# Integrating Pavement Information Modeling (PIM), Pavement Sustainable Design, Climate Changing Adaptation and Cost Estimating at The Conceptual Design Stage of Roads

By

Hanguang Huang

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Under the Supervision of Dr. Ahmad Jrade

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Ottawa-Carleton Joint Institute of Engineering Faculty

Department of Civil Engineering

University of Ottawa

Ottawa, Canada

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# Abstract

Pavements are critical parts of infrastructure projects for urban growth since cities rely significantly on numerous kinds of transportation for access and movement. To fill the gaps in the knowledge and methodology used in the construction of sustainable, resilient, and cost-effective pavements, a model is developed based on the concept of Pavement Information Modeling (PIM), with the ability to develop roads at the conceptual design stage by providing ample versatility to influence stakeholders' decisions toward better pavement design. The model consists of five modules: 1) pavement design selection (PDS) module; 2) updated sustainable and resilient pavement design (SRPD) module; 3) Pavement Information Modelling (PIM) module; 4) conceptual cost estimating module; and 5) a preliminary scheduling analysis module. The model uses traffic data as inputs to run the Equivalent Single Axle Load (ESAL) determination model, which was developed based on AASHTO 1993 design recommendations, to estimate the traffic's impact on the pavement. The PDS module determines the recommended design for flexible and rigid pavements, including paving materials and structural design, based on various provincially published pavement design manuals, guidelines, specifications, and mechanistic-empirical pavement design guide (MEPDG) manipulated results. The sustainability's capability of the model is achieved by the SRPD module, which was developed based on the Canadian flexible pavement adaption measures gathered from the literature that investigated flexible pavement performance under future climate conditions in Canada. Various adaptation tactics, such as changing paving material at different lifts and varying lift thicknesses, are assigned to flexible pavement designs generated by the PDS module based on the project's location, construction period, and future climate conditions. A user-friendly platform is built,

allowing users to select and alter the pavement design within the ranges specified by the design criteria. The PIM module provides the pavement 3D design outputs within Building Information Modeling (BIM) environment via its tool (i.e., Autodesk Revit), allowing users to visualize their design's details in 3D Scenes. The cost estimation module generates three types of approximate cost estimates for the conceptually designed pavement based on the design inputs, historical bidding prices, RS Means cost data, and TAC recommended rehabilitation schedule, that includes materials' cost, labor and equipment costs, and rehabilitation cost. The fifth module provides an analysis for the project's construction duration by generating a preliminary linear schedule (Line of Balance, LOB). Both the cost estimate and schedule analysis reports are ideal for conducting projects' feasibility assessment.

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## Glossary

| | |
|---|---|
| 2D | Two Dimensions |
| 3D | Three Dimensions |
| AADT | Annual Average Daily Traffic |
| AASHTO | American Association of State Highway and Transportation Officials |
| AC | Asphalt Concrete |
| ACPA | American Concrete Pavement Association |
| ADT | Average Daily Traffic |
| ARA | Applied Research Associates |
| ATPD | Alberta Transportation Pavement Design |
| BIM | Building Information Modelling |
| CAC | Cement association of Canada |
| CBR | California Bearing Ratio |
| CMIP | Coupled Model Intercomparison Projects |
| CRCP | Continuously Reinforced Concrete Pavement |
| C&D | Construction and Demolition |
| DCG | Dense Course Graded |
| DFG | Dense Fine Graded |
| DOT | Department of Transportation |
| ESAL | Equivalent Single Axle Load |
| FHWA | Federal Highway Administration |
| GHG | Greenhouse Gases |
| GIS | Geographic Information System |

| | |
|---|---|
| HMA | Hot Mixed Asphalt |
| IGB | Intermediate-Graded Base |
| IPCC | Intergovernmental Panel on Climate Change |
| IRI | Roughness |
| JPCP | Jointed Plain Concrete Pavement |
| JRCP | Jointed Reinforced Concrete Pavement |
| LCA | Life Cycle Analysis |
| LCCA | Life Cycle Cost Analysis |
| LOB | Line of Balance |
| LTPP | Long-Term Pavement Performance |
| PCA | Portland Cement Association |
| LCCA | Life Cycle Cost Analysis |
| ME | Mechanistic Empirical |
| MEPDG | Mechanistic-Empirical Pavement Design Guide |
| MTO | Ministry of Transportation of Ontario |
| M&R | Maintenance and Rehabilitation |
| NCHRP | National Cooperative Highway Research Program |
| NPV | Net Present Value |
| OGB | Open-Graded Base |
| OGFC | Open-Graded Friction Course |
| PA | Porous Asphalt |
| PCA | Portland Cement Association |
| PCC | Portland Cement Concrete |
| PDS | Pavement Design Selection |

| | |
|---|---|
| PG | Performance Grading System |
| PIM | Pavement Information Modelling |
| PSI | Pavement Serviceability Index |
| RAP | Reclaimed Asphalt Pavement |
| RCM | Regional Climate Model |
| RCP | Representative Concentration Pathway |
| SMA | Stone Matrix Asphalt |
| SN | Structural Number |
| SRPD | Sustainable and Resilient Pavement Design |
| TAC | Transportation Association of Canada |
| TF | Truck Factor |
| USGCRP | United States Global Change Research Program |
| USGS | United States Geological Survey |
| WGB | Well-Graded Base |

# Chapter One

## Introduction

### 1.1 General

With the population's growth, the demands for a competent transportation system have increased over the past hundred years. Pavement as being one of the most common type of assets used in road infrastructures provides convenience to society while facing inadequate development to the growth of mobility (Costin et al., 2018).

Meanwhile, the emission of Greenhouse Gases (GHG) caused by human activities leads to global warming and causes changes in the temperature, precipitation, humidity, and groundwater table (Knott et al., 2019). Pavements are subject to deterioration due to aging while the changes in climate factors are aroused by the excessive GHG impact on the pavement design, performance, and service life (Swarna et al., 2021a).

Like Building Information Modeling (BIM) that is used in building projects, the concept of Pavement Information Modeling (PIM) is introduced for pavement projects to provide adequate comfort and safety for motorists. Although, the application of BIM concept has been successively used in building projects in the past decades, the adoption of BIM concept to pavement projects is still at the beginning (Maltinti et al., 2021). Up to now, the application of BIM concept in pavement projects is not comprehensive where most of the models presented in the literature focused only the traffic impacts without considering the impacts of the changing climate factors. Thus, the adoption of BIM concept on pavement projects along with the consideration of climate change adaptation has great potential to the improvement of pavement design and could fill in the gap between conventional pavement design and sustainable pavement design (Costin et al., 2018).

PIM methodology is an efficient guidance to develop a tool that can simplify the design and construction of pavement projects. To achieve this, PIM takes into consideration both the pavement design information and climate information in the corresponding construction location (Maltinti et al., 2021). The adaptation of design strategies to climate change is gaining momentum for the development of sustainable pavement, especially in the economic aspect, but no technological progress has been integrated into this concept. To maximize the pavement design's efficiency and reach an optimum pavement performance, sustainable measures must be applied in each stage of pavements' life cycle. PIM can integrate sustainable considerations into the general pavement design (Swarna et al., 2021a; Costin et al., 2018).

## 1.2 Research Objectives

The vast volume of information makes it a challenge to manage the design and implementation of paving projects. Traditional management employs empirical approaches that necessitate constant upkeep, resulting in wasteful time and capital. Adding sustainability adaptation to a pavement's design necessitates the access of additional information management. To increase efficiency, PIM is used, which is capable of automatically store and analyze the necessary information. By implementing PIM with a long-term perspective, information transition and communication among projects' stakeholders become more successful. As a result, the primary objective of this research thesis is to develop an integrated model that interrelates a pavement design system with economic consideration, and a pavement information modeling along with cost estimating and scheduling at the conceptual design stage.

The sub-objectives are:

- Identify the climate impacts on the current pavement construction projects,
- Identify, search and understand the current characteristics of PIM for model development,
- Identify the Canadian pavement design methodologies and limitations,
- Identify and outline the improved design and management methods for pavements under climate change,
- Identify the relationship between the conceptual design of pavement projects and economic and resilient development.

## 1.3 Research Methodology

The research followed a series of steps that outline the adopted methodology to achieve the set list of objectives. The procedure is as follows:

### *1.3.1 Literature Review*

A thorough literature review is performed to identify recent advances and limitations in PIM and its application, pavement design methodology, and sustainable pavement development research studies. This stage is critical since it helps clarify the study objectives and methodology.

### *1.3.2 Data Collection*

Several modules of the integrated model require a database to be established to implement design and forecasting values. The collected data are used for the preliminary design of pavements and the corresponding life cycle cost analysis. The variables required for different types of pavement designs are identified and samples of historical design data are compiled to restrict and minimize the range of inputs. The reliability of the design guides and historical design data is imperative for future estimates. Therefore, the data is acquired from reliable public database sources such as the governmental or provincial database of Statistics Canada or the department of transportation in the U.S.

### *1.3.3 Modules Implementation and Integration*

From the literature review, the implementation methodologies for each of the five modules are clearly outlined and followed. The five modules are: 1) a pavement design selection module; 2) a sustainable and resilient pavement design module; 3) PIM module; 4) conceptual cost estimating module; and 5) construction scheduling module. The pavement structural design module along with the climate change adaptation and PIM module are directly implemented and coded by using C# programming language, while the cost estimating module and the construction scheduling module are implemented using C# coding along with Microsoft Excel spreadsheets due to their needs for compiled databases. Each of those modules is integrated with the model following a logical design sequence to enhance the model's overall capabilities.

## 1.4 Thesis Organization

This thesis is organized through six chapters as follows:

**Chapter Two: Literature Review**

Covers the substantial literature review performed to define the terms and concepts associated with revised pavement design strategies, pavement sustainability, and PIM. Advances in the industry are recognized, while gaps in the literature are identified. The first highlighted gap in the literature is the absence of an accurate and user-friendly conceptual pavement design process. There are pavement design approaches, however they are either inaccurate or too complex. According to the literature, few studies have examined pavement sustainable design that takes into account the temperature, precipitation, and traffic variables at the same time, and the integration of an adapted strategy with the conceptual design of pavements has been uncommon. Furthermore, literature indicates that the application of pavement information management is still at the development stage, and that its implementation has a major impact on the project's life cycle cost. As a result, the conceptual design stage is identified as the best period for pavement information modelling.

**Chapter Three: Research Methodology**

Clearly defines the development processes for the five modules, which are: 1) Pavement Design Selection, 2) Sustainable and Resilient Pavement Design, 3) Pavement Information Modelling, 4) Cost Estimating, and 5) Construction Scheduling. Each module's methodology describes its development process, its data and sources in the data flow process, the techniques used in its development, and how each module is incorporated with the others. Furthermore, the general model components and dataflow development approaches are discussed.

**Chapter Four: Model Development**

Illustrates the development process of the model and its integrated modules and describes how the model achieves its user-friendliness goal.

**Chapter Five: Model Testing**

Test the full capabilities of the model by simulating the conceptual design of a pavement project.

**Chapter Six: Conclusion and Future Work**

Presents the research findings by demonstrating how the pavement design selection module, sustainable and resilient pavement design module, PIM module, cost estimating module, and construction scheduling modules incorporate sustainability design features and PIM capabilities at the conceptual design stage. Similarly, the chapter discusses how the designed model contributes to the conceptual design of sustainable and economical pavement. Furthermore, the constraints of the current model are discussed, as well as proposes future tasks for the model improvement and enhancement.

## 1.5 Research Contribution

To provide a reliable and simplified decision-making process, the research contributions are:

1. The development of a simplified and complexity-understandability-balanced pavement design procedure that provides users the options to conceptually design pavement projects in Canada.

2. The creation of a PIM module within BIM environment via Autodesk Revit through set of C# algorithms to accomplish the 3D visualization of the generated conceptual pavement parameters.

3. The execution of maintenance plans, LCCA, and construction schedules of the pavement projects and integrating them with the developed cost estimation and scheduling module then connecting them to the PIM module to facilitate the decision-making process.

## 1.6 Summary

This chapter provides an outline of the current problems that pavement construction projects are facing and lists the proposed solutions reflected by the set research objectives and the adopted methodology. A brief overview of the research methodology used to achieve the study objectives is provided, and the thesis arrangement is broken down with a brief explanation of what each chapter contains.

# Chapter Two

## Literature Review

### 2.1 Introduction

The first step in this research study is to conduct a comprehensive literature review to identify the existing gaps that exist in the industry and to highlight the research areas that were left behind in the published studies, which will help clearly set the current study objectives. Pavement is an important part of any transportation infrastructure project because of its effects on commuting and economy on societies. Applying PIM concepts could potentially have good consequences, such as introducing sustainable and resilient design, saving the costs, improving the pavement quality, and enhancing productivity and efficiency. Pavement projects are expensive and complex, and their designs are subject to the fluctuation of climate change. This chapter will discuss how the climate affects the pavement, it will provide definitions for important concepts, and will describe the design techniques for applying PIM and considering resilient and sustainable pavement.

### 2.2 Impacts of Climate Change on Pavement

With the increased value of GHG, climate has changed during the past decades. The different impacts brought by climate change are a rise in the global temperature, extreme weather fluctuations, major change in the quantity of precipitation, which lead to a variation in the underground water table, those are the main factors that accelerate the pavements' deterioration (FHWA, 2015; USGCRP, 2018; Government of Canada, 2014). Several studies revealed that the majority of highways and roads pavements in Canada are facing severer challenging. The government of Canada (2007) and Canada's Core Public Infrastructure Survey (2018) stated that permafrost zones underline more than half of Canada's highways, especially in the Northern territories. Due to the increase in temperature, permafrost degradation becomes the main source for pavement damage (Boyle et al., 2013). In Eastern and Western Canada, more than 80% of the coastline is submerged due to the rising sea levels. Greater storms with higher frequency could accelerate the erosion of coastline, which results in additional deterioration in the pavement (Natural Resources Canada, 2007; Government of Canada, 2011).

Presently, pavements are constructed from materials that are suitable for the environment. Studies showed that 94% of the roads in the United States have flexible pavements, while more than 90% of the pavement in Canada are covered by asphalt. Historic data and assumptions made about a fixed climate were the basis for the traditional pavement's design, which took into consideration the materials used in construction (Qiao et al., 2020; Qiao et al., 2023; Underwood et al., 2017). Therefore, climate change caused the rapid failure of infrastructures and accordingly the increased cost of maintenance. It is well known that the lifespan of Canadian pavement is less than 30 years (Tighe, 2015).

### *2.2.1 Impacts of High Temperature and Extreme Weather on Pavement*

The Canadian government claims that the northern part of the country has the highest midsummer average temperatures. The average summer temperature will rise by 2 to 5℃ within the next 50 years as well as the winter temperature (Government of Canada, 2010; Boyle et al., 2013). The literature showed that extreme weather has a substantial correlation with the pavement disruption. In Canada, where water content plays a significant role in the stability of pavements, Alberta and Saskatchewan experience the greater water scarcity as described by Boyle et al., (2013). In Canada, asphalt accounts to more than 90% of paved surfaces. Due to the temperature-dependent modulus of asphalt concrete (AC) being reduced by severe heat, bituminous materials are more susceptible to deformation or rutting. Flexible pavement softening, rutting, bleeding of asphalt, longitudinal cracking, raveling, and fatigue are more likely to occur due to the increased frequency of hot days during the summer and the lack of water content in the pavement layers (Boyle et al., 2013; Haslett et al., 2021; Qiao et al., 2013; Qiao et al., 2019). Thus, in Canada, the expected rutting of asphalt concrete ranges from 0 to 16 mm within a 20 year period, with 25% of the AC rutting is between 6 to 7 mm. A total rutting ranges from 5 to 25 mm, with more than 70% of the sections display a total rutting between 13 to 19 mm. Over a 20 years period, the rate of cracking due to asphalt concrete fatigue is predicted to reach 5.46%, however it will be close to 11.2% in 50 years (Mills et al., 2009; Saha et al., 2012; Swarna et al., 2022).

According to Underwood et al., (2017), the type of asphalt utilized in the U.S for the Superpave Performance Grading System (PG) is designed based on determining a temperature-related grade related to the maximum and minimum temperature. In the U.S, 35% of the stations have a different high or low temperature grade (6% for high, 26% for low, and 3% for both). Pavements degrade more quickly and require more maintenance during hot weather. Generally, the capabilities of a pavement designed for a life of 20 years will need to be maintained after 16 to 17 years (Underwood et al., 2017; Zeiada et al., 2022). In the United States, the effects of climate change on pavement subgrade and asphalt binder pushed the average pavement life to drop from 16 to 4 years, and to increase the maintenance cost by 100% (Gudipudi et al., 2017; Stoner et al., 2019).

A study done by Gudipudi et al., (2017), which took into account the traffic impacts, found that rutting is more likely to occur when the traffic volume is high and the speed is slow. Bituminous binder’s viscos-elastic nature is influenced by temperature, whereas age hardening is another component that impacts pavement flex under traffic. Migration brought by climate change changed the population and the associated traffic patterns, causing pavement degradation, which is difficult to predict. Hence, the pavement is affected directly and indirectly by climate change. On another side, the stress-strain response is constrained by both the high temperatures and the heavy traffic load, which are making asphalt surfaces brittle and susceptible to thermal fatigue cracking (Gudipudi et al., 2017; Qiao et al., 2013; Tighe et al., 2015). A study presented by Qiao et al., (2013), showed that a 5% increase in temperature had caused a 20% loss in the service life of pavement in the U.S.

Over 90% of the pavements in North America (Canada and U.S) are paved with asphalt, however, the literature is lacking discussion on how the climate change affects the rigid pavements. The slabs stretch and curl because of the high temperature and moisture's differences in Portland cement concrete. The Jointed Plain Concrete Pavement (JPCP) slab cracking sensitivity is very high to the daily average range of temperature. The temperature gradient in the slab from top to bottom and the associated critical stresses during the day (upward curling) and night (downward curling) are significantly influenced by ambient temperature. Transitory temperature and moisture gradients in Portland cement concrete cause curling and warping in the slabs. Therefore, both transient and permanent alterations are required to calculate the joint opening and closure, as well as the fatigue damage accumulation of JPCPs (Chai et al., 2012; Shafiee et al., 2019).

### *2.2.2 Changing the Precipitation Impacts on Pavement*

Over the next three decades, it was predicted that a substantial number of locations in North America would experience an increase in aridity, which would lead to drought conditions and the subsequent salinization of land. On the other hand, warm temperatures during the winter cause the process of water vapor condensation to occur, which causes substantial amounts of precipitation to be formed. Based on an information released by Environment Canada in 2005, it is anticipated that precipitation levels could rise in a range of 0% to 10% during a 45-year time period (Environment Canada 2005; Meyer et al., 2014).

One of the main effects that an increase in the precipitation causes is the raising of sea levels. The elevation of the road has an impact on the possible effects of the increase of the sea level on road infrastructure, particularly on coastal highways. In such cases, the increased danger of erosion, slope instability, and structural damage of pavements is a significant concern (Saleh et al., 2022). In North America, pavements of lower functional class are typically built to sustain storm events that are occurring once every 10 to 25 years. Due to the variations in the precipitation duration and its frequency, there has been an observed increase in the formation of major storms. Consequently, these intense storm events have the potential to weaken the bearing capacity of the pavements (FHWA 2016; Saleh et al., 2022).

The presence of water is a significant factor in determining the stability of pavement. Premature pavement's failure could occur in coastal places due to the combined effects of sea level rise and the storm events, which lead to the infiltration of the surface water into the unbound layers of the pavement structure. Excessive groundwater infiltrates the pavement structure via cracks and potholes, mostly impacting the surface layer of the pavement. The presence of pore water pressure under traffic conditions disrupts the adhesion between aggregate and bitumen, hence increases the occurrence of stripping and raveling in the asphalt layers (Haslett et al., 2021; Qiao et al., 2020). The resilience modulus and stiffness of unbound and subgrade materials are also influenced by moisture. Furthermore, the infiltration of water into pavements has been observed to diminish the shear strength of unbound materials, rendering pavements susceptible to persistent deformation (Gudipudi et al., 2017; Qiao et al., 2013). Moreover, with the increasing frequency of precipitation, the flow rate during instances of unexpected heavy precipitation exceeds the intended design flow rate, resulting in the occurrence of flooding. Based on the finding of Lu et al.'s study (2018), the impact of flooding on the service life of pavement varies depending on the magnitude of the flooding event, which may result in either short-term or long-term consequences. Excess loading, which is primarily caused by overflow, can result in instability of the pavement that in turn may lead to a sudden structural failure. In terms of long-term consequences, such as delayed effects, the immediate impact of flooding is rather minimal. Nevertheless, the degradation of pavement performance is expedited after the incidence of flooding (Ghani et al., 2016; Lu et al., 2018). A study by Ghani (2016) examined the enduring effects of floods on pavement where the results obtained from conducting the California Bearing Ratio (CBR) test on sample pavements subjected to repeated submergence showed that the soil strength experienced a reduction of 61.1% after being submerged in the water for a duration of seven days. Pavements experience accelerated settlement and quick performance degradation due to reduced soil strength and following floods, which would result in the formation of a jump effect. The occurrence of the jump and delayed effect is observed when rapid settlements and water degradation concurrently affect pavement, resulting in both immediate and prolonged consequences (Ghani et al., 2016; Lu et al., 2018).

In a study conducted by Shafiee et al., (2019), it was found that pavements' performance exhibit less sensitivity to precipitation when compared to other climate conditions. However, it should be noted that the performance of rigid pavements is still influenced by the presence of the water content. The spread of cracks in slabs is influenced by the moisture gradient and lower temperature, which in turn affect the phenomena of curling and warping. The Load Transfer Efficiency (LTE) of Portland Cement Concrete (PCC) aggregates interlock is expected to decrease because of the winder joint aperture induced by slab shrinkage. Consequently, the occurrence of joint faulting is heightened in response to elevated levels of precipitation (Shafiee et al., 2019).

The occurrence of precipitation also contributes to the formation of sinkholes, hence exerting an additional influence on pavement infrastructure. Precipitation is identified as the primary factor contributing to the natural occurrence of sinkholes, as stated in a study conducted by the United States Geological Survey (USGS) in 2007. The infiltration of water content into pavement occurs through cracks or openings in joints, resulting in long-term effects on the pavement. Throughout its life cycle, the pavement undergoes expansion, contraction, and blow-ups, leading to the formation of deeper fissures. In the absence of remedial measures to address the issue, the formation of cracks allows stormwater to infiltrate the underlying subgrade layers, exacerbating the process of degradation (Adham et al., 2010; USGS, 2007).

Soil composition is an additional component contributing in the formation of sinkholes. As stated by Adham (2010), the process of dissolution occurs when limestone reacts with rainwater, resulting in the formation of an aqueous bicarbonate salt. The combined influence of elevated temperature and higher inflow of the water velocity results in a heightened effect of acidic stormwater on the breakdown of limestone. The process of limestone deterioration would be exacerbated by global warming, leading to an increase in the porosity of the subgrade layer. The recurring pattern of traffic pressure and fluctuations in the subsurface water levels speed up material dissolution and displacement, ultimately resulting in the formation of cavities. Consequently, the mechanical integrity of unbounded layers is diminished, rendering them more susceptible to collapse (Adham et al., 2010; Linares et al., 2017).

### *2.2.3 **Changing in Permafrost Zone Impacts on Pavement***

In contrast to road infrastructure in other geographical areas, Canadian pavements encounter distinct challenges caused by the presence of permafrost regions. Canada is identified as having a significant presence of snow, with approximately 65% of its land area experiencing snow cover for a duration exceeding six months annually (Government of Canada, 2021; Haslett et al., 2021). Climate change has been shown to cause an increased level of precipitation during the winter, leading to wetter conditions on pavement structures. The freeze-thaw cycle in snowy regions is a result of the combined influence of temperature and moisture, whereas the presence of wind has been found to cause a reduction in pavement surface temperature ranging from 2 to 10℃. In regions characterized by cold temperatures, the presence of moisture in unbound layers and subgrade soils undergoes a freezing process, resulting in the formation of ice lenses. These ice lenses then thaw during the spring. The thawing process can cause an excessive amount of moisture to be released, which becomes trapped in the pavement's base layers until the thawing is completed. As a result, the pavement's capacity to endure external forces and resist deformation declines due to the infiltration of water into its pores, eventually causing distress in the pavement. Salour et al., (2012) and Qiao et al., (2020) observed a reduction in resilience modulus due to thawing that ranges between 48% and 63%, respectively.

More than half of the road infrastructure in Canada is situated within areas that have the presence of permafrost (Canada's Core Public Infrastructure Survey 2018). The impacts of climate change led to the occurrence of warmer winters and a reduction in the winter duration, resulting in the disappearance of permafrost zone and alterations in the frequency of freeze-thaw cycles (Barbi et al., 2021; Saha et al., 2012). The permafrost zone, which is mostly located in the northern areas of Canada such as Northwest Territories, Nunavut, and Yukon, poses a significant challenge to the stability of pavements due to the presence of permafrost layers (Saha et al., 2012). In addition, it is worth noting that the western region of Canada is recognized as a high freeze-dry zone, while the eastern region is classified as a high freeze-wet zone.

Climate change has the effect of accelerating the freeze-thaw cycle, hence diminishing the stability of pavements. The occurrence of transverse cracking is seen to be significantly elevated in stations located in Saskatchewan, Quebec, and Yukon. In the permafrost zone, it is anticipated that there will be a rise in the longitudinal cracking. The presence of severe frost and a high freezing index (FI) in the area results in a subgrade that is classified as highly rigid. Consequently, this rigidity leads to a significant tensile strain at the surface layer, resulting in the occurrence of longitudinal cracking (Maadani et al., 2021; Saha et al., 2012; Tighe et al., 2015).

## 2.3 The Conceptual Design Stage of Pavement

Pavements are multilayered structures mainly made of concrete, asphalt, and stone. To withstand forces from the climate and traffic, each underlayer serves as the basis for the layers above it (Ahammed et al., 2018). Since each layer must be sufficiently stiff to prevent overstressing during its service life, the goals of pavement design include finding the proper paving materials and calculating the thickness of pavement layer for load support (Ahammed et al., 2018; Moreno et al., 2017).

To standardize the design process, pavement designs approaches adopted in North America are mainly developed by National Transportation Associations (NTA) such as the American Association of State Highway and Transportation Officials (AASHTO) and Federal Highway Administration (FHWA), and Transportation Association of Canada (TAC), or industry associations such as the Asphalt Institute and Portland Cement Association (PCA) (Ahammed et al., 2018). According to a TAC study in 2011, 75% of Canadian provinces follow the AASHTO 1993 guidance for flexible pavement, compared to 100% for rigid pavement design (Boone, 2013; TAC, 2011).

However, the design guides remain problematic since most of the existing designs are either over-designed or under-designed. Most of the rehabilitation projects have over-designed components, which caused an increase in the costs and a reduction in the performance of rigid pavements. On the other hand, a shorter service life is the result of a thinner pavement structure. Records from the Ontario Ministry of Transportation (MTO) show that the cost of rehabilitation is substantial because of the environment and traffic's impacts. In 2007, Ontario's Road system had a replacement cost of $46 billion. As a result, the primary goal of the pavement design is to provide structural alternatives that are both technically and financially viable. According to the specification of the pavement layer's thickness with appropriate materials under certain traffic and environmental conditions, a Life Cycle Cost Analysis (LCCA) must be used to achieve the goal (Ahammed et al., 2018; Mills et al., 2009; Ontario Ministry of Transportation, 2013; Tighe et al., 2007).

To present more effective pavement designs, the design approach is divided into three groups: a) A design that is based on empirical pavement performance models; b) A design that is based on mechanistic analysis; and c) A design that is based on Mechanistic-Empirical (ME) models (Tighe et al., 2007).

### *2.3.1 AASHTO 1993*

In 1920, Yoder was the first to propose the idea of pavement design. That idea was not frequently applied at that time since the pavement design was primarily based on prior experience. To give a fundamental information about the pavement design, an experiment was conducted in an area close to Ottawa, Illinois, between 1958 and 1960. In 1962, at the first International Conference on the Structural Design of Asphalt Pavements, the principles of pavement design were further introduced. The AASHTO interim guide for pavement structural design was created in 1972 to improve the construction specifications. In 1986, that design manual was revised to provide new information about enhanced materials and their properties. The most recent version was released by AASHTO in 1993, which included rehabilitation design for both flexible and rigid pavement (Boone et al., 2013; Haas et al., 2007; Transportation Research Board 2007). That design is known as an empirical pavement design because the AASHTO 1993 design process is based on empirical equations established from experiment done in 1958. The design guides set a truck class distribution, traffic loads, subgrade modulus, and road roughness conditions as user inputs. Then AASHTO 1993 used empirical equations to calculate the changing in Pavement Serviceability Index (PSI) during the pavements' service life by relating pavement longitudinal cracking length or area, patched area, and rut depth. The outputs of the overall design procedure is a Structural Number (SN) for determining the pavement thickness, while the procedure clearly defines the relationship between pavement distresses and pavement design (AASHTO 1993; Boone et al., 2013; TAC 2011). A study performed in Manitoba showed that the application of AASHTO 1993 can minimize the calculated Equivalent Single Axle Loads (ESALs) if compared to the previous design approaches, which is an improve in the design efficiency and the pavement performance (Ahammed et al., 2018). However, only 45% of the U.S. agencies confirmed that the pavement's actual performance matches the expected design life from the design procedure they are using (AASHTO 1993, AASHTO 1972, AASHTO Design Procedure), therefore the AASHTO design procedure remains holding some limitations (Aguib et al., 2021). First, the AASHTO 1993 design analyzed how the traffic load affected the pavement deterioration and losing parts of its serviceability by using empirical models that were created from field performance data based on AASHTO road tests. An update of such data is necessary because the experiment was conducted in the 1950s, which means it is an outdated design information due to the relatively low traffic loads (up to 2 million ESALs), single climate, and single subgrade material used at that time. (Boone et al., 2013). Furthermore, it is specific to one design scenario that only considered one traffic mode and one unique temperature condition. The design is constrained by the area in which it was created, which makes it challenging to implement in areas with different climatical or material characteristics. AASHTO 1993 design also lacks the ability to predict how the pavement structure would perform during later maintenance and rehabilitation phases since it ignored the issue of material aging (Ahammed et al., 2013; Bayomy et al., 2012; Boone et al., 2013; Carvalho et al. 2006; Hall & Beam 2005; Schwartz 2007; Li et al., 2011; TAC 2011; Tighe et al., 2007).

Results from studies done by Carvalho and Schwartz in 2006 indicated that the state of pavement that had been designed by AASHTO 1993 and constructed in warm locations had lower performance satisfactorily than those in mild to low-temperature regions. The AASHTO 1993 guidance was created in a cold climate with flexible pavements, hence the design for flexible pavement layers in hotter climates underestimates the thickness needed (Carvalho & Schwartz, 2006). The pavement behavior predicted by AASHTO 1993 was tested in 2011 by Perraton by using a similar test. As a result, the findings show that the AASHTO technique is insufficient for pavement designs that utilize high-performance materials. Additionally, according to Perraton (2011), AASHTO 1993 only takes the resilient modulus value into account when evaluating the pavement performance, which is inadequate since pavement's performance over the time is also crucial for the pavement design. Therefore, AASHTO 1993 methodologies can be used after considering additional updates and modifications to improve the reliability in pavement design (Ahammed et al., 2013; Boone et al., 2013; Perraton 2011).

### *2.3.2 Mechanistic Pavement Design*

In the 1920s, Westergaard developed the first mechanical study for rigid pavements, which analyzed the critical loads and deflections in a PCC slab. To calculate stresses, strains, and deflections in flexible pavements, Burmister introduced the mechanistic theory in 1940s based on the multilayer linear elastic theory (Christopher, 2007). Figure 2.1 illustrates the design procedure that acquires climate factor such as moisture content, temperature, as well as paving material properties and traffic conditions from users.

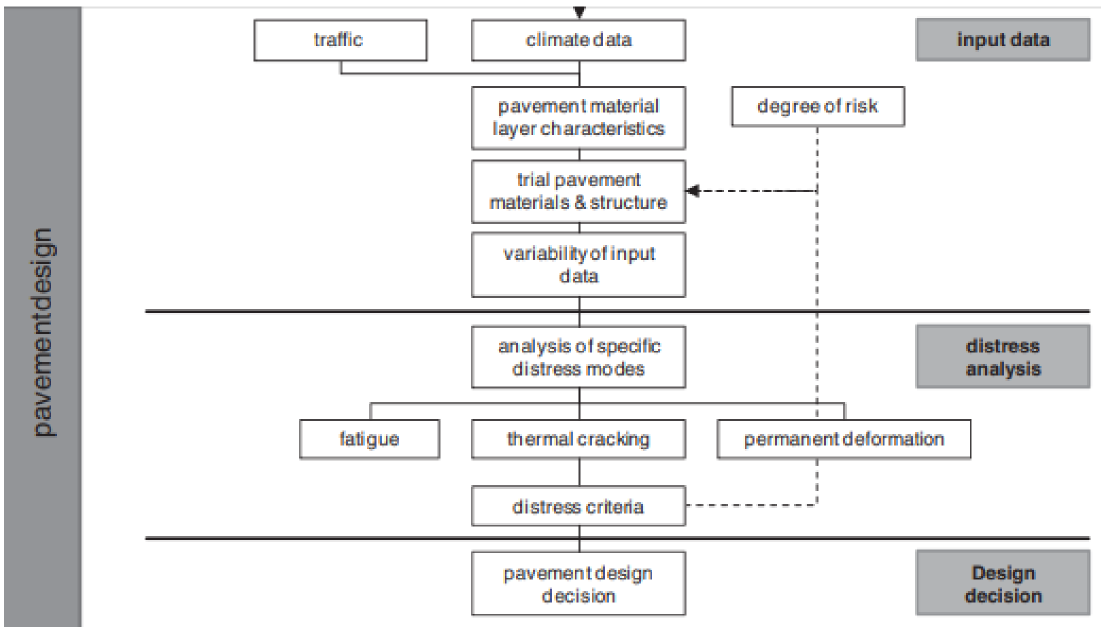



**Figure 0.1 Mechanistic Pavement Design's Procedure (Wistuba and Walther, 2013)**

By applying linear elastic theory, the approach determines pavement fatigue, thermal cracking, permanent deformation, and other distresses. The outcome could be used by the users to determine whether iterative steps are necessary to satisfy the design criteria (Wistuba and Walther, 2013).

Based on numerous studies, the Strategic Highway Research Program (SHRP) stated in the early 1990s that the assumption of linear elastic for pavement layers holds a number of limitations since the approach cannot handle the nonlinear or inelastic distresses. Meanwhile, mechanistic design relates traffic loads, environmental conditions, and pavement performance by using engineering mechanics theories, but this method does not consider any observed performance of the pavement. In other words, pavement structural response (stress, strain, deflection, rutting, raveling, etc.) is not taken into consideration by the mechanistic design, but yet only the pavement distresses are investigated. Therefore, there is currently no fully mechanical pavement design's approach to match the design criteria where linkages between pavement performance and distress theory are necessary for future development (Christopher, 2007)

### *2.3.3 Mechanistic-Empirical Pavement Design*

All of AASHTO, the National Cooperative Highway Research Program (NCHRP), and the Federal Highway Administration (FHWA) did study the Mechanistic-Empirical (ME) design in 1996 as a combination of mechanistic model and empirical design method to improve the mechanistic pavement design with the consideration of empirical pavement performance (Boone et al., 2013). A semi-empirical approach was applied by relating certain mechanistic characteristics to fatigue and permanent deformations to better integrate the empirical and mechanical design. To create a new pavement design platform, AASHTO released the AASHTO ME Pavement Design Guide (MEPDG) in 2008 along with the software program AASHTOWare Pavement ME Design by incorporating more dynamic environment and material information (AASHTO, 2008; Perraton et al., 2011).

As shown in Figure 2.2, MEPDG requires the user to enter data related to the traffic, environmental conditions, material characteristics, and structural specifications.

