (mm)	50	Thickness (mm)	70
PG	PG 70-28	PG	PG 58-22
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 4**	
Thickness (mm)	60	Thickness (mm)	200
PG	PG 64-28	Aggregate base layer (Granular A) Crushed Stone	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	127.2
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2

Layer 5	
Thickness (mm)	250
Aggregate subbase layer (Granular B) Permeable layer	
Maximum dry unit weight (pcf)	127.2
Specific gravity of solids, Gs	2.7
Saturated hydraulic conductivity (ft/hr)	0.05054
Optimum gravimetric water content (%)	7.4
Calculated degree of saturation (%)	61.2

Table 5-4: MEPDG Inputs for Perpetual Pavement Design for Low Traffic Volume Roads

Layer 1		**Layer 3**	
Thickness (mm)	50	Thickness (mm)	100
PG	PG 70-28	PG	PG 58-22
Volumetric Properties as Built		Volumetric Properties as Built	
Mixture VMA (%)	17	Mixture VMA (%)	16
Air voids (%):	4	Air voids (%):	4
Volumetric binder content (%):	13	Volumetric binder content (%):	12
Total unit weight (pcf):	153	Total unit weight (pcf):	151
Layer 2		**Layer 4**	
Thickness (mm)	80	Thickness (mm)	300
PG	PG 64-28	Open Graded Drainage Layer (OGDL)	
Volumetric Properties as Built		Maximum dry unit weight (pcf)	125.8
Mixture VMA (%)	16	Specific gravity of solids, Gs	2.7
Air voids (%):	4	Saturated hydraulic conductivity (ft/hr)	0.05054
Volumetric binder content (%):	13	Optimum gravimetric water content (%)	7.4
Total unit weight (pcf):	152	Calculated degree of saturation (%)	61.2

The layer’s physical and mechanical characteristics were assumed as these are used in the high and moderate traffic volume models and based on the MTO recommendations. The OGDL inputs used to model that layer were based on previous research work performed at the CPATT test track [Smith 2009].

5.3.2.1.2 MEPDG Model Results

The MEPDG model has simulated the two pavement designs for a 50 year analysis period. Traffic loads and climate conditions are taken into account by the MEPDG program. A prediction for the two pavement designs’ performance is presented in this section.

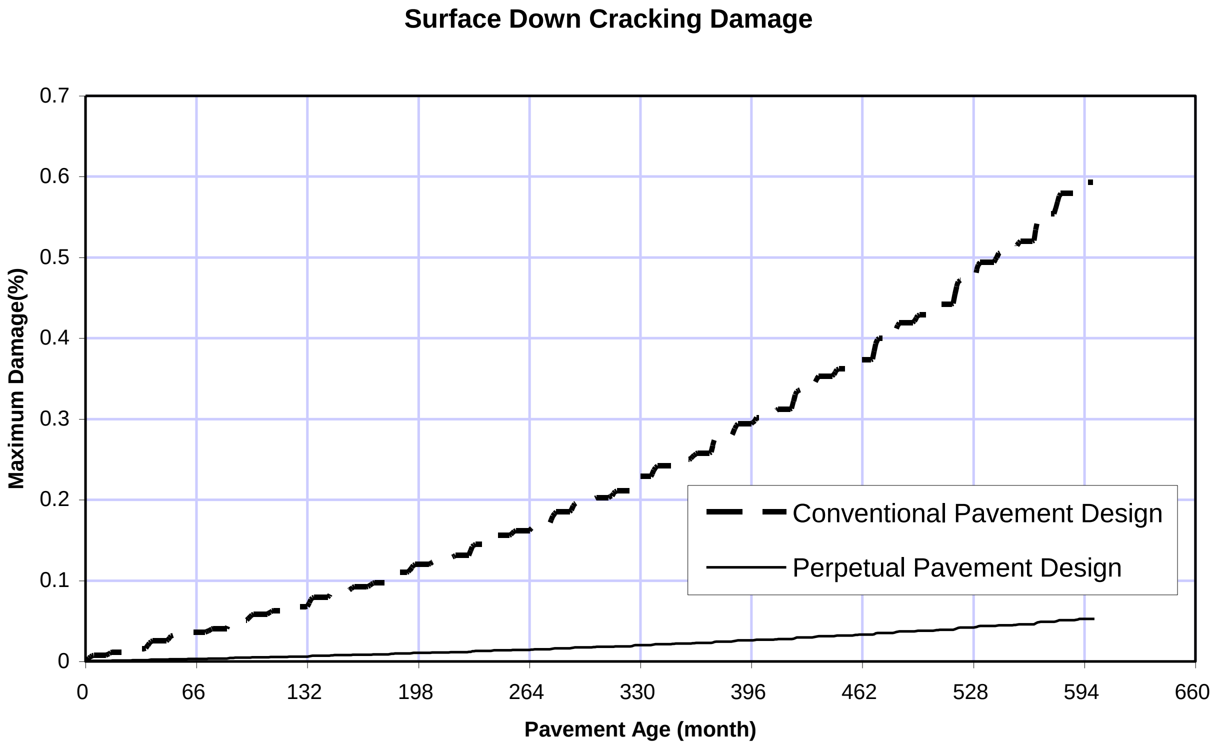


Figure 5-15: Surface Down Cracking Damage for Low Traffic Volume Designs

The conventional pavement design is showing higher damage percentage due to the top down cracking over the 50 year analysis period. The damage caused by top down cracks in the conventional design is six times greater than that occurring in the perpetual design.

Figure 5-16 presents the surface down cracking rate per kilometer. It is obvious that the surface down cracking rate of the conventional design is increasing more rapidly compared to that of the perpetual design. The surface down cracking rate model results are showing reasonable results that explains the high damage percentage expected by the conventional pavement design due to the surface down cracking. This surface down cracking can be resolved by crack sealing the top down cracks before they propagate to the following asphalt layers. The surface course should be milled and replaced every ten to twenty years as crack sealing is insufficient for retaining the surface coarse condition and preventing the crack propagation.