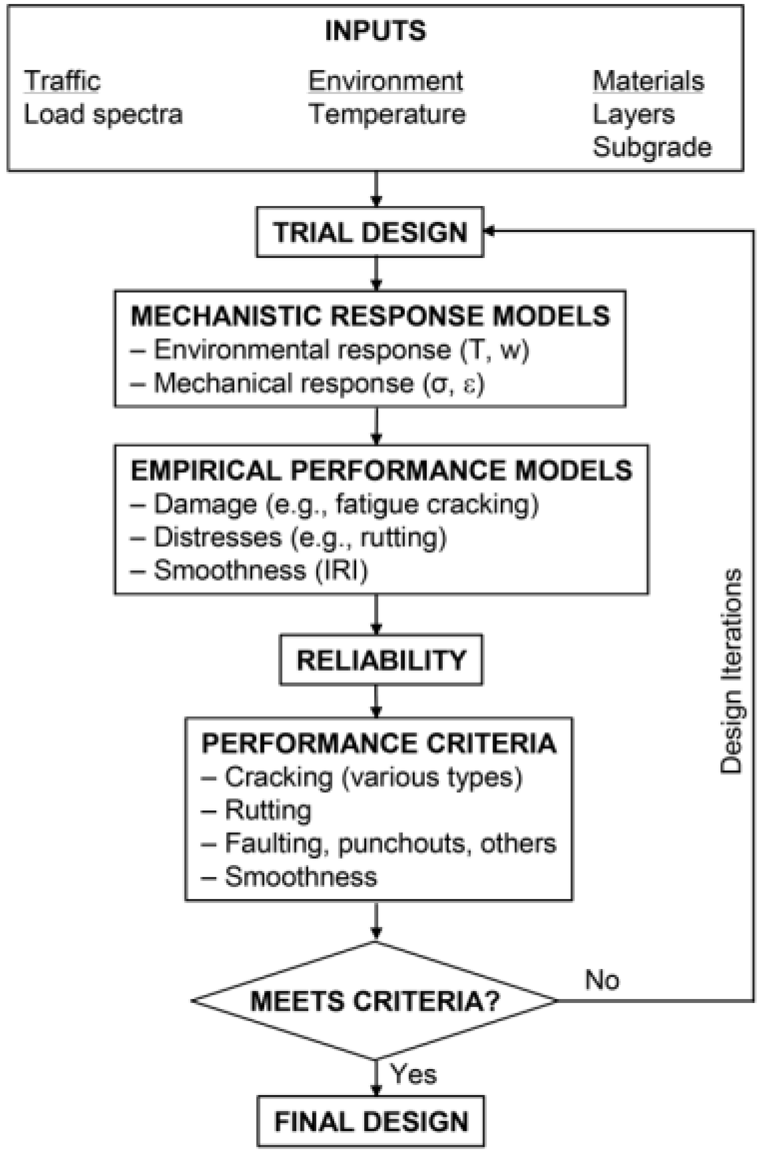



**Figure 0.2 MEPDG Design Procedure (Schwartz et al., 2007)**

ME design concentrates on the effects of axle group loads, tire types and pressures, axle spacing, driving speed and repetitions when it comes to traffic conditions. Moreover, ME design takes into account a number of environmental factors, such as the minimum and maximum temperatures, moisture content, radiation levels, and length of the freeze-thaw cycle. Another essential part of the ME design input is information about the material and the structural data, which include the qualities of the paving materials, the types and thicknesses of the layers, and the types of subgrades (AASHTO 2008; Boone et al., 2013; Haas et al., 2007; Schwartz 2007).

Every component in the input section is involved in the development of a trail pavement; these elements interact with each other to create an iterative design process. The pavement response (surface tensile stress or strain, lateral shear strain or deformation, tensile strain, or stress at the bottom of the AC layer, vertical stress, strain, or deflection at the surface of subgrade) is then calculated at the pavement critical location thru the use of the principles of engineering mechanics (either linear elastic or multi-layer elastic theory). By using the Enhanced Integrated Climate Model (EICM), environmental reactions such as the distribution of heat and moisture throughout the pavement structure are also mechanistically determined (AASHTO 2008; Haas et al., 2007; Schwartz 2007). Following that, the empirical performance models receive the output from these mechanistic models as input. The empirical models were calibrated using a database that stores information collected from hundreds of pavement tests that were conducted across the United States. The empirical model connects reactions from the mechanical model to distresses in the pavement including joint faulting, fatigue cracking, heat cracking, permanent deformation, and roughness. Reliability concepts are used to verify the output from empirical and mechanistic models, determining whether the expected performance would surpass the design standards or not. An iterative procedure is carried out to redefine the design once the trail design does not fulfil the design standards (Boone et al., 2013).

In comparison to the AASHTO 1993 design approach, the pavement layers designed by using MEPDG are around 5% to 10% thinner while maintaining the same level of reliability (Timm 2006). A reduced pavement thickness leads to increased pavement performance and decreased pavement cost (Mashayekhi et al., 2011). The literature indicates that because MEPDG took structural reaction and pavement distress into account when designing the pavement, the pavement met higher performance standards. Pavements developed by MEPDG performed better in terms of resistance to bottom-up fatigue cracking and rutting under the same traffic and environmental conditions and reliability levels. As a result of many studies, AASHTO 1993 designs had overestimated the performance of flexible pavement, and therefore were unable to meet the MEPDG paving criterion (Ahammed et al., 2013; Boone et al., 2013; Carvalho et al., 2006; El-Badawy et al., 2011; Li et al., 2011, Mulandi et al., 2006). Furthermore, only low-traffic and relatively low-temperature conditions are appropriate for the AASHTO 1993 design, as compared to its rationale stemming from experiments conducted in the 1950s. For comparison purposes, the MEPDG is used in Alberta, Canada, together with Alberta Transportation Pavement Design (ATPD). With minor modifications to the AC mix design, structural layer thickness, and design reliability, the ATPD primarily follows AASHTO 93. Even though ATPD offered a high dependability design, pavement did not fulfil MEPDG at high traffic intensities and warm climate. The findings indicate that in high-traffic and warm climate conditions, the AC and unbound layer thickness designed by ATPD are underestimated (Jhuma et al., 2012). Similarly, AASHTO 93 and MEPDG were compared for layer thickness design in Manitoba, Canada. According to the findings, the MEPDG design overestimated the layer thickness for moderate to high truck loads but underestimated it for low traffic volumes (Ahammed et al., 2018).

Although MEPDG-designed pavements perform better than AASHTO-1993 pavements because this method considers the generated damage based on stress and strain variation at various points in the pavement over time, MEPDG has limitations that limit the adoption of ME design across Canada (Perraton et al., 2011). Due to the variations in weather, traffic patterns, paving materials, and construction techniques, MEPDG used pavement across the U.S for calibration. As a result, a substantial amount of efforts will be needed to guarantee that the pavement design inputs is accurately reflecting the conditions in the target location. Moreover, Haas et al. (2007) and Tighe et al. (2007) noted that the environment is changing because of climatic impacts, thus regular updates to the calibration will be required (Boone et al., 2013; Haas et al., 2007; Perraton et al., 2011; Tighe et al., 2007).

In addition, Life Cycle Cost Analysis (LCCA) is necessary throughout the entire design process due to the variation in the value of labor and material prices, which will be helpful in the final decision-making process. Whereas LCCA is not included in the current MEPDG, therefore additional work is required when using this program. Likewise, to implement the program, MEPDG needs inputs with a relatively high level of accuracy (either through extensive laboratory or field measurement), which necessitates the need for more personnel, tools, and training. Consequently, applying MEPDG leads to high cost in both budget and time, which is not adoptable across Canada (Haas et al., 2007; Tighe et al., 2007).

As shown in Figure 2.3, MEPDG can be used to adopt a new empirical method or to maintain and update an existing empirical procedure.

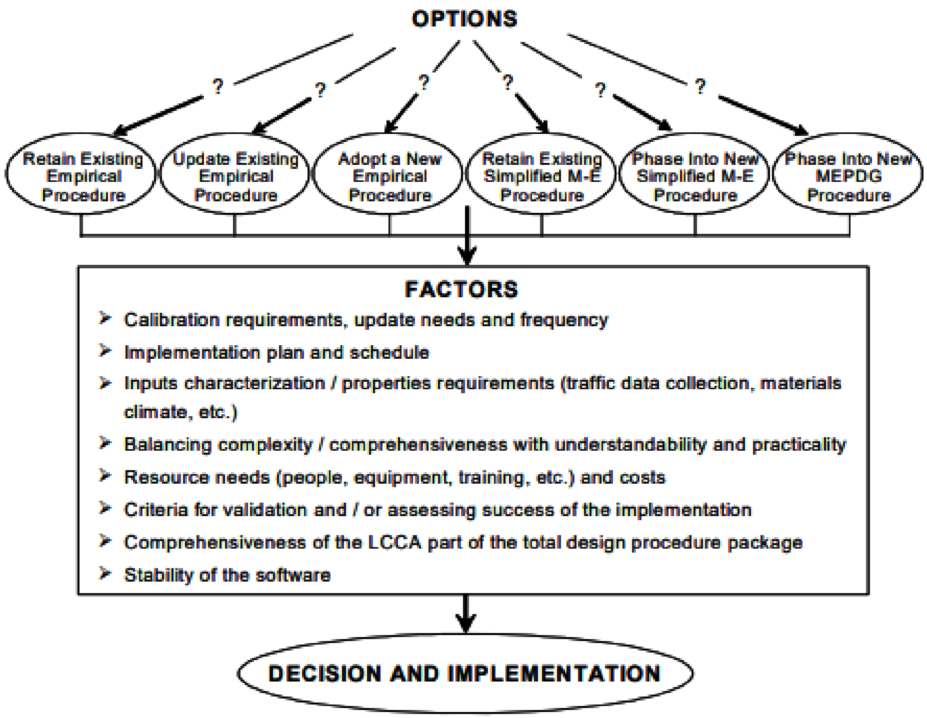



**Figure 0.3 Challenges in Future MEPDG Development (Haas, 2007)**

To fill the MEPDG's gap to the next phase, Haas et al. (2007) suggested that the existing MEPDG can be further developed. According to Haas, the Pavement ME design can be applied to both new and existing pavement construction projects. This can be done by using the current MEPDG as a standard design, which will be used to verify the reliability of any new empirical technique (Aguib et al., 2021; Boone et al., 2013; Haas et al., 2007).

The literature identified another gap in the current MEPDG's simplicity, which is the need for calibration and updates for the existing MEPDG. Both the input complexity and the calibration process can be made simpler by MEPDG's future research and implementation. Not only should the ME techniques themselves be made simpler, but designs that are originated from ME analysis should also be evaluated. In addition, together with the construction planning and scheduling, LCCA can be used in MEPDG’s development to better control design and construction duration and cost and facilitate the decision-making process (Aguib et al., 2021; Boone et al., 2013; Haas et al., 2007; Tighe et al., 2007).

## 2.4 Climate Adaptation Measures

Current studies indicate that temperatures and precipitations are expected to rise in the next few years, and therefore the national and provincial transportation centers are beginning to focus more on adapting and mitigating additional measures (Boyle et al., 2013; FHWA, 2016).

A study by Fletcher (2016) revealed that while temperature alone has little effect on the structure, those effects can increase when the moisture is present. However, that study does not deeply explore the impact of precipitation; it mainly examined the temperature changes brought on by climate change (Fletcher et al., 2016; Swarna et al., 2022a). Moreover, Fletcher et al., (2016) study showed that the future rise in the temperature in Canada will impact the asphalt behavior. Nevertheless, this study only considered the temperature changes caused by climate change, and not the impact of moisture. Barbi et al. in 2021 employed two case studies to explore the impacts of increased ground water table on pavement design, the findings, like Fletcher's study, showed that the temperature by itself has little effect on structure, while it can increase when moisture is present. (Barbi et al., 2021). In 2022, Swarna et al. built on Fletcher and Barbi's findings to do additional research, they found that temperature and precipitation both affect pavement behavior and cause problems like cracking and rutting. As a result, adaptation methods are used to increase the resistance in various pavement layers to account for future climate circumstances (Swarna et al., 2022a).

The objectives at each stage of construction can be successfully met by incorporating climate change into pavement design. However, the best practices for incorporating novel climate stressors that reflect risks associated with climate change have not yet been established. Most of the current research works on adaptation strategies specific to pavements are focusing on integrating the climate change in the development process of transportation projects or in predicting the pavement performance in future climates (Mills et al., 2009; Saleh et al., 2022).

Pavement design incorporated with climate adaptation follows a general framework, as shown in Figure 2.4, which are Top-Down (scenario based) approach and Bottom-Up (asset-based) approach.

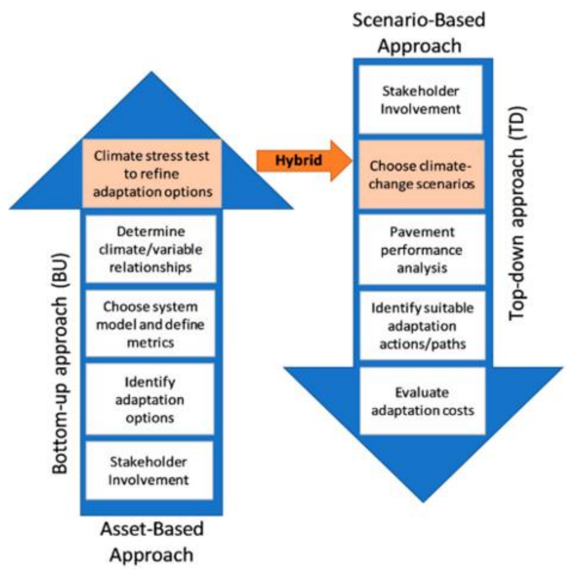



**Figure 0.4 Pavement Adaptation Design Framework (Haslett et al., 2021)**

Based on the evaluations of the sensitivity of pavement and how the performance would be impacted by climate stressors, pavement designs are developed by using a bottom-up method. Whereas the Top-down approaches choose climatic scenarios first and identify how the pavement will function over its lifetime in those situations. The scenario-based method provides information on the key environmental change to guide the design and planning of the M&R, while the asset-based approach gives a fuller picture of the pavement's response to incremental environmental change. Both strategies supply decision-makers with relevant information, and a combination of the two can deliver the data required for flexible adaptation planning that incorporates both design and M&R components (Brown et al., 2018; Jacobs et al., 2018; Taner et al., 2017; Haslett et al., 2021). If many adaptation’s pathways are to be evaluated, LCCA is used to optimize the budget (Knott et al., 2019; Rattanachot et al., 2015). Knott et al. conducted a study in 2019 to examine the pavement lifecycle cost with and without adaptation, the findings indicate that, in the absence of adaptation, climate impacts in 2060 will raise agency costs by 21.7% under normal climate change and by 31.4% under severe climate conditions, while its total costs will be raised by 21% under normal climate change and 30.6% under severe climate conditions. These findings demonstrate that early adaptation through the utilization of specified pavement overlays to avoid the high expense of base-layer restoration has a significant financial benefit. It is critical for pavement management to take distinct climate change conditions into account during both the construction and rehabilitation stages (Knott et al., 2019). To better understand the relationship between climate change and the specific adaptation strategy, comprehensive literature review is performed. Based on the literature review, problems encountered in the current adaptation strategies and suggestions of new adaptation strategies are concluded in Table 2.1.

**Table 0-1 Problems and Improvements in Pavement Design Adaptation Strategy**

| Facing Problem | Adaptation Strategies and Solutions |
|---|---|
| ● Soil resilient modulus used in ME design is affected by shorter frozen periods and decreased soil resistance.<br>● Asphalt modulus is affected by temperature, previous temperature used in AC modulus test is not representative for the future condition.<br>● Except for the provinces of Ontario and Quebec, Pavement ME uses worldwide calibration variables. Resulting in erroneous estimates for all Canadian provinces but Ontario and Quebec.<br>● Current climate estimating model LTPP2.1 underpredicts the lowest annual air temperatures for all considered locations, which affects PG determination.<br>● Climate models applied in current study only consider temperature factor. Whereas precipitation also plays an important role in pavement design.<br>● Climate models used for adaptation planning exists uncertainties to accurately capture climate condition and predict the human behavior in future.<br>● Many studies assess the potential challenges using future climate estimates rather than determining the most effective solution pathway for engineers to pursue to avoid the potential impacts.<br>● Current studies do not consider cost-effectiveness, which is required in latter study. | ● Determine the material stage, compute soil resilient modulus of base and subbase based on regional factor.<br>● Based on climate change prediction, adjust the temperature used in AC modulus test and the following coefficient design.<br>● Change in Superpave binder grade to resist IRI, AC deformation, fatigue cracking in extreme weather conditions.<br>● Updating climate related design value using most appropriate climate data/scenario.<br>● Updating historical data as frequently as possible.<br>● Evaluating benefits and costs of alternative designs.<br>● Applying alternative materials; Upgrading properties of asphalt mixture; Adjusting pavement structural design to adapt future climate; Changing in maintenance |

| | |
|---|---|
| | plain; Triggering earlier maintenance. |

(Barbi et al., 2021; Basit et al., 2022; Fletcher et al., 2016; Hodakova et al., 2022; Qiao et al., 2020; Saleh et al., 2022; Shafiee et al., 2020; Swarna et al., 2022c; Underwood et al., 2021; Zanoni et al., 2019)

Pavement adaptation strategies have been carried out in recent years, which remains flaws in decision process. The current issue in applying adaptation was identified by the literature of current pavement design methodologies to be:

1. The current adaptation strategy is carried out depending on the material and environmental features. Climate change has an impact on the material's qualities including the strength of the subgrade soil and the asphalt modulus, which results in inadequate strategies,

2. The current adaption technique is based on a simulation of the pavement ME design calibrated in the U.S, which has limitations when applied to the Canadian settings,

3. The current adaptation strategy considers climate aspects that have been simulated by several climate models; nevertheless, these models have errors and flaws that have an impact on the accuracy of the adaptation planning,

4. The current studies on adaptation strategies concentrate on assessing the possible pavement behavior in future climatic conditions, but they rarely offer effective solutions, and the cost aspect is rarely considered in these studies.

After reviewing the literature, one can conclude that suggestions and improvements to the current adaptation strategies are to be:

1. Adjust the AC grade and modify the material parameters in the pavement ME design simulation according to the local climate,

2. Consider the effects of the paving materials' rate of deterioration during the pavement

lifecycle and modify the rehabilitation intervals,

3. Simplify the ME design procedures and minimize the impact from the location calibration,

4. Use the most recent climate data and consider the various future climate paths when simulating climate conditions,

5. Incorporate evaluation of benefits and costs to current adaptation strategies while considering alternative materials and structural designs.

The present pavement adaptation strategies hold gaps in the aspects of material properties, structural and climate simulations, and the economic consideration as discussed in the following section.

### *2.4.1 **Climate Model***

The framework to be used in developing adaption strategies states that to simulate the pavement performances under various climate modes, climate models and ME design tools must be used at various time intervals. Consequently, a crucial step in the decision-making process is to consider the climate change condition into account based on climate change models (Swarna et al., 2022a).

Regional climate models, or RCMs are commonly used in conducting climate change simulations. RCMs imitate processes specific to a continent by concentrating on a certain area (e.g., Provinces or cities). Climate models are subject to uncertainty since they rely on a variety of anthropogenic and natural parameter inputs as well as assumptions about the potential future scenarios. To better understand the past, current, and future climate changes resulting from natural or other unforced variability, the Coupled Model Intercomparison Projects (CMIP) and the Intergovernmental Panel on Climate Change (IPCC) release different climate models based on RCMs (Flato et al., 2014; Taylor et al., 2012).

The most recent version is CMIP6, which is a combination of over 100 different climate models, which was released by CMIP to help minimize uncertainty in the modelling process. CMIP6 uses a newer framework called Shared Socioeconomic Pathways (SSPs) to classify future scenarios, which builds upon the Representative Concentration Pathways (RCPs) introduced in CMIP5. The RCPs, including RCP 2.6, RCP 4.5, RCP 6.0, and RCP 8.5, were used in CMIP5 to represent different greenhouse gas (GHG) emission scenarios based on radiative forcing values. The most optimistic set of criteria, RCP 2.6, predicted that the annual global greenhouse gas (GHG) emissions would be at its peak between the year of 2010 and 2020, after which they will begin to drop. Emissions would peak close to the year 2040 in RCP 4.5 and then begin to drop. Whereas for RCP 6.0, the peak will be near the year of 2080 and then will start to fall. RCP 8.5, the most pessimistic scenario, suggested that emissions would continue to rise throughout the century with no peak until after 2100 (Stocker et al., 2013; Swarna et al., 2022a; Shafiee et al., 2020; Taylor et al., 2012; Underwood et al., 2021).

Under CMIP6, Swarna et al. in 2022 used two individual models, the Coupled Global Climate Model 2 (CGCM2A2x) and Hadley Climate Model 3 (HadCM3B21) climate models in conjunction with the Long-Term Pavement Performance (LTPP) data set in their recent study. These two models consider the effects of temperature and precipitation on the climate. To show a climate change model with more accuracy, the CGCM2A2x and HadCM3B21 models consider extra variables such as Degree-Days (DD), Mean Annual Minimum Air Temperatures (MAMAT), and Mean Annual Air Temperature (MAAT). (Swarna et al., 2022a).

### *2.4.2 **Flexible Pavement Adaptation Strategy***

Over 90% of the pavement in North America is flexible pavement, and this type of pavement is sensitive to the surrounding conditions. Therefore, adaptation strategy of flexible pavement to face the climate change is necessary for the next decades. Flexible Pavements are generally composed of surface course, binder course, base course, subbase course, and natural subgrade underneath these layers. As mentioned earlier, climate has serious impacts on the pavement structure that cannot be ignored. Therefore, actions including adjusting asphalt binder grade and asphalt mixture gradation, changing paving material, adopting proper pavement structural design must be taken into consideration against the deterioration at each pavement layer (Basit et al., 2022; Fletcher et al., 2016; Hassim et al., 2005; Qiao et al., 2020; Saleh et al., 2022; Shafiee et al., 2020; Swarna et al., 2022a).

#### *2.4.2.1 Upgrading Asphalt Binder Grade*

According to the literature, the temperature rise conveyed by climate change is the most obvious effect. Higher temperatures make asphalt pavement more prone to loading and more prone to deformation. Mill et al. (2009) estimated the changes in Superpave binder grade for 17 sites in southern Canada to investigate the relationship between future climate and deformation of asphalt pavement. The findings showed that eight of the seventeen sites required a low-temperature grade, and six of the seventeen sites required an upgrade to a high-temperature grade.

Concerns have been raised worldwide on how pavement behavior may be affected by climate change. In 2013, a parametric analysis was carried out in Europe by Wistuba et al., to determine how the pavement mechanistic design should respond to climate change. They concluded that the temperature plays a significant role in the structural design. However, their study did not provide any information or recommendations for which design elements should be modified.

Fletcher et al. (2016) carried out an additional investigation that was based on Mill's findings. For the chosen Superpave asphalt binder grade, Fletcher et al., projected the temperature change in the pavement for the anticipated climate in Canada. Fletcher's study employed a different climate model, based on the findings of Mill et al., study. As a result, nine of the seventeen sites showed an increase in the asphalt binder grade.

Similar research was done in 2017 by Underwood to determine the upgrading requirement for the asphalt binder grade in the United States. The findings demonstrated that out of 799 observed stations, 35% of the climate records from 1985 to 2014 suggested an asphalt binder grade that differed from the binder grade indicated by the climate data from 1965 to 1996. As a result, extensive follow up research was conducted by the FHWA in 2016, which stated that an improvement in the asphalt binder grade is necessary to reduce the fatigue damage and rutting (FHWA, 2016; Underwood et al., 2017).

Many studies have been published over the last three years, which have confirmed the significance of improving the asphalt binder. The effect of climate change on Canada's short-, medium-, and long-term future decisions about performance-grade asphalt cement (PGAC) was examined by Shafiee et al. in 2020 who evaluated sixteen locations and determined that most of them needed a significant PGAC upgrade (Shafiee et al., 2020).

Qiao et al. (2020) did an in-depth review of the literature to demonstrate the effects of high temperatures on the pavement design. Given that flexible pavements are extremely vulnerable to high temperatures and that these effects can develop over the span of their service life, Qiao et al., indicated that high temperatures represent the biggest climate issue. To adapt to the future climates, pavement design must consider the changes in high temperatures. Upgrading asphalt binder to withstand higher temperatures may be useful, but its financial viability must be further examined and justified (Qiao et al., 2020).

To thoroughly confirm the significance of updating the asphalt binder, York University evaluated the effects of climate change on PG throughout Ontario in various time periods in 2022 by using both the LTPP and government-developed models. The outcome indicated that by the middle of the 20th century, a change in PG could be necessary for the majority of southern and northern Ontario, and an average of one grade pump is recommended for the whole province. A study by Basit et al., (2022) claimed that by the year 2100, nearly every place in Ontario may need a grade increase of one to two in both high and low temperature PG. It should be mentioned that towards the end of the century, comparing to the PG 52-40, 52-34, and 58-28, which are currently widely applied in Canada, the most appropriate PG recommended by both models might be 52–34, 58–28, 64–28, and 70–22.

According to a study by Swarna et al.'s (2022), ten of the sixteen Canadian cities would need to upgrade their asphalt binder grade before the year of 2040, thirteen of the sixteen cities would need to upgrade their asphalt binder between the year of 2040 and 2070, and all cities would need to change their asphalt binder grade by the end of the century. Based on the outcome, a change in the binder grade is required in all the Canadian locations apart from Corner Brook, Saguenay, and Quebec City. All other cities require one binder grade increment, except for Saskatoon and Brandon. Brandon and Saskatoon require two raises. By altering the asphalt binder grade, permanent deformation in the AC layer dropped by 10 – 40%, while IRI fell little (< 2%) for nine out of sixteen sections. This resulted in an extended pavement's service life across Canada (Swarna et al., 2022).

#### *2.4.2.2 Adjusting Asphalt Mixture Aggregate Gradation Type*

In flexible pavements, bituminous materials are extensively used to provide a smooth and dense surface, such as Performance-Graded Asphalt Cements (PGACs). To increase the service life of flexible pavements, Superior Performing Asphalt Pavements (SUPERPAVE) was developed. As said earlier, significant PGAC upgrade is required due to climate change (Shafiee et al., 2020). Furthermore, because of the rise in the temperatures and flooding levels, the HMA modulus in the surface layer tends to decrease. Temperature increases that are related to climate change accelerate the aging process of AC, reduce the stiffness of the HMA layer, and cause the surface layer to break down brittlely (Mallick et al., 2014; Mallick et al., 2016; Shao et al., 2017). In addition to raising the asphalt binder grade in response to climate change, another method is used to improve the asphalt's performance, which consists of changing the asphalt mixture's gradation type. In 2001, the Federal Highway Administration (FHWA) declared that the types of HMA pavement mixes include Open-Graded Friction Courses (OGFC), Stone Matrix Asphalt (SMA), Dense Fine Graded Asphalt (DFG), and Dense Coarse Graded Asphalt (DCG). In Canada, specialist mixes (such as SMA and OGFC) can be used to meet specific needs at greater traffic levels, whereas DFG and DCG are advised for both low and moderate traffic levels. On the other hand, OGFC is a kind of porous asphalt that is intended to be water-permeable, setting it apart from SMA and dense-graded mixtures, which are comparatively impermeable. Due to its porous structure and frequent freeze-thaw cycles, OGFC experiences harsh winter conditions (Uzarowski et al., 2008). Therefore, according to FHWA and Uzarowski et al., unless there is a unique necessity, OGFC is not often considered in the Canadian Pavement design (FHWA, 2001; Uzarowski et al., 2008). The well-graded mixtures of DFG and DCG have a uniform distribution of aggregate particles ranging in size from coarse to fine. These two varieties of mixtures are intended to be applied in most of the traffic scenarios and at all pavement levels. Nonetheless, according to the thorough literature review, popular asphalt mixtures are rutting-prone. To optimize rutting resistance and longevity with a stable stone-to-stone interforce, the SMA, a gap-grade HMA, can be utilized (FHWA, 2001).

Applying SMA can reduce irreversible deformation and assist in resisting rutting. SMA uses premium materials to achieve its outstanding performance. The design uses cubical, low-abrasion crushed stone combined with manufactured sands since the stone-on-stone aggregate skeleton provides most of the mixture's strength. However, SMA is a premium mix with a greater quantity of asphalt, and using more durable aggregates means that the initial cost becomes higher. A study by Yu et al., (2000) stated that the initial cost of SMA is higher by 20 to 25% than that of the traditional mix. Nonetheless, according to a study by UPM's Road Safety Research Center in year 2000, the total cost of construction for SMA is roughly 10% to 15% less than that of the conventional mix, with a 1.5-times longer pavement life, and the layer thickness of SMA surfacing is 30% thinner than that of conventional wearing course layer (UPM’s Road Safety Research Center, 2000; Yu et al., 2000).

However, Hassim et al., (2005) compared the construction costs of SMA with conventional asphalt concrete wearing course ACW20 in a study they did in Malaysia. The results of their study showed that, in contrast to Yu et al.'s research, the mean construction cost of SMA is 61% more than that of ACW20. According to their research, the two most important elements influencing the cost of building SMA are the thickness of the surface layer and the cost of the materials (Hassim et al., 2005). Consequently, additional research must be done with the focus on using SMA to pursue economic construction.

A study performed by Yin and West in 2018 revealed that flexible pavement that was built by using SMA had a 16.7% longer service life than the pavement built by using standard HMA (Yin and West, 2018). Also, Yin and West investigated SMA's economic concerns and thus they found that the weighted bid prices for SMA are 9 to 45% higher per ton than dense-grade mixtures after comparing the bidding prices in the Department of Transportation (DOT) in the United States (Yin and West, 2018). In addition, Yin and West used Life-Cycle Cost Analysis (LCCA) to compare the traditional densely graded pavement with the SMA pavement. Their findings recommended that a case-by-case evaluation of the SMA cost-effectiveness is necessary. Variations in the service life and discount rate produce variations in the Net Present Value (NPV) of SMA pavement in various locations and under weather conditions. A key consideration in determining whether SMA is cost-effective or not is the trade-off between the increased initial cost and the longer life expectancy (Yin and West, 2018).

According to the most recent study done by Swarna et al. (2022), SMA can be considered as a climate change adaptation technique. AC rutting predicted for baseline data ($R_b$) and AC rutting predicted for climate change data ($R_{cc}$) are compared based on their study. When $R_{cc}$ exceeds $R_b$ this means that the current design is not suitable for the future climate. Consequently, adaptation steps are implemented, such as upgrading the binder grade and altering the type of mixture gradation. According to Swarna et al., various mixture gradations (SMA) to be considered in cases where the upgraded binder was unable to reduce rutting. Pavement performance (resilience) in Canada to AC cracking, AC rutting, subbase/base rutting, roughness (IRI), and total rutting is predicted to increase by 38.92%, 14.57%, 6.51%, 5.96%, and 8.54%, respectively, within the next 100 years from 2010 to 2100 when considering various adaptation strategies (Swarna et al., 2022b). Also, Swarna (2022c) performed a life cycle assessment (LCA) for pavements with varying adaptations. By including different adaptation techniques, the average LCA results for Canadian pavements adopting different adaptation strategies dropped by 23.2% when compared to not taking any measures under future climate conditions (Swarna 2022c).

#### *2.4.2.3 Changing Paving Material*

Except for upgrading asphalt binder grade and changing asphalt mixture gradation, adopting Reclaimed Asphalt Pavement (RAP) and porous material are considered as adaptation strategy towards sustainable pavement.

The elements that make up RAP are sorted from the wasted asphalt pavements. It typically comprises finely graded, well-quality aggregates. RAP can be utilized in asphalt binder and pavement design to reduce the amount of virgin material used. RAP was used in asphalt mixtures in Canada at a rate of about 25% in 2013. A study by Milad et al. (2019) predicted that by 2021 that percentage will rise to 30%.

The literature showed that the three reasons to use RAP are: i) mechanical, ii) environmental, and iii) economic. A thirty to fifty percent substitution of RAP can result in a thirty percent decrease in the price of virgin material (Milad et al., 2019). RAP can also be used to lower the cost of the asphalt binder, which is the costliest component of asphalt pavement. Huang et al. (2010) stated that to attain the same stiffness and workability as the traditional mixture, up to 40% of RAP can be employed with a bio-modified binder. Consequently, the amount of virgin material used in asphalt pavement can be reduced by up to 30% and cause a drop of 13–14% in the environmental effects of mining and processing, such as burning fossil fuels, (Huang et al., 2010; Vidal et al., 2013). Due to the innovative bio-modified binder and the classification process, the usage of RAP asphalt mixture and binder would result in a higher initial cost but a reduced overall life-cycle cost because of its steady performance. In comparison to the traditional pavement, RAP has superior mechanical qualities. Milad et al. (2019) claimed that a minimum of 30% RAP has the same strength as a standard asphalt pavement. Furthermore, the RAP is better to be used in areas with heavy traffic and freezing temperatures because it is made of high-quality aggregates (Huang et al., 2010; Milad et al., 2019; Vidal et al., 2013). However, there are certain restrictions in utilizing RAP. A particular percentage of RAP can increase the rigidity in an asphalt mixture, while a high percentage of RAP leads to a reduction in its strength (Bennert et al., 2005; Taha et al., 1999). A study by Senior et al., was done in 2008 to compare the performance of RAP with conventional pavement. The results indicated that the overall performance of RAP does not meet 80% of the traditional pavement performance. RAP fails the CBR and permeability tests, as well as the standard guidelines for an engineered road base material (Senior et al., 2008). A similar investigation was carried out in 2016 where the findings indicated that the shear strength and durability decreased when the proportion of RAP in the asphalt mixture is above 50%. As a conclusion, before construction, the asphalt mixture containing RAP should generally be tested to ensure that its strength is equal to or greater than that of the traditional mixture (Moghaddam et al., 2016).

Another environmentally and economically beneficial alternative to regular pavement is the application of permeable or porous material. Lebens et al., (2012) carried out a comparison study between dense graded hot mix asphalt and porous asphalt (PA) pavement. The study showed that several US DOTs have chosen PA pavements for areas with less traffic volume. The conclusion they derived identified the following benefits that may be obtained by using porous material: 1) PA reduces hydroplaning, using PA pavement in rainy areas improves pavement friction, lessens splash and spray, and improves visibility; 2) PA application can lower tire wear and increase vehicle fuel efficiency; 3) Recycled materials can be included in porous pavement (Lebens et al., 2012). A study conducted by Zanoni et al., (2019) on the advantages of PA pavement found that reducing stormwater runoff is the primary goal of using porous materials. Furthermore, PA pavement is accepted by the US Environmental Protection Agency as a suitable method for fulfilling Section 438's standards for lowering stormwater flow (Zanoni et al., 2019).

On another side, the literature shows that permeable pavements may be constrained by geographical variables including climate and local materials. Lindow et al., (2015) stated that porous pavement is not able to support the same weight as standard impermeable pavement. As a result, the thickness of porous pavement frequently must be greater than that of impermeable pavement sections, and the type of load is one of the most significant structural factors that influences material choice. Meanwhile, porous pavements suffered from reduced durability due to water freezing in the pores. Because of this, porous pavement is limited by vertical elements and breaks down more quickly when subjected to loads, particularly angular loads from turning motions (Lindow et al., 2015).

Meanwhile, to guarantee that the porous pavements keep working properly, regular maintenance is needed. The benefits of porous structures are limited or even eliminated if sediments block the pores. Conventional design procedures do not consider the maintenance for porous pavements because roadways are made to accommodate maintenance equipment without excessive effort or risk to the equipment operators. Hence, PA pavement faces difficulties not only during the design phase but also during the maintenance phase (Houle et al., 2010).

Consequently, the use of porous pavements is still holding drawbacks, such as: 1) Less structural resistance for less dense, open-graded PA pavement due to porous structure; 2) Clogging and raveling issues with porous pavements during maintenance; and 3) Higher costs associated with PA because of high asphalt content, higher-quality aggregates, and extensive site preparation. An increased requirements for equipment, location, mixing, and transportation result in an increased life cycle costs (ACI 2010; Hein et al., 2016; Houle et al., 2010; Lebens et al., 2012; Miller et al., 2004; Zanoni et al., 2019).

Additional review of the literature in that perspective resulted that asphalt can be used as a foundation or surface course in porous pavement to lower the total costs. However, the mechanical bearing capacity of this combined pavement is rarely investigated. Prior studies indicated that the mechanical qualities of PA can be maintained with an additional 15% of recycled asphalt aggregates (Frigio et al., 2013; Goh et al., 2012). A recent study done by Xiao et al., (2023) showed that porous pavement performance is like the control group mixed at 30% RAP content. RAP increases, however, can lower performance at high temperatures. Furthermore, according to Frigio et al., (2013) the Canadian technical specifications do not currently recommend the use of recycled materials in porous asphalt surface layers. Therefore, more research studies are needed to determine whether porous pavement can be used in Canada as a climate change adaptation method.

Rubberized asphalt and sulfur-modified asphalt are two more elements that are taken into consideration to lower the virgin contents (FHWA, 2016). But these materials are rarely used in Canada due to many factors such as the total cost, workability, bearing capacity, and social and environmental impact (FHWA, 2016). Therefore, a longer evaluation period is required to verify the application of rubberized and sulfur-modified asphalt in Canada.

#### *2.4.2.4 Adopting Proper Pavement Structural Design*

The adaptation alternatives for flexible pavement that were covered in the literature included updating the asphalt binder, altering the type of asphalt gradation, and using new paving materials. Differential mixture gradation (SMA) was considered in cases when the improved binder was unable to reduce rutting. To ascertain whether the approach was appropriate, the rutting with the modified mixture gradation was evaluated in comparison with the baseline situation. The last option of action was to thicken the asphalt layer if the rutting remained an issue (Swarna et al., 2022b; Uzarowski et al., 2008).