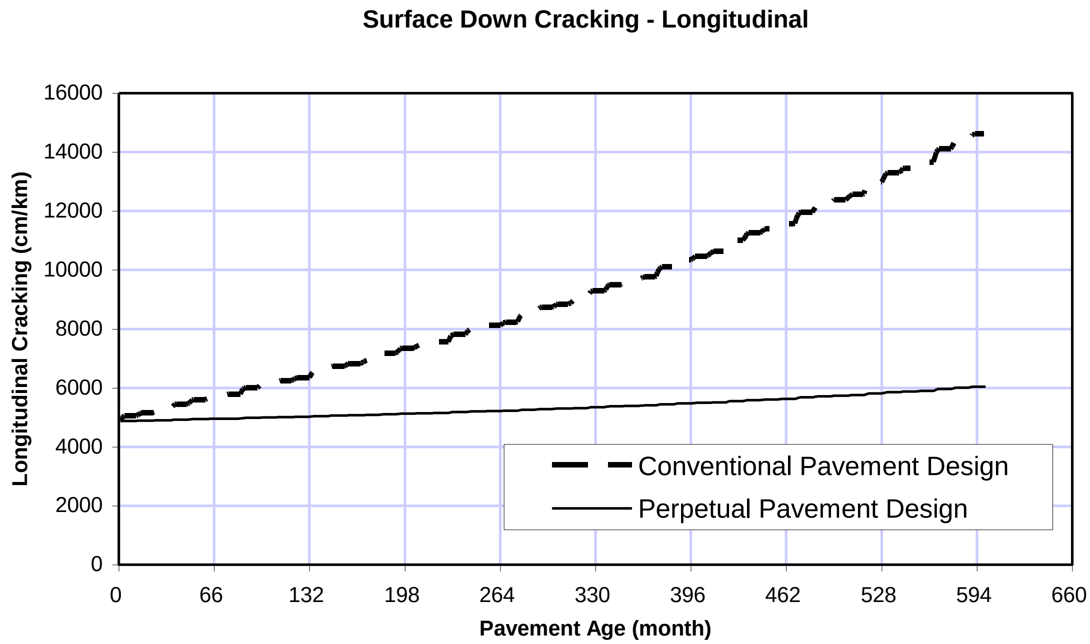


Figure 5-16: Surface Down Cracking Rate for Low Traffic Volume Designs

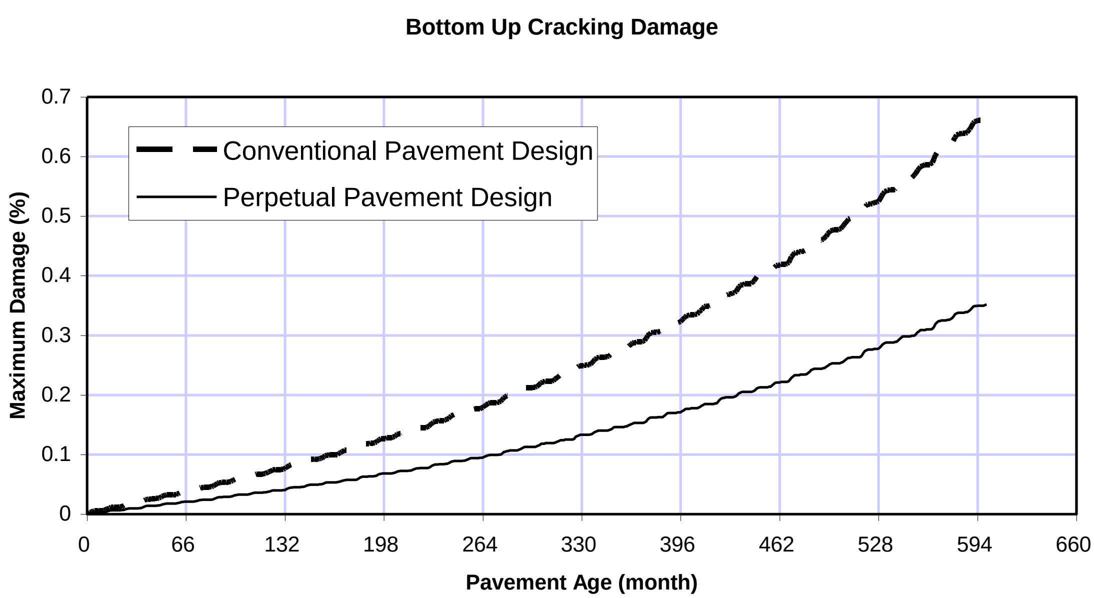


Figure 5-17: Bottom Up Cracking Damage for Low Traffic Volume Designs

Figure 5-17 shows the bottom up cracking model for both the conventional and perpetual designs. It is obvious that the perpetual pavement design is having higher resistance to the bottom up cracks compared to the conventional pavement design. The Open Graded Drainage Layer (OGDL) acted as a strong base protecting the perpetual pavement design from the freeze thaw cycles and their impact on propagation of the bottom up cracks. It is believed that the replacement of the traditional Granular A and Granular B layers by the OGDL have improved the pavement performance in addition to the existence of the Rich Bottom Mix (RBM) layer placed at the bottom of the perpetual pavement Asphalt layer.

Figure 5-18 presents the alligator cracking MEPDG model results and shows the noticeable difference between both pavement resistances for alligator cracking. The perpetual pavement design is expected to have higher resistance to the alligator cracking than that of the conventional pavement design. The reasons behind this high resistance are the existence of the OGDL and the RBM layer installed at the bottom of the perpetual asphalt section which controlled the bottom up cracking and minimized its effect. In addition, the relatively higher own weight of the perpetual asphalt section assisted in resisting the alligator cracking phenomenon.