To improve the pavement's shearing and bearing capacity so that it can withstand severe weather and high traffic, the FHWA, in 2016, recommended long-life pavement. According to FHWA, to achieve a greater bending resistance in longer-lasting asphalt pavement, layer thickness or material stiffness is raised through the type of material and design advancements. As mentioned in previous sections, depending on the various regions, surface type and material type can be changed to adapt to the climate. Another possible strategy is adjusting the layer combination of the pavement, where the base type and layer thickness can be modified. An ideal percentage of RAP (if applied) and layer thickness are reached to satisfy the design requirement by an iterative process of designing the pavement thickness using the ME design approach (FHWA, 2016).

Nevertheless, FHWA did not offer any recommendation for changing the thickness in its 2016 study. To get more insight into the effects of variables like temperature-dependent resilient modulus, seasonal average temperature, and variations in season length on pavement thickness adjustment, Knott et al. (2019) proposed a case study under several climate scenarios employing various climate paths. According to the findings, the current base layers are operating properly, with 85% reliability. To mitigate the effects of climate change, the thickness of the surface HMA layer must be increased by 7 to 32%. Additionally, Knott et al. expanded their research in 2020 to calculate the base layer enhancement required because of the rise on the groundwater level. Their study showed that the pavement service life will be shortened by rising temperatures and groundwater levels, especially in coastal areas. To avoid this, thicker base layers and HMA layers might make the pavements be more resilient (Knott et al., 2019, Knott et al., 2020; Saleh et al., 2022; Swarna et al., 2022b).

Studies by Knott et al., done in 2019 and 2020, primarily addressed the effect of climate change on pavement surface course and suggested strategies for HMA layer adaptation. In 2022, Swarna et al., conducted a study drawing on Knott's findings and offered more thorough recommendations for pavement adaptation plans. In agreement with Knott's study, Swarna et al., indicated that the asphalt binder may age more quickly due to rising pavement temperatures. Aging causes the asphalt mixture to become stiffer, especially in thin pavements, which exacerbates bottom-up fatigue cracking. Consequently, increasing the thickness of the AC layer was thought to be an adaptive technique. Furthermore, the findings demonstrate that an increase in this thickness can lessen the tensile strains at the AC layer's base, which in turn lessens the potential of AC BU fatigue cracking (Knott et al., 2019, Knott et al., 2020; Swarna et al., 2022b).

Moreover, Swarna et al., in 2022b, revealed the base and subbase layer adaption, and reported that the average increase in BU fatigue cracking in AC across all tested sections in Canada because of the climate change was 11.46%. Therefore, to enhance the base and subbase rutting, an adaptation technique is needed. The same preliminary procedure is followed by applying the climate model and ME design to assess whether an adaptation plan was required. If base and subbase rutting for data related to climate change exceeded baseline data, the base layers' thickness was increased (Swarna et al., 2022b).

In both studies that have been done by Knott and Swarna, adaptability was accomplished by altering the thickness of the pavement layer, which produced an adaptable and durable design. Raising the base-layer thickness would increase the resistance to the temperature rise, while rising the groundwater level would reduce this benefit, which indicate the necessity of both HMA and base-layer thickness improvements (Knott et al., 2019; Knott et al., 2020; Swarna et al., 2022b).

On the other hand, increasing the thickness is the most expensive and emitting adaptation strategy out of all of those that have been examined. Therefore, this strategy is only required in extreme coastal climate change zones, such as British Columbia and Newfoundland. However, the expense of applying a thicker base course or asphalt is much greater than the initial rise in agency costs associated with adopting an upgraded binder or mixture gradation. For almost all the locations in Canada, updating the asphalt binder grade and asphalt mixture type is the first step towards adapting to climate change, as they are the least expensive, least emitting, and easiest to execute (Knott et al., 2019; Swarna et al., 2022b; Swarna et al., 2022c).

### *2.4.3 **Rigid Pavement***

In Canada, flexible pavement covers more than 90% of the road surface, while rigid pavement is mostly used on high-traffic roads. Traditional rigid pavement is rarely considered for low to moderate traffic roads due to its high initial cost as well as substantial maintenance complexity (Bakash et al., 2013; Memon et al., 2016). According to the literature review, most of the studies on rigid pavement evaluated the effects of climate change on the pavement but hardly offered recommendations for adaptation. Just like flexible pavement, rigid pavement is made up of several layers, such as a base layer, subbase layer, and concrete slab surface layer. Less pressure is applied on the lower structure because the load acting on the concrete slab is spread uniformly, which makes rigid pavement to have a greater bearing capacity if compared to the flexible surface course. For this reason, the subbase course is typically not needed for the rigid pavement (Cement Association of Canada, 2020). Therefore, the adaptation can be applied to base course and surface course designs.

Much like with flexible pavement, recycled materials can be put into the base course and concrete slab to cut down on the amount of virgin materials used. In 1976, Bergren et al., conducted a study in Iowa to examine the behavior of recycled aggregates containing 25% RAP. The findings of their investigation first supported the use of RAP in rigid pavement (Bergren et al., 1977; Fergus 1981). The Austrian Highway Authority carried out a test, similar to Bergren's study, in 1991. In that study, 10% of the RAP was utilized to build new roads, and the Austrian Highway Authority has stated that up to 20% of the RAP may be used in the bottom base course (Al-Oraimi and Hassan, 2009; Sommer et al., 1994). A further investigation was conducted to assess the viability of using recycled materials. According to a report by Bakash et al., (2013) the recycled aggregates obtained from fly ash, cement-stabilized recycled crushed aggregate, building material, and other construction and demolition (C&D) waste can be applied as alternative materials to be added to concrete mixtures (Bakash et al., 2013).

Numerous scholar studies have validated the idea of using C&D material in base courses and slabs of concrete pavement. According to most of those studies, using RAP in place of new aggregates in concrete pavement is not only a viable option in terms of cost and strength, but it also presents the chance of enhancing the pavement's performance due to its increased toughness. Furthermore, concrete pavements of intermediate and low strengths can be built directly by using the RAP (Brand et al., 2013; Hassan and Brooks, 2000; Hossiney, 2010; Huang, 2006; Mathias et al., 2004; Memon et al., 2016; Okafor et al., 2010).

However, the over adding of RAP in pavements may lead to a decrease in the pavement performance. Thus, it is also necessary to consider how much RAP can be applied to the rigid pavement. Prior studies have shown that the replacement of RAP in rigid pavement base course is feasible for up to 40–50%, while more RAP would result in a decrease in the pavement performance (Brand et al., 2013). Furthermore, a study fulfilled by Arulrajah et al., in 2011, demonstrated that recycled materials can be used in the pavement’s construction to some extent. However, the application of recycled materials in construction is limited because there are no performance-based specifications or defined guidelines (Arulrajah et al., 2011). Thus, adding RAP to rigid pavement partially can lower the costs while increasing the performance. However, more research is needed on this method because it does not specify recycled materials.

Other methods for preventing climate change in rigid pavement include adjusting the dowel bar, rearranging dowel position and spacing, and implementing different mix designs. Nevertheless, these tactics are not going to be discussed in this study because there is not enough research or definition to support them (Memon et al., 2016).

## 2.5 Pavement Information Modelling

The application of information modelling can facilitate the integration of several tactics with the construction processes of projects. An organization's approach to gathering, organizing, and preserving asset-related data is called an information management process (IMP) (Oliveira, 2020). One way to ensure the necessary production standards in the building architectural, structural, and MEP (mechanical, electrical, and plumbing) domains in Canada is attained by using Building Information Modelling (BIM). But, the application of BIM to the infrastructure (I-BIM, or Infrastructure-Building Information Modelling) cannot be deemed adequate (Dell et al., 2016; Ding et al., 2017; Liao et al., 2017). Presently, most of the road sector uses computer-aided design, or CAD, technology. Despite that BIM technology is still developing and the changes in the involved process are not all-inclusive, operators have been gradually shifting from the traditional CAD-based approach towards BIM methodologies because of the inaccuracies, design errors, and delays caused by modifications and corrections that are common in the CAD design procedures (Bosurgi et al., 2020; Ding et al., 2017).

### 2.5.1 Developing BIM towards IBIM

Recently, studies have concentrated on evaluating the advantages of employing digital tools and procedures to support road infrastructures and transport facilities throughout their whole life cycle, from strategic planning, design and construction to performance management and maintenance (Oreto et al., 2021). As shown in Figure 2.5, the manufacture of material, pavement design, road building, using phase, preservation and upkeep throughout use, and end of life (EOL) treatments are the six stages of the pavement life cycle, according to FHWA in 2016. Like the findings of Oreto et al., (2021) study, BIM can be used at different phases of the pavement life cycle to measure and control road performance and cost (FHWA, 2016).

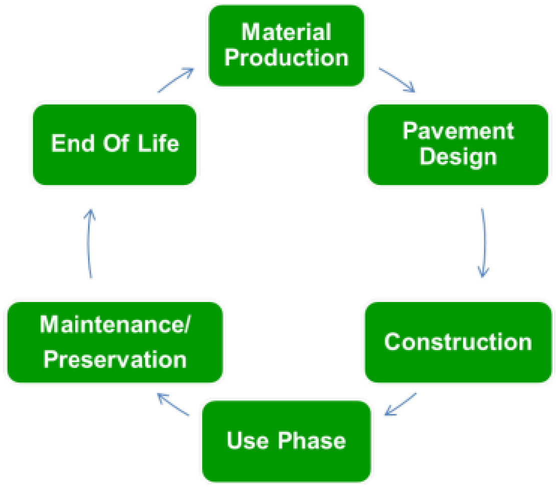



**Figure 0.5. Infrastructure Life Cycle (FHWA, 2016)**

A thorough literature review concluded that the capacity of applying BIM to transport infrastructures at various stages of their service life has significant potential in the future design and construction of those type of projects. However, there are modifications needed to the BIM process in infrastructure domains due to the shortage of objects that necessitate substantial work to create them (Bosurgi et al., 2020; Dell et al., 2016; Fontul et al., 2021; Marttinen et al., 2015; Oreto et al., 2021; Tang et al., 2020).

Autodesk Revit as one of BIM tools is widely used in creating effective models for proposed projects. Autodesk Revit became as one of the most helpful tools for quickly and precisely capturing design intents in three dimensions, as well as for analyzing, simulating, or connecting in the cloud to enhance design quality (Tang et al., 2020). As a matter of fact, procedures that enhance data flow and communication among project participants, as well as the quality of the completed project, can facilitate the digitization of road infrastructure projects (Oliveira et al., 2020). Consequently, to visualize, query, and process survey data, BIM needs to be used in the infrastructure industry for both the overall infrastructure and each individual component. When using BIM concept in road construction, intricate analyses and simulations of the features and interactions of the structure are taken into consideration (Bosurgi et al., 2020; Bosurgi et al., 2022).

To use BIM concept in infrastructure projects, the idea of IBIM has been recently researched. According to Oliveira et al. in 2020, IBIM is seen as a tool that can aid in the early detection of omissions and errors, increase productivity, structure simulation and analysis, and improve communication between the process's actors through more informed participation and data sharing. However, as numerous studies have shown, the infrastructure sector's adoption of IBIM and the problems associated with linear asset management continue is facing major obstacles to future advancements and modifications that would fully realize the advantages indicated (Oliveira et al., 2020).

Oliveira's study has also recognized by Biancardo et al. (2020) and Wang et al. (2021). Recently, IBIM tools have started to proliferate among infrastructure engineers as claimed by Biancardo (2020) and Wang (2021), with the goal of supporting decision-making in road asphalt pavement design and management. These tools enable the efficient archiving, storing, managing, and analysis of large amounts of data generated by various actors and analytical tools involved in the project (Biancardo et al., 2020; Wang et al., 2021).

To improve the decision-making process and expedite projects' efficiency at all stages of their life cycle, a full development of IBIM yields a digital and intelligent representation of data-enriched objects created via effective collaboration between the involved parties. To evaluate the financial and environmental sustainability of a road project considering reaching a more effective decision-making, however, not much work has been put to fully integrating IBIM potentialities with lifecycle-based approaches (Oreto et al., 2023; Safari et al., 2021).

### *2.5.2 IBIM and PIM Benefits*

Scholars proposed an overall PIM process approach, despite that BIM procedures applied to infrastructure facilities have not yet reached a sufficient degree of maturity. According to Oreto et al. (2023) and based on Figure 2.6, PIM can be used throughout the pavement life cycle, including the stages of conception, design, construction, operation and maintenance, and end-of-life management (Oreto et al., 2023).

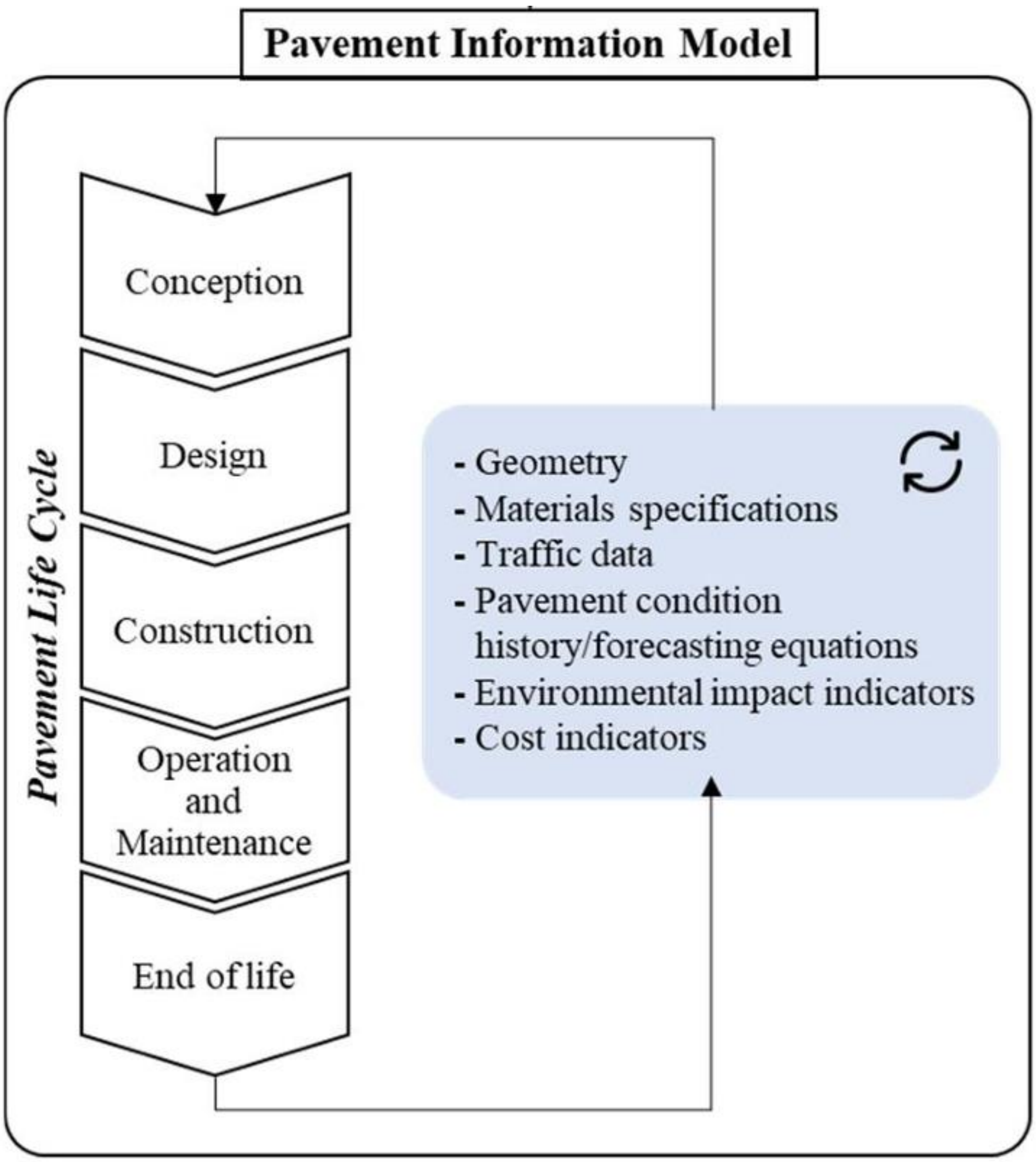



**Figure 0.6. Pavement Information Model Process (Oreto et al., 2023)**

Pavement geometry, paving material specifications, traffic statistics, pavement condition, environmental effect indicators, and cost indicators are among the details that are acquired before the construction and for analysis afterwards (Oreto et al., 2023). More specifically, Figure 2.7 summarizes the facts that should be considered for simulating the pavement as per Oliveira et al., (2020).

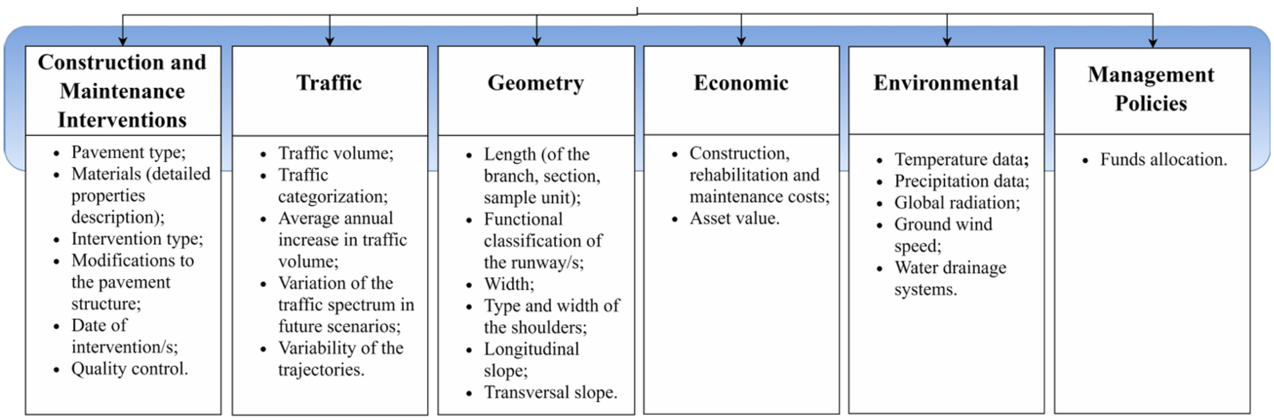



**Figure 0.7. Details in Pavement Information Model (ENAC, 2015; Oliveira et al., 2020)**

The information that needs to be included for the pavement at the different life cycle stages was highlighted by two studies conducted by Oreto et al., (2023) and Oliveira et al., (2020). Oreto et al., (2023) stated that using an IBIM analysis tool to produce a variety of outputs comes after the PIM procedure. To create a safe, interoperable, and visually appealing pavement model to help stakeholders make their decisions based on life cycle cost and road service life, Oreto et al., (2023) stated that three main outcomes must be achieved for pavement projects:

- Sequence of maintenance actions,
- LCCA and LCA of maintenance alternatives, and
- Visualization of results in a BIM environment (Oliveira et al., 2020; Oreto et al., 2023).

A thorough literature review has been conducted concerning the use of PIM and IBIM to realize a visualized 3D pavement model. According to a study by Leone et al., (2017), using IBIM in pavement projects can effectively overcome procedural delays brought on by design errors and provide designers complete control over scheduling and costs (Leone et al., 2017). Sankaran et al., (2018) created a comparable pavement model, in which they claimed that in their assessment, the automation of pavement modelling offers the opportunity to collect and update as-built data for generating a digital information archive to support management and future project development (Sankaran et al, 2018). A study by Tang et al., (2020) supported the design process by implementing an empirical model for the analysis of permanent deformation of the asphalt pavement by using the visual programming software Dynamo for Autodesk Revit. This enables users to choose the pavement that best fits the intended service life.

In a recent study done by Hasan et al., (2022) demonstrated the benefit of using PIM with a visualized model in cost management. The minimum cost strategy, according to their study, involves the use of different asphalt mixtures and aggregate types for the base layer and binder in order to maximize the service life and reduce the maintenance costs by 47% when compared to the baseline reactive strategy with traditional HMA (Hasan et al., 2022).

As a conclusion, based on the afore comprehensive literature review, the use of IBIM in pavement projects would result in significant PIM improvements in relation to the followings:

- Develop a road network model that is safe, interoperable, and capable of storing and analyzing pertinent data to estimate the pavement's service life,
- Put forward an easier-to-use and more optimized decision support model,
- Quick confirmation that the features of the materials match the technical requirements,
- Remarkable material savings when compared to the conventional approach,
- Facilitate the handling, maintenance, and display of a three-dimensional, detailed model of the road infrastructure,
- Decrease mistakes, inaccuracies, and inadequate definitions of the information that is accessible to increase the productivity of road agencies and technicians, and
- Connect a 3D model of the road pavement to a personalized ranking algorithm for the hierarchy of routine maintenance, which only needs small adjustments when used on a new

project.

(Bosurgi et al., 2020; Bosurgi et al., 2022; Marttinen et al., 2015; Oliveira et al., 2020; Oreto et al., 2021; Oreto et al., 2023; Sankaran et al., 2018)

However, PIM is unable to precisely estimate the true pavement service life under the effects of traffic and climate change because it lacks an efficient analysis tool for tracking the performance of the pavement in real time. Therefore, several real-time ME analysis tools can be used for future studies (Tang et al., 2020; Oreto et al., 2021).

### *2.5.3 Application of PIM*

One of the main goals for road agencies is to keep pavements, structures, and other components of the road infrastructure in a safe and effective manner to guarantee the comfort and safety of motorists. To maintain the pavement in the intended structural and functional conditions and to reduce its life cycle costs, road pavement management is a complicated procedure that entails ongoing monitoring and planning of maintenance operations. Road agencies have shifted to "proactive" maintenance rather than "reactive" maintenance since this duty becomes more crucial when the costs associated with maintenance and rehabilitation activities are considered (Bosurgi et al., 2022; Nautiyal and Sharma, 2021; Oreto et al., 2023).

Prior to implementing the ideas of PIM and IBIM, Chen et al., (2012) assessed the efficiency of maintenance operations by measuring the interval between the occurrence of new damage and the treatment of existing damage by using a Geographic Information System (GIS) - based map to forecast the maintenance costs based on the degree of distress found on the road pavement (Chen et al., 2012). Also, based on the findings of Chen et al., (2012) study, Marttinen et al., (2015) applied BIM to the process of road maintenance by integrating the 3D point cloud of uneven road surfaces with the model of the existing road structure in order to optimize the costs of the rehabilitation work and to maintain continuous control over the milling and paving machines throughout the process (Chen et al., 2012; Marttinen et al., 2015). In 2017, Bazlamit et al., conducted a study on the model by creating an integrated model in order to help the government logically plan the maintenance and rehabilitation of the paved network (Bazlamit et al., 2017).

In conclusion, PIM offers the practical advantage of assessing several design or management options with the goal of reducing the consumption of nonrenewable raw materials and fossil fuels in the construction and maintenance of asphalt road pavements. Once PIM is associated with LCCA, a thin overlay that is laid as late as possible is necessary to preserve the road in an acceptable condition. By doing so, the NPV associated with the treatment will be minimized. According to a study by Qiao et al. (2015), depending on climate scenarios, maintenance was triggered by 8 to 16% earlier than baseline. The effects of climate change on pavement maintenance and the agency cost or the whole LCC can be mitigated with well-designed plans for frequent maintenance. This could entail performing some treatment(s) sooner rather than later, which is currently considered as an optimal practice (Oreto et al., 2023; Qiao et al., 2015).

## 2.6 Summary

This chapter provided an overview of the thorough literature review that was fulfilled to identify the most recent studies related to this thesis topic. The idea was to include a pavement information model in the pavement design process, which introduces the concepts of pavement sustainability and resiliency under climate change. First, an overview about how climate change affects the pavement was discussed to identify the issues with the way pavement is currently designed. A review of previous research works done on conceptual design techniques for pavement that discussed the benefits and drawbacks of several pavement design techniques. Subsequently, the definition of the climate simulation module and its significance for pavement design were expounded. After that, adaptation techniques for both flexible and rigid pavements were presented, along with the concepts of resilience and sustainability. The issue at hand and the solutions for both kinds of pavements were listed, discussed, and evaluated. The establishment of the approach and development of the adaptation module in pavement information modelling was largely influenced by literature. The concepts of infrastructure building information modelling and pavement information modelling were presented and identified. A summary of the use and benefit of this concept was also provided. The result of this literature review chapter shows the importance of creating information models for pavements that aids in the construction of the pavement information model linked with life cycle cost analysis.

# Chapter Three

## Methodology

### 3.1 Introduction

This chapter provides detailed information about the methodology that will be followed in order to achieve the set objectives of this study, and yet the development of the integrated model. The model consists of different modules that are interrelated to each other in a dynamic manner to simplify the transmission of data from one to the other. The said modules are: a pavement design selection module (PDS); an updated sustainable and resilient pavement design (SRPD) module for the future climate; and the PIM modeling module, will be created and connected together to form the overall integrated model. Each module will be developed according to a set of guidelines retrieved from the literature, and the steps that will be taken to develop them.

### 3.2 Model Components

Pavement information modeling follows the sequence of conception, design, construction, operation, maintenance, and end of life. At the conception and design stage, geometry, material specifications, traffic data, pavement condition, and cost indicators need to be considered (Oreto et al., 2023). To do the initial pavement design, the model must first be developed by using an Equivalent Single Axle Loads (ESALs) determination module. Based on users' inputs, the module first collects fundamental pavement data. Truck factors, truck distribution, road and traffic group, and traffic information are collected and applied to calculate the ESAL result. Later, the value of the computed ESALs is used to determine the pavement design selection.

To realize the pavement design and integrate resilient and sustainable design elements, the subsequent module will be created and combined with the earlier ESAL module. Overall the model consists of three modules: 1) PDS, 2) SRPD, and 3) PIM. The PDS module consists of a database that holds the pavement structural design guidelines for conventional flexible pavements, jointed plain concrete pavement (JPCP), and continuous reinforced concrete pavement (CRCP) that are collected from the ten Canadian provinces, the outcomes of the ME pavement design is for six Canadian provinces, while the typical pavement thickness designs carried out for the majority of Canadian provinces by the local Ministry of Transportation and Infrastructure, Applied Research Associates (ARA), the Cement Association of Canada (CAC), and Thurber Engineering Ltd. These studies mainly focused on the equivalency of rigid pavements compared to flexible pavements. To provide a recommendation for the conceptual pavement design, the PDS module compares the paving location, traffic data, and subgrade condition with the pavement structural design criteria. PDS provides ME pavement design outcomes for regions that adopted MEPDG for pavement design, while for regions applying other design methods, official proposed designs are considered. If the target region lacks design information, pavement designs collected from recent literatures are used. Strategies for climate adaptation and recommendations for Canadian pavements derived from the literature review section are implemented into the SRPD module. Based on the construction time and location, three adaptation strategies collected from the literature are integrated in the SRPD module, including changing in asphalt binder grade, changing in pavement thickness, and changing in base course type. Adjustments are made to the pavement design generated from the PDS module. The PDS and SRPD modules improve the usability of the interface by allowing users to customize the pavement design within a predetermined range that is based on typical pavement designs in Canada.

The results from PDS and SRPD will serve as the foundation for building the PIM module. The selected paving material and the corresponding lift dimensions are presented in 3D view by the PIM module. Based on the pavement type, dimensions, and construction materials that result from the PDS and SRPD modules, the conceptual cost estimating module generates cost estimates for the designed pavement. Using case projects constructed in North America and cost data collected from multiple sources related to bidding prices, the cost estimating module applies the required time and location adjustment factors along with the linear interpolation method to generate the material cost estimating report. Crew and equipment cost data and their daily output information are collected from R.S. Means construction handbook and database. The scheduling module uses the pavement dimensions, rates of different crews, and daily output, to select the most cost-effective crew and establish a schedule for the pavement project based on the Line of Balance (LOB) method. The selected crew types in the scheduling module and the crew rate are combined to form reports for the labor and equipment cost by the cost estimating module. Finally, using the rehabilitation schedules and costs proposed by the cement associates of Canada (CAC), and with the consideration of inflation rate, the future rehabilitation costs are converted into present cost. The cost estimating module generates rehabilitation cost report based on different ESAL results and road conditions. Figure 3.1 illustrates the components of the model.

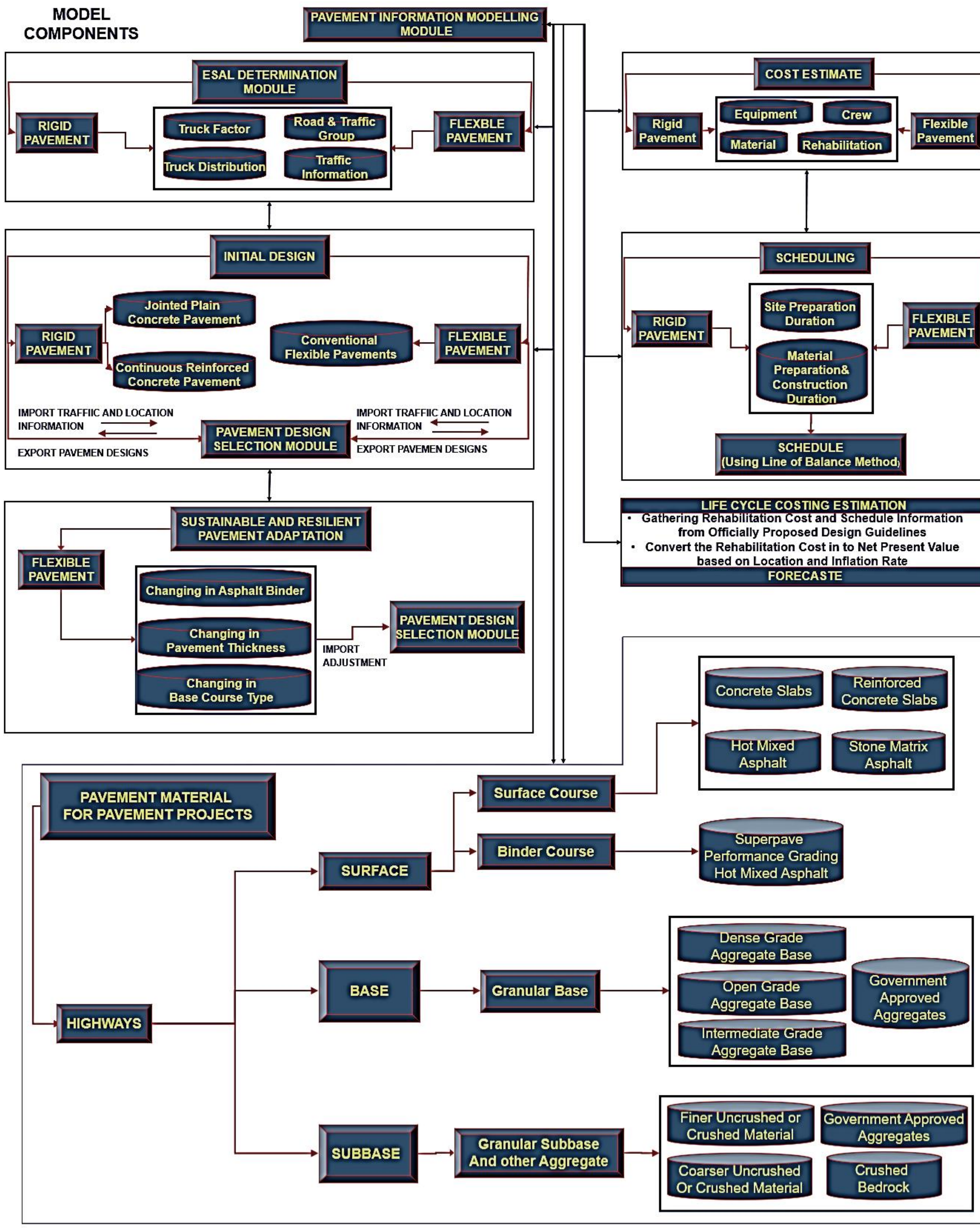



**Figure 0.1. Model's Component**

The pavement design selection process necessitates a methodical approach that adheres to the information management guidelines. This is because of the project's complexity and the necessity to prevent mishaps. As such, the modules will be developed in accordance with a rational flowchart that illustrates the different stages of the conceptual design stage. Figure 3.2 exhibits the model's architecture.

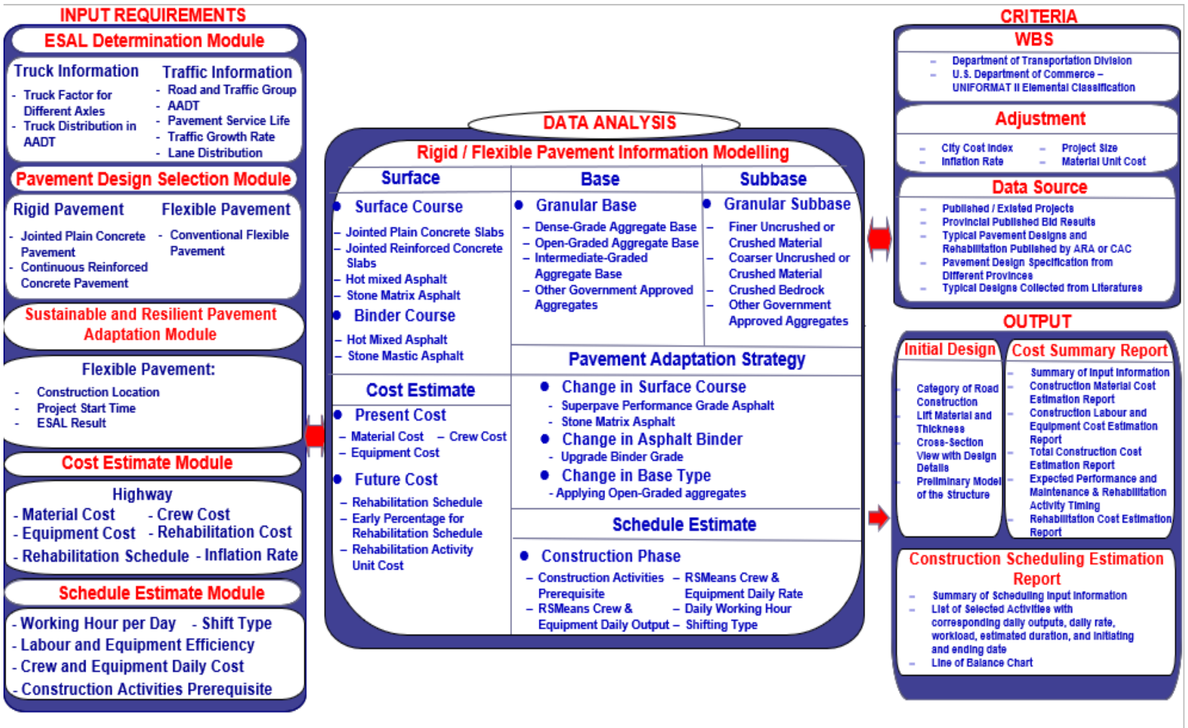



**Figure 0.2. Model's Architecture**

## 3.3 ESALs Determination

One of the factors that impact pavement service life is the traffic acting on it. To design the pavement for bearing the loading during its service life, the total traffic impact on the pavement must be determined. The AASHTO 1993 method first characterized traffic loading in terms of number of Equivalent Single Axle Loads (ESALs). An ESAL represents the damage experienced by a pavement structure because of loading from an 18,000 lb. single axle. All traffic loading from a mixed stream of traffic of different axle loads and axle configurations predicted over the design life of the pavement is converted into an equivalent number of ESALs for design (AASHTO, 1993). With every application of an ESAL, the surface of a pavement structure experiences a quantifiable amount of structural damage. In other words, every application of the ESAL would slightly decrease an amount of the pavement structure's available life. Moreover, the calculated values of ESAL are associated with the subgrade and climate conditions for later PDS module synthesis.

### *3.3.1 Calculating the ESALs*

AASHTO 93 proposes that the traffic loads acting on the pavement can be converted into an equivalent number of 18,000-lb single axle loads by applying Equation - 1 as follow:

$$\sum ESALs = T_f TGL(365)AADT \qquad \textbf{Eq. 1}$$

Where:

$T_f$ represents the Truck Factor (TF). The effect of heavy loads on pavement damage, such as fatigue cracking and rutting of asphaltic concrete pavements is expressed using the concept of axle load equivalency factors (AFs or EALFs). EALFs attempt to capture the combined influence of the entire traffic mix on pavement damage caused by different axle loads and axle groups that are caused by a standard axle load. For convenience, EALFs are related to a standard axle load of 80 kN (18,000 lb) carried by a single axle with dual tires called the equivalent single-axle load (ESAL) (AASHTO, 1993). A Truck Factor represents the summation of all EALFs for a given truck and a TF is equal to the number of ESALs per truck, which can be calculated by applying Equation – 2, as follow:

$$T_f = \sum_i^m \left(\frac{Load\ on\ given\ axles\ (actual\ load)}{Load\ on\ standard\ axles}\right)^{n'} \qquad \textbf{Eq. 2}$$

Where,

i = first axle in truck,

m = m $_{th}$ axle in truck,

Load on given axles = the actual load acting on the wheel axle, Load on standard axles = the standard load acting on the wheel axle for same number of axles, e.g. based on AASHTO, 80 kN for single axle, 148 kN for tandem axles, and 224 kN for tridem axles,

n' = 4, according to AASHTO.