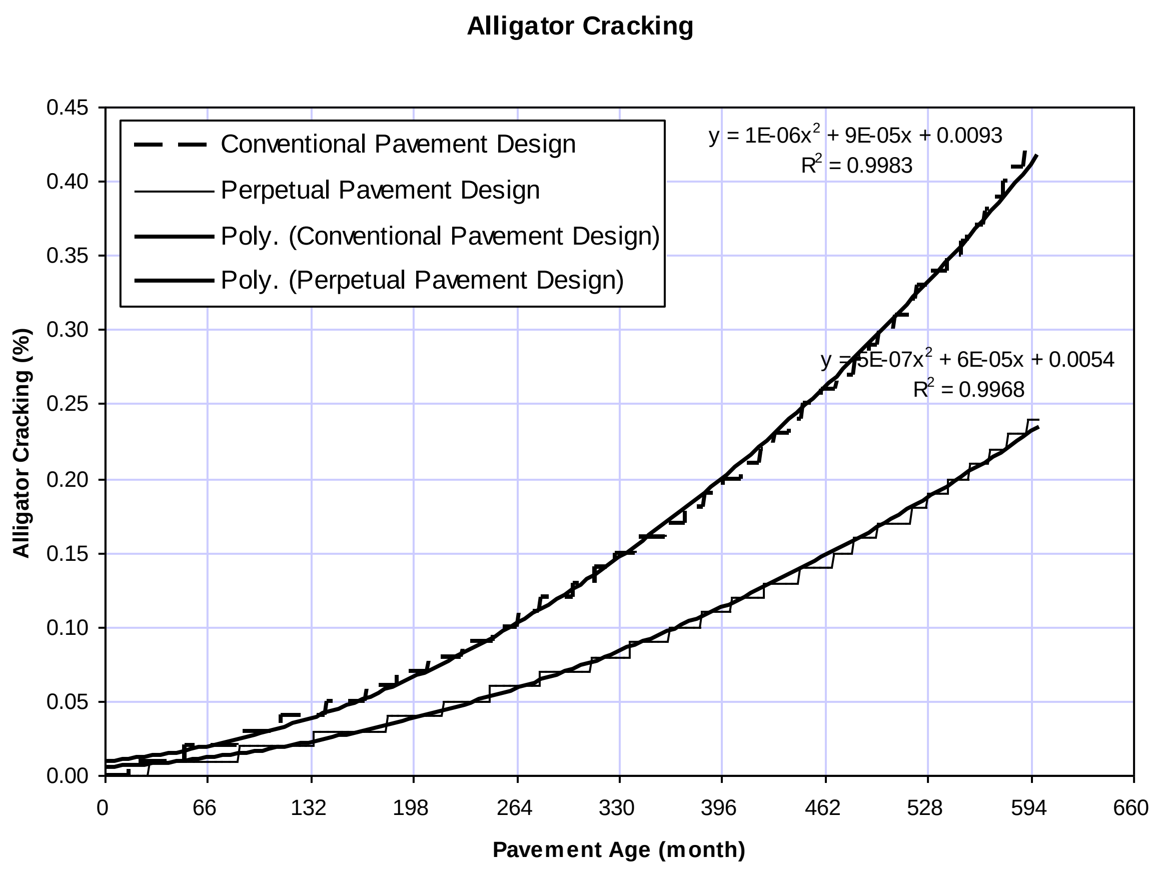


Figure 5-18: Alligator Cracking for Low Traffic Volume Designs

Figure 5-19 shows the rutting MEPDG model results. The results are showing higher rutting values in the conventional pavement design compared to that in the perpetual pavement design. The perpetual pavement design have showed higher rutting resistance in all its layers as the granular rutting value occurring in the Granular A and B of the conventional pavement design is causing to higher rutting values than the OGDL in the perpetual design. The strong rutting resistance of the perpetual design was also reflected through the high rutting resistance in the asphalt layers as the perpetual asphalt layers were subjected to less AC (Asphalt Concrete) rutting values than that occurring in the conventional pavement design.