The load on standard axles in North America is normally given by AASHTO, however, the local design may also use different standards for different situation. Meanwhile, the load on given axles may also change from one location to another. To facilitate the design procedure, most of the Canadian provinces provide suggested values for TF according to the local traffic condition as mentioned in the provincial construction and specification guidelines. Moreover, due to the different paving material and structure, recommended TF values are different for rigid and flexible pavements. TF values on rigid pavements are 50% to 82% higher than that on flexible pavements (FHWA, 1995). This current study assumes that the TF values on rigid pavements are 50% higher than the one on the flexible pavements.

In **Equation 1**, "T" represents the percentage of trucks in average daily traffic (ADT). The results of "T" usually depend on the group of the road. The percentage of trucks in average daily traffic for industrial and freeways is higher than that of commercial road and higher than that of residential road, which is also affected by the road classification. For expressway and major arterial, the percentage of trucks in daily traffic is obviously higher than that in minor arterial, collector, and local roads. Therefore, such a variable is easy to change from one location to the other, and it depends on the real traffic data.

The amount of traffic changes proportionally to the increase in population and the increased number of vehicles. In **Equation 1,** a traffic growth rate (G) is used to expand the existing traffic count to estimate the forecasted volume for a future year at specific locations, and is determined by using Equation – 3 as follow:

$$G = \frac{(1+g)^n - 1}{g}$$ **Eq. 3**

Where:

g = annual growth rate,

n = analysis period in years.

In **Equation 1**, "L" is a constant used to determine the ESAL value, where:

"L" represents the lane distribution factor, which indicates the percentage of traffic driving on the target lane, and is used to determine the distribution of the traffic on a specific pavement section.

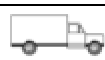
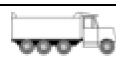
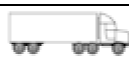
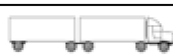
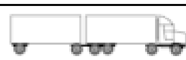
The last factor that impacts the ESALs is the Annual Average Daily Traffic (AADT). AADT is a measured number used primarily in transportation planning, which is the total volume of vehicle traffic on a highway or road per year divided by 365 days.

### *3.3.2 Equation Constants and Assumptions*

The calculation of ESALs contains several variables such as the truck factor, truck percentage, growth factor, traffic distribution factors, and AADT. For major regions in Canada, most of these variables are determined based on the historical local data, except for the truck factor, which is calculated by using two variables that are: i) the load acting on the given axles; and ii) the load acting on standard axles. The loads acting on the given axles change from case to case, whereas the load acting on standard axles are quantified based on AASHTO 93 for Canadian pavement design (AASHTO 1993). The following constants are used for the load acting on standard axles (AASHTO, 1993):

- Loads on Single Axle = 80 kN,
- Loads on Tandem Axles = 148 kN,
- Loads on Tridem or more Axles = 224 kN,

Since the actual load acting on the axles is difficult to determine for each case, the values shown in Figure3.3 will be used for that purpose since it provides a predetermined value of the traffic factor that was developed by Applied Research Associates (ARA) and adopted by Transportation Association of Canada (TAC) for pavement design with various vehicle classes. However, adjustments to the truck factor are made by some Canadian provincial transportation ministries to adapt to local traffic conditions. To simplify the design procedure, the ESAL determination module proposed in this study will first consider the truck factor used in provincial transportation ministries for later calculation. If the provincial guideline does not mention the truck factor, then a default value will be taken from Figure 3.3 based on the different design locations.

| FHWA Vehicle Class | Schema | Truck Factor | | | |
|---|---|---|---|---|---|
| | | Western Canada | | Eastern Canada | |
| | | Typical | Range[a)] | Typical | Range[a)] |
| 4 |  | 1.1 | 0.3 – 2.2 | 1.1 | 0.3 – 2.7 |
| 5 |  | 0.3 | 0.05 – 1.7 | 0.3 | 0.05 – 2.3 |
| 6 |  | 0.8 | 0.07 – 2.3 | 1.1 | 0.07 – 2.7 |
| 7 [b)] |  | n/a | n/a | 4.0 [c)] | 0.2 – 8.0 [c)] |
| 8 [b)] |  | 1.0 | 0.2 – 3.3 | 1.1 | 0.2 – 4.3 |
| 9 |  | 1.3 | 0.3 – 3.4 | 1.6 | 0.3 – 4.2 |
| 10 [b)] |  | 2.3 | 0.4 – 3.3 | 4.2 [c)] | 0.4 – 6.2 [c)] |
| 11 [b)] |  | 1.2 | 0.4 – 4.8 | 1.2 | 0.4 – 6.4 |
| 12 [b)] |  | 1.7 | 0.5 – 4.8 | 2.7 | 0.5 – 6.4 |
| 13 |  | 2.2 | 0.5 – 4.8 | 3.5 | 0.5 – 6.4 |

***Notes:***
*Western Canada: Provinces west of the Ontario-Manitoba border.*
*Eastern Canada: Provinces east of the Ontario-Manitoba border.*
*n/a Not applicable: This vehicle type may not exist in Western Canada.*
*a) The range may not include overloaded axles.*
*b) These types of trucks are relatively infrequent. Truck factors were based mainly on calculations rather than on surveys.*
*c) The configuration may include one or more liftable axles.*

**Figure 0.3. Recommended Truck Factors for FHWA Vehicle Classes (ARA, 2001; TAC, 2007)**

In the ESAL determination procedure, the percentage of trucks in the total AADT (T), is a factor based on historical data of local traffic conditions, and assumptions are made about this variable during the calculation's process.

ESAL represents the loads acting on the pavement structure from the mixed traffic, including motorcycles, passenger cars, buses, two-axle vehicles (except trucks), and different types of trucks. However, many studies conducted in the U.S. indicated that a truck would do 1,000 times as much damage to the road as a passenger car (Alaska Department of Transportation, 2020). Therefore, the developed module will only consider the loads from trucks that lead to most of the damages on the road, while the other traffic components such as automobiles, pickup trucks, and other small vehicles will not be considered in the determination process.

The traffic growth rate (G) follows a linear growth pattern, and the lane distribution factor (L) is also used in the calculation. Although these two variables are subject to change based on the location and traffic condition, historical data shows default values of 3.5% and 50% for G and L, respectively. However, unlike in AASHTO 1993, ESAL is no longer used in the MEPDG; ESAL calculations are now primarily for comparison purposes. The results of ESAL calculation in this study are used to determine pavement thickness through a simplified process.

## 3.4 Pavement Design Selection System Synthesis

The mechanistic-empirical pavement design process looks at both short-term damage, such as changes in stress and strain at different points in the pavement over time, and long-term damage, such as fatigue cracks, rutting, and loss of surface smoothness. As a result, the overall consideration makes MEPDG dominate in the selection of the pavement design methods. Moreover, in 2007, Haas et al., conducted a focused literature review on MEPDG and thus identified several gaps in this design method for future evolution and implementation. The gaps are listed as:

- Simplifying calibration procedure,
- Balancing complexity with understandability and practicality,
- Implementing construction planning and scheduling,
- Identifying the resources needed (e.g., people, equipment, etc.) and costs, and
- Implementing a LCCA for total design procedure.

The traditional procedure for ME pavement design has a complex calibration and iterative process that utilizes traffic load spectra, environment temperature, and materials of each layer as inputs to form a trial design. Then an iterative process is performed to test the mechanistic response and empirical performance of the trail design until the design meets all the performance criteria (AASHTO, 2008; Schwartz et al., 2007). PDS provides a simplified procedure to select the ME pavement design. In the PDS procedure, traditional procedures for the pavement design are obeyed, and gaps are integrated with the existing ME design results that were performed at different locations in Canada through the following five steps:

1. Define the basic pavement information (type, location, soil property),
2. Define the traffic information,
3. Define the material used for pavement design at each lift based on provincial pavement design guideline and literature review,
4. For flexible pavement, develop the proper pavement thickness design based on the existed ME design results, government verified results, and literature proofed results, and
5. For rigid pavement, develop the proper pavement thickness design based on the existed ME design results and provincial design specification; develop a proper joint spacing design and

reinforcement placement design for concrete surface lift based on the ME design results and literature review.

This proposed methodology is based on a general pavement design procedure, pre-existing ME pavement design results, and adaptation strategies covered in the literature review. Since all the information is collected from either official verified design or proofed literature and the traditional procedures followed in the pavement design, it will be adopted as a universal methodology to develop the PDS in Canada. The module provides pavement designs with different specifications so that users can make a selection within a certain range.

### *3.4.1 Definition of Basic Pavement and Traffic Information*

This study focuses on pavements as parts of infrastructure projects, as they vary in scale and are complex to design and execute. Pavement design is comprised of two important components: 1) pavement structure; and 2) paving material, so it is important to identify the factors that define resilience and sustainability in the pavement design. Thus, the components that affect the pavement design are classified as follows:

- Pavement Type
  - Flexible pavement is typically composed of four layers from top to bottom, which are: i) a surface asphalt course (sometimes called a wearing course); ii) a binder course usually made of asphalt concrete or bituminous material; iii) a base course; and iv) a subbase course that comprise different sizes of aggregates. In this study, only conventional flexible pavement is considered.
  - Rigid pavement is typically composed of two layers, which are: i) a Portland cement concrete slab with or without reinforcement, which is usually taken as the surface course; and ii) a base course that comprises aggregates with different sizes. Rigid pavement usually does not have a subbase course due to its high resistance. The

most common types of rigid pavement used in Canada are the JPCP and CRCP and thus they are considered in this study.

- Location
  - The location of the paving projects is classified into cities and provinces. Different locations have different climate information that affects the pavement design. For regions with higher precipitation, the material used for the surface course may change to porous material depending on the traffic loads. While for regions with extreme high or low temperatures, the asphalt grade applied in the binder course is adjusted.
- Soil Properties
  - The type of subgrade soil based on the percentage of coarse and fine particles and the result of the plasticity test can be classified into five groups as follows: i) crushed stone; ii) sandy soils; iii) silty soils; iv) clay soils; and v) organic soils. Each group has a different value of the coefficient of curvature (Cc), uniformity coefficient (Cu), or Atterberg limits, which leads to different bearing capacities. Pavement design is affected by the subgrade condition, for instance for a low-strength subgrade the pavement tends to have a thicker lift, whereas for a high-strength subgrade the pavement usually has a thinner base course.
- Traffic Composition
  - Traffic composition represents the percentage of vehicles with respect to the total number of vehicles. Specifically, because heavy vehicles cause much more damage to the road than small vehicles, a pavement designer is interested in truck traffic composition, which includes the percentage of heavy vehicles in total vehicles.
  - Truck traffic composition affects pavement design as it is adopted for the calculation of ESAL. For roads with a higher percentage of heavy vehicles, the road type is

prone to rigid pavement, as this type of pavement has better performance under extreme high traffic loads. For the same reason, it is preferrable to use coarser aggregates as the base course material, and under heavy traffic loads, the pavement lift thickness becomes greater to provide higher resistance.

- Traffic Loads
  - Traffic loads are brought on by the traffic passing through the road section over a period of time. To quantify the impact of the traffic load on pavement structure, the value of ESAL, which was mentioned in previous sections, is applied. Both pavement material and structural design are affected by the value of ESALs.
  - For pavements with high ESALs, which is used to provide more strength, the paving materials tend to use coarse-graded aggregates rather than fine-graded ones, whereas the thickness of the pavement layer is increased for the same purpose.
  - For low-traffic regions, paving material is likely to adopt finer aggregate to increase the driving experience and pavement service life and therefore, the thickness of the base course and subbase course is adjusted to lower values for economic purposes.
- Road Classification
  - FHWA (2000) classified roadways as: expressway, major arterial, minor arterial, collector, local, and laneway by road function based on the type of service the road provides to the public, which is adopted in North America for the purpose of urban planning and pavement design. Some of the Canadian's pavement design guidelines also refer to this classification. From the perspective of pavement design, the main difference among each type of roadway is the traffic composition. For expressway and major arterials, it is more likely to have a higher percentage of heavy trucks than the other types of roads. As a result, pavement design for each road type is varied due to different traffic loads and the corresponding ESALs.

The classification of roadways is changing based on different aspects. In Canada, another classification of pavement is adopted by some provinces that differs from the FHWA classification by classifying the roads based on their constructed region. By adopting this classification method, roads are first classified into four groups: i) residential roads; ii) commercial roads; iii) industrial roads; and iv) freeways. Then, under each group, roads are categorized based on their function (locals, collectors, and arterials). Since both roadway classifications are applied in different provinces of Canada, they are all adopted in the development of this proposed model. As a result, different pavement design strategies are established for each road group and classification.

### *3.4.2 **Paving Material Categories and Selection***

The selection of material in pavement design largely influences its resiliency and sustainability. Proper material used in the design not only provides resistance to current climate conditions but also to future changes in climate. Pavement is classified into two groups: 1) Flexible Pavement; and 2) Rigid Pavement. Both types of pavements have different structures. Flexible pavements are usually composed of four layers: i) surface course; ii) binder course; iii) base course; and iv) subbase course. While rigid pavements have a simpler structure, usually composed of a concrete surface course and base course. Therefore, the paving materials utilized in Canada for each type of pavement needs to be considered separately, as follows:

- Flexible pavements are usually surfaced with bituminous (sometimes called asphalt) materials. The following materials are commonly considered for the surface course:
  - Hot Mixed Asphalt
    - Dense Fine Graded Asphalt,
    - Dense Coarse Graded Asphalt,
    - Open Graded Friction Course Asphalt (OGFC),

- Stone Matrix Asphalt (SMA), and
- HMA with provincial specified gradation.

As stated by the FHWA in 2001, most of the design strategies approved by the Canadian provincial ministry of transportation suggest HMA as the paving material for surface courses due to its stable behavior.

HMA pavement mix types include fine and coarse-graded dense mixes, SMA, and OGFC, in addition to the two main components in the asphalt mixture, which are the aggregates and asphalt binder. Based on the aggregate nominal maximum size (NMS), each asphalt mixture has the following types: 4.75 mm, 9.5 mm, 12.5 mm, 19.0 mm, 25.0 mm, and 37.5 mm.

For a low traffic level, FHWA recommends only densely graded mixes. For a moderate traffic level, dense-graded mixes are again highly recommended. However, a SMA or OGFC can also be considered, especially when the road approaches a high traffic level. For high traffic levels, all the mix types are appropriate. For the different levels of traffic, the NMS of asphalt mixes can be adjusted to best fit the traffic conditions. Generally, asphalt mixes with a higher value of NMS have a higher resistance due to the larger and tightly packed particles (FHWA, 2001).

The selection of the paving material and its specifications changes from one location to the other. Most of the pavement design manuals issued by the Canadian Ministry of Transportation indicates that the selection of paving material must be based on the local climate and traffic conditions. For Ontario, especially in Toronto, Superpave Asphalt (SP) with various gradation is applied for asphalt surface course. Therefore, different provinces recommended HMA with different gradation for surface course.

Other material, such as porous asphalt, has an unstable mechanical structure. Porous asphalt pavement is limited by vertical loads, and is sensitive to angular loads. As a result, porous asphalt pavement is utilized in areas with low traffic or high surface runoff. However, the literature listed that few provinces have stated a design strategy for porous asphalt pavement. British Columbia, which is abundant with precipitation, the normal HMA is adopted as a surfacing material. Therefore, porous asphalt is not considered in this study.

Emulsified asphalt material is sprayed over to create a seal coat, which is then instantly covered in a thin layer of stone. Seal coat can be utilized as a surface course in areas with low traffic. On the other hand, seal coats in surface paving methods are rarely included in the Canadian design requirements. Therefore, the seal cost is not considered in this study.

On the other hand, the application of asphalt binder in HMA is regulated by the government based on the climate and traffic conditions.  There is a binder course with bitumen-bound aggregate placed between the base layer and the bottom of the surface layer. This course is used to transfer the loads from the surface course to the base course and to provide lateral resistance, and it usually has similar components and properties with the surface course. The following asphalt binder classifications are commonly considered for the binder course:

- Superpave Performance-Graded Asphalt Binder (PG),
- Penetration-Graded Asphalt Binder, and
- Viscosity-Graded Asphalt Binder.

Asphalt binder course can be deemed as a second layer of asphalt that sit under the surface course, and it has the similar composition with the surface course. One way to gauge the performance of asphalt binder is through the Performance Graded (PG) system. It was first created in early 1990 by the Strategic Highway Research Program (SHRP). Most of the Canadian provinces used PG asphalt for pavement surface course construction, except in Saskatchewan and some areas of Alberta.

Two numbers are used to report the Superpave performance grading: a) the lowest pavement design temperature that is expected to be experienced; and b) the average seven-day maximum pavement temperature in Celsius degrees. Thus, the type of asphalt cement used in the binder varies from place to place due to variations in the climate at various construction sites. Most of the Canadian provinces indicated the appropriate PG binder types that is commonly adopted by the local governments, as a result, the PG binder recommended by the governments are taken as binding material in this study.

Certain cities in western Canada have adopted penetration-grade asphalt binder. Developed in the 1900s, the penetration grading system predates the PG system by a significant margin. To confirm the binder’s performance, the penetration grading system measures the ductility, solubility, and penetration depth of asphalt concrete. The penetration grading method employs numbers to reflect binder performance. A high penetration number is used to indicate soft asphalt binders in cold regions, and a low penetration number is used to indicate hard asphalt binders in warm climates. However, due to the adoption of the PG system in most provinces in Canada, the Government of Alberta conducted a study to compare the penetration grade system with the PG system (Government of Alberta, 2012). As a result, most penetration grade commonly used in western Canada has a corresponding equivalent PG asphalt. Therefore, the penetration grade system is substituted with the PG system in this study.

Viscosity-graded asphalt was introduced in the early 1960s as an improved asphalt grading system based on a penetration grade system. In Canada, the viscosity grading system is usually combined with the penetration grading system to indicate the binder performance, but few regions in Canada have adopted this combined grading system for the pavement design. Therefore, viscosity-graded asphalt is not considered in this study.

To provide higher load resistance, conventional concrete pavement is surfacing with a concrete slab to distribute the heavy loads over a relatively wide area and to minimize the pressure on the subgrade. The following surfacing types are commonly considered for the concrete surface course:

- Jointed Plain Concrete Pavement (JPCP),
- Continuously Reinforced Concrete Pavement (CRCP), and
- Jointed Reinforced Concrete Pavement (JRCP).

JPCP is the most common conventional concrete pavement. According to a survey conducted by the American Concrete Pavement Association (ACPA) in 2005, over 80% of the pavement in North America is applying JPCP.

Concrete slabs are built directly on top of the aggregate base construction in JPCP. Subsequently, transverse joints are used to divide the concrete into panel parts and are situated in the expected natural cracking locations. JPCP can be doweled or un-doweled depending on the traffic volume and pavement thickness. When the slab is 200 mm or thicker, smooth steel dowel bars are positioned halfway through the thickness, parallel to the direction of traffic. Moreover, to maintain the pavement lanes together, tie bars (so-called deformed rebar) are positioned at the longitudinal joint perpendicular to the flow of traffic. There is no need for dowel bars in joints in the case of low-traffic areas because aggregate interlock facilitates load transfer. Designers may choose the size of the reinforcement (often ranging from 32mm to 38mm in diameter) according to the actual traffic condition; however, adjustments for the application of dowel bars and tie bars are always possible (ACPA, 2005; Cement Association of Canada, 2020). Hence, with the high adoption of JPCP in Canada, this surface type is considered for concrete pavement design in this study.

Instead of using construction joints, CRCP uses reinforcement steel to closely hold together the anticipated transverse cracks. Because reinforcement is added, CRCP is usually utilized in heavy-duty applications. A study done by the ACPA in 2005 found that 10% of the provinces and cities in North America are using CRCP to build new pavement. Particularly in Alberta, Canada, CRCP is applied to nearly all concrete pavement in Calgary and more than 70% of concrete pavement in Edmonton. In addition, Quebec uses CRCP in the Montreal region because of the crowded highway system. As a result, pavements in Montreal have a low need for maintenance. Therefore, this study choses CRCP as a surfacing option in Canada (ACPA, 2005; Cement Association of Canada, 2020).

JRCP is an outdated pavement design that makes use of dowel bars along the transverse joints as well as a layer of thick wire mesh positioned in the upper third of the concrete panels. However, according to studies conducted by ACPA in 2005, only 6% of North American communities use JRCP when building concrete pavements. Furthermore, according to ACPA, this type of concrete surface is no longer advised for new construction in North America because the other two forms provide a better performance over the long term and are more cost-effective than JRCP (ACPA, 2005; Cement Association of Canada, 2020).

A base layer, sometimes called the structural layer or load-spreading layer, is set under the surface course (concrete surface for rigid pavement, asphalt surface, and binder course for flexible pavement) to distribute the traffic-induced stresses. The materials commonly considered for the base course are Compacted, High-quality Unbound Granular Aggregates.

Based on the literature, the most commonly available aggregates used in the construction of pavement base course in Canada have the similar physical properties with 25mm Well-Graded Base (WGB), 25mm Intermediate-Graded Base (IGB), and 25mm Open-Graded Base (OGB). However, the base course is composed of different sizes of crushed stone, the aggregate gradation adopted by the local government is changing depending on the location, climate, and the traffic load. Some Canadian provinces provided the recommended paving aggregates in their design guidelines. For instance, Granular (A) is recommended in Ontario’s design guideline to be used as base course, and MG 20 is commonly used in paving construction in Quebec. For cities that do not specify the aggregate type or the aggregate information is missing, since 25mm WGB, 25mm IGB, and 25mm OGB have similar material properties to other paving materials used in Canada with similar aggregate gradations, these three types of aggregates are applied for base course construction. Thus, at the conceptual modelling stage, the material of the base course adopted by this study takes WGB, IGB, and OGB as representative gradation types used in Canada, and other aggregates for locations that specified the paving material.

In some cases, if the base course strength is insufficient to resist the traffic loads, a stabilized base course is considered. Pavement base course stabilization requires adding cement or asphalt to the aggregate base course to provide extra strength, which is a cost-effective method to enhance the pavement's performance. However, few design guidelines in Canada specify the design thickness based on the local conditions. Therefore, a stabilized base course is not considered in this current study.

Located on top of the natural subgrade, the lowest layer in pavement design is called a subbase layer. Building a subbase serves to strengthen the pavement's structure, reduce the number of fines that infiltrate into the pavement structure, and to enhance drainage. However, a subbase course is not always required in the pavement design. For concrete pavement design, based on the collected design guidelines all over Canada, few regions consider the subbase course into the design. The materials commonly considered for the subbase course are the Crushed Aggregates. The subbase is the main load-bearing layer of the pavement. Due to the role of subbase in distributing the load uniformly over the subgrade and improving drainage, aggregates of lower quality and larger particle sizes are usually applied. Based on the collected pavement design manual over Canada, with slight variation due to local conditions, fine crushed or uncrushed material and coarser crushed or uncrushed material are commonly adopted for the pavement design in most of the provinces in Canada. For locations with higher drainage demand, crushed bedrock with larger aggregate spacing can be used. However, if local government recommends the paving material for subbase course, for example, Granular (B) is commonly used in subbase paving in Ontario, that type of aggregates is used in this study.

The model proposed in this study covers typical paving materials that are used for conventional pavement. New materials or paving methods are always introduced. However, this study will follow the design methods and specifications provided by different government and published pavement design manuals to ensure its workability. New materials or paving techniques not tested by the official organization to provide a complete design specification will not be considered in the study at this stage. The ultimate selection of paving materials to be used in this study will be based on the design guidelines and typical designs in different Canadian provinces. Since different types of materials can be used in the paving process, the model will propose multiple options for paving materials to users in a user-friendly environment.

### *3.4.3 Selecting the Design Thickness for Flexible Pavement*

Determining the pavement thickness is crucial to pavement design. According to AASHTO MEPDG, climate factors like temperature, precipitation, underground water table, freeze and thaw cycles, wind speed, and traffic loads acting on the pavement affect the design thickness of the pavement. Both climate factors and traffic conditions vary from one location to another; pavement design in British Columbia differs from the design in Ontario. Therefore, the design manual, construction specification, and official verified ME pavement design results at different Canadian provinces and cities are followed to recommend the appropriate pavement thickness based on the local condition.

This study will focus on pavement projects located in ten Canadian provinces. The literature showed that in Canada, AAHTO 93 guidelines are applied in the design procedure of pavement.

Selecting the pavement thickness in this study will follow the methodology provided by Oliverira et al., in 2020 and Oreto et al., in 2023. Performing information management for pavement projects requires the collection of geometry information for the target pavement (Oliveira et al., 2020; Oreto et al., 2023). Therefore, to ensure the pavement resiliency and sustainability, an up-to-date pavement design guidelines or representative pavement designs from each province will be collected. If the province does not provide an updated design strategy, then previous pavement design guides will be followed. If no design guides are found, typical pavement designs in the province provided by either the government documents or official verified research will be collected as a reference for the thickness design. In case if both pavement design guidelines and typical designs are not found, based on the climate condition, similar design thickness used in other provinces with similar climate conditions (temperature and precipitation information) will be applied to the target province. Table 3.1 represents the different methods for the thickness design of flexible pavement in each province.

**Table 0-1 Collected Flexible Pavement Thickness Design Over Canada, Methods, and Sources**

| Province | Design Method | Published Year | | Source | Comments |
|---|---|---|---|---|---|
| AB | MEPDG | 2014 | | CAC | - |
| BC | MEPDG | 2016 | | CAC | - |
| MB | MEPDG | 2013 | | TAC Conference | Typical Thickness Design Provided |
| NB | AASHTO 93 | 2002 | | C-SHRP | Minimum Thickness Provided |
| NL | Empirical Method | 2022 | | Government of NL | Thickness Taken from Typical Cross-Section |
| NS | MEPDG | 2015 | | ARA | - |
| ON | MEPDG | 2015 | | ARA | - |
| PEI | Empirical Method | 2020 | | Government of Charlottetown | Thickness recommended for Charlottetown, PEI |
| QC | MEPDG | 2015 | | ARA | - |

| | | | | | |
|---|---|---|---|---|---|
| SK | AASHTO 93 | 2022 | | Government of Saskatoon | Thickness recommended for Saskatoon, SK |

(ARA: Applied Research Associates; CAC: Cement Association of Canada; C-SHRP: Canadian Strategic Highway Research Program; TAC: Transportation of Canada)

Based on the information collected about the pavement design over Canada, half of the provinces do not provide a well-organized pavement design based on MEPDG, hence the typical thickness collected from the literature review will be applied in this study.

The local government in Manitoba does not offer typical thickness design for the pavement. In 2013, Ahammad et al., conducted research that summarized the typical flexible pavement structure designs under different traffic volumes in Manitoba (Ahammad et al., 2013).

In New Brunswick, typical pavement thickness design is not provided by the local government, and yet little literature has been published to discuss the thickness design. The Canadian Strategic Highway Research Program (C-SHRP) summarized the minimum thickness specification for NB, which will be adopted in this study. However, C-SHRP did not provide a typical thickness design (C-SHRP, 2022). Another study conducted by Hutchinson et al., in 1994 compared the typical flexible pavement structure in NB and ON, however they summarized typical pavement thickness design in NB and provided a range for thickness variation for each lift. Since little literature has discussed the typical thickness of pavement in NB, the results of the study conducted by C-SHRP and Hutchinson et al., will be combined to form the pavement thickness design in this study.

In Newfoundland and Labrador, the government of NL does not take AASHTO 93 or MEPDG into account in pavement design. Instead, the NL government uses a standard road cross-section with modifications based on future values for climate and traffic. Since both the AASHTO method and MEPDG are not conducted in NL, the typical thickness adopted in this study for that area will be taken from the standard road cross-sections under different traffic conditions.

In Prince Edward Island, the low population and traffic density lead to a low demand for pavement construction. The government of PEI only provides the minimum requirement for pavement lift thickness in its PEI pavement construction specification document. For typical pavement thickness information, the Architects Association of Prince Edward Island conducted a geotechnical investigation in the city of Charlottetown in 2020, which proposed a typical pavement thickness design under light and heavy traffic conditions. Since it is the only representative pavement thickness design found in PEI, the proposed model in this study will combine results from both the PEI pavement construction specification and the geotechnical report in Charlottetown to form the pavement thickness selection in PEI.

In Saskatchewan (SK), although the government of Saskatchewan only proposed the pavement construction specification to address the minimum life thickness, the main city, Saskatoon, provided a design and development standards guide for surface infrastructure. Since Saskatoon is the largest city in SK and few design guides were found in other cities, the design guide in Saskatoon is adopted to be used for the pavement thickness design in SK.

According to the construction specification of pavement design in each province, most regions have limitations on the design thickness of pavement. Depending on the subgrade condition and traffic loading, pavements have a minimum thickness for each layer to ensure fundamental resistance. While the cost-effectiveness of pavements is another important factor to be considered, the approximate range of maximum pavement thicknesses is determined by most government design specifications and therefore, both the minimum and maximum pavement thicknesses in each province will be considered in this study.

### *3.4.4 **Design of Rigid Pavement***

Rigid pavement has a different design strategy if compared to flexible pavement. Traditional rigid pavement commonly adopted in Canada is divided into two types: JPCP and CRCP. Both types of pavements are composed of a concrete surface course with reinforcement and a base course to provide enough resistance under traffic. Therefore, both lift thickness and reinforcement arrangement need to be considered in the concrete pavement design. The rigid pavement design adopted in this study will strictly follow the pavement design guide proposed by provincially approved institutes from different provinces. However, if a design guide is not found for the province under consideration, typical designs stated in the relative literature will be adopted, whereas if no literature is found, typical designs from adjacent provinces will be applied.

#### *3.4.4.1 Rigid Pavement Thickness Design Selection*

Governments in different provinces in Canada provide design guides for rigid pavement. In rigid pavement's design, a logic sequence similar to the one used in flexible pavement thickness design is followed. However, since more than 90% of the pavement in Canada is paved with asphalt, rigid pavement has less construction and design demand. In most cases, newly constructed rigid pavement is applied in areas with extremely high traffic loads or in industrial areas. Moreover, rigid pavement in some regions of Canada is constructed either based on previous design manuals or experience-based, where updated pavement design guidelines are rarely applied and followed. Table 3.2 shows the rigid pavement thickness design that the governments of various Canadian provinces have adopted.

**Table 0-2 Collected Rigid Pavement Thickness Design Over Canada, Methods and Sources**

| Province | Design Method | Published Year | Source | Comments |
| --- | --- | --- | --- | --- |
| AB | MEPDG | 2014 | CAC | - |
| BC | MEPDG | 2016 | CAC | - |
| MB | MEPDG | 2013 | MB Infrastructure and Transportation | Typical Thickness Design Provided |
| NB | - | - | - | - |
| NL | - | - | - | - |
| NS | MEPDG | 2015 | ARA | - |
| ON | MEPDG | 2015 | ARA | - |

| | | | | |
|---|---|---|---|---|
| PEI | - | - | - | - |
| QC | MEPDG | 2015 | ARA | - |
| SK | AASHTO 93 | 2023 | City of Regina | Minimum Concrete Thickness recommended for Regina, SK |

(ARA: Applied Research Associates; CAC: Cement Association of Canada)

Rigid pavement has less application in Canada compared to flexible pavement. According to the data shown in Table 3.2, six out of ten provinces have government-approved rigid pavement designs. Based on C-SHRP's results, in SK, Portland Concrete Cement (PCC) pavement is generally not used (C-SHRP, 2002). However, the second largest city in SK, Regina, published a pavement design standard in 2023 that listed the minimum thickness requirement for concrete pavement. Since this is the only design guide found in SK, the minimum thickness requirement is adopted for concrete pavement design in SK.

According to C-SHRP in 2002, PCC pavement is typically not used in four provinces, which are NB, NL, SK, and PEI for rigid pavement design (C-SHRP, 2002). Since there is no typical concrete pavement design or design guidelines found in these four provinces, available pavement design strategies from adjacent provinces will be applied in this study for these four provinces.

For NB and PEI, the design manual from NS is applied, including the lift thickness design for both the concrete slab and the base course under different traffic conditions.

The design manual approved by the QC government is adopted in NL, containing both lift thickness design and reinforcement arrangements for different traffic types.

SK is adjacent to two provinces, AB and MB. However, since the design guides adopted in MB considered less traffic conditions as well as subgrade conditions than the design guides used in AB, this study will apply Alberta's rigid pavement thickness design strategy to Saskatchewan.

#### *3.4.4.2 Jointed Plain Concrete Pavement Reinforcement Design*

Rigid pavement applied in Canada is divided into two types: JPCP and CRCP. These two types of rigid pavements have different structures in the concrete surface course. JPCP is composed of a concrete slab jointed by dowel bars and tie bars, and the arrangement of reinforcements becomes crucial to resist the traffic loads. This study will follow the JPCP design guide proposed by FHWA in 2019, with modifications based on local design guides in different provinces. Based on FHWA, the following factors need to be considered in JPCP reinforcement design: 1) dowel bar diameter; 2) dowel bar length; 3) dowel bar spacing; 4) transverse joint spacing; 5) tie bar diameter; 6) tie bar length; 7) tie bar spacing; and 8) longitudinal joint spacing (FHWA, 2019).

A dowel bar is placed parallel to the traffic direction and at the mid-depth of the slab, provides connections for joints between adjacent concrete slabs. According to TAC (2020) and FHWA (2019), the general rule of thumb is to use dowel bars with a diameter equal to 1/8 $^{th}$ of the concrete slab thickness. Consequently, the dowel bar diameter applied for highways usually ranges from 32 to 38 mm. While a length of 450 mm for the dowel bar is recommended by FHWA for normal traffic conditions. In Canada, depending on the paving location and traffic conditions, the dowel bar can either not be applied in a low-traffic area or be placed with 25 to 32M in sizes and a 300 mm spacing (ARA, 2015; CAC, 2014).

Since cracks are present throughout the concrete slab, cracking is managed by adjusting the transverse joint spacing, which determines where the dowel bars are placed next to each other. Concrete pavements can be joined at a rate lower than the rate at which the concrete would naturally fracture to limit the cracking, according to the ACPA in 2018. To reduce internal pressure and eliminate mid-slab cracking, the pavement is cut into small enough slabs or panels. Transverse joint spacing in North America ranges from 3.5 to 5.5 m, according to Kivi et al. and ACPA, although in Canada, it is often 3.7 m, 4.0 m, 4.3 m, and 4.5 m (ACPA, 2018; Kivi et al., 2020). As a result, this study will use the joint space's range that Kivi's team supplied for joint designs in Canada and makes adjustments in accordance with ACPA's findings.

Placed perpendicular to the direction of traffic at the longitudinal joint, the tie bar, also known as deformed rebar, keeps the pavement lanes together (CAC). AASHTO 93 states that tie bars are intended to hold the faces of adjacent slabs together rather than to transfer weight. AASHTO 93's earlier data collection indicates that tie bars are normally between 0.6 and 1.0 meters long and 12.5 mm in diameter in North America. According to a study conducted by ACPA, the average tie bar diameter is between 12 and 25 mm, the average tie bar length is between 600 and 1000 mm, and the average tie bar spacing is between 450 and 1200 mm. Consequently, to improve the performance of the pavement, the tie bar designs used in this study will utilize the standard design that is frequently used in North America and is supplied by ACPA (AASHTO, 1993; ACPA, 2018).

Longitudinal joint spacing represents the placement of tie bars in a longitudinal direction. The design of longitudinal joint spacing used in this study will be based on the typical joint spacing provided by ACPA in 2018. Depending on the lane width, a 3.5 to 4.5 m longitudinal joint space is generally recommended to be applied for pavements constructed in North America (ACPA, 2018).

The design for dowel bar, tie bar, and joint spacing can be altered depending on agencies within a certain modification range. Table 3.3 presents the collected data for reinforcement design for JPCP in Canada that will be adopted in this study.

**Table 0-3 Collected JPCP Reinforcement Design Over Canada, Designs, Ranges and Sources**

| Reinforcement Design | Typical Design | Typical Design Range | Source | Comments |
|---|---|---|---|---|
| Dowel Bar Diameter | 25 – 32 mm | 16 mm – 1/8 Slab thickness | ACPA, CAC | Round in shape |
| Dowel Bar Length | 450 mm | 457 mm or 18" | ACPA, FHWA | Subject to adjust based on DOT decision |

| | | | | |
|---|---|---|---|---|
| Dowel Bar Spacing | 300 mm | 12" | ARA, CAC | Subject to adjust based on DOT decision |
| Transverse Joint Spacing | 3.7 – 4.5 m | 3.5 – 5.5 m | ACPA, TAC | Ratio of transverse to longitudinal spacing less than 1.5 |
| Tie Bar Diameter | 12.5" | 12 – 25 mm | AASHTO 93, ACPA | Round in shape |
| Tie Bar Length | 600 mm | 600 – 1000 mm | AASHTO 93, ACPA | Subject to adjust based on DOT decision |
| Tie Bar Spacing | 750 mm | 450 – 1200 mm | AASHTO 93, ACPA | Subject to adjust based on DOT decision |
| Longitudinal Joint Spacing | 3.6 m | 3.5 – 4.5 m | ACPA | Ratio of transverse to longitudinal spacing less than 1.5 |

(ACPA: American Concrete Pavement Association; CAC: Cement Association of Canada; FHWA: Federal Highway Administration; TAC: Transportation Association Canada)

#### *3.4.4.3 Continuous Reinforcement Concrete Pavement Reinforcement Design*

Another kind of rigid pavement that is frequently employed for heavy-duty applications in the eastern regions of Canada is called CRCP. In contrast to JPCP, CRCP uses reinforcement steel to tightly hold the anticipated transverse cracks together rather than construction joints and dowel bars. The quantity of reinforcement employed in CRCP is roughly 0.6 to 0.7% by cross-section area in a longitudinal orientation, according to AASHTO 93 and CAC (AASHTO, 1993; CAC, 2020).