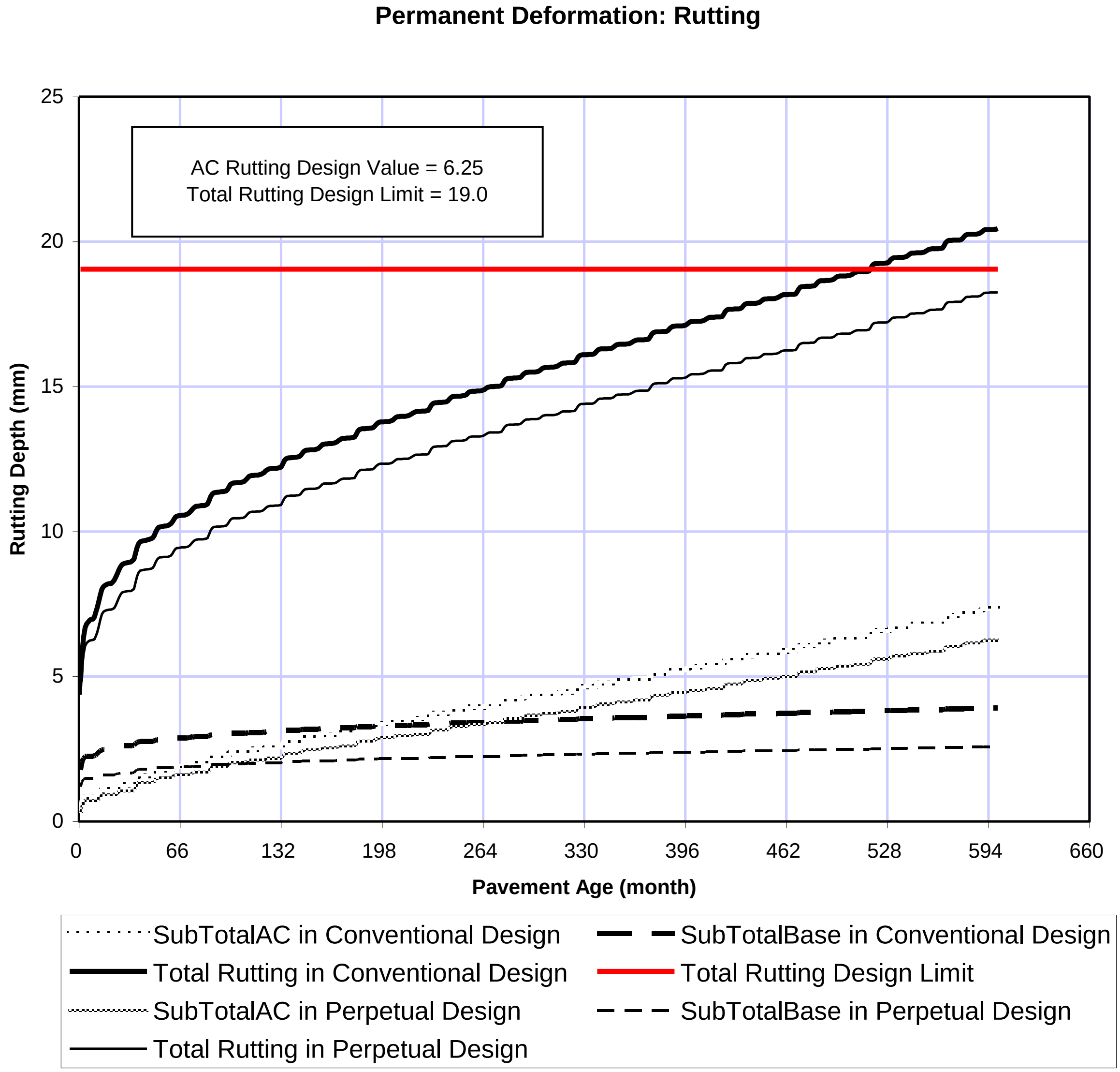


Figure 5-19: Rutting Results for Low Traffic Volume Designs

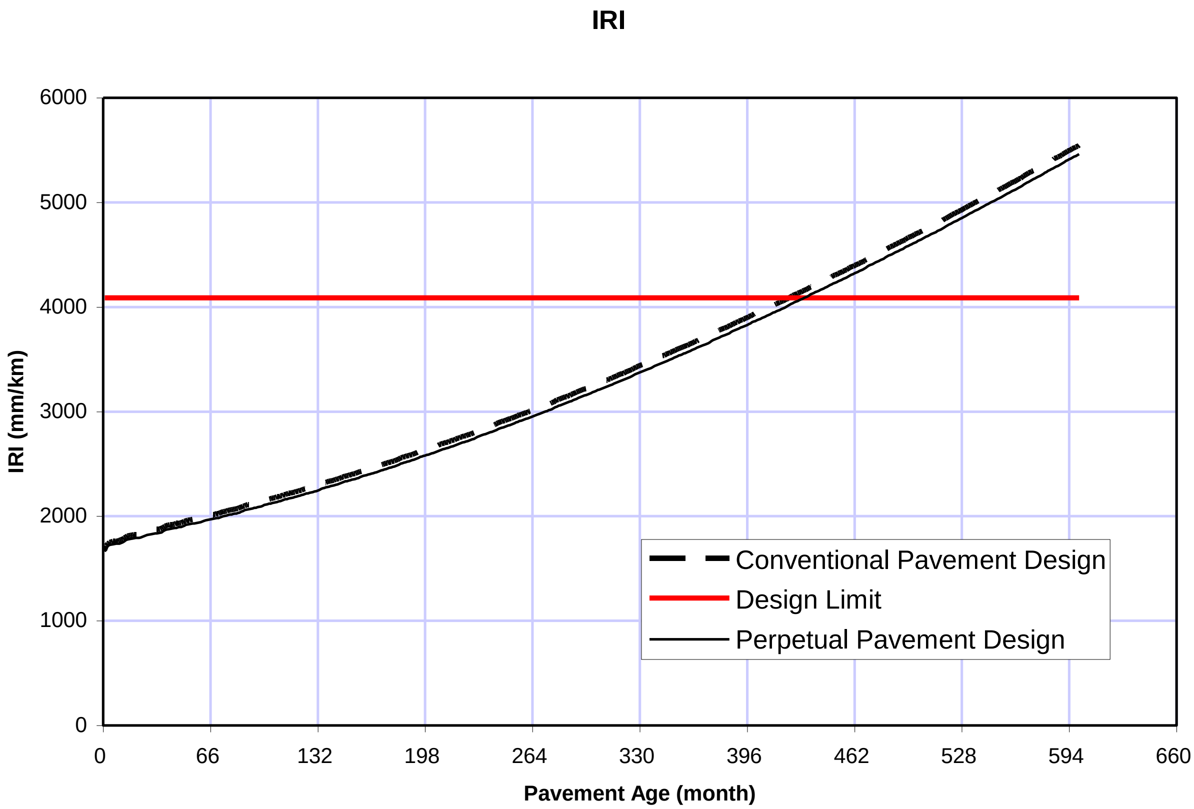


Figure 5-20: International Roughness Index Results for Low Traffic Volume Designs

Figure 5-20 shows the International Roughness Index (IRI) MEPDG model result. The pavement design methodology does not have a great impact on the IRI deterioration rate. The pavement roughness in the perpetual design has shown slightly better performance compared to the conventional pavement design.

5.4 WESLEA for windows Model

WESLEA for windows software was used to create computer models simulating the conventional and perpetual pavement designs for moderate and low traffic volume roads. The aim of this model is to evaluate the perpetual pavement design based on the normal strain value at the bottom of the asphalt layers. The normal strain value reflects the Long life pavement and shows whether the pavement section is behaving as a perpetual section or not. An introduction and brief explanation of the WESLEA software inputs and expected outputs was previously presented in section 4.3 of the thesis. The data used for the WESLEA model creation was assumed as that used for the MEPDG model. It is essential to highlight that the properties of all pavement layers (modulus of elasticity and Poisson's ratio) were assumed based on a comprehensive literature review of Hot Mix Asphalt (HMA) dynamic modulus data obtained from the FHWA Design Guide Implementation Team (DGIT).

5.4.1 Moderate traffic Volume Road Designs

Figure 5-21 shows the WESLEA model results for the moderate traffic volume road designs. The normal strain results are showing the high fatigue resistance characterizing the perpetual pavement design. The construction of thick asphalt layers with a RBM at the bottom of the asphalt layer has shown significant impact on the fatigue resistance of the pavement section. The normal strain value at the bottom of the asphalt layers is 58.6 µs. While the normal strain at the bottom of the asphalt layers in the conventional design is 72.3 µs. The result of the WESLEA model for conventional and perpetual designs is showing a higher fatigue resistance of the perpetual design than the conventional one. The WESLEA model results proves the results obtained from the MEPDG as they are both showing the better structural performance of the perpetual design compared to the conventional design.