Like the design of JPCP, the next step in CRCP design is to consider the arrangement of reinforcement in the concrete layer. This study will adopt the following reinforcement design elements in accordance with the design guide that both CAC and FHWA provided: 1) cover depth; 2) steel percentage; 3) bar size; and 4) bar spacing.

Reinforcements in CRCP are embedded in the concrete slab; the depth of reinforcement is crucial in the pavement design to prevent premature failure. The climate and traffic impact have an influence on the vertical position of the reinforcement, according to the CRCP design guide that FHWA proposed in 2016.

Drying shrinkage and temperature fluctuations are typically more pronounced at the pavement surface, which leads to wider cracks at this location. Therefore, by positioning the reinforcement closer to the surface, narrower crack widths and higher load transfer efficiency can be achieved. However, according to FHWA, keeping the reinforcement closer to the surface increases the likelihood of exposure to chlorides from deicing salt, which causes increased corrosion. Thus, it is common to position the reinforcement between one-third (1/3) and one-half (1/2) of the slab thickness measured from the pavement top surface. Effective cover depth measures the distance from the pavement surface to the center of the steel bar, which represents the depth of reinforcement in the concrete slab will be considered in this study. Based on FHWA and ACPA, a typical range of 76 to 90mm is used for the vertical position design of steel bars, and this value can be adjusted according to the local ministry of transportation judgement (CAC, 2020; FHWA, 2016).

This study will assume that the vertical arrangement of reinforcement in the reinforced concrete slab consists of only one layer of steel bar. The assumption is based on the consideration of pavement thickness. A thicker CRCP may need a second layer of longitudinal steel to provide enough strength, whereas adding another layer of steel may not satisfy the steel ratio as well as the bar spacing determination. A study conducted an AASHTO Pavement ME Design program to test the reinforcement arrangement in CRCP, resulted in a finding that many CRCP designs would not require two layers of reinforcement. Therefore, only one layer of steel bar is assumed to exist in CRCP in this model (FHWA, 2016).

The ratio of the area of longitudinal steel to the area of concrete across a transverse cross-section is known as the longitudinal steel reinforcement content, or reinforcement ratio. This ratio is commonly stated as a percentage and may be computed using Equation - 4 as follow:

$$p_s = \frac{A_s}{A_c} \qquad \textbf{Eq. 4}$$

Where,

$A_s$ represents the area of longitudinal steel in transverse cross-section,

$A_c$ represents the concrete area of transverse cross-section.

Adding reinforcement to the concrete layer increases the slab's strength. However, excess strength causes the opposite effect on CRCP strength. According to FHWA, a lower amount of steel reinforcement results in widely spaced transverse cracks, large crack widths, and high tensile stresses in the steel. Whereas excess steel reinforcement results in closely spaced cracks and intersecting cracks, which could develop into punchouts under poor supporting conditions. As a result, to produce a desirable crack pattern and keep reinforcement stresses within allowable levels, a range of 0.7 to 0.8% for reinforcement percentage is recommended by FHWA to be used in the United States. In Canada, CAC recommended 0.6 to 0.7% for CRCP reinforcement design, and in cold regions, TAC in 2010 proposed 0.7 to 1% for reinforcement ratio can be reached (CAC, 2020; FHWA, 2016; TAC, 2010).

Bar size is another factor that needs to be considered in the design procedure. Based on FHWA 2016, typical steel bar diameters used in CRCP range from 12.7 mm (13M) to 22.2 mm (22M). This study will assume that the longitudinal reinforcements in CRCP will have the same total length as the road length, and the transverse reinforcements will have the same total length as the road width. The study will also assume the reinforcements in both directions have 10% of the total length as the overlapped length.

Furthermore, the study will consider steel spacing in both longitudinal and transverse directions. The determination of reinforcement in the longitudinal direction will be governed by the selected size of the bars, steel percentage, slab thickness, and cover depth, which will be determined by Equation – 5, as follow:

$$n = \frac{4p_s DW}{\pi\varphi^2} \quad \textbf{Eq. 5}$$

Where (n) is the number of bars in longitudinal direction, ($p_s$) is the longitudinal reinforcement ratio, (D) is the slab thickness, (W) is the slab width (equals to road width in this model), and (φ) is the bar diameter, all units are in mm. The spacing will be calculated by using Equation – 6, as follow:

$$S = \frac{W-2t}{n-1} \quad \textbf{Eq. 6}$$

Where (t) is the concrete cover depth in horizontal direction. The reinforcement spacing determined from Equation - 6 should be considered as the maximum allowable value to maintain the required longitudinal reinforcement percentage. If the spacing needs to be adjusted, it should be done by rounding down to a practical spacing according to the pavement geometry (FHWA, 2016).

Transverse reinforcements in CRCP are used to serve the following purposes: 1) to support the longitudinal steel and ensure proper bar spacing and elevation; and 2) to function as tie bars across longitudinal joints. Based on FHWA, the transverse reinforcements have the same diameter range as the longitudinal reinforcements. The transverse reinforcement used for CRCP is normally spaced at standard increments of 0.6, 0.9, or 1.2 m, which is taken as the typical value for transverse steel spacing in CRCP design in Canada.

Table 3.4 presents the reinforcement design data for CPCP, which is adopted in Canada.

**Table 0-4 Collected CRCP Reinforcement Design, Designs, Ranges and Sources**

| Reinforcement Design | Typical Design | Typical Design Range | Source | Comments |
|---|---|---|---|---|
| Longitudinal Reinforcement Diameter | 13 M | 12.7 – 22.2 mm | FHWA 2016 | - |
| Longitudinal Reinforcement Ratio | 0.7 – 0.8% | 0.6 – 1% | CAC, FHWA 2016, TAC | Higher Reinforcement ratio in Cold Region |
| Concrete Cover Depth | 76 – 90 mm | $\frac{1}{3}$ to $\frac{1}{2}$ of slab thickness | FHWA 2016 | - |
| Longitudinal Reinforcement Spacing (S) | $n = \frac{4p_S DW}{\pi\varphi^2}, S = \frac{W-2t}{n-1}$ | | CAC, FHWA | n = number of bars in longitudinal direction |
| Transverse Reinforcement Diameter | 13 M | 12.7 – 22.2 mm | FHWA 2016 | - |
| Transverse Reinforcement Spacing | 1.2 m | 0.6 – 1.2 m | FHWA 2016 | - |

(CAC: Cement Association of Canada; FHWA: Federal Highway Administration; TAC: Transportation Association Canada)

## 3.5 Sustainable and Resilient Pavement Upgraded Design Synthesis

This study will consider pavement design methods based on historical traffic and climate conditions. However, as population increases, traffic and climate have deeper impacts on pavement performance. Therefore, the proposed Sustainable and Resilient Pavement Upgrade Design (SRPD) module will provide pavement design for future climate.

The SRPD module will be a decision-making tool that will provide adaptation strategies for the pavement design constructed from the proposed PDS module. The technique will be derived from peer-reviewed literature that provides various strategies for pavement in Canada under different climate conditions. The steps to be considered in developing the SRPD module are:

1. Paving location and construction time determination, and
2. Upgrade strategy determination.

However, since more than 90% of the pavements in Canada are flexible, few studies have discussed adaptation strategies for rigid pavement. In the meantime, rigid pavement has better resistance to the changing climate than flexible pavement, which means rigid pavement usually has better performance in future climates and fewer actions can be taken against rigid pavement. Therefore, this proposed module will focus on providing adaptation strategies for flexible pavements.

### *3.5.1 Paving Location and Construction Time Determination*

Pavement design strategies change from one construction location to the other. The literature conducted for pavement design conditions within Canada showed that the current pavement designs based on previous climate and traffic information are facing various problems. Decreasing terminal International Roughness Index (IRI), increasing AC bottom-up (BU) fatigue cracking, total rutting, subbase and subgrade rutting, and AC rutting are commonly found in pavement design in the ten Canadian provinces.

For locations with a future reduced freezing index and freeze-thaw cycles, such as Saskatoon, IRI is more likely to decrease to form a rough surface. In regions with high precipitation, such as BC, the design strategies focus more on mitigating subgrade and subbase rutting. For locations with higher future temperatures, pavement designs are facing more challenges from AC rutting. While for locations with a significant increase in future traffic, bottom-up fatigue cracking becomes critical to the pavement design. At the same time, pavement has different conditions over different time periods. In most cases, rutting and cracking problems deteriorate over time. Therefore, pavement location and construction time will be the first to be determined in this proposed module for later decision-making procedures (Barbi et al., 2021; Basit et al., 2022; Qiao et al., 2020; Shafiee et al., 2020; Saleh et al., 2022; Swarna et al., 2022b). The integrated pavement performance in ten Canadian provinces in the next 100 years is presented in Table 3.5.

**Table 0-5 Collected Pavement Performance in Canada at Three Different Time Period – 2010 to 2040, 2040 to 2070, and 2070 to 2100 (Modified from Swarna et al., 2021)**

| Province | AC Rutting (%) | Subbase and Subgrade Rutting (%) | Total Rutting (%) | AC BU Fatigue Cracking (%) | Terminal IRI (%) |
|---|---|---|---|---|---|
| AB | 6.3/15.2/22 | -4/-5.3/-6.2 | -1.2/-0.3/0.7 | 1/2.3/2.5 | -0.8/-0.17/-0.4 |
| BC | 5.9/17.8/17.8 | 10.6/10.6/12.1 | 9.8/12/13.3 | 10/13/19 | 3.2/4.3/3 |
| MB | 14.4/25.4/25.4 | -3.2/-5.3/-7.9 | -0.11/-0.11/-2.4 | 1.25/1.25/1.25 | 0/0.3/-0.7 |
| NB | 9.3/19.4/16.1 | 0/-2.06/-2.06 | 3.9/6.5/5.2 | 0/0/0 | 0.11/0/-2.3 |
| NL | 8.9/20.7/24.1 | -0.89/-3.1/-5 | 2/3.8/3 | 6/12.8/20.8 | -0.2/-0.7/-1.5 |
| NS | 5.9/11/17.8 | 0/0/-3.82 | 3.3/6.5/8.3 | 5.5/18/17.5 | 3/1.3/1.1 |
| ON | 3.4/19/34.6 | -2.2/-4.56/-4.56 | 0/2.5/6.3 | 5.3/12.5/13 | -1/1.3/1.7 |
| PEI | 24.5/49/73.5 | 0/0/0 | 3.7/7/10.4 | 0/0/0 | 0.1/-0.2/-0.7 |
| QC | 10.4/18.6/36.6 | -1.6/-3.6/-8.3 | 2.2/3.3/5.7 | 5.2/8.8/15.8 | 0.2/0.2/0.4 |
| SK | 6.8/9.3/12.7 | -5.3/-7.9/-10.6 | -1.7/-2.6/-3.7 | 1/8/16 | -4.5/-4.9/-5.1 |

(AC = Asphaltic Concrete, BU = Bottom-UP, IRI = International Roughness Index)

The data presented in Table 3.5 is modified from the study results conducted by Swarna et al. in 2021. According to Swarna, the AASHTOW ME pavement design along with the climate condition of RCP 8.5 was applied to test the pavement behavior in the future (Swarna et al., 2021). Pavements at different locations are facing reductions in performance to varying degrees. For example, the occurrence of AC rutting on pavement in PEI increased by 24.5%, 49%, and 73.5% in 2040, 2070, and 2100, respectively. Whereas, the pavement performance for AC rutting in ON at three different periods of time has increased by 3.4%, 19%, and 34.6%, respectively, which is significantly lower than the occurrence in PEI. Pavement design in PEI should consider additional strategies to mitigate AC rutting than the design in ON (Swarna et al., 2021; Swarna et al., 2022b). On the other hand, pavement performance in the same location is facing different degrees of reduction at different time periods. For pavement in QC, pavements are estimated to have increased AC BU fatigue cracking by 5.2% from 2010 to 2040. Without any adaptation measures, the AC BU fatigue cracking on QC pavements increased by 8.8% from 2040 to 2070 and 15.8% from 2070 to 2100, respectively. The occurrence of AC BU fatigue cracking at the end of the century is three times higher than the occurrence from 2010 to 2040.

Therefore, to improve pavement performance in different regions and at different times, the proposed module will consider both location and time factors to be applied in the pavement design strategy selection process.

### *3.5.2 Upgrade Strategy Determination*

Flexible pavements are deteriorating mainly due to traffic loads, changing temperatures, precipitation, and underground water table changes. The literature listed solutions to mitigate the deterioration that include adjusting binder course, adjusting mix gradation, changing paving materials, and changing pavement structural design.

However, many scholars looked at only one potential adaptation solution to flexible pavement in a certain region. Since pavement aging and deterioration caused by changing climates vary with time and location, the adaptation solutions proposed in the literatures are not representative to be used and adopted in Canada.

Additional research was conducted by Swarna et al., in 2021 and 2022, addressing various adaptation measures to mitigate the impacts on the Canadian flexible pavement from future climates. The results presented by Swarna et al., showed solutions to enhance the pavement performance in ten provinces in Canada at different time intervals. The recommended measures were tested under RCP 8.5 climate conditions through the AASHTOW ME pavement design to prove their feasibility as presented in Figure 3.4 (Swarna et al., 2021; Swarna et al., 2022b).

| City ID | Province | City | Adaptation Strategy | | |
|---|---|---|---|---|---|
| | | | 2010–2040 | 2040–2070 | 2070–2100 |
| BC_00 | BC | Vancouver | BG + BT | BG + MG + BT | BG + BT |
| AB_01 | AB | Calgary | BG | BG | BG |
| AB_02 | AB | Edmonton | BG | BG | BG |
| SK_03 | SK | Saskatoon | BG | BG | BG |
| MB_04 | MB | Brandon | BG | BG | BG |
| MB_05 | MB | Winnipeg | MG | BG + MG | BG |
| ON_06 | ON | Toronto | MG | BG + MG | BG + MG |
| ON_07 | ON | Ottawa | BG | BG | BG + MG |
| QC_08 | QC | Montreal | MG | MG | BG |
| QC_09 | QC | Quebec City | MG | BG | BG + MG |
| QC_10 | QC | Saguenay | MG | MG | BG + MG |
| NB_11 | NB | Fredericton | BG | BG | BG |
| PEI_12 | PEI | Charlottetown | BG + MG | BG + MG | BG + MG + CT |
| NS_13 | NS | Halifax | BG | BG | BG |
| NL_14 | NL | Corner Brook | MG | MG | BG + MG |
| NL_15 | NL | St. John's | BG | BG + CT | BG + MG |

Note: AB = Alberta, BC = British Columbia, BG = Change in asphalt binder grade, BT = Change in the base type, CT = Change in asphalt layer thickness, MB = Manitoba, MG = Change in asphalt mixture gradation, NB = New Brunswick, NL = Newfoundland and Labrador, NS = Nova Scotia, ON = Ontario, PEI = Prince Edward Island, QC = Quebec, SK = Saskatchewan.

**Figure 0.4 Appropriate Adaptation Strategies for Canada Provinces (Swarna et al., 2022b)**

Based on Figure 3.4, Swarna et al., in 2022, proposed four methods to improve the pavement performance based on different surrounding conditions, which are: 1) changing the asphalt binder grade; 2) changing the base type; 3) changing the asphalt layer thickness; and 4) changing the asphalt mixture gradation to SMA. Each single adaptation measure was previously mentioned by other scholars to be applied in a specific region, however, the final integration of these methods to be applied in Canada was performed by Swarna et al., in 2022. Since the adaptation strategies provided by Swarna et al., considered both the time and location factors, which are rarely considered in other studies, the results of Swarna's study will be utilized in this proposed module as upgrade design strategies for flexible pavements.

By adopting the adaptation suggestions conducted by Swarna et al., in 2022, the following assumptions and expected limitations will be considered to ensure the proposed model is universal and can be applied all over Canada:

1. The proposed model will only consider design and adaptation strategies in ten provinces in Canada. While the territories will not be included because previous studies rarely discussed pavement designs in these regions.
2. Except for the three territories, the approaches suggested by Swarna et al., in 2022 will be used in the proposed model. By doing so, the proposed model will make the assumptions recommended by the adopted strategies for each capital city in each province and the major cities in that province. The proposed model will make the assumption that cities close to those included in Swarna's results will have similar weather and traffic patterns, which will allow to apply the same adaption techniques.

Since different Representative Concentration Pathways (RCP) would give different adaptation measures, the proposed module will adopt RCP 8.5 as the climate condition in Canada from 2010 to 2100.

#### *3.5.2.1 Changing Binder Grade*

As shown in Figure 3.4, a change in the binder grade was proposed by Swarna for most of the Canadian provinces. As Swarna et al., stated in 2021, climate change increases the air temperature and precipitation, which brings a hotter summer. To improve the performance of the flexible pavement under hotter conditions, many studies discussed the importance of upgrading binder grade. However, previous studies provided upgrading strategies for a few locations in Canada, which is not enough to construct an adaptation strategy to be generally used in Canada. Based on a previous study, Swarna et al., (2021) proposed a recommendation for upgraded binder grade to be used in Canada in the coming 100 years, as illustrated in Figure 3.5.

| Province | City | Base Binder | Upgraded Binder Grade | | |
|---|---|---|---|---|---|
| | | 1980–2010 | 2010–2040 | 2040–2070 | 2070–2100 |
| **BC** | Vancouver | PG 52-16 | PG 58-16 | PG 58-16 | PG 64-10 |
| **AB** | Calgary | PG 52-40 | PG 58-40 | PG 64-34 | PG 64-34 |
| **AB** | Edmonton | PG 52-46 | PG 58-40 | PG 58-40 | PG 58-34 |
| **SK** | Saskatoon | PG 52-52 | PG 58-40 | PG 58-34 | PG 64-34 |
| **MB** | Brandon | PG 52-46 | PG 58-34 | PG 58-34 | PG 58-28 |
| **MB** | Winnipeg | PG 58-40 | PG 58-40 | PG 58-34 | PG 58-34 |
| **ON** | Toronto | PG 58-28 | PG 58-28 | PG 64-28 | PG 64-22 |
| **ON** | Ottawa | PG 58-34 | PG 64-34 | PG 64-34 | PG 64-28 |
| **QC** | Montreal | PG 58-34 | PG 58-34 | PG 58-28 | PG 64-28 |
| **QC** | Quebec City | PG 58-34 | PG 58-34 | PG 58-28 | PG 64-28 |
| **QC** | Saguenay | PG 58-34 | PG 52-34 | PG 58-34 | PG 58-28 |
| **NB** | Fredericton | PG 58-34 | PG 58-34 | PG 58-28 | PG 58-22 |
| **PEI** | Charlottetown | PG 52-34 | PG 52-28 | PG 52-22 | PG 52-22 |
| **NS** | Halifax | PG 52-28 | PG 64-22 | PG 64-22 | PG 64-22 |
| **NL** | Corner Brook | PG 52-28 | PG 52-28 | PG 52-28 | PG 52-22 |
| **NL** | St. John's | PG 52-28 | PG 58-22 | PG 58-22 | PG 58-16 |

Note: AB = Alberta, BC = British Columbia, MB = Manitoba, NB = New Brunswick, NL = Newfoundland and Labrador, NS = Nova Scotia, ON = Ontario, PG =Performance grade, PEI = Prince Edward Island, QC = Quebec, SK = Saskatchewan.

**Figure 0.5 Change in Asphalt Binder Grade in Canada for Future Climate (Swarna et al., 2021)**

The results of the change in asphalt binder grade in Canada were conducted by using RCP 8.5 with AASHTOWare ME pavement design. Since the results covered strategies for most of the provinces in Canada, and time factors are also considered, this strategy will be adopted as an asphalt binder grade adjustment by this proposed module.

#### *3.5.2.2 Changing in Base Type*

Changing the base types in Canada includes applying either cement-treated base (CTB) or adopting open-grade aggregate as the base course. As mentioned by Swarna et al., (2022b), due to high precipitation, BC (Vancouver) becomes the only region that requires a change in the base type to control the subbase and subgrade rutting. However, since CTB is rarely used in Canada, few specifications were found in the pavement design manuals. Meanwhile, due to the differences in the strengthes between CTB and traditional aggregate base, the base course thickness needs to be reconsidered if CTB is applied. Therefore, CTB will not be considered in this proposed module. The government of British Columbia counseled using a base course with larger aggregate sizes in flexible pavement design to provide additional drainage. According to the government of BC, the traditional base course can be replaced by a 25mm open-graded base in circumstances where maximum drainage is required without making a thickness adjustment. Therefore, for the change in the base type in BC, the proposed module will adopt a 25mm open-graded base as an adaptation strategy.

#### *3.5.2.3 Changing in Asphalt Layer Thickness*

Swarna et al., (2022b) stated that to decrease the AC rutting, a change in the thickness of the asphalt layer is recommended for PEI and NL (St. John's). Pavement design in PEI used the Highway Pavement Management Application (HPMA) in the late 1970s. However, because the system was not regularly updated, the pavement design in PEI continued to use the design strategy based on the traffic and weather conditions in the late 1970s. Like in PEI, NL used cross-section design based on historical traffic and weather information. As the population increases, the former design parameters are not satisfied with that condition. Therefore, an increase in the thickness of the asphalt layer is required for both regions.

Reviewing the literature showed that no typical pavement thickness design was found in PEI and NL. Thus, assumptions will be made to these two provinces in this proposed module. To assume a better range for increasing the thickness of asphalt in PEI and NL, the pavement design in capital cities near Charlottetown and St. John's was collected. As Table 3-6 shows, by comparing the population, location, and weather condition, Halifax in NS has the closest population and geographic condition to Charlottetown. Whereas St. John's has a similar population and climate condition to Quebec City. Therefore, the typical asphalt pavement design in NS (Halifax) and Quebec City will be taken as a reference to make assumptions for the pavement design in Charlottetown and St. John's, respectively. As a result, with the consideration of actual traffic conditions, the proposed module will assume the asphalt thicknesses in Charlottetown and St. John's will be increased by 35% to 50%.

**Table 0-6 Comparison of Pavement Design Near to PEI and NL (Environment Canada, 2010; Government of Canada, 2017)**

| Location | PEI | NS (Halifax) | NL | QC |
|---|---|---|---|---|

| | (Charlottetown) | | (St. John's) | (Quebec City) |
|---|---|---|---|---|
| Population (Metro) | 78,858 | 465,703 | 205,955 | 839,311 |
| Metro Density | 70.9/km$^2$ | 64/km$^2$ | 255.9/km$^2$ | 234.8/km$^2$ |
| Temperature ℃ (Average High / Daily Mean / Average Low) | 9.9 / 5.7 / 1.3 | 11.3 / 6.6 / 1.9 | 9.0 / 5.0 / 1.0 | 9.2 / 4.2 / -0.8 |
| Precipitation mm (Average Annual) | 1158.2 | 1388 | 1534.2 | 1189.7 |
| Typical Flexible Pavement Design mm (Asphalt Layer / Binder) | 50 / 75 (Heavy Traffic) | 50 / 150 (Heavy Traffic) | 50 / 60 (Heavy Traffic) | 50 / 170 (Heavy Traffic) |

#### *3.5.2.4 Changing in Asphalt Mixture Gradation*

The last part of the flexible pavement upgrade strategy is changing the asphalt mixture gradation. As stated by FHWA in 2001, recommended general mix types for surface courses include dense graded courses, SMA, and OGFC. Specifically, dense fine graded (DFG) and dense coarse graded (DCG) are usually applied for medium to low traffic and high traffic conditions, respectively. Whereas OGFC is affected by a high freeze-thaw cycle in Canada, which is applied to high traffic conditions for special purposes. The proposed module will adopt another mixture gradation as an adaptation strategy, which is SMA. As stated by FHWA in 2001, SMA is recommended to be used for medium- to high-traffic. Especially in high-traffic conditions, SMA has a better performance in relation to durability, fatigue and rutting resistance, and cracking resistance than the traditional dense-graded mixture.

The application of SMA was also suggested by Swarna et al., in 2022. Based on their study, pavement paved with SMA have a better resistance to AC rutting, and the overall performance can be increased from 5% to 70% depending on the construction location and application time (Swarna et al., 2022b; Swarna et al., 2022c). Therefore, the proposed module will adopt SMA as an adaptation strategy to improve pavement performance.

## 3.6 PIM Modeling

To realize the conceptual pavement design, incorporate resilient and sustainable design elements, and develop an efficient model with Pavement Information Modeling (PIM) capabilities, it is proposed to integrate 3D modeling of the conceptual pavement design with the above listed modules. To include PIM features, Autodesk Revit is chosen due to its flexibility and ease of manipulation. Furthermore, Autodesk products are currently predominantly software used by design engineers in the industry, while Autodesk Revit is the main application that falls under BIM or I-BIM tools. After considering multiple options, Autodesk Revit will be used since it has quick response to C# commands due to its open architecture, therefore entire proposed model will be coded via C# programming language. The integration process will be established through three steps: 1) C# coding to be applied to gather construction information from the PDS and SRPD and store the inputs for later modelling processes; 2) C# coding to be used to construct the 3D model based on the output from the first step; and 3) cost estimation and scheduling analysis will be formed and will export the construction information to an Excel Spreadsheet.

### *3.6.1 Construction Information Integration*

To develop the proposed PIM module, the first step is to gather the construction information that will be selected by the users in the proposed PDS and SRPD modules. The proposed PDS and SRPD modules will provide pavement information as follows:

1. Construction time,
2. Construction location,
3. Traffic classification,
4. Calculated ESALs from the ESAL calculation module,
5. Pavement dimensions for each lift (Width, Length, Lift thickness), and
6. Paving material.

This information will be temporarily stored in C#. Due to the different layer structure between flexible and rigid pavement, input data will be classified into two groups. The pavement design information will be stored under either flexible or rigid pavement classes based on the selected pavement type. Once the user verifies the construction information, Autodesk Revit will read the stored information to produce a 3D model.

### *3.6.2 PIM Modeling Integration*

The PDS and SRPD will process the user input and then will recommend dimensions of the conceptual pavement stored in C#. The integration will be based on the "CreateExtrusionGeometry" (CEG) methodology outlined by Autodesk Revit. The CEG method consists of two steps: i) using the "Curveloop" function to draw the framework; and ii) using "SolidOption" to fulfill the entity.

The CEG method involves "stretching" a certain shape in one direction to form a 3D frame. To ensure the integrity of the 3D frame, a closed shape will be defined through a C# code. The shape used in the CEG method will refer to the cross-sections of the inner structure (e.g., cross-sections of different lifts, reinforcement, etc.). To define the shape, pavement information will be read by Autodesk Revit via a code to get the structure dimensions (lift width, depth, and length). Points will be assigned at the vertex and midpoint on each side of the cross-section. As shown in Figure 3.6, a 3D coordinate axis (X, Y, Z) will be defined first, where X will represent the transverse direction (width), Y will represent the longitudinal direction (length), and Z will represent the vertical direction (depth). Based on the lift width and thickness, the vertexes and mid-points of the cross-section will be defined on the XZ plane.

```
var binderBasePt = new XYZ(0, 0, -surfaceCourse);
var binderLeftTopPt = new XYZ((-width / 2 - offset), 0, -fullThickness);
var binderLeftBottomPt = new XYZ((-width / 2 - offset), 0, -fullThickness - binderCourse);
var binderRightBottomPt = new XYZ((width / 2 + offset), 0, -fullThickness - binderCourse);
var binderRightTopPt = new XYZ((width / 2 + offset), 0, -fullThickness);

var binderCoursePts = new List<XYZ>
{
    binderBasePt,
    binderLeftTopPt,
    binderLeftBottomPt,
    binderRightBottomPt,
    binderRightTopPt,
    binderBasePt,
};

var binderCourseTextPt = binderRightBottomPt.Add(binderRightTopPt).Multiply(0.5);

var binderCourseCurs = this.GetCurveLoops(binderCoursePts);

offset += 50 / 304.8;
fullThickness += binderCourse;
```

**Figure 0.6 Using Curveloop Method to Define Shape Code Snippet**

Then, as illustrated in Figure 3.7, the Curveloop method will be applied to connect these points to form a closed shape. To form a 3D frame, the shape will be stretched by the Curveloop method along the Y axis to the pavement length.

```
matElemId = this.CreateOrGetMaterial(doc, "Binder Course", new Color(255, 255, 159));

var binderSolid = GeometryCreationUtilities.CreateExtrusionGeometry(
   new List<CurveLoop> { binderCourseCurs }, XYZ.BasisY, length,
   new SolidOptions(matElemId, ElementId.InvalidElementId));
```

**Figure 0.7 Using Curveloop and SolidOption Method to form an Element in Revit Code Snippet**

Once the frame is formed, the last step in CEG will be to fill the frame with entities to form an element. As presented in Figure 3.7, a Revit code will read the pavement information stored in C#, the paving material information for the specific lift will be taken by the SolidOption method as the entity property. A full 3D geometry will be made up of all the different parts of the structure working together. This will be achieved by using the CEG method for every layer and any reinforcements that will be built into the pavement.

After the 3D model is constructed, the next step will be to export the pavement and construction information to an Excel file. The pavement information stored in Revit will be ready to be read by the Excel file. Whereas construction information such as rehabilitation plans and selection of labor and equipment during the construction will be stored in C#. Revit coding will read the construction information and will store it in Revit for later a calculation. The cost estimation and scheduling proposed modules will be developed in Revit. The results, along with the pavement and construction information, will be presented in the form of cost estimating and scheduling reports and will be exported to Microsoft Excel for the decision-making process.

## 3.7 Cost Estimation

To supply stakeholders with cost information for their decision-making process, a parametric cost estimation method will be applied in this study, along with executing adjustments for time and location to improve accuracy. Three types of costs will be considered in the proposed model: 1) costs of material; 2) costs for maintenance, and 3) costs of labor and equipment during the construction phase.

### *3.7.1 Cost of Material*

By performing parametric estimation, material unit cost data will be gathered from multiple databases storing material bidding price and project constructed in North America. The unit cost data will be collected at different times and locations for past projects. Therefore, adjustments will be made to the collected unit price data to increase its accuracy and dependency.

Since the purchasing power varies with time, the inflation rate will be used to adjust the past cost records to reflect the current or future costs by using the Equation – 7, as follow:

$$F = P\ (1 + i)^{n} \quad \textbf{Eq. 7}$$

Where,

i = Inflation rate

n = Number of years between known and forecasted year,

F = Forecasted cost of the proposed project (Future Value),

P = Past cost of completed project (Present Value).

The unit price data will be collected from bidding results occurred in previous years, which will be as the P value. (n), the number of time periods between the past bidding year and the proposed construction year. As a result, this method will adjust the cost of materials from past years into the cost at the time the project's construction will be performed.

Since the costs vary from one city to the other, another factor will be considered to reflect the differences between the different locations, which is known as the city cost index that will adjust the costs to be compatible to the current project's location. To adjust the previous project's cost data for the city cost index Equation – 8 will be used, as follow:

$$C_A = C_B \frac{I_A}{I_B} \qquad \textbf{Eq. 8}$$

Where,

$C_A$ = Cost in dollars for city A,

$C_B$ = Cost in dollars for city B,

$I_A$ = Index for city A,

$I_B$ = Index for city B.

In the proposed cost estimation module, $C_B$ represents the collected cost information from city B, and $C_A$ represents the adjusted cost in the city where the construction to be performed. The city indices $I_A$ and $I_B$ will be retrieved from R.S. Means' *Building Construction Costs Data*. The project's time and location factors will both be used to adjust the collected material unit prices.

Most unit cost data can be found from different databases; however, interpolation method will be applied if the material cost is missing. For most cases, the missing cost data is asphalt from top layers. For surface and binder course, these two layers are composed of aggregate mixes and asphalt binder. If asphalt with same gradation but specific binder type is missing, the unit cost will be estimated based on the price of asphalt with adjacent binder grade.

### *3.7.2 Maintenance and Rehabilitation Cost*

Rehabilitation costs will be considered in this proposed module. The method to determine the rehabilitation cost will be based on studies conducted by ARA and CAC. According to ARA and CAC, three factors determine the rehabilitation cost estimation: 1) rehabilitation schedule; 2) rehabilitation quantity; and 3) rehabilitation unit cost. The rehabilitation schedule includes different rehabilitation measures to be taken at different times. Rehabilitation quantity determines the amount of workload to be performed for corresponding rehabilitation measures, and the rehabilitation unit costs will be based on the universal cost collected over Canada (ARA 2015, CAC 2014, CAC 2016).

As stated by ARA and CAC, the rehabilitation schedule is affected by the climate and traffic conditions. Therefore, judgments based on climate and traffic conditions will be applied to determine the rehabilitation schedule. When climatic conditions are more severe, ARA and CAC take road repair measures earlier than anticipated. For pavements under higher traffic, additional repair measures will be taken with higher frequency, and the rehabilitation quantity will be increased to adapt to the intense traffic. Since the rehabilitation measures are going to be taken in the future, the determination of rehabilitation costs will consider time adjustments to calculate the NPV for each repair measure in the calculation process. The proposed module will assume the adopted repair unit prices based on the universal costs across Canada, thus, location adjustment will not be required in this case.

### *3.7.3 Labor and Equipment Cost*

The labor and equipment costs are the costs of the construction resources (manpower and machine) needed for different tasks during the construction stage, including earth stripping, compaction, excavation, construction for pavement lifts, and other activities. The cost will be determined based on 1) the workload required for the project; 2) labor and machine efficiency; and 3) labor and machine cost.

The required workload is directly determined by the scale of the project. Both pavement type and size affect the volume to be treated in each activity. The information on labor and machine efficiency (daily output) and cost will be gathered from the RS Means cost database, which is commonly used in North America. To determine the total labor and equipment costs, Equation – 9 will be used as follow:

$$Total\ Cost = \sum\left(\frac{Workload}{Labour\ \&\ Equipment\ Efficiency} \times labour\ and\ machine\ Daily\ cost\right) \qquad \textbf{Eq. 9}$$

To determine the total cost for each construction activity, the workload will be divided by the daily output of labor and equipment to calculate the duration required for each activity. The calculated duration will be rounded up to one day and be multiplied by the daily cost of the construction group to get the total labor and equipment costs.

## 3.8 Scheduling Analysis

To determine the construction duration, the Line of Balance (LOB) method will be applied. This scheduling method will be based on the project scale, construction group efficiency, and duration of each construction activity.

Before generating the LOB schedule a lead-time chart will be predetermined to optimize the cost and time efficiency. The LOB method shows the project work continuity, which is done repeatedly and shown as a single line on a graph, where the X-axis represents the number of days and the Y-axis represents the progress of construction. In the proposed scheduling module, the construction starting time on the LOB chart (X-axis) will be set based on the construction duration and activity sequence based on the lead-time chart. The construction progress will be presented on the Y-axis, which will indicate the length of pavement in a longitudinal direction that would have been constructed. The unit applied in the Y-axis will be in meters and the unit used in the X-axis will be in days, the slope of the line will have a unit of meter per day (in longitudinal direction). Therefore, the construction group efficiency will be converted into meters per day in the longitudinal direction so that it can be applied in the LOB chart.

With a fixed construction task efficiency, the LOB schedule will be generated by setting the construction start and finish times to satisfy the prerequisites among activities. In the proposed scheduling module, to avoid interruptions and waiting among activities, the line of each construction task should be single and continuous to meet all the prerequisites. If the lines do not meet the prerequisites or a line with multiple sections is formed for one task, the starting or completion time will be adjusted until it satisfies the prerequisite condition.

The proposed cost estimation and scheduling modules will be stored and presented in Autodesk Revit and their generated data will be saved in Microsoft Excel. Autodesk Revit's coding will first read the database from the existing Excel files to get and store the required information in Autodesk Revit. The cost estimating and scheduling calculations will be performed in Revit. As a result, the LOB chart, along with the cost estimating and scheduling reports will be presented in Autodesk Revit, and the necessary results will be exported from Revit to be saved in a new Excel workbook. This will simplify the decision-making process for stakeholders to read and compare the results between different pavement designs based on different specifications. Therefore, the data flow among C#, Revit, and Excel will be developed following an easy, straightforward sequence to establish a user-friendly platform. Figure 3.8 illustrates the data flow logic within the modelling process of the proposed model.