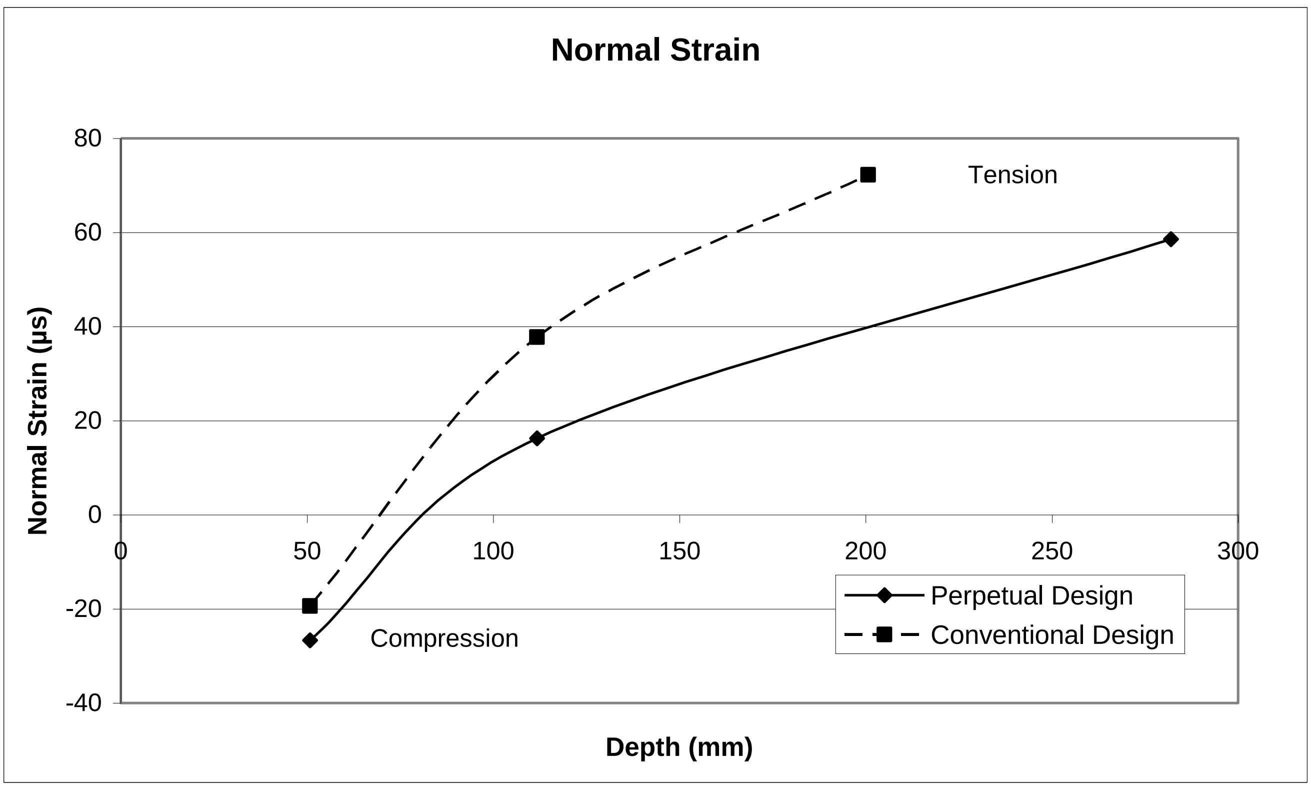


Figure 5-21: WESLEA Results for Normal Strain Values of Moderate traffic Volume Road Designs

5.4.2 Low Traffic Volume Road Designs

The WESLEA software was used to model the perpetual and conventional pavement designs for the low traffic volume roads. The WESLEA model results are presented in figure 5-22.

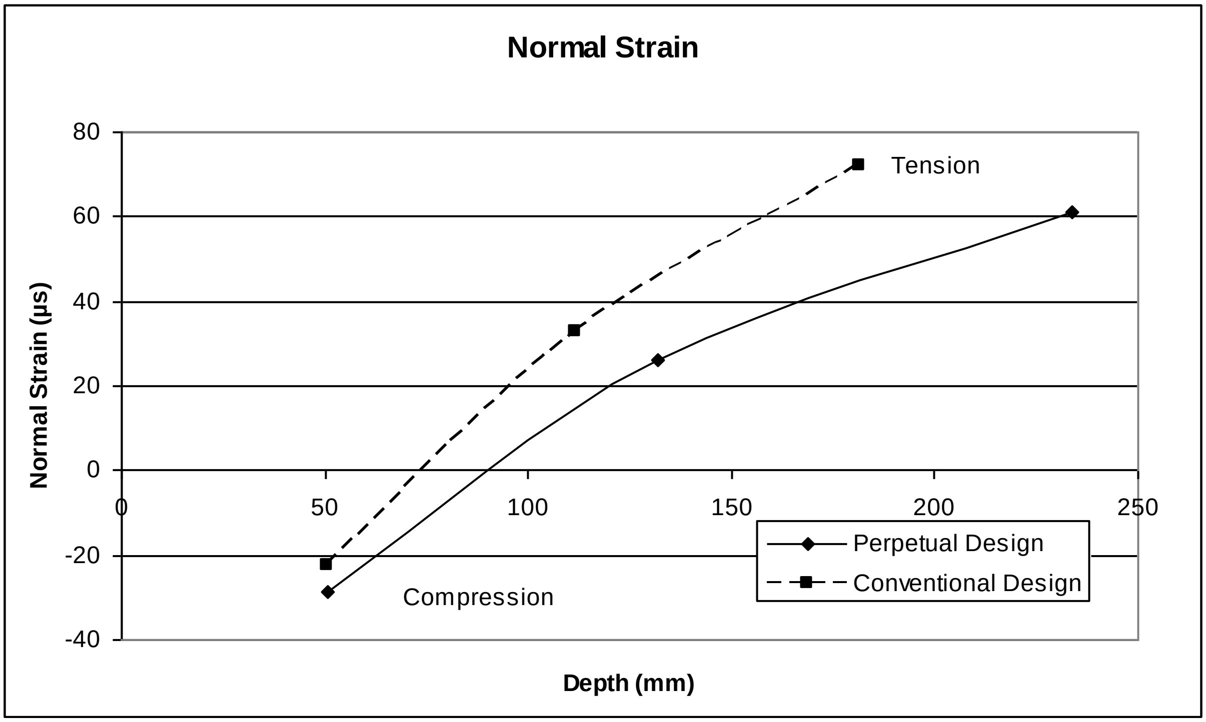


Figure 5-22: WESLEA Results for Normal Strain Values of Low Traffic Volume Road Designs

The WESLEA results are showing the normal strain value under the wheel path at the bottom of the asphalt layers in both perpetual and conventional designs. The normal strain value at the perpetual design is 61 µs which proves the perpetual design is having long term performance characteristics as it is below the 70 µs normal strain limit. The conventional design normal strain value is 71.9 µs. This value shows the ability of conventional design to be subjected to fatigue stresses due to its lower structural capacity compared to the perpetual design. The MEPDG model have proved the WESLEA model results by predicting it will be subjected to high percentage of bottom up cracks damage.

5.5 Life Cycle Cost Analysis (LCCA)

The pavement designs performed for moderate and low traffic volume roads were evaluated economically by implementing Life Cycle Cost Analysis (LCCA) on both the conventional and perpetual designs. The assumptions for the LCCA performed for both moderate and low traffic volume roads are similar to those made for performing the LCCA for high traffic volume roads in chapter 4. Maintenance and rehabilitation schedules were assumed based on the MTO recommendations and maintenance reports prepared for similar projects. The initial cost for all materials and labor required for construction were assumed based on the MTO Life Cycle Cost Analysis (LCCA) reports [Smith 1998][Hein 2007].

5.5.1 Moderate traffic Volume

Tables 5-5 and 5-6 present the expected construction cost of the conventional and perpetual pavement sections. It is noticed that the construction cost of the perpetual pavement design is higher than that of the conventional pavement design by twenty two percent.

Table 5-5: Initial Construction cost of the Conventional Pavement Design

Length (m)	(50 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(60 mm) Superpave 19 Density = 2.41 t/m^3	(90 mm) Superpave 25 Density = 2.34 t/m^3	(200 mm) Granular A Density = 3.12 t/m^3	(300 mm) Granular B Density = 2.05 t/m^3	SUM
1,000	$212,625	$171,990	$238,140	$238,680	$138,375.0	$999,810.0
1,000	$212,625	$171,990	$238,140	$238,680	$138,375.0	$999,810.0
					Avg	$999,810.0
					SUM	$1,999,620.0

Table 5-6: Initial Construction Cost of the Perpetual Pavement Design

Length (m)	(50 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(60 mm) Superpave 19 Density = 2.41 t/m^3	(70 mm) Superpave 25 Density = 2.34 t/m^3	(100 mm) RBM Layer Density = 2.44 t/m^3	(250 mm) Open Graded Drainage Layer (OGDL) Density = 2.02 t/m^3	SUM
1,000	$298,496	$248,134	$212,612	$348,920	$113,625	$1,221,787.00
1,000	$298,496	$248,134	$212,612	$348,920	$113,625	$1,221,787.00
					Avg	$1,221,787.00
					Sum	$2,443,574.00

The maintenance and rehabilitation programs were assumed for the conventional and perpetual pavement designs based on the MEPDG model results and MTO maintenance and rehabilitation programs recommended in their reports [Smith 1998][Hein 2007].