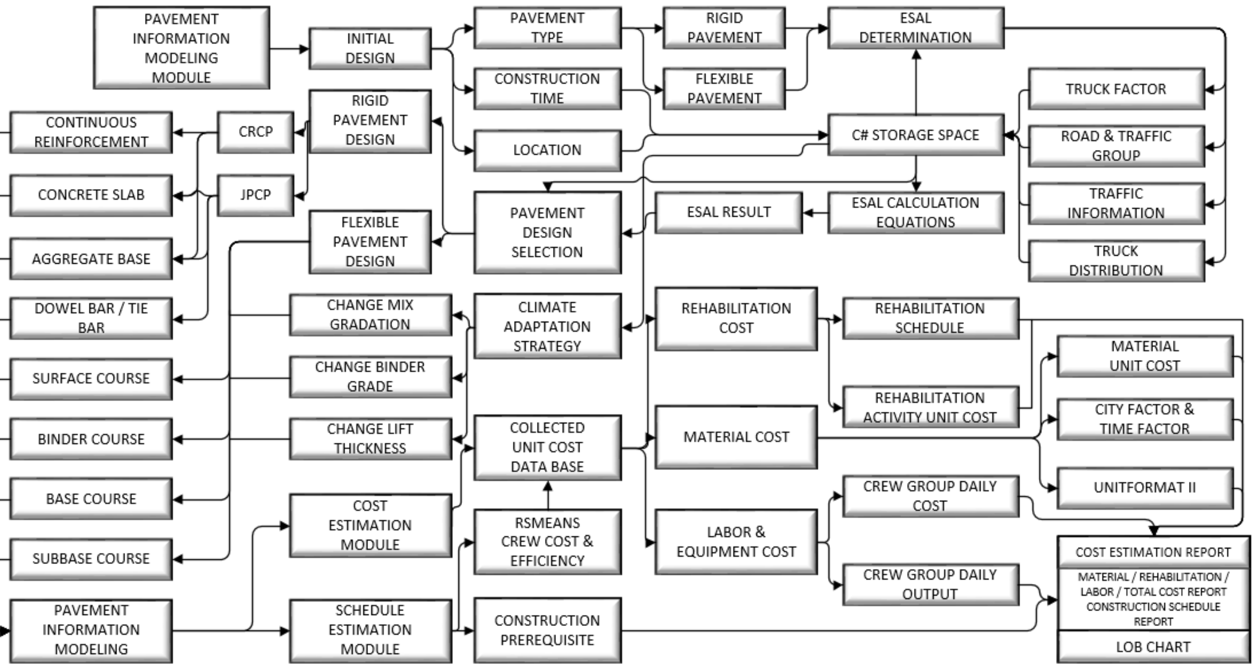



**Figure 0.8 Data Flow in the Proposed Model**

## 3.9 Summary

The methodology to be used in creating the proposed model and its three integrated modules, which are: 1) a PDS; 2) an SRPD; and 3) a PIM, was described in this chapter. Initially, an ESAL module will be utilized to determine the influence of traffic load on the pavement. The PDS will be used in five-step procedures to build the design selection system based on the computed ESAL result and the pavement design criteria supplied by the Canadian governments. To build a pavement design selection system for Canada, the effects of traffic and the environment will be considered while choosing paving materials and designing pavement structures. AASHTOWare MEPDG or AASHTO 1993 conducted a variety of pavement design standards, which will be examined and employed in the creation of PDS. Comparably, the SRPD, which is a theoretical method for enhancing pavement performance that is compiled from several adaptation techniques were used in peer-reviewed studies. At the end, based on the construction location and time, the SRPD will provide users appropriate solutions for climate adaptation to the present pavement design generated by the proposed PDS module.

The proposed PIM module adhered to the CEG technique, which will first instruct Revit to draw a cross-section shape and then stretch the shape to the length of the pavement to create a 3D frame. C# coding will be used for this process. Revit will read the pavement information, and the paving material will be poured into the frame to create an element. The 3D model will ultimately be formed by combining the pieces for the pavement components. Revit will read the pavement information, cost information, and construction efficiency to deliver information for construction management. The cost estimating and scheduling reports and LOB chart will be displayed in Revit, and the results will finally be saved in a new Excel file for future use.

# Chapter Four

## Model Development

### 4.1 Introduction

To enhance the model's capabilities, the proposed modules are thoroughly investigated, and their development process is described in this chapter. The proposed model will be implemented by integrating the PDS, SRPD, and PIM modules in a dynamic manner. The implementation will be executed through three main phases. Since each module serves different tasks, they are going to be individually created and then merged. Furthermore, the flowchart presented in the methodology section of chapter 3, which outlined a logical sequence, will also serve as a guide for the model's development and integration logics. The methodology chapter presented the overall methods in the synthesis of the PDS and SRPD and addressed the code to be executed to form the modules, their presentation, and limitations. To display the initial 3D pavement design, the PIM module will be incorporated using the outputs from PDS and SRPD modules. The modules for cost estimating and scheduling will be created to show the costs at the various stages of construction as well as generating a schedule to show the paving process. C# code will be used in the Visual Studios interface to integrate all the model’s components.

### 4.2 ESAL Determination Module Integration

The ESAL determination module will be created by gathering the location and traffic data from users, as described in the methodology section. The location data is read and used to calculate the truck factor and truck dispersion to construct the ESAL computation. The equations to calculate the ESAL are straightforward and thus, they are embedded directly in the C# code to enable the module to quickly produce the results. The result for the ESAL determination module is presented as a single number for ESAL that is applied in the target location. The calculation and input results are stored in C#, so they are used in later modules. Moreover, the module provides users with the ability to adjust the input values they wish to use in the calculation procedure. Figure 4.1, Figure 4.2, and Figure 4.3 illustrate the required inputs for the ESAL calculation in the model's interface.

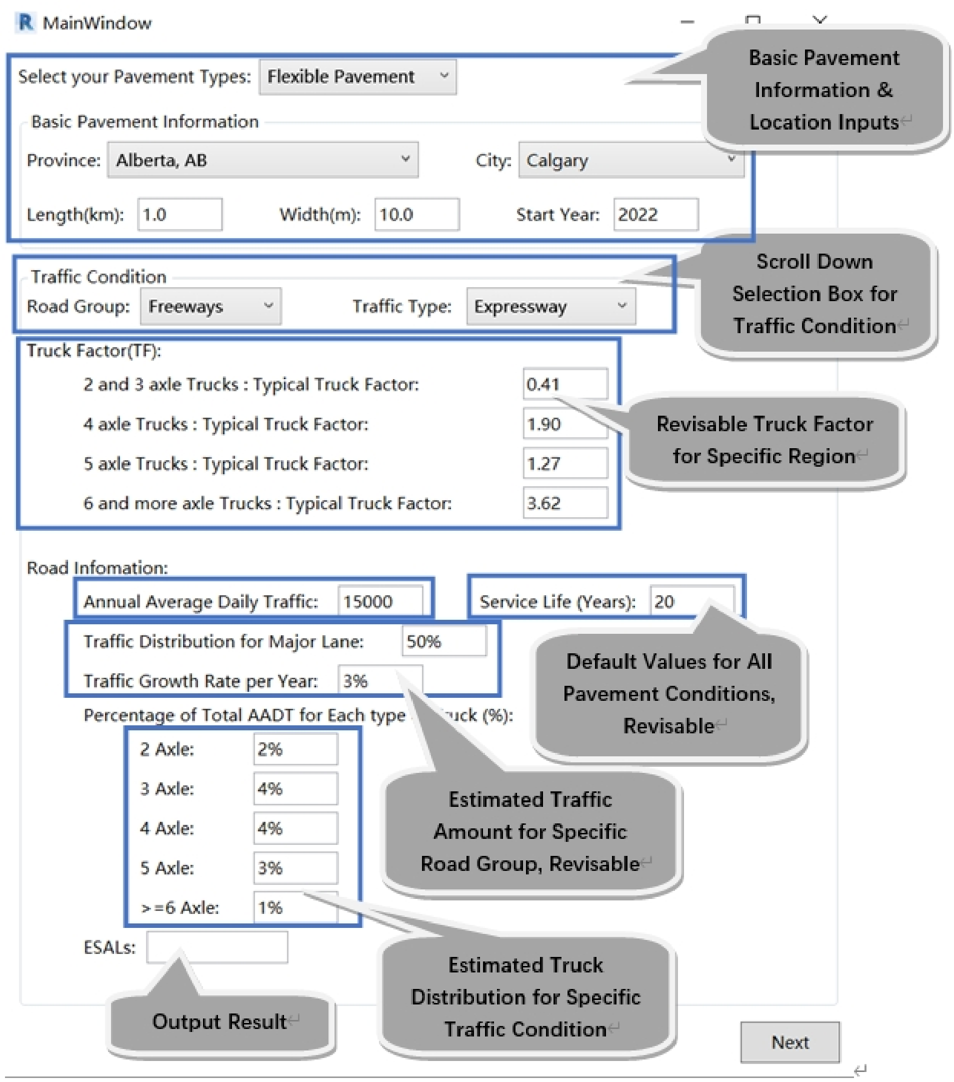



**Figure 0.1 ESAL Determination Module Information Display**

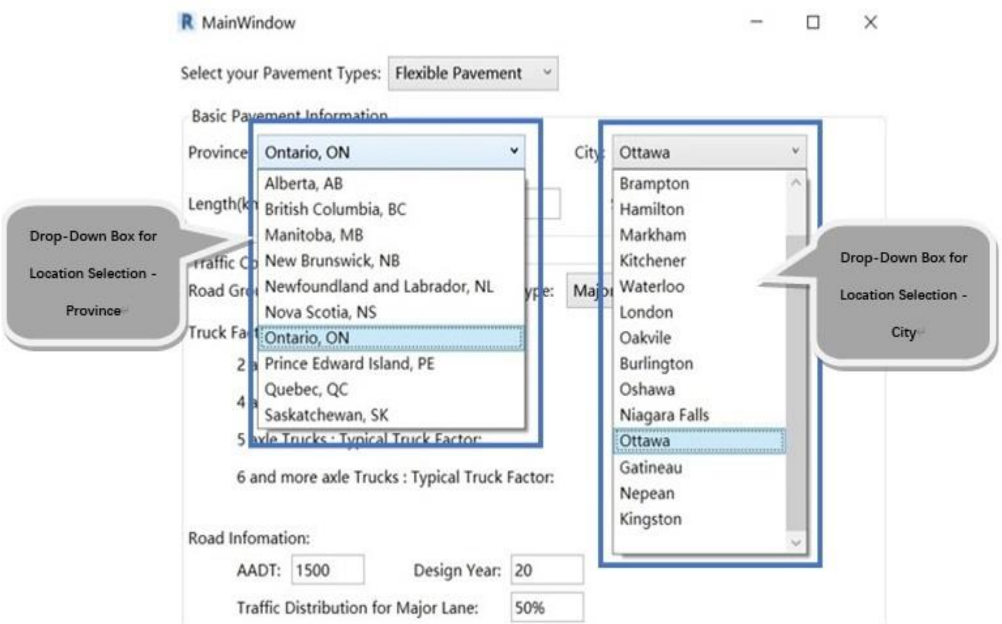



**Figure 0.2 Expanded Location Information Inputs Sample**

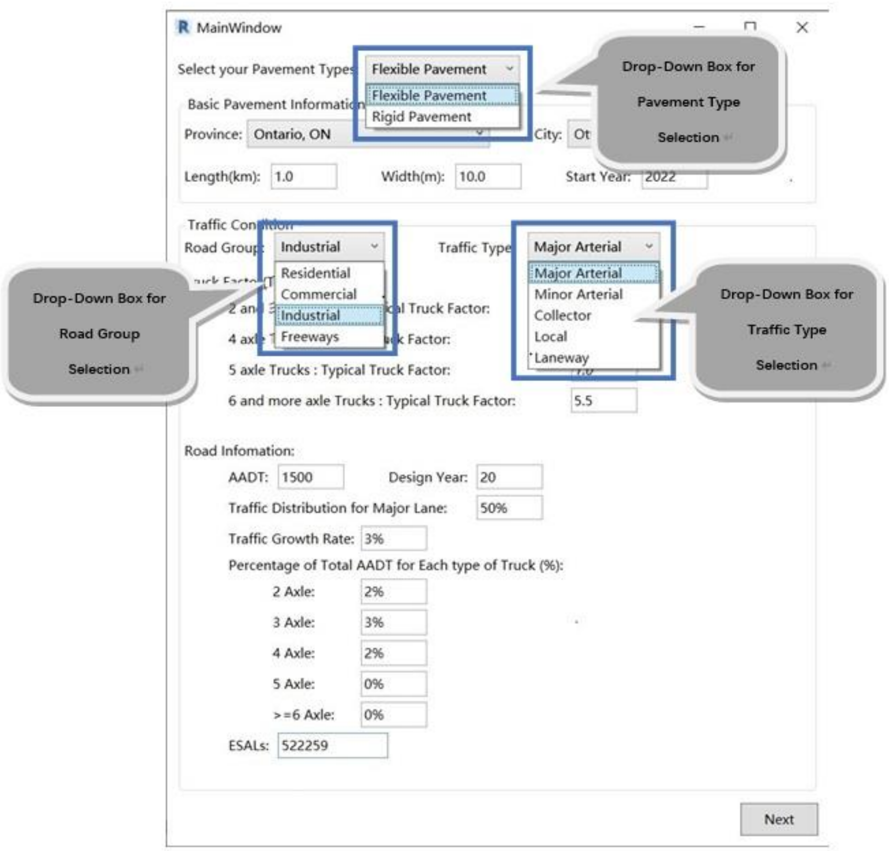



**Figure 0.3 Expanded Traffic Information Inputs Sample**

The ESAL determination is the first step in the model's process; that module receives and analyses the user inputs and accordingly presents the ESAL value as outputs. The ESAL outcome and the user inputs are stored in C# for the pavement design selection and adaptation strategy's selection. As illustrated in Figure 4.2, drop-down boxes are placed for provinces and most of the cities within a province to select construction locations, which affect the truck factor. By selecting the pavement type and clicking on the target location, the truck factors adopted for that region are automatically displayed. The truck factor for rigid pavement is 50% higher than the value of flexible pavements because AASHTO claimed in 1993 that tandem axles were shown to have a bigger influence on rigid pavements (AASHTO 1993). Users may adjust the truck factors to fit the actual traffic conditions. However, limits are set based on the truck factor records within Canada to constrain the range of input.

Similarly, as shown in Figure 4.3, drop-down boxes are provided to select a road group and traffic type. After consideration, matching judgements are set to make the selection between the road group and traffic type reasonable. If a freeway is selected under the road group, due to speed limits and freeways or expressways having no intersections, major arterials or other lower classifications are not assigned to the traffic type's selection. Therefore, for the selection of freeways under the road group, only expressways are assigned under the traffic group. Whether the road group is industrial or commercial, since these two groups may contain any type of traffic (except for expressways) in the city, all the selections under the traffic type are provided except for expressways. For the selection of residential road groups, both major arterials and expressways are not provided under traffic types because these two types of traffic are rarely used in residential areas.

Both road group and traffic type are used to estimate the distribution of trucks in AADT. Under the road group selection, traffic is more likely to have more trucks for freeways than the rest of the options. Traffic in industrial and commercial areas contains more trucks than that in residential areas. Similarly, under the traffic type selection, the truck percentage for each traffic type is ranked from high to low as expressways, major arterial, minor arterial, collector, local, and laneway. As presented in Figure 4.4, the combination of road group and traffic type determines the estimation of the truck percentage in AADT at the target location.

```
if (this.roadGroupCB.SelectedIndex == 0)    // "Residential"
{
    this.trafficDist.Text = "100%";

    if (this.aadt != null)
    {
        this.aadt.Text = "500";
    }

    if (this.trafficTypeCB.SelectedIndex == 0
        || this.trafficTypeCB.SelectedIndex == 1) //"Minor Arterial", "Collector"
    {
        this.twoAxle.Text = "2%";
        this.threeAxle.Text = "2%";
        this.fourAxle.Text = "0%";
        this.fiveAxle.Text = "0%";
        this.sixAxle.Text = "0%";
    }
```

**Figure 0.4 Truck Distribution Determination Sample Code**

The "road group selected index" represents the user input for the road group. Index 0 is assigned for residential roads, index 3 is assigned for expressways, and other conditions are assigned for commercial and industrial roads. As shown in Figure 4.4, index 0 is assigned for residential roads, traffic distribution is assumed to be 100% on the lane, and AADT is assumed to be 500 for this type of road. While a residential road is selected, the "traffic type selected index" is applied to determine the traffic type. Indices 0 and 1 are assigned to represent minor arterial and collector roads, index 2 is assigned for local roads, and index 3 is assigned for laneways. Based on the "road group selected index" and "traffic type selected index", truck percentages distribution are assigned to the specific case. As a result, the truck factor, truck distribution, and other road information are coded with C# to calculate the ESAL, as shown in Figure 4.5.

```
double axle2Trk, axle4Trk, axle5Trk, axle6Trk;
double.TryParse(this.twoAndThrTrk.Text, out axle2Trk);
double.TryParse(this.fourTrk.Text, out axle4Trk);
double.TryParse(this.fiveTrk.Text, out axle5Trk);
double.TryParse(this.sixTrk.Text, out axle6Trk);


double axle2 = this.CalulatePercentageValue(this.twoAxle.Text);
double axle3 = this.CalulatePercentageValue(this.threeAxle.Text);
double axle4 = this.CalulatePercentageValue(this.fourAxle.Text);
double axle5 = this.CalulatePercentageValue(this.fiveAxle.Text);
double axle6 = this.CalulatePercentageValue(this.sixAxle.Text);

double growthRate = this.CalulatePercentageValue(this.trafficGrowth.Text);

var result1 = aadt * trafficDist;

var result2_2Axle = result1 * axle2Trk * axle2 * 365;
var result2_3Axle = result1 * axle2Trk * axle3 * 365;
var result2_4Axle = result1 * axle4Trk * axle4 * 365;
var result2_5Axle = result1 * axle5Trk * axle5 * 365;
var result2_6Axle = result1 * axle6Trk * axle6 * 365;

var result3 = result2_2Axle + result2_3Axle + result2_4Axle + result2_5Axle + result2_6Axle;

double.TryParse(this.designYear.Text, out double designYear);

var esal = result3 * (Math.Pow(1 + growthRate, designYear) - 1) / growthRate;
return esal;
```

**Figure 0.5 ESAL Calculation Sample Code**

As presented in Figure 4.5, the model performs all the calculations in the background and presents the ESAL value in the interface. C# first reads the user inputs from the input boxes to ensure the inputs are available by using the “tryParse” logic. If all inputs are available, the codes proceed to the calculations step by using the following parameters:

- Result1 = AADT × Traffic Distribution (%), traffic in the target lane,
- Axle_Trk = Truck factor for each type of truck,
- Axle_ = Percentage of each type of truck in AADT,
- Result2_Axle = ESALs for each type of truck calculated in construction year,
- Result3 = Total ESALs in the construction year,
- Growthrate = Traffic growth rate (%),
- Designyear = Service life of the pavement in years.

By using the mentioned parameters, the ESAL calculation process, which was explained in chapter 3, is applied to estimate the ESAL on the target pavement segment. By pressing “Enter” on the keyboard and clicking the “Next” button on the interface, the calculations are executed and stored in C# to be used in the other modules.

## 4.3 PDS Module Integration

The development of the PDS module is divided into two stages, one for flexible pavements and the other for rigid pavements. For flexible pavements, the development encompasses two paces: 1) collecting and setting paving information, including paving materials and lift thicknesses for different cities and provinces; 2) matching the paving information with the local subgrade soil condition, traffic condition, and road classification to provide a list of design recommendations. For rigid pavements, the development encompasses three steps: 1) collecting and setting paving information, including paving materials and lift thicknesses for different cities and provinces; 2) matching the paving information with traffic conditions to provide a recommendation for the design; 3) calculating the reinforcement design based on the reinforcement type and road information.

The selection of the paving materials and lift thicknesses was gathered from different government-approved sources. A link is required to incorporate the design information into the database. Due to technical constraints, the design information is directly coded in C#. Figure 4.6 and Figure 4.7 illustrate samples of the embedded code for the flexible pavement design in the model.

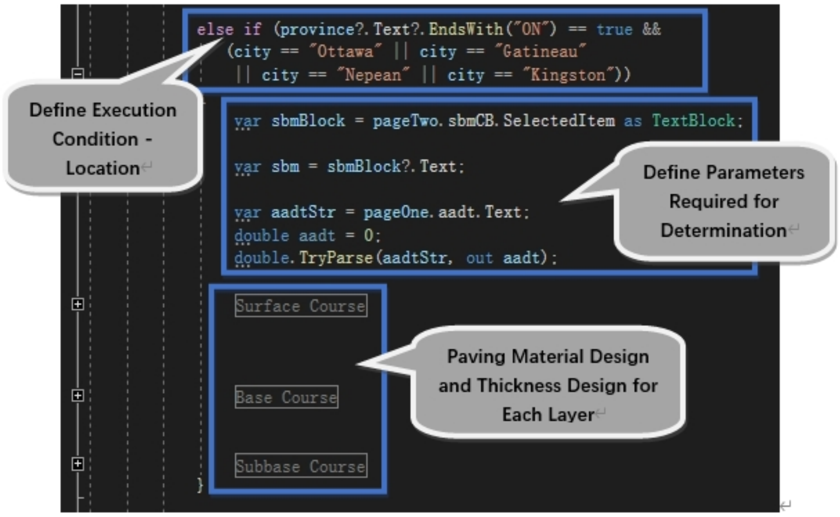



**Figure 0.6 PDS Flexible Pavement Design Sample Code**

In the methodology section of the PDS module it was mentioned that the guidelines adopted by each province are used for the pavement design. For Ontario, design guidelines were found for both Ottawa and Toronto. Therefore, those design guidelines are adopted by the adjacent cities to Ottawa and Toronto based on the distance and climate conditions. As illustrated in Figure 4.6, Ottawa, along with the other three adjacent cities, adopt the same guidelines for the flexible pavement design. The parameters used to determine the paving materials and lift thicknesses in Ottawa and adjacent cities mainly include subgrade soil property (represented by “sbm” in the code) and the ESAL result. However, based on the design guidelines from different provinces, other determinants such as AADT value, road groups, and traffic types are applied for the pavement design in that province.

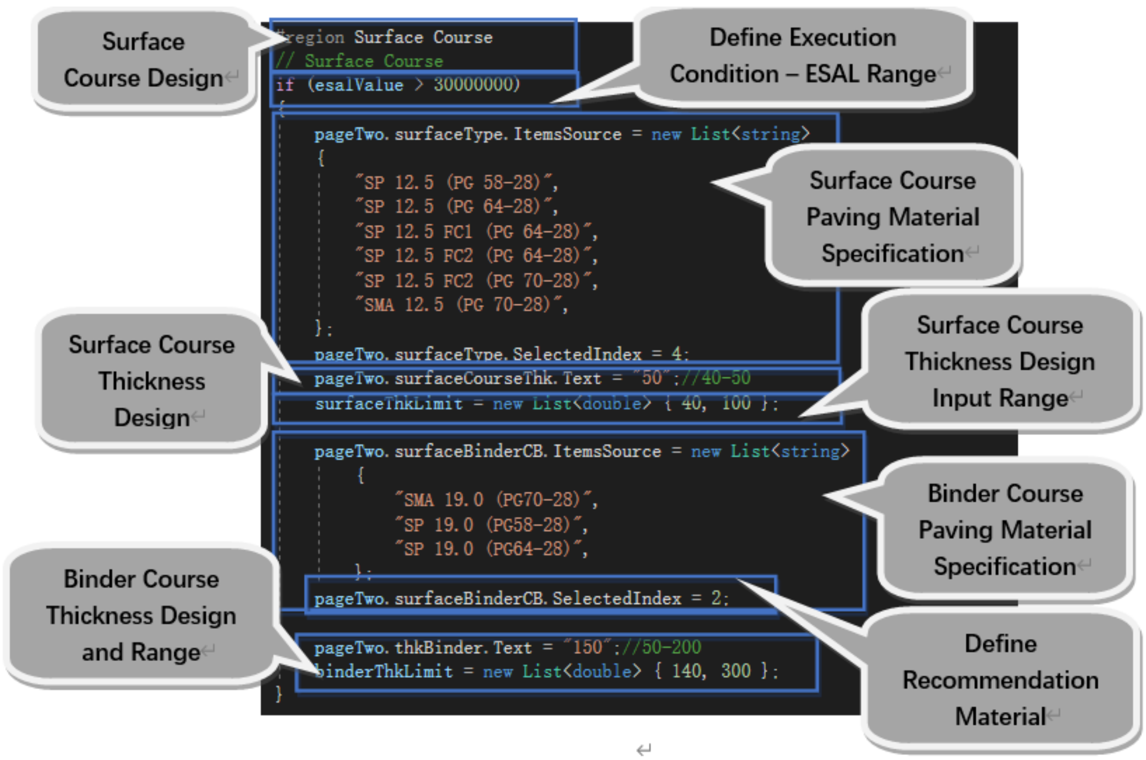



**Figure 0.7 PDS Flexible Pavement Surface Course Design Sample Code**

Based on the location, traffic conditions, and the subgrade soil properties, as shown in Figure 4.7, the PDS outputs include the paving materials and lift thicknesses recommended for the project. As displayed in Figure 4.7, PDS first reads the calculated ESAL result from the ESAL determination module; for each traffic case and soil property, the recommended paving materials and lift thicknesses are predetermined and coded in C#. A drop-down box lists the paving materials recommended for the project, as it gives users the ability to freely customize the design. A text box is provided to display the recommended layer's thickness. With the consideration of a flexible model, as shown in Figure 4.11, users are allowed to adjust the lift thicknesses within a certain range that is taken from the design specification.

For rigid pavement, lift thickness design followed the same development logic and execution methodology as flexible pavement. Additionally, reinforcement design is developed in the PDS module. There are two types of rigid pavements considered in this model: JPCP and CRCP. For

JPCP, typical designs for the reinforcements are applied in Canada, and the typical design values are adopted in the model development. Like the development of PDS for flexible pavement, the reinforcement designs of JPCP are integrated into the model by considering both location and traffic conditions, as shown in Figure 4.8.


**Figure 0.8 PDS Rigid Pavement JPCP Reinforcement Design Sample Code**

The design specifications for JPCP are predetermined and coded in C#. PDS first reads the selected location and ESAL value to display the proper designs adopted in that region with similar traffic conditions. The reinforcement diameters for the dowel bar and tie bar are presented in a drop-down list. Users may select different values to customize their design. The recommended values for the bar length, spacing, and joint spacing are displayed in text boxes. The displayed results can be modified by users within a certain range.

The development of PDS for CRCP follows the described methodology in chapter 3. As presented in Figure 4.9 and Figure 4.10, typical design values for reinforcement size, reinforcement ratio, and cover depth are predetermined and coded in C#; the number of rebars and spacing between bars are calculated based on the road information. As shown in Figure 4.12, all the presented outcomes are adjustable within certain ranges adopted from the Canadian governments' guidelines, as addressed in the methodology chapter.

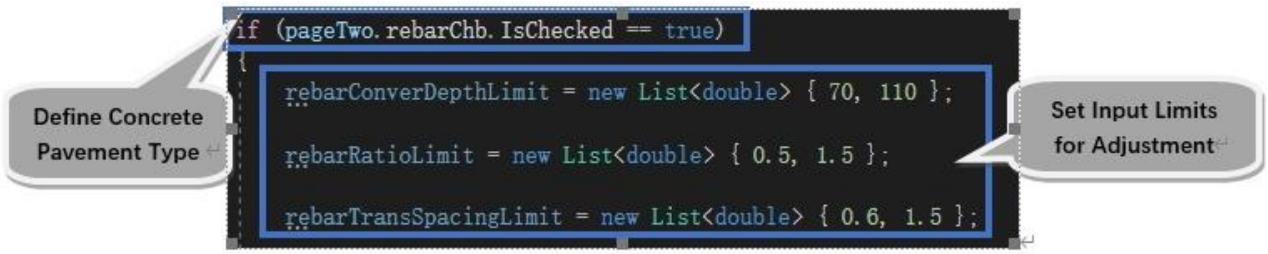



**Figure 0.9 PDS Rigid Pavement CRCP Design Constrains Sample Code**

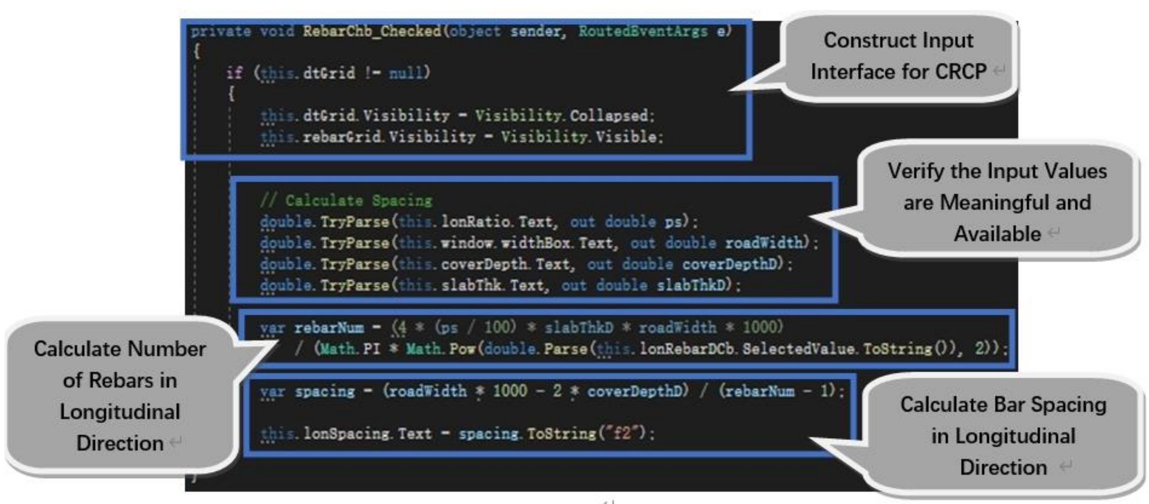



**Figure 0.10 PDS Rigid Pavement CRCP Rebar Design Sample Code**

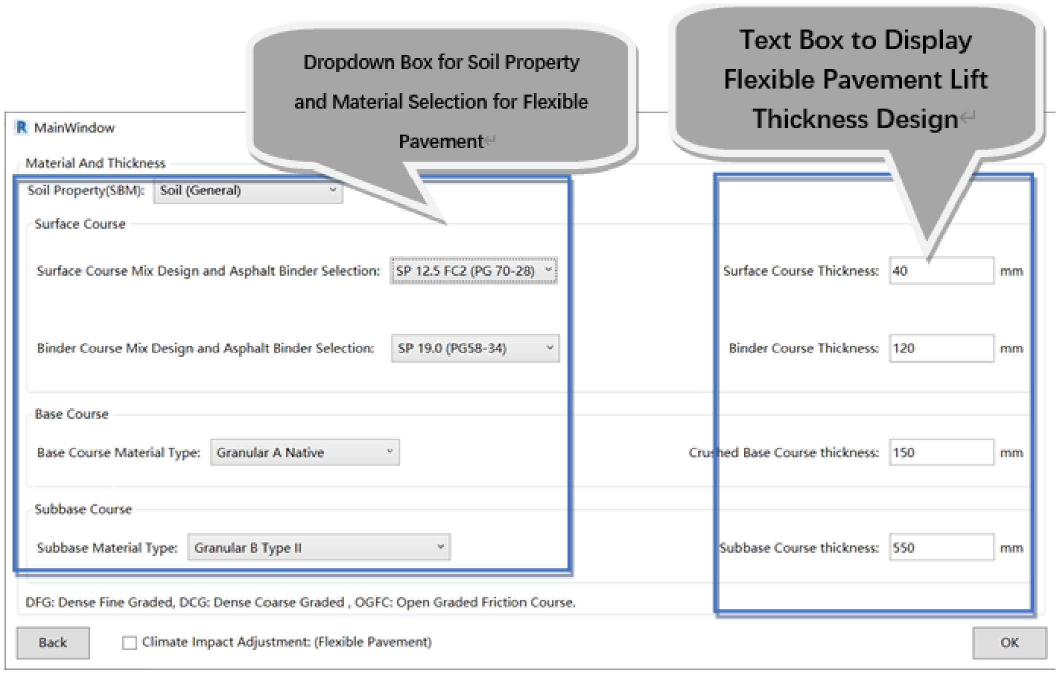



**Figure 0.11 PDS Module Information Representation for Flexible Pavement**

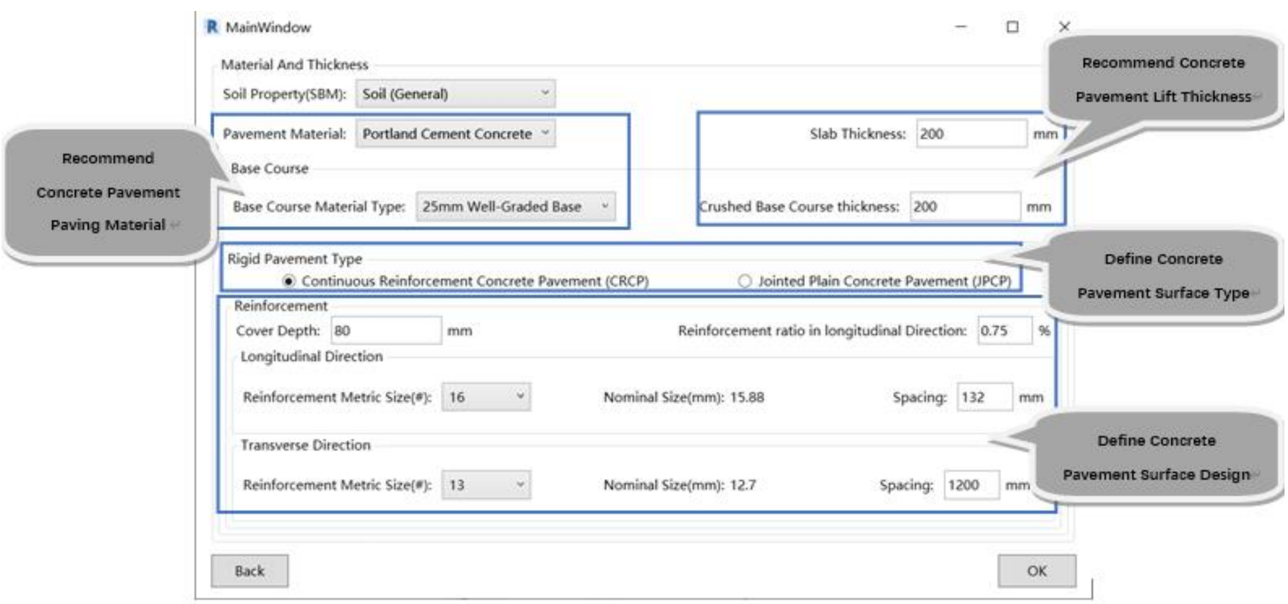



**Figure 0.12 PDS Module Information Representation for Rigid Pavement - CRCP**

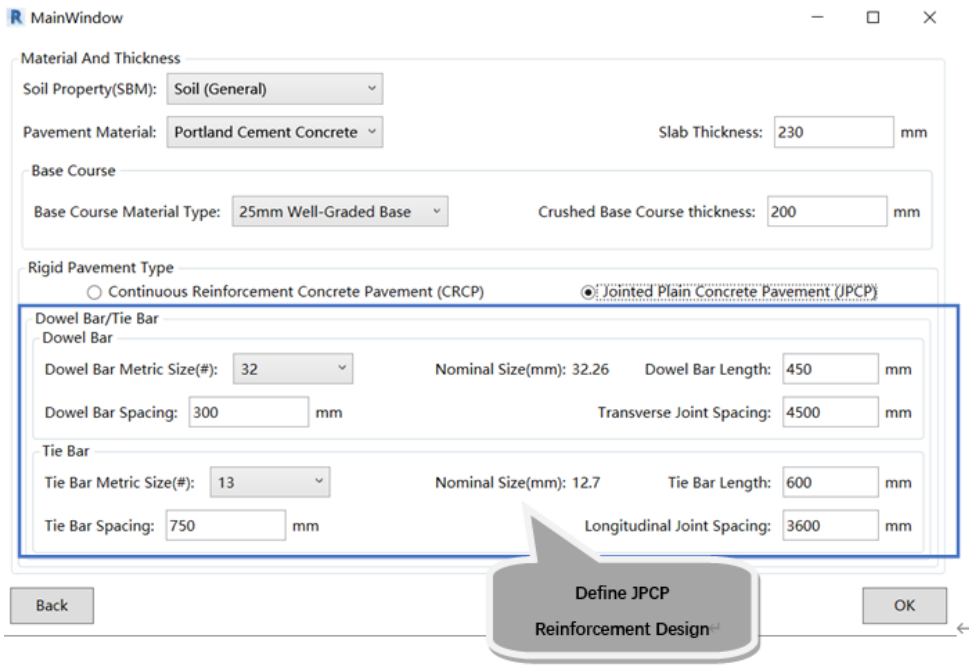



**Figure 0.13 PDS Module Information Representation for Rigid Pavement - JPCP**

Based on the construction location, traffic condition, and subgrade property, the PDS module provides design recommendations for both flexible and rigid pavement. As presented in Figure 4.11, Figure 4.12, and Figure 4.13, the recommended designs are displayed, and users can customize the design to fit the actual conditions. For flexible pavements, as illustrated in Figure 4.11, material and thickness designs for each lift are generated. For rigid pavements, the paving material and thickness designs are provided for both CRCP and JPCP. The reinforcement designs are provided by the PDS as well. Based on Figure 4.12 and 4.13, reinforcement quantity, dimensions, and spacings in different directions are provided.