The maintenance and rehabilitation programs for the conventional and perpetual pavement designs are presented in tables 5-7 and 5-8 respectively.

Table 5-7: Maintenance Program for Conventional Design in Moderate traffic Volume Road

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (352 m/km)	4
Rout and Crack Sealing (352 m/km)	8
Rout and Crack Sealing (352 m/km)	12
5% Mill and patch 50 mm	15
Rout and Crack Sealing (704 m/km)	19
20% Mill and patch 50 mm	23
Rout and Crack Sealing (704 m/km)	27
Tack Coat	31
Mill 50 mm Asphalt Pavement	32
Superpave 12.5 FC2 - 50 mm	32
Rout and Crack Sealing (352 m/km)	36
Rout and Crack Sealing (352 m/km)	39
Rout and Crack Sealing (352 m/km)	42
20% Mill and patch 50 mm	45
Partial Reconstruction of Pavement	48
Rout and Crack Sealing (352 m/km)	52
Rout and Crack Sealing (352 m/km)	56
Rout and Crack Sealing (352 m/km)	60
5% Mill and patch 50 mm	63
Rout and Crack Sealing (704 m/km)	67

The maintenance and rehabilitation program for the conventional pavement design is designed to treat several structural pavement distresses as the bottom up cracks, rutting, top down cracks and alligator cracking in addition to some serviceability deterioration factors as the International Roughness Index (IRI). Rout and crack sealing is performed every four years in addition to mill and patch of the surface coarse every ten years to retain the surface layer condition and overcome the top down cracks before their propagation to the following layers. The bottom up cracking is treated by partial reconstruction of the pavement section every forty eight years. This partial reconstruction activity projects a reconstruction of the asphalt layers (surface HMA, intermediate HMA and HMA base layers). The alternative to this partial reconstruction activity is usually a thick asphalt overlay to increase the pavement thickness. Based on the structural and economic evaluation of both alternatives, the overlay solution will overcome some structural deformations but it will not be able to treat the bottom up cracks. These cracks will continue to propagate due to load repetitions and freeze thaw cycles and the pavement deterioration after the overlay is expected to be faster than the partial reconstruction alternative. Thus the partial reconstruction rehabilitation treatment alternative is expected to be more cost effective in the long term.

Table 5-8: Maintenance Program for Perpetual Pavement Design in Moderate traffic Volume Road

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (280m/km)	5
Rout and Crack Sealing (280m/km)	10
3% Mill and Patch 40 mm	14
Rout and Crack Sealing (560m/km)	18
15% Mill and patch 40 mm	22
Mill 50mm Asphalt pavement	26
SMA- 50 mm	26
Tack coat	26
Rout and Crack Sealing (280m/km)	30
Rout and Crack Sealing (280m/km)	35
15% Mill and patch 40 mm	39
Rout and Crack Sealing (560m/km)	43
Mill 50mm Asphalt pavement	48
SMA- 50 mm	48
Tack coat	48
Rout and Crack Sealing (280m/km)	53
Rout and Crack Sealing (280m/km)	58
15% Mill and patch 40 mm	62
Rout and Crack Sealing (560m/km)	66

The maintenance and rehabilitation program for the perpetual pavement design is designed to treat the expected distresses occurring in the perpetual pavement design and based on the MEPDG model results for the perpetual pavement design. The maintenance schedule includes several mill and patch treatments to retain the initial condition of the surface course and eliminate the top down crack before their propagation in the intermediate layers.

The LCCA results for the perpetual and conventional pavement designs have shown very similar results. Although the initial construction cost of the perpetual pavement design is higher than that of the conventional design by twenty two percent, the overall Net Present Value (NPV) of both pavement designs is almost equal after 70 year analysis period. The NPV for the conventional and perpetual pavement designs are $2,947,085 and $3,061,968 respectively. This calculation is based on a three percent annual discount rate and neglecting the user delay costs for the maintenance and rehabilitation operations. The difference between the NPV of both pavement designs is around 3.5% without including the user delay costs. The LCCA result is expected to give more advantage to the perpetual pavement design if user delay costs are included in the LCCA analysis as the number of maintenance operations included in the LCCA conventional design program is higher than that in the perpetual pavement maintenance program. Thus, more delay times are expected due to lane closers for longer periods and with shorter durations between the maintenance activities. The user delay cost was not included in this LCCA analysis to simplify the LCCA process and due to lack of precise user delay cost data on the proposed roads. However, it is recognized to be important. In addition, the extension of the LCCA analysis period for over 70 years is expected to give more advantage for the perpetual design as it is expected to perform better from a structural prospective over life cycle.

5.5.2 Low Traffic Volume

The construction cost of the conventional and perpetual pavement designs is showing noticeable difference compared to that of the moderate traffic volume designs. The reason behind that is the reduction in asphalt thickness in the both conventional and perpetual designs due to reduction of traffic loads. The number of asphalt layers in the perpetual pavement design was even reduced to three asphalt layers. The construction cost of the conventional pavement design was reduced by eight percent due to the reduction of the asphalt and granular layers' thickness. On the other hand, the reduction in the perpetual pavement design construction cost was even higher and is expected to reach ten percent.

Tables 5-9 and 5-10 presents the initial construction cost of the perpetual and conventional pavement designs respectively.

Table 5-9: Initial Construction Cost of the Perpetual Design for Low Traffic Volume Road

(50 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(80 mm) Superpave 19 Density = 2.41 t/m^3	(100 mm) RBM Layer Density = 2.44 t/m^3	(300 mm) Open Graded Drainage Layer (OGDL) Density = 2.02 t/m^3	SUM
$298,496	$330,845	$348,920	$136,350	$1,114,610.80
$298,496	$330,845	$348,920	$136,350	$1,114,610.80
			Avg	$1,114,610.80
			Sum	$2,229,221.60

Table 5-10: Initial Construction Cost of the Conventional Design for Low Traffic Volume Road

Length (m)	(50 mm) Superpave 12.5 FC 2 Density = 2.56 t/m^3	(60 mm) Superpave 19 Density = 2.41 t/m^3	(70 mm) Superpave 25 Density = 2.34 t/m^3	(200 mm) Granular A Density = 3.12 t/m^3	(250 mm) Granular B Density = 2.05 t/m^3	SUM
1,000	$212,625	$171,990	$185,220	$238,680	$115,312.5	$923,827.5
1,000	$212,625	$171,990	$185,220	$238,680	$115,312.5	$923,827.5
					Avg	$923,827.5
					SUM	$1,847,655.0

A maintenance and rehabilitation program was assumed to perform a Life Cycle Cost Analysis for a 70 year analysis period. The maintenance and rehabilitation activities included in the program are designed to address the estimated structural distresses based on the MEPDG model created to structurally evaluate the pavement performance of the conventional and perpetual pavement designs. The pavement designs are expected to be subjected to the same structural distresses as in the moderate traffic volume road designs but the deterioration rate of both pavement designs is less than in the case of moderate traffic volume roads due to the reduction in traffic loads. Thus, maintenance and rehabilitation programs for the low traffic volume road designs are having the same maintenance and rehabilitation activities while increasing the duration between maintenance activities.