Since the design is being recommended based on the selected traffic and location, and limits being set for the range of input, by clicking the “Back” button, users may go back to the ESAL module to modify the traffic and location information. The PDS module does the calculation and recommendation repeatedly until the design is satisfied and the “OK” button is clicked.

## 4.4 SRPD Module Integration

The development of the SRPD module follows the methodology addressed in **Section 3.5** in chapter 3, which provides adaptation strategies for the pavement designs generated from the PDS module. The adaptation strategies for different cases are predetermined and coded in C#. Depending on the different construction location and construction time, measures and adjustments are assigned to the outcomes from the PDS module. Figure 4.14 presents the integration of pavement adaptation strategies with the designs provided by the PDS module.

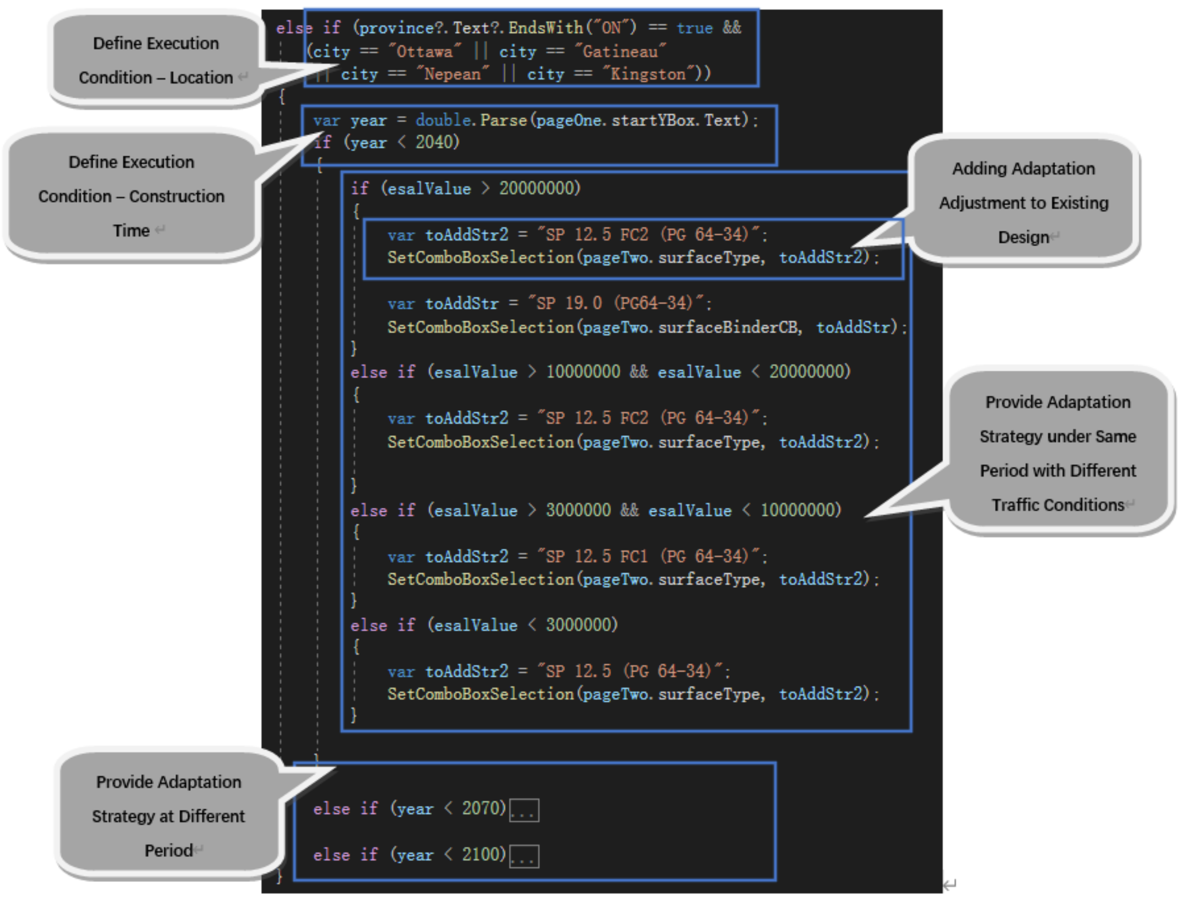



**Figure 0.14 SRPD Strategy Recommendation Sample Code**

The development of SRPD uses pre-set conditions coded in C# to determine the selection of adaptation measures. The recommended measures and adjustments are added to the existing designs produced by the PDS module. To provide adaptation suggestions for pavement designs, the SRPD module first reads the construction location and matches the location with adaptation measures from the results of Swarna et al., study for each case. Based on that study, adaptation measures are taken at different time periods to provide resilience; therefore, the construction time is read secondarily by the SRPD module to determine the type of measures to be taken at different times. For locations and time periods where a strategy of change in asphalt mixture gradation is applied to the pavements, the traffic condition is read by SRPD. Based on the outcome of the ESAL determination module, SRPD provides additional paving material to the existing design. For the condition where a change in the thickness of the asphalt layer is applied, the lift thickness is increased by multiplying a certain index by the previous value. Figure 4.15 presents the relationship between the PDS module and the SRPD module.

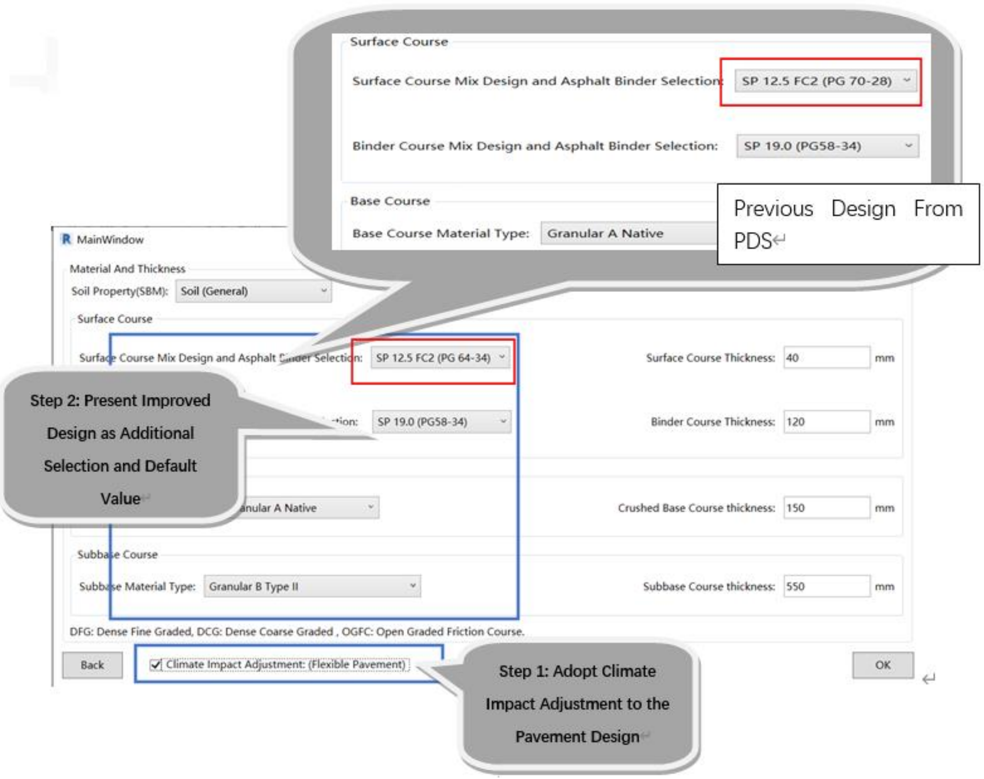



**Figure 0.15 SRPD Information Display, Comparing to PDS Outcomes**

By checking the "Climate Impact Adjustment" box, SRPD provides improved designs to users. The improved designs for paving material are set as default values and added to the drop-down list for users to make selections. In cases where the improved designs are proposed for lift thickness change, the adjusted thicknesses are displayed in the text boxes as default values. With the consideration of the model’s flexibility, users can modify the thickness within a revised range to ensure the pavement's resilience. The design information from both PDS and SRPD is saved in C# by clicking the “OK” button on the right bottom corner.

## 4.5 PIM Module Integration

Both the dimension outputs from the PDS and SRPD modules are needed to construct the conceptual pavement frame, and the paving material information for each layer is required to fill the frame with associated entities. These are the two inputs that the PIM module requires. For users to view the conceptual pavement design before creating the conceptual cost estimating and scheduling, the module is positioned as the third-to-last phase in the model's workflow. As previously illustrated in Figure 4.1, Figure 4.11, Figure 4.12, and Figure 4.13, pavement width and length are set by users on the input page, and lift thickness and material information are displayed in the PDS and SRPD modules. Once users verify the design information, the information is transferred from C# into Autodesk Revit by clicking the “OK” button on the PDS and SRPD modules. The PIM module applies the CEG method to construct the pavement frame, and material information is indicated for each lift. As a result, Figure 4.16, 4,17, and 4.18 illustrate the PIM’s implementation.

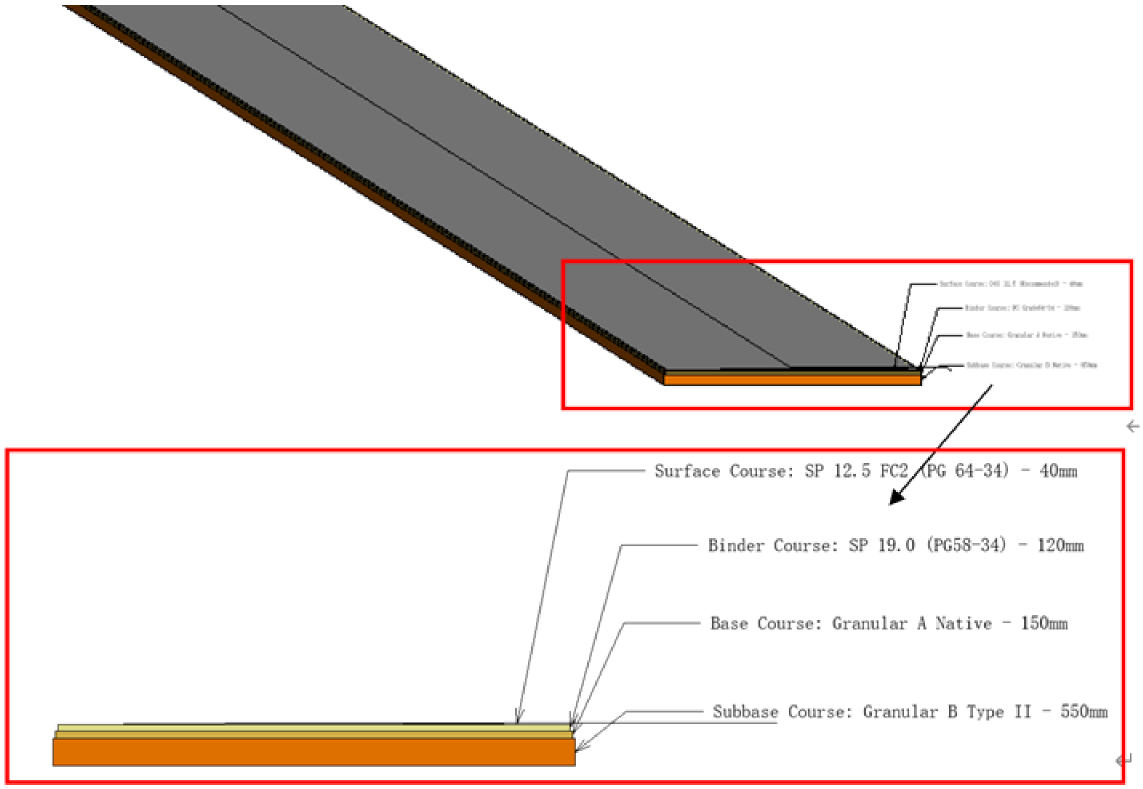



**Figure 0.16 PIM Model for Flexible Pavement Design**

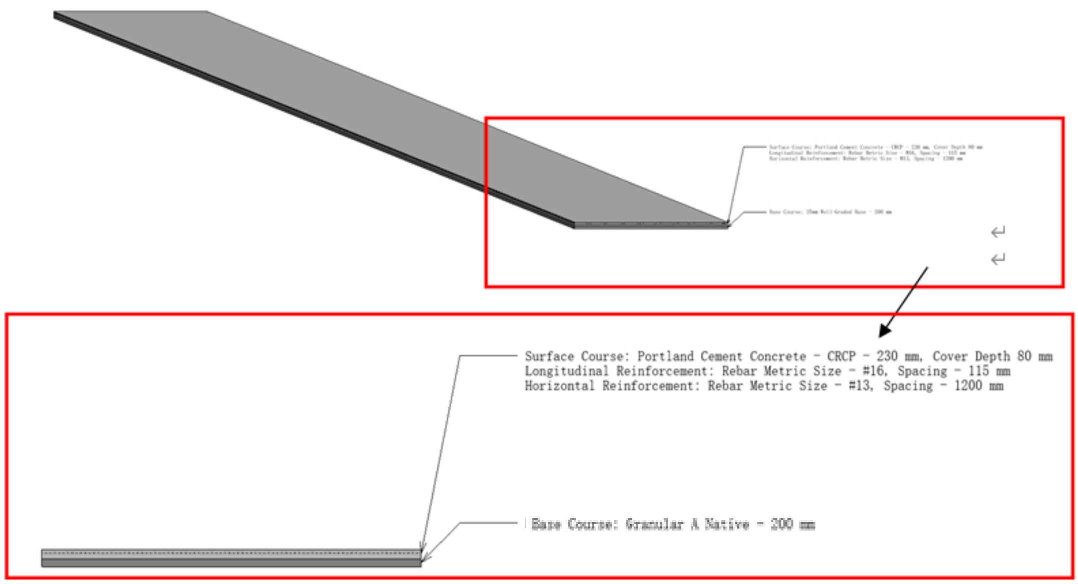



**Figure 0.17 PIM Model for Rigid Pavement Design - CRCP**

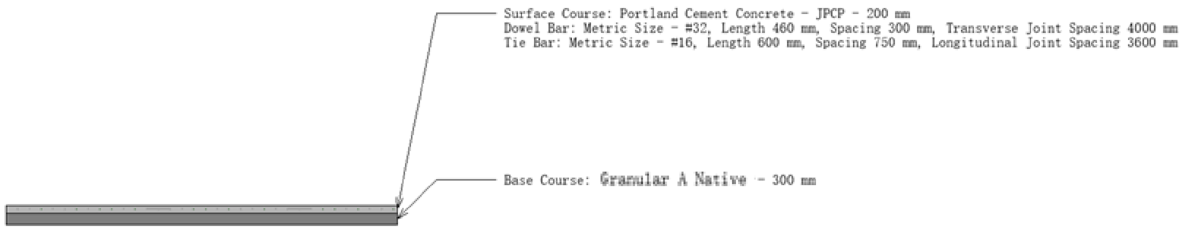


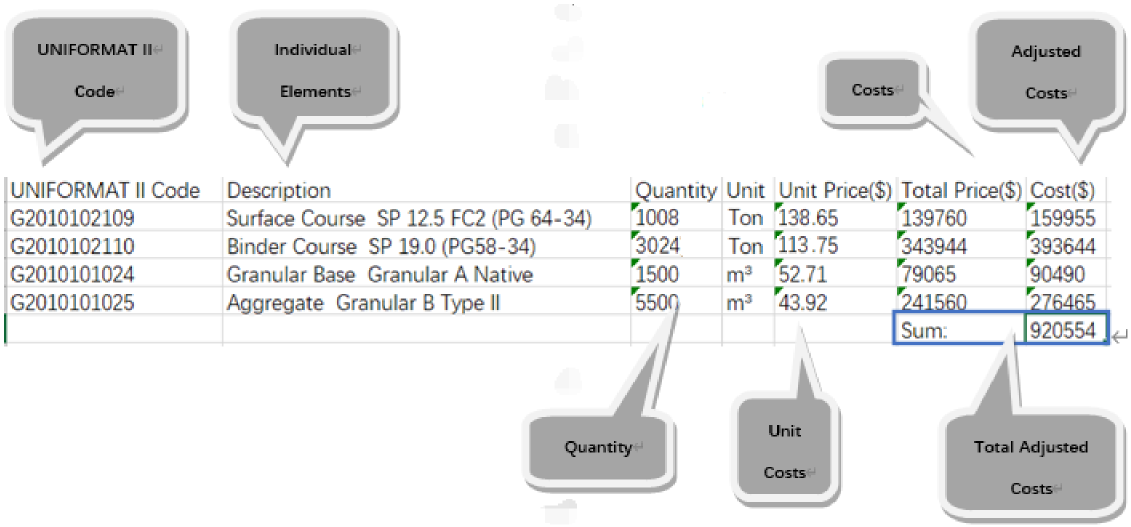

**Figure 0.18 Screenshot of PIM Model for Rigid Pavement Design - JPCP**

The PIM module in Revit creates a 3D model, as shown in Figure 4.16. The dimensions of the flexible pavement are constructed based on the design inputs provided by users in the ESAL determination, PDS, and SRPD modules. The material and lift thickness information of the pavement cross-section is provided in the front view. Like flexible pavements, Figures 4.17 and 4.18 illustrate the PIM module for CRCP and JPCP. In addition to the paving material and thickness design, PIM module provides information about the reinforcement design for CRCP and JPCP. If the current design is not satisfied, users need to go back to the first input page, the ESAL determination module, to re-design the pavement.

## 4.6 Conceptual Cost Estimate and Scheduling Module Integration

The final two phases in the model's development process are cost estimating and scheduling. The cost-estimating module generates the pavement project's cost summary using the dimension output values obtained from the PDS and SRPD modules as input. The following methods are used to achieve this: 1) developing an Excel spreadsheets for both pavement types that include the material unit prices collected from previous pavement projects and average those prices; 2) developing another Excel spreadsheets for both pavement types that include rehabilitation plans and unit prices collected from project studies proposed by the Canadian governments; 3) developing Excel spreadsheets for labor and equipment daily output and daily cost gathered from the RS Means construction cost database.

The paving materials' unit price is collected based on past bidding prices and projects from different locations, where various standards are followed. After consideration, ASTM UNITFORMAT II Work Breakdown Structure (WBS) is applied to create the materials unit price spreadsheets. Figure 4.19 exhibits the formatted cost-estimating Excel spreadsheets for paving material.




| UNIFORMAT II Code | Description | Quantity | Unit | Unit Price($) | Total Price($) | Cost($) |
|---|---|---|---|---|---|---|
| G2010102109 | Surface Course SP 12.5 FC2 (PG 64-34) | 1008 | Ton | 138.65 | 139760 | 159955 |
| G2010102110 | Binder Course SP 19.0 (PG58-34) | 3024 | Ton | 113.75 | 343944 | 393644 |
| G2010101024 | Granular Base Granular A Native | 1500 | m³ | 52.71 | 79065 | 90490 |
| G2010101025 | Aggregate Granular B Type II | 5500 | m³ | 43.92 | 241560 | 276465 |
| | | | | | Sum: | 920554 |

**Figure 0.19 Conceptual Cost Estimating Module Excel Spreadsheet Sample – Material Cost**

As shown in Figure 4.19, the specifications of the paving materials are gathered from PDS and SRPD modules and matched with the existing database. UNIFORMATT II coding system is matched with the standards applied in Canada. The quantity for each lift is calculated by multiplying the dimensions taken from the pavement design. The costs are calculated by simply multiplying the quantity by the unit costs. Finally, the materials' costs are adjusted for time and location differences.

Before performing the cost estimation for the pavement rehabilitation, as displayed in Figure 4.20, users are allowed to select the rehabilitation measures and adjust the rehabilitation periods and quantities.

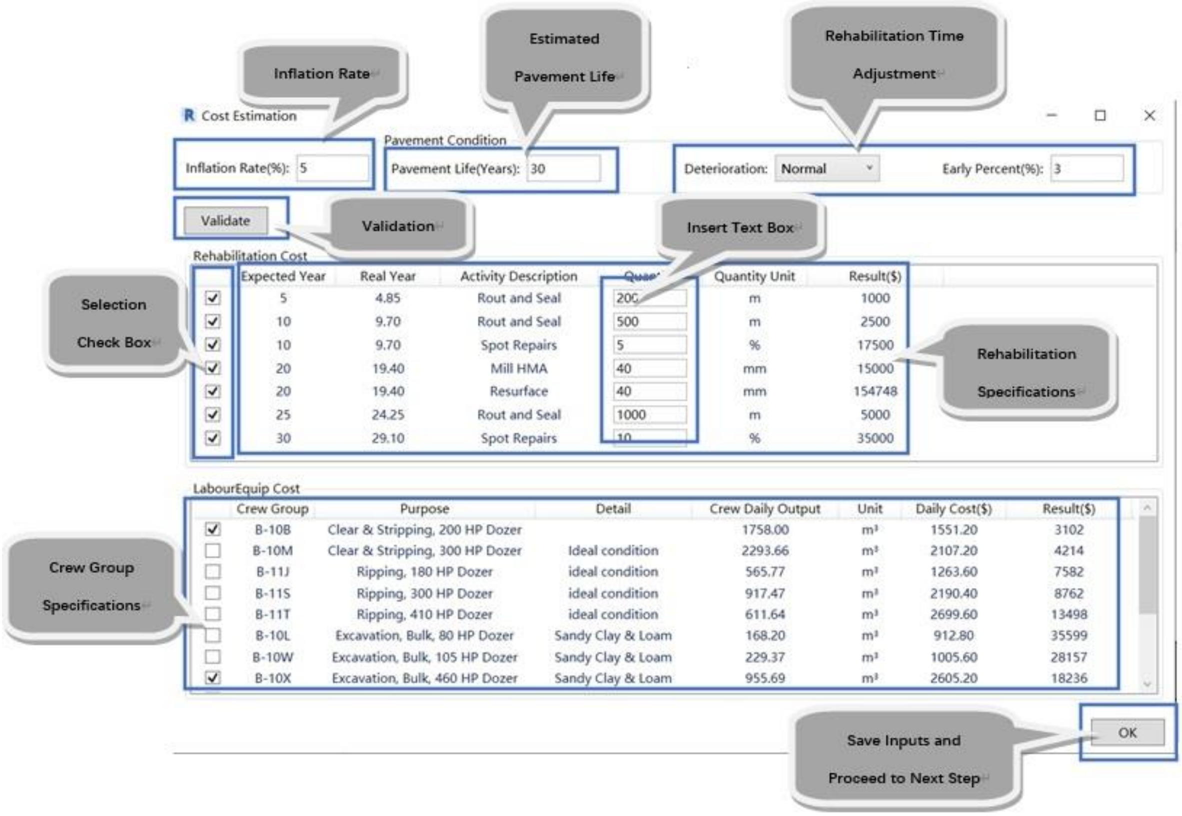
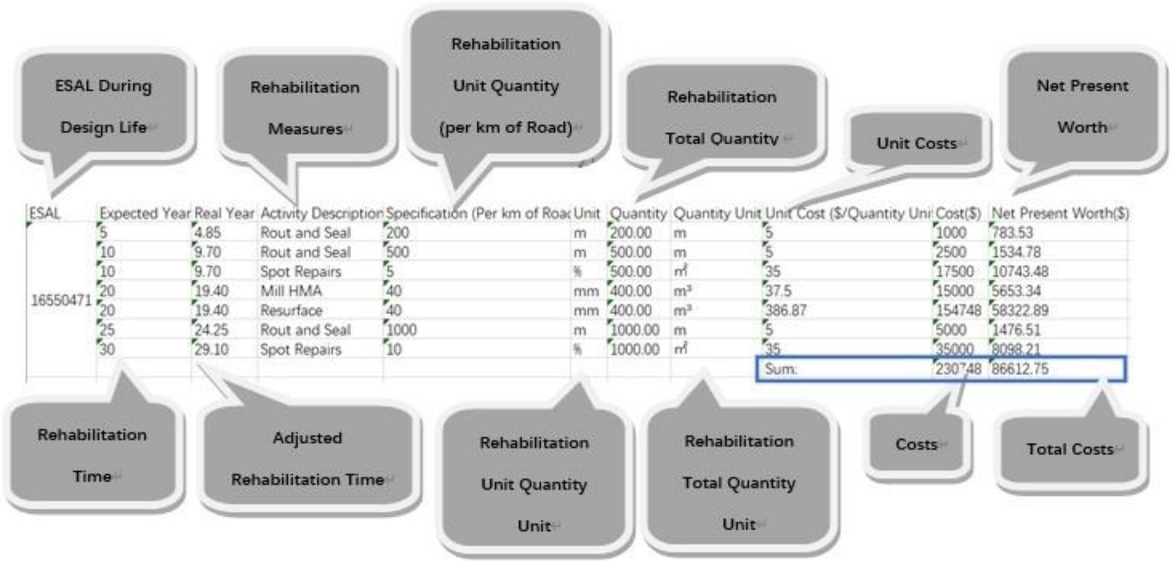



**Figure 0.20 Conceptual Cost Estimating Module Interface (Flexible Pavement)**

Rehabilitation actions are taken in the future; therefore, the module considers inflation rate that impacts the NPV of work that is taken at different time periods. Besides, based on project studies conducted by the Canadian governments, rehabilitation actions are taken periodically throughout the design life of pavement. Due to climate impacts, pavements will be facing severe deterioration in the future. Based on the literature, it is crucial to trigger the maintenance earlier to preserve the pavement's performance. Based on future climate condition, the module considered three surface conditions, "normal", "average", and "severe". Grounded on the reviewed literature, for these three situations, rehabilitation measures need to be taken at different degrees in advance with a range of 0% for ideal conditions and up to 20% for severe conditions. Thus, users may select the estimated degrees of deterioration in the future to adjust the rehabilitation cycle. Moreover, with the consideration of the model's flexibility, check boxes and text boxes are applied for users to make selections among the different maintenance plans and to adjust the maintenance quantities. Users can update and save the input results by clicking the “Validate” button. Figure 4.21 illustrates the conceptual cost estimation for the rehabilitation activities.




| ESAL | Expected Year | Real Year | Activity Description | Specification (Per km of Roa | Unit | Quantity | Quantity Unit | Unit Cost ($/Quantity Uni | Cost($) | Net Present Worth($) |
|---|---|---|---|---|---|---|---|---|---|---|
| 16550471 | 5 | 4.85 | Rout and Seal | 200 | m | 200.00 | m | 5 | 1000 | 783.53 |
| | 10 | 9.70 | Rout and Seal | 500 | m | 500.00 | m | 5 | 2500 | 1534.78 |
| | 10 | 9.70 | Spot Repairs | 5 | % | 500.00 | m² | 35 | 17500 | 10743.48 |
| | 20 | 19.40 | Mill HMA | 40 | mm | 400.00 | m³ | 37.5 | 15000 | 5653.34 |
| | 20 | 19.40 | Resurface | 40 | mm | 400.00 | m³ | 386.87 | 154748 | 58322.89 |
| | 25 | 24.25 | Rout and Seal | 1000 | m | 1000.00 | m | 5 | 5000 | 1476.51 |
| | 30 | 29.10 | Spot Repairs | 10 | % | 1000.00 | m² | 35 | 35000 | 8098.21 |
| | | | | | | | | Sum: | 230748 | 86612.75 |

**Figure 0.21 Conceptual Cost Estimating Module Excel Spreadsheet Sample – Rehabilitation Cost**

As displayed in Figure 4.21, the rehabilitation cost estimate contains five components: 1) ESAL; 2) maintenance time; 3) maintenance description; 4) maintenance quantity; and 5) maintenance cost. The ESAL value is displayed because it is used to determine the rehabilitation and maintenance schedule. Based on project studies conducted by ARA and CAC, rehabilitation plans are scheduled depending on the traffic condition; rehabilitation and maintenance activities are less frequent in light-traffic regions than in heavy-traffic regions. Considering the deterioration conditions, the adjusted rehabilitation schedule for each activity is presented. According to studies conducted by ARA and CAC, the recommended maintenance quantity applied per kilometer of road is displayed in different units and is proposed under the average climate condition. To adopt the quantity of maintenance work in the project, the quantity and unit are changed based on the purpose of the activities. For example, for spot repair on the pavement, ARA and CAC addressed 5% per kilometer of road, which represents that for every kilometer of road, under average climate and road surface condition, 5% of the road surface area will need spot repair. The actual quantity of maintenance for spot repair is calculated by multiplying the maintenance unit quantity (5%) by the total surface area. Another example is to mill HMA and resurface. According to ARA and CAC, 40 mm is applied for both milling and resurfacing activities. Milling is removing the current HMA layer, while resurfacing is paving the surface with previous or new materials. Therefore, the actual quantity for maintenance, which is the volume of the surface to be removed or resurfaced, is calculated by multiplying the construction thickness (40mm) by the pavement length and width (surface area). For the maintenance cost estimate, the unit costs are provided by ARA and CAC, which were collected from various project studies. Whereas, the rehabilitation cost for each activity is calculated by multiplying the unit costs by the maintenance quantities. Since rehabilitation and maintenance activities are planned as future work, the NPV is calculated by applying the Equation – 10, as follow:

$$P = \frac{F}{(1+i)^n} \qquad \textbf{Eq. 10}$$

Where,

i = Inflation rate

n = Number of years between the maintenance year and the construction year,

F = Future cost of the proposed activity in the maintenance year,

P = Present cost of the proposed activity in the construction year.

The estimated cost for labor and equipment relates to the project schedule. Working hours per day and the working type (single shift or double shift) affect both the cost estimate and scheduling. Therefore, before estimating the costs of labor and equipment, working information needs to be identified, as illustrated in Figure 4.22.

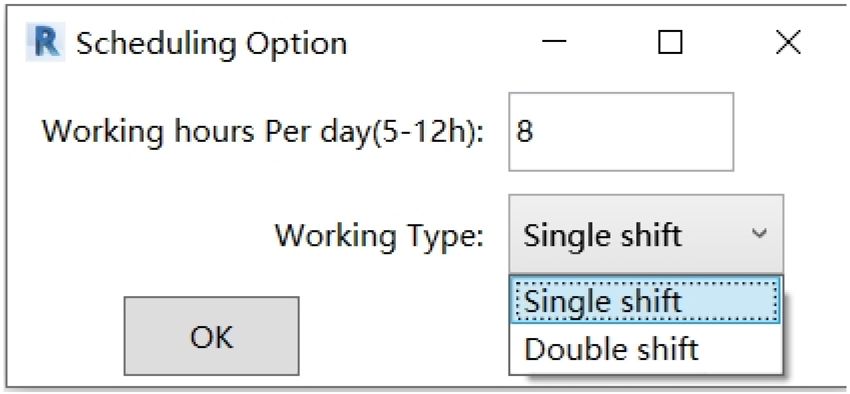



**Figure 0.22 Working Information Input Interface**

The working hours and working types applied in the cost estimating and scheduling modules are taken from the RS Means construction cost data. According to RS Means, the default values for working hours per day and working type are 8 hours and single shift, respectively. Based on the research on Canadian pavement projects, the working hours adopted in Canada vary from 5 hours to 12 hours, and the working types of single shift and double shift are commonly used in Canada. Therefore, the input is set and limited t based on that information. The working information input interface presented in Figure 4.22 determines the labor and equipment costs as well as the schedule. In cases where users made an input less than the set working hours per day, the crew's daily costs as well as their daily outputs will proportionally decrease. If the users select double shift, both the crew daily cost and output for each activity are doubled, and the slope of the LOB chart is correspondingly doubled. The interface is performed before the conceptual cost estimating module as illustrated in Figure 4.20. Once users input the working information and click the "OK" button, the cost estimating module recommends the most cost-effective crew type to be used for the project construction. As displayed in Figure 4.20, multiple crew types with different daily outputs and costs are presented to users. The module recommends construction activities at different construction stages, including stripping, excavation, grading, subbase paving, base paving, and surface paving, by comparing the total labor and equipment costs. Check boxes are provided to users so that they can select other construction work for the project. Figure 4.23 illustrates the cost estimation for labor and equipment. The information on crew type, working purpose, crew daily output, and crew daily cost is collected from the RS Means database, and the construction quantity and quantity unit of measurement are determined based on the project scale. The scheduling module determines the working duration to calculate the costs of the selected crew.


| Crew Group | Purpose | Detail | Material Amount | Unit | Crew Daily Output (Unit of Material / d | Daily Cost($) | Result($) |
|---|---|---|---|---|---|---|---|
| B-10B | Clear & Stripping, 200 HP Dozer | | 3000 | m³ | 1758.00 | 1551.20 | 3102 |
| B-10X | Excavation, Bulk, 460 HP Doze | Sandy Clay & Loa | 6400 | m³ | 955.69 | 2605.20 | 18236 |
| B-11L | Grading (rough) | | 10000 | m² | 3019.35 | 1180.40 | 4722 |
| B-32 | Subbase Course | | 10000 | m² | 3134.00 | 3181.20 | 12725 |
| B-36C | base Course | Well Graded | 10000 | m² | 4114.00 | 5106.00 | 15318 |
| B-25 | Binder Course | | 10000 | m² | 3390.00 | 5785.20 | 17356 |
| B-25B | Surface Course Inst. | | 10000 | m² | 6530.00 | 6372.20 | 12744 |
| | | | | | | Sum: | 84203 |

**Figure 0.23 Conceptual Cost Estimating Module Excel Spreadsheet Sample – Labor and Equipment Cost**


| Crew Group | Purpose | Detail | Pavement Length(m | Daily output activitie | Duration of activities(Hours | Task Duration(Day) | Task Begins | Task Ends |
|---|---|---|---|---|---|---|---|---|
| B-10B | Clear & Stripping, 200 HP Dozer | | 1000 m | 586 m/day | 13.65 | 2 | 2023-05-01 | 2023-05-0 |
| B-10X | Excavation, Bulk, 460 HP Doze | Sandy Clay & Loa | 1000 m | 149 m/day | 53.57 | 7 | 2023-05-02 | 2023-05-0 |
| B-11L | Grading (rough) | | 1000 m | 302 m/day | 26.50 | 4 | 2023-05-06 | 2023-05-1 |
| B-32 | Subbase Course | | 1000 m | 313 m/day | 25.53 | 4 | 2023-05-07 | 2023-05-1 |
| B-36C | base Course | Well Graded | 1000 m | 411 m/day | 19.45 | 3 | 2023-05-09 | 2023-05-1 |
| B-25 | Binder Course | | 1000 m | 339 m/day | 23.60 | 3 | 2023-05-10 | 2023-05-1 |
| B-25B | Surface Course Inst. | | 1000 m | 553 m/day | 12.25 | 2 | 2023-05-12 | 2023-05-1 |

**Figure 0.24 Conceptual Scheduling Module Excel Spreadsheet Sample**

To calculate the construction duration for each activity, as shown in Figure 4.24, the construction progress is set in the longitudinal direction, and the construction group efficiency is converted into meters per day in the longitudinal direction. The duration is calculated by simply dividing the pavement length by the crew daily output, and the results are rounded up to one day. The results for construction duration in days are adopted by the model for two purposes: 1) the construction duration for each activity is multiplied by the corresponding crew daily cost to accomplish the labor and equipment cost estimates; and 2) the construction duration for each activity is adopted in generate the LOB chart. While calculating the construction duration, the cost estimating module calculates the total costs for each activity. For construction work with the same purpose, the one with the lowest cost is automatically checked and recommended for users. However, users may always select other options depending on the actual construction conditions.

A LOB chart is constructed by using the calculated construction duration and crew daily output for each activity, as displayed in Figure 4.24. The crew daily outputs represent the slope of the line for each activity. The construction duration, along with the start time, is used to determine the starting and finishing times of the activities. Since all construction activities considered in this model have a start-to-start relationship, the minimum start-time interval between adjacent activities (time buffer) is set at one day. The chart is developed under three principles: 1) the minimum start-to-start interval between adjacent activities is one day; 2) construction activities follow the pre-set prerequisites; and 3) construction activities cannot interfere with others to cause any delay. For example, if the starting date of the construction is assumed to be on May 1$^{st}$, then the LOB schedule will be generated as illustrated in Figure 4.25 that considers the recommended crew, 8 hours of daily work time, and a single-shift working type.