Tables 5-11 and 5-12 presents the maintenance and rehabilitation programs designed for the perpetual and conventional designs respectively.

Table 5-11: Maintenance Program for Perpetual Pavement Design in Low Traffic Volume Road

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (280m/km)	6
Rout and Crack Sealing (280m/km)	12
3% Mill and Patch 40 mm	17
Rout and Crack Sealing (560m/km)	22
15% Mill and patch 40 mm	27
Mill 50mm Asphalt pavement	32
SMA- 50 mm	32
Tack coat	32
Rout and Crack Sealing (280m/km)	38
Rout and Crack Sealing (280m/km)	43
15% Mill and patch 40 mm	48
Rout and Crack Sealing (560m/km)	53
Mill 50mm Asphalt pavement	58
SMA- 50 mm	58
Tack coat	58
Rout and Crack Sealing (280m/km)	64
Rout and Crack Sealing (280m/km)	69

The maintenance and rehabilitation program designed for the perpetual pavement design included rout and crack sealing every six years at the beginning of the pavement life cycle. After several years and due to the pavement aging, the period between the crack sealing should be decreased to five years as the pavement deterioration rate increases due to pavement aging. Several mill and patch activities are essential for restoring the pavement section surface coarse condition. The crack sealing alone is insufficient to maintain the pavement section condition.

The maintenance and rehabilitation program designed for the conventional pavement design included a rout and crack sealing for the pavement surface course every 5 years at the beginning of the pavement life time. The rate of operating the crack sealing maintenance should be increased to every 4 years due to aging of the pavement section. In addition, the bottom layer of the conventional design is not designed to be a fatigue resistance layer. Thus, the bottom layer is expected not to be able to resist the fatigue loads and be subjected to fatigue cracking due to freeze thaw cycles. The conventional pavement design is expected to require a partial reconstruction by replacing the asphalt layers after fifty five years from construction. The partial reconstruction is the only rehabilitation activity that can treat the bottom up cracks caused by freeze thaw cycles and retain the pavement section structural strength.

Table 5-12: Maintenance Program for Conventional Pavement Design in Low Traffic Volume Road

Maintenance and Rehabilitation Activity	Year
Rout and Crack Sealing (352 m/km)	5
Rout and Crack Sealing (352 m/km)	10
Rout and Crack Sealing (352 m/km)	14
5% Mill and patch 50 mm	18
Rout and Crack Sealing (704 m/km)	22
20% Mill and patch 50 mm	26
Rout and Crack Sealing (704 m/km)	30
Tack Coat	34
Mill 50 mm Asphalt Pavement	34
Superpave 12.5 FC2 - 50 mm	34
Rout and Crack Sealing (352 m/km)	38
Rout and Crack Sealing (352 m/km)	42
Rout and Crack Sealing (352 m/km)	46
20% Mill and patch 50 mm	50
Partial Reconstruction of Pavement	55
Rout and Crack Sealing (352 m/km)	60
Rout and Crack Sealing (352 m/km)	65
Rout and Crack Sealing (352 m/km)	69

The LCCA results for the conventional and perpetual pavement designs in low traffic volume road have generated a similar result to that of the moderate traffic volume road designs. Although the construction cost of the perpetual pavement design is expected to be higher than that of the conventional design by eighteen percent, the overall Net Present Value (NPV) of both pavement designs for a 70 year LCCA period is showing very small difference. The (NPV) of both conventional and perpetual designs is $2,652,188 and $2,711,181 respectively. Conventional asphalt pavement design is having a (NPV) of two percent less than that of the perpetual design. The result of the LCCA is expected to provide more advantages to the road user as the user delay cost is reduced as mentioned in previous analysis in this research. In addition the perpetual pavement design is expected to perform for longer period without the need for a major rehabilitation or reconstruction. Thus, the increase of the LCCA period would provide more benefits that are not included in this analysis.

Chapter 6
Conclusions and Recommendations

6.1 Summary

This research has involved the design and instrumentation of three pavement test sections on Highway 401. In addition, two numerical and LCCA models have been developed to predict pavement performance under high traffic loading on Highway 401. Finally, perpetual pavement design has been analyzed for two moderate and low traffic volume roads.

Construction of three test sections on the Eastbound of Highway 401 approaching Woodstock, Ontario area was completed in summer 2009. Different pavement cross sections and asphalt mixes are used in the three test section. The pavement design procedure for two of the test sections are perpetual structural pavement designs while the third test section was designed as a conventional asphalt pavement section. The two perpetual pavement test sections are identical in terms of structural pavement layers characteristics and thicknesses with exception of the asphalt binder content in the bottom asphalt layer. One of the perpetual pavement sections is constructed with a Rich Bottom Mix (RBM) layer which means the binder asphalt content is increased by 0.5%. The other perpetual pavement section is constructed without having the RBM layer at the bottom asphalt layer. To monitor and evaluate the pavement structural performance of the three different pavement sections, six monitoring stations have been designed and installed with various sensors that are capable of measuring dynamic strain, vertical pressure, moisture in the subgrade soil and the temperature profile in the pavement layers. Two monitoring stations are constructed to monitor the pavement performance of each pavement design. The monitoring stations were constructed on the left and right lanes of the Highway 401. The right lane is expected to be subjected to heavy truck loading while the left lane is expected to be subjected to higher average speed. The construction of the three monitoring stations on the left and right lane will permit the evaluation of heavy truck impacts on the structural deterioration of these three unique pavement designs. The construction of the three pavement sections was successfully completed in summer 2009 and all the sensors were installed in the six monitoring stations and are showing reliable performance. The data collection phase has started. The longer term structural evaluation and analysis will be performed using the data collected from the test sections in a future doctoral research program.

Preliminary pavement structural evaluation and analysis was performed for the three pavement designs using the AASHTO-Mechanistic Empirical Pavement Design Guide (MEPDG). The 50 year analysis period simulation models were created using input level three in the MEPDG software. The mechanical, physical and chemical properties of the different pavement layers were based on state of the art practice and in partnership with the Ministry of Transportation of Ontario (MTO). The WESLEA software was used to structurally validate the Long life performance of the perpetual pavement designs by predicting the maximum tensile strain value at the bottom of asphalt layers in different pavement designs. Life Cycle Cost Analysis (LCCA) was performed for the perpetual and conventional pavement designs by preparing a maintenance and rehabilitation program based on the structural evaluation and performance prediction generated by the 50 year analysis period MEPDG models. The material and maintenance activities costs were assumed based on the MTO recommendations and price estimation in several reports.