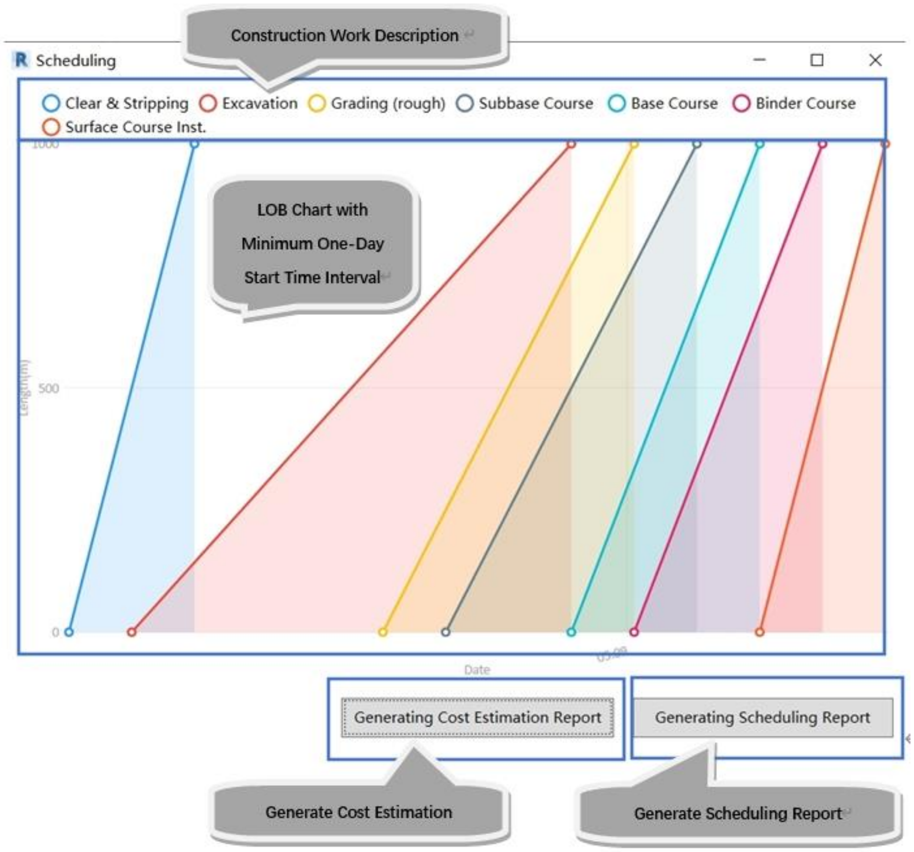
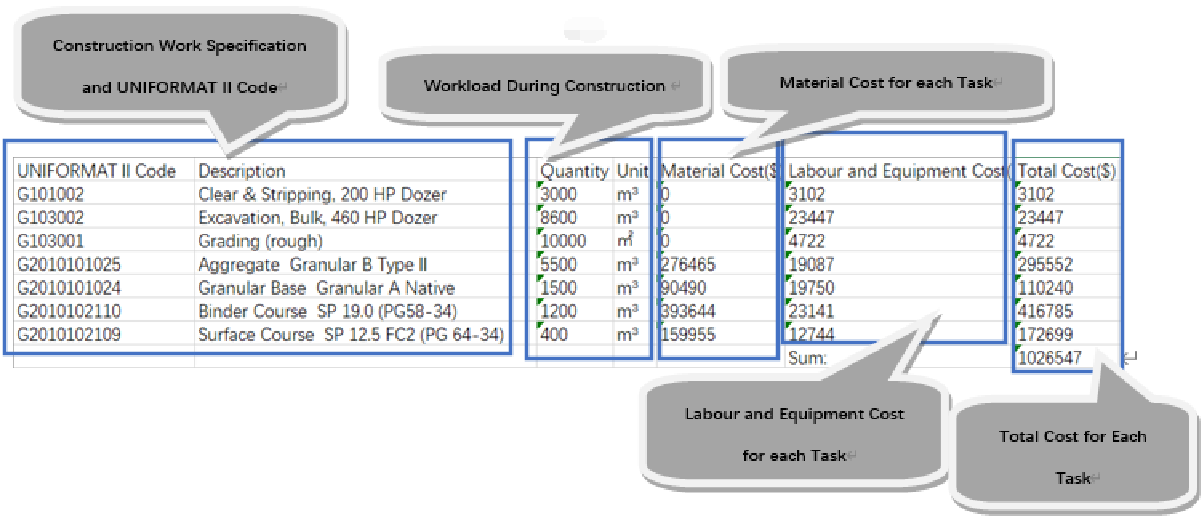



**Figure 0.25 Conceptual LOB Chart Sample**

The cost estimation module and scheduling module are integrated into the model via the C# interface. Once users input the required data for both modules, a LOB chart is formed, and the cost and schedule data are saved in C# and exported to an Excel spreadsheet. Then the results are taken from C# to be displayed in five different reports formats: i) a conceptual material cost estimation report; ii) a conceptual rehabilitation cost estimation report; iii) a conceptual labor and equipment cost estimation report; iv) a total construction cost; and v) a scheduling report.

The total construction cost report is formed based on the conceptual material cost estimation report and conceptual labor and equipment cost estimation report as displayed in Figure 4.26.



| UNIFORMAT II Code | Description | Quantity | Unit | Material Cost($ | Labour and Equipment Cost( | Total Cost($) |
|---|---|---|---|---|---|---|
| G101002 | Clear & Stripping, 200 HP Dozer | 3000 | m$^3$ | 0 | 3102 | 3102 |
| G103002 | Excavation, Bulk, 460 HP Dozer | 8600 | m$^3$ | 0 | 23447 | 23447 |
| G103001 | Grading (rough) | 10000 | m$^2$ | 0 | 4722 | 4722 |
| G2010101025 | Aggregate Granular B Type II | 5500 | m$^3$ | 276465 | 19087 | 295552 |
| G2010101024 | Granular Base Granular A Native | 1500 | m$^3$ | 90490 | 19750 | 110240 |
| G2010102110 | Binder Course SP 19.0 (PG58-34) | 1200 | m$^3$ | 393644 | 23141 | 416785 |
| G2010102109 | Surface Course SP 12.5 FC2 (PG 64-34) | 400 | m$^3$ | 159955 | 12744 | 172699 |
| | | | | | Sum: | 1026547 |



**Figure 0.26 Conceptual Cost Estimating Module Excel Spreadsheet Sample - Total Construction Cost**

As illustrated in Figure 4.26, UNIFORMAT II coding system is assigned to each construction task based on various literatures. The workload for each construction task is gathered from the labor and equipment cost report. The material, labor and equipment cost for each task is displayed according to the calculated cost values shown in the material cost and the labor and equipment cost reports. For the earth work construction, since these tasks do not consume any material, the material costs for these tasks become zero. For the other construction work that consumes material, both material and labor and equipment costs are considered. The total construction cost for each task is achieved simply adding the material and labor costs together, and the added results are summed to have the total construction cost for the whole project.

## 4.7 Summary

This chapter discussed the development of the model and the integration of the three associated modules (PDS, SRPD, and PIM) with the ESAL determination, Conceptual Cost Estimation, and Scheduling modules through C# coding. To recommend appropriate paving materials, lift thicknesses, and reinforcement designs (if required) at a given location under specified traffic impacts and subgrade soil condition, the PDS is displayed in drop-down lists and text boxes. Users can choose and modify the designs within pre-set parameters. The flexible pavement designs that were previously created using the PDS are adjusted for climate change by the SRPD module. Once users decide to adopt the adaptation tactics, modifications are made to the existing designs regarding the paving materials and lift thicknesses depending on the construction duration, location, and traffic conditions. The PIM module uses the dimension outputs and paving material from the other modules to build the 3D model in Autodesk Revit by using the pavement designs that have been saved for it. Users can view the design's details by choosing from a variety of view angles. When estimating the cost of a project, users can modify and choose the rehabilitation plans and crew types in order to create reports for paving materials, rehabilitation, labor and equipment, and total construction costs. Additionally, schedule reports are created and given to users, along with a LOB schedule based on the information provided by the crew types and the project scale.

# Chapter Five

## Model Testing

### 5.1 Introduction

This chapter aims to test the overall capabilities of the designed model. Although there is a dearth of documentation for any existing pavement project that was built with consideration of future climate and extensive information management, the model attempts to propose pavement project designs and information management with the consideration of climate change factors in Canada. As a result, it is a challenge to evaluate the model's output because there is not much data of real projects with which to compare the output. Given this, the goal of testing the model in this thesis is to evaluate the estimated cost and time schedule to assess the model's workability. A pre-existing conventional pavement project proposed by Smith et al. during the TAC conference in 2012 is chosen, and the reliability of the model is tested by comparing cost and time schedule. Maintenance details for both flexible and rigid pavements are included in the rehabilitation project that was chosen for the pavement. The selection of construction information for both pavement types discussed in the TAC conference (2012) was based on the availability of the conceptual data, as governmental projects involving pavements are typically confidential and challenging to obtain. The OR-174 pavement in the City of Ottawa, Ontario, was chosen as the pavement. Figure 5.1 displays the position of the pavement, and Tables 5.1, 5.2, and 5.3 display the pavement data that was loaded into the model.

**Table 0.1 OR-174 Pavement Project Information**

| Project Parameters | Model Input Data |
|---|---|
| **Country** | Canada |
| **Province** | Ontario |
| **City** | Ottawa |
| **Road Group** | Freeways |
| **Traffic Type** | Expressway |
| **Length** | 3.9 Kilometer |
| **Width** | 14.8 Meter |
| **Number of Lanes Per Direction** | 2 |
| **Traffic Distribution for Major Lane** | 50% |
| **Direction** | Two Way |
| **Annual Average Daily Traffic (AADT)** | 21318 |
| **Service Life (Years)** | 20 for Asphalt; 25 for Concrete |
| **Traffic Growth Rate per Year** | 1.1% |
| **Percentage of 2 Axles Truck in AADT** | 0.95% |
| **Percentage of 3 Axles Truck in AADT** | 0.95% |
| **Percentage of 4 Axles Truck in AADT** | 0.63% |
| **Percentage of 5 Axles Truck in AADT** | 2.84% |
| **Percentage of 6 and more Axles Truck in AADT** | 0.93% |
| **Subgrade Condition** | 25MPa – Fine Grained Soil, Silty Clay |

**Table 0.2 OR-174 Pavement Project Flexible Pavement Design**

| | Material | Thickness |
|---|---|---|
| Surface Course | SP 12.5 FC2 (PG 70-34) | 50 mm |
| Binder Course | SP 19.0 (PG 70-64) | 150 mm |
| Base Couse | Granular A | 150 mm |
| Subbase Course | Granular B | 600 mm |

**Table 0.3 OR-174 Pavement Project Rigid Pavement Design**

| | Material | Thickness |
|---|---|---|
| Surface Course | Portland Cement | 250 mm |
| Reinforcement | 32mm Dowels, 4.5m Joint Spacing | - |
| Base Couse | Granular A | 300 mm |


**Figure 0.1 – OR-174 Pavement Project Limits (TAC 2012)**

The testing process followed the same sequence of the model's step, which is as follows:

1. Pavement project scope: project's type, location, and limits input,
2. Traffic conditions: road group and traffic type selection and truck factor input,
3. Road Information: project's road and traffic information input,
4. ESALs determination: project's ESALs input,
5. PDS: selecting proper pavement designs,
6. SRPD: recommending pavement designs based on future climate conditions,
7. PIM: rendering the selected pavement design as a conceptual model on Revit, and provide step 8 and 9,
8. Conceptual cost estimation: generating the pavement project's conceptual cost estimate at both construction and rehabilitation stages, and
9. Conceptual scheduling analysis: generating the pavement project's conceptual scheduling analysis at construction stage.

## 5.2 Pavement Project Scope

Flexible and rigid pavements have various layer structures and therefore have different design strategies. The first step is to determine the type of pavement required for the project. In this model, conventional flexible and rigid pavements are considered. A drop-down menu is provided to make the selection(s). Meanwhile, because different provinces and cities use different design guidelines, the first step requires entering the location information of the project to determine the basic traffic condition. This involves selecting the province and city from the drop-down menus. Depending on the different pavement types and location inputs, different traffic conditions and pavement design guidelines will be followed. Besides, project limits such as pavement length and width are defined and provided by the users and filled into the text boxes. Figure 5.2 illustrates the data entry for the first step for flexible pavement construction.

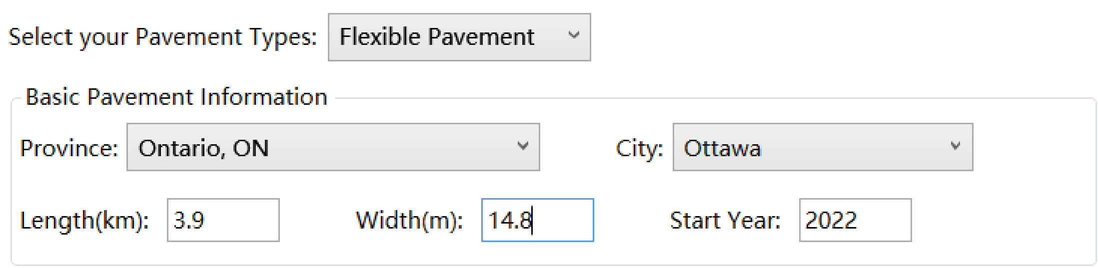



**Figure 0.2 – Entry of Location and Limits Variables**

## 5.3 Traffic Condition

The second step is to enter the traffic conditions in the target area required by the ESAL determination process to determine which guidelines and code criteria are satisfied. This includes the selection of the road group and traffic type from the drop-down list and entering the truck factors for the construction region. Different combinations of selections for the road group and traffic type lead to different road information. The project used in this study is highway construction, freeway and expressway are set for road group and traffic type, respectively. The truck factors are set based on the design guidelines in Ontario. Figure 5.3 illustrates the data input for this step for flexible pavements.

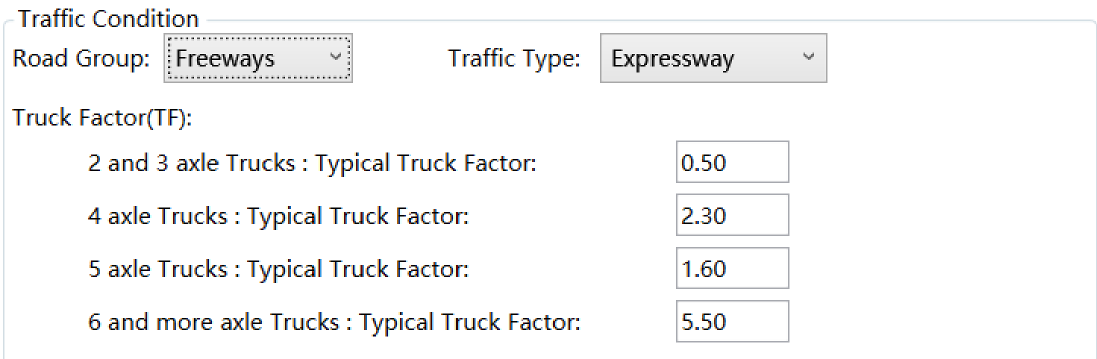



**Figure 0.3 – Traffic Condition Data Entry**

## 5.4 Road Information and ESALs Determination

The third step is to enter the estimated road information, including the Annual Average Daily Traffic (AADT), service life, traffic distribution for major lanes, traffic growth rate per year, and distribution of each type of truck in total AADT in the construction region. Following the selected road group and traffic type, the AADT set for freeways and highways is 15,000. However, the estimated AADT in this case project is proposed at 21,318. Therefore, AADT of 21,318 is entered. The rest of the road information inputs are modified based on the project requirements. The entered road information, along with the previously entered traffic conditions, are considered in the ESALs determination process. The ESALs are calculated by pressing the “Enter” button in the ESALs textbox. Figure 5.4 illustrates the road information that was input for the project and the corresponding ESAL outcome, in case of flexible pavements.

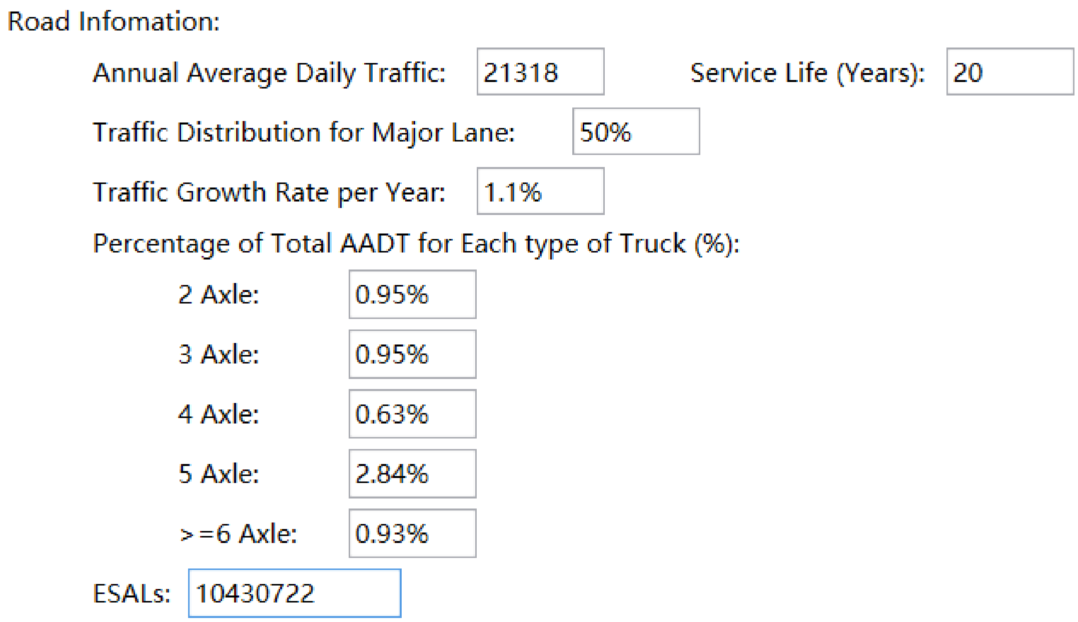



**Figure 0.4 – Road Information Data Entry and ESALs Output**

Once the ESALs and traffic conditions are identified, the information is stored, and the model proceeds to the pavement structure determination process by clicking the “Next” button at the bottom. Figure 5.5 summarizes the data inputs before the flexible pavement structure design.

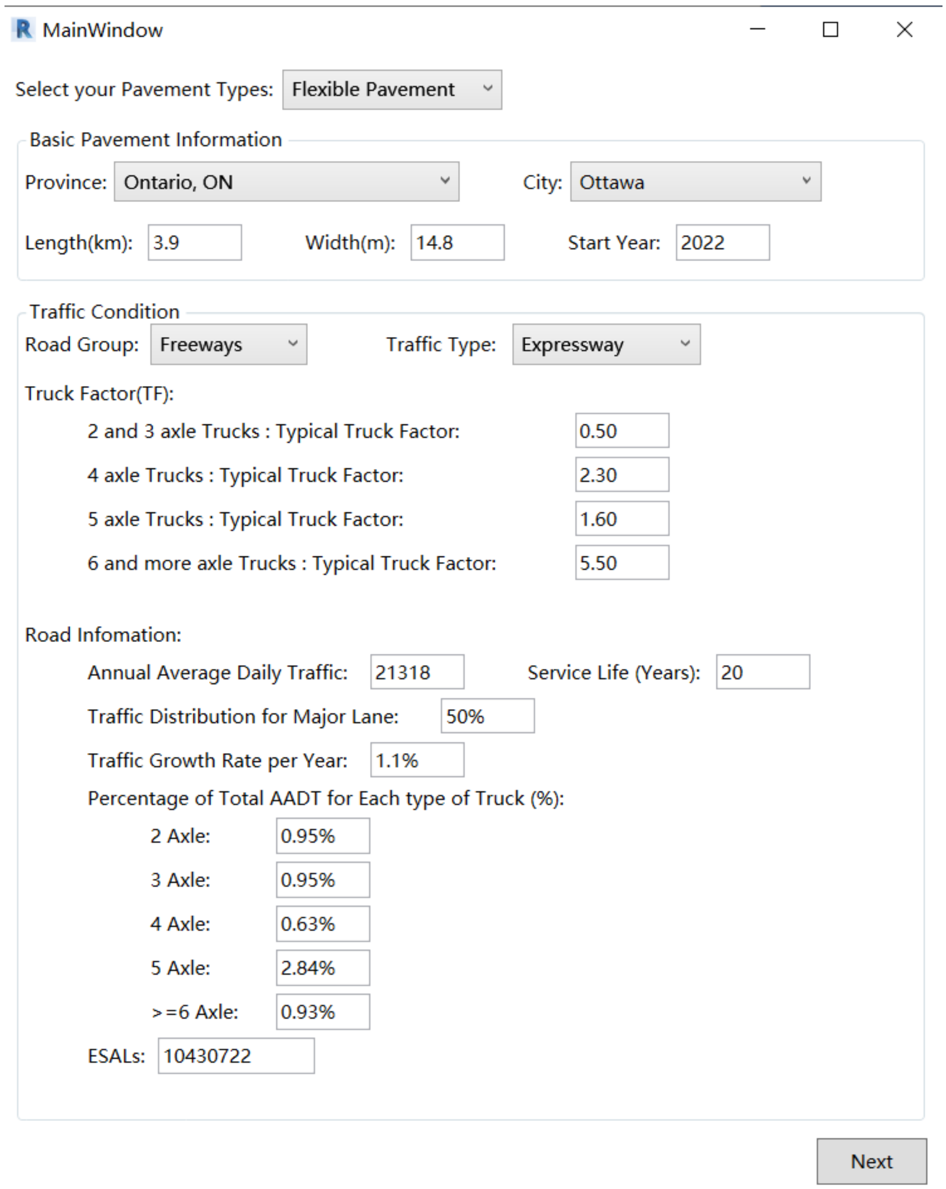



**Figure 0.5 –Summary of Traffic Condition and Road Information Data Entry for Flexible Pavement**

For the design of rigid pavements, the entire input procedure is like the design of flexible pavements, except for the pavement type, truck factors, and service life. First, users are required to change the pavement type to rigid pavement, and the truck factor is changed automatically. The truck factor can be adjusted before calculating the ESAL value. Based on Table 5.1, the service life for rigid pavement is set to be 25 years. Other inputs follow the same procedure performed for flexible pavements, and Figure 5.6 illustrates the data inputs before rigid pavement structure design.

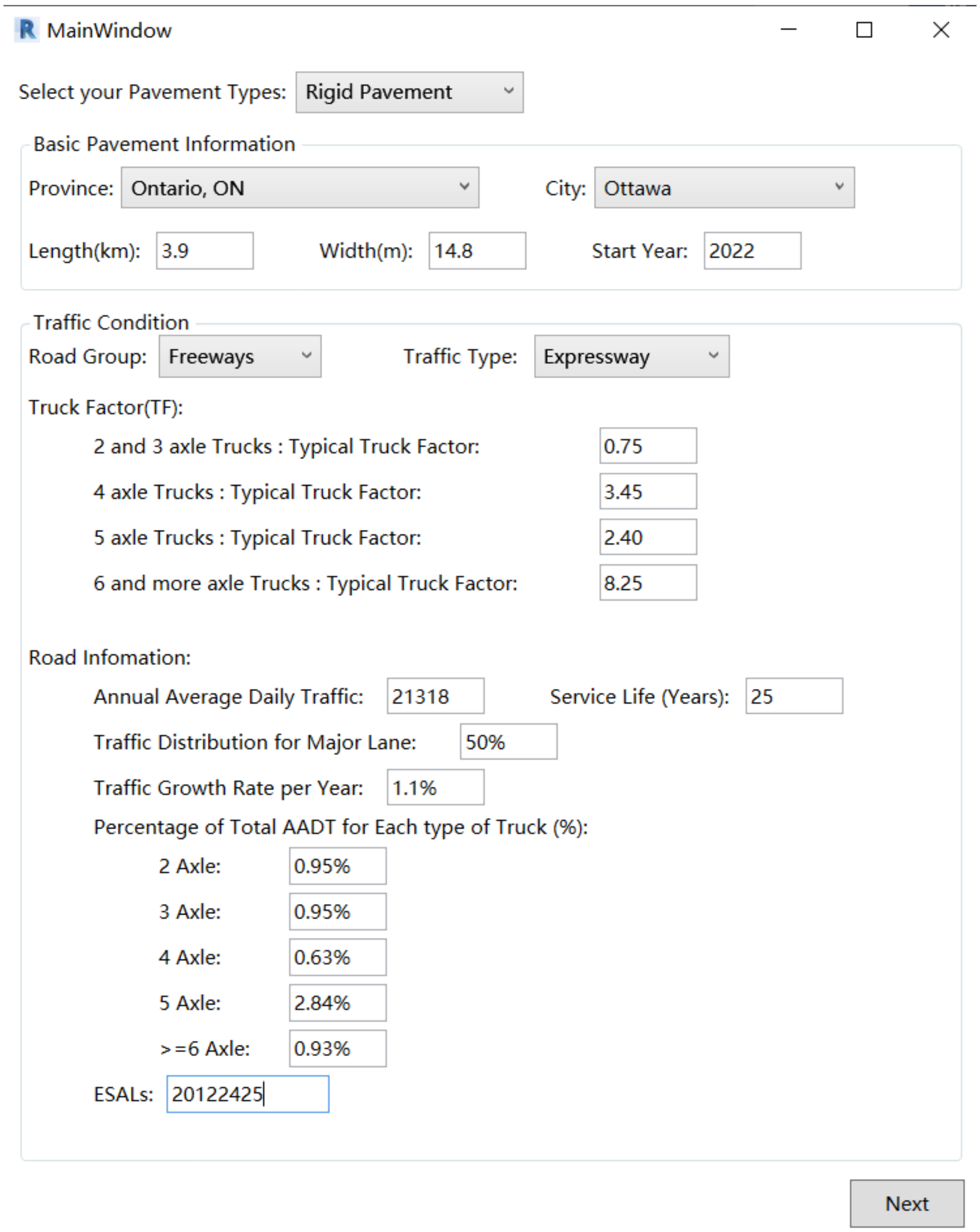



**Figure 0.6 – Summary of Traffic Condition and Road Information Data Entry for Rigid Pavement**

## 5.5 PDS

The fourth step is to select and identify the structural design of the pavement. Before proceeding to the structure design, the soil property of the subgrade needs to be identified by selecting from the drop-down list. Based on different locations, traffic conditions, ESAL results, and soil properties, specific design criteria under certain design guidelines are selected and displayed. Based on the provided project information, 25MPa – Fine Grained Soil is assumed for the subgrade. The model provides minimum pavement designs based on the design guidelines from official documents; however, users may adjust the input values within a certain range based on the design manual. Figure 5.7 illustrates the outputs of the PDS for flexible pavement structure design for this project based on project inputs.

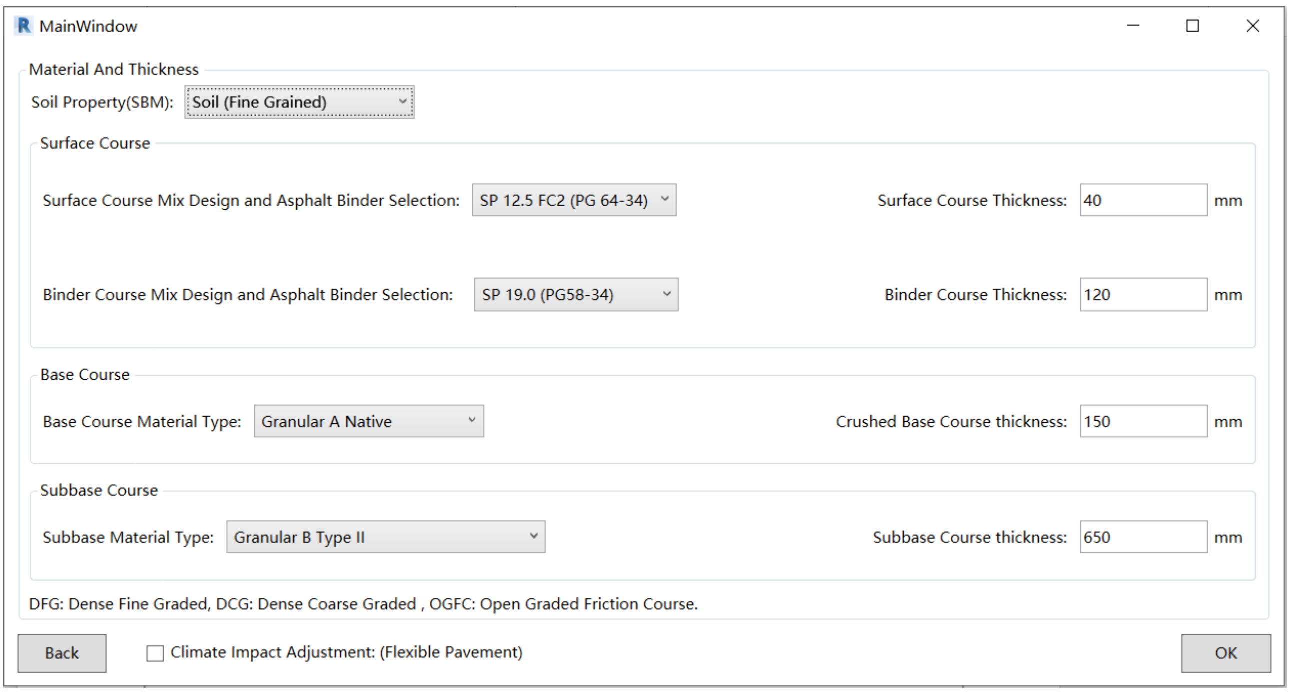



**Figure 0.7 – Pavement Structure Design Data Entry for Flexible Pavement**

For rigid pavement design, the material and thickness design follow the same steps as for flexible pavement design. For reinforcement design, as indicated by the case project, the pavement uses 32M dowels with 4.5 meters transverse spacing. Therefore, JPCP is selected for rigid pavement type, and other reinforcements are designed according to the Ontario design manual since they were not mentioned by the case project in Table 5.3. Figure 5.8 illustrates the inputs of rigid pavement structure design for this project.

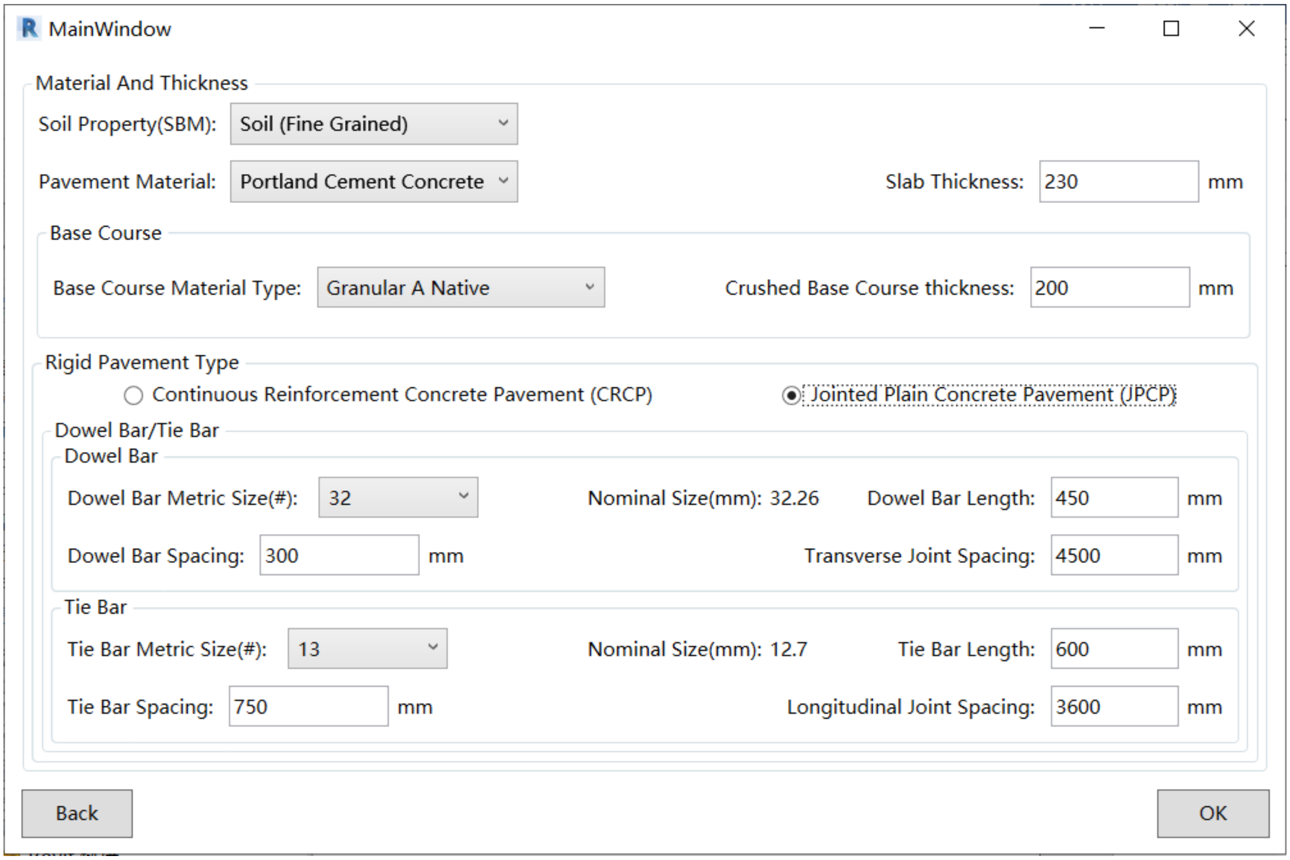


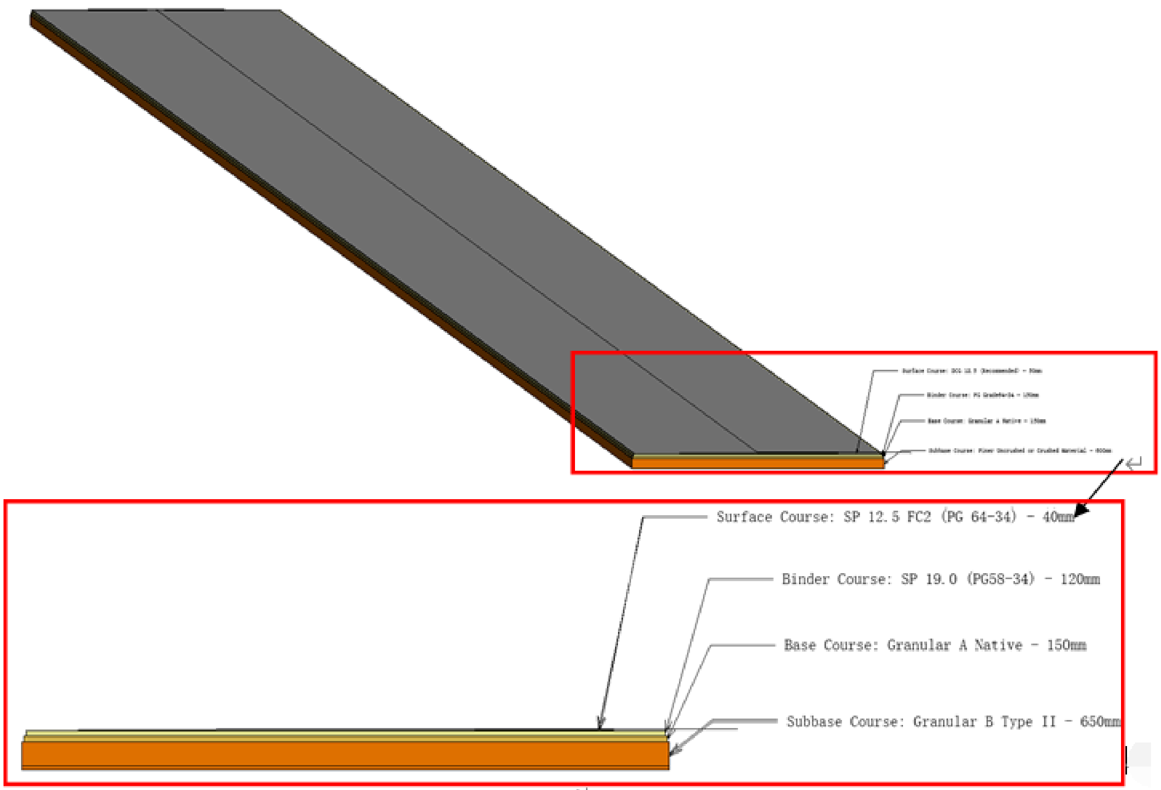
**Figure 0.8 – Pavement Structure Design Data Entry for Rigid Pavement (JPCP)**

## 5.6 SRPD

The fifth step is the SRPD module, which is activated only if users check in the check box at the bottom of the flexible pavement design page when they want to design a sustainable pavement with consideration of future climate change. The material or thickness design that makes the pavement sustainable or resilient would be shown on the pavement structure design input page, as indicated in Figure 5.7. However, the SRPD is only available for flexible pavements where upgrade measures are required. For pavement designs in Ottawa, the material and thickness designs provided by the Ontario Pavement Design Manual were already adapted to climate change by 2040. Therefore, since the construction of the project is planned before 2040, no additional measures are required, and the flexible pavement design remains the same as provided by PDS. Users may go back to the last step and modify the inputs by clicking the "Back" button, as shown in Figures