Comparison between perpetual and conventional asphalt pavement designs under moderate and low traffic volume road conditions were also evaluated in this research. Comparing both pavement structural design methodologies when applied on heavy, moderate and low traffic volume roads provides a complete structural and economic evaluation for the two pavement structural designs. The pavement cross sections for the moderate and low traffic volume roads were designed using the AASHTO- DARWin software. The structural evaluation of the pavement designs on moderate and low traffic volume roads was performed using MEPDG models and the WESLEA for windows softwares. The economic evaluation of conventional and perpetual pavement designs in moderate and low traffic volume roads was performed using LCCA for both pavement designs. The LCCA was based on state of the art maintenance and rehabilitation programs for both pavement designs.

6.2 Conclusion

Structural analysis of pavement designs in heavy traffic volume roads have shown that perpetual pavement designs are subjected to lower stresses and corresponding strain values in the lowest and highest asphalt layers compared to the conventional designs. This result reflects the better structural performance of the perpetual pavement designs on the long term. Perpetual pavement designs are having high structural resistance to different fatigue stresses due to their thick asphalt layers. The perpetual pavement designs are showing high performance in resisting the bottom up fatigue cracking, top down cracking, rutting in asphalt layers and the base/subbase rutting. The International Roughness Index MEPDG model results are showing no significant improvement in the asphalt surface roughness by using the perpetual pavement design. The MEPDG models have evaluated the benefits of using a Rich Bottom Mix layer at the bottom of the asphalt layers. This layer was found to have higher fatigue resistance and is preventing the initiation of bottom up cracks compared to the regular asphalt layer used in the perpetual pavement design without RBM. The additional 0.5% of the asphalt cement in the bottom asphalt layer has increased the elasticity of bottom asphalt layer and decreased the air voids. It also improved the rutting resistance in the asphalt layers of the pavement section.

The economic analysis performed for the perpetual and conventional pavement designs under the heavy traffic volume resulted in a high initial construction cost for the perpetual pavement design due to the thick asphalt layers constructed in comparison to the conventional pavement design layers. Although the construction cost of perpetual pavement design is 70% higher than that of the conventional pavement design, the overall Net Present Value (NPV) of the conventional pavement design was found to be less than that of the perpetual pavement design by four percent at the end of 70 year analysis period. The LCCA did not include the user delay cost. However, the perpetual pavements are expected to have less NPV when user delay costs are taken into account in the LCCA and by increasing the LCCA analysis period. Although this design requires a high construction cost, it is expected to require less maintenance and rehabilitation activities through its life cycle. The top down cracking is the main distress affecting the perpetual pavement design and it can be resolved. The pavement section condition would be retained to its initial condition by milling and patching the surface asphalt layer every five to ten years in order to prevent the propagation of surface cracks to the following asphalt layers. While the conventional pavement design is expected to exhibit more bottom up cracks which require a partial reconstruction of the pavement section to restore the pavement condition.

The perpetual and conventional designs under moderate traffic volume conditions were structurally and economically evaluated. The perpetual pavement design is expected to have higher structural capacity and would be able to better resist fatigue cracking. Unlike the perpetual pavement design, the conventional design would exhibit a more rapid deterioration rate. The high fatigue resistance characteristic of the perpetual pavement design enables it to retain its feasible condition by applying minimum maintenance and rehabilitation activities. While the conventional pavement design does require more frequent maintenance activities including a partial reconstruction of the asphalt layers during the 70 year analysis period of the LCCA. Although the construction cost of the conventional pavement design is expected to be twenty two percent less than that of the perpetual pavement design, the difference in maintenance and rehabilitation has decreased the NPV difference between the two pavement designs to be 3.5% at the end of 70 year analysis period.

For low traffic volume, both pavement designs were again evaluated both structurally and economically. The perpetual pavement design MEPDG model showed an ability to withstand the fatigue stresses throughout life cycle. On the other side, the conventional pavement design is expected to exhibit bottom up cracking and high rutting values. The sound structural performance of the perpetual pavement design as modeled decreased the maintenance and rehabilitation activities. The perpetual pavement design required crack sealing activity to limit the crack propagation expected in the models. Mill and patch maintenance activities are implemented every ten years. The conventional pavement design will require more frequent pavement maintenance and rehabilitation activities to recover the pavement top down cracks and fatigue stresses. Although the initial construction cost of the conventional pavement design is lower than that of the perpetual pavement design by eighteen percent, the NPV at the end of 70 year analysis period for the conventional pavement design is less than that of the perpetual pavement design by two percent. The LCCA assumptions have neglected the user delay cost to simplify the LCCA calculations. If user costs were included, it is expected to provide even more of an advantage to the perpetual pavement design by increasing the LCCA analysis period and taking into account the user delay cost.

Overall, the perpetual pavement design proved to have better structural performance and require less expensive maintenance and rehabilitation program to maintain the pavement condition compared to the conventional pavement design.

6.3 Recommendations

This research resulted in a various recommendations that would benefit the pavement designers, ministry of transportations, consultants, contractors and all parties involved in the pavement production and road industry.

The recommendations can be summarized as follows:

1. Based on preliminary analysis and modeling, the use and construction of perpetual asphalt pavement designs in all road levels by increasing the asphalt layers thickness by 25% more than in the conventional pavement designs. The structural benefits of using perpetual

pavements will be verified by monitoring its performance and analyzing the data obtained from the field in the future.

2. Increasing the asphalt cement content in the bottom asphalt layer by 0.5% than the optimum binder content to create a Rich Bottom Mix (RBM) layer at the bottom of asphalt layers.
3. Construction of test sections of perpetual and conventional asphalt sections on moderate and low traffic volume roads to monitor the pavement designs performance and document the maintenance and rehabilitation activities needed for each accurately through their life time.
4. Applying the latest technology in sensor manufacturing in the test sections construction to monitor and evaluate the structural deterioration of the different pavement designs.
5. Making periodic pavement distress analysis using the Automatic Road Analyzer (ARAN) and the Portable Falling Weigh Deflectometer (PFWD). In addition, core sampling should be done whenever needed to investigate the stresses and deformations in different layers.
6. Further research and investigation is required to analyze the data collected by the sensors installed on the Highway 401.
7. Further research and development of a numerical model that simulates the pavement deterioration for perpetual pavements should be pursued which is based on the Canadian climate and uses the monitoring data available by different transportation agencies across Canada.

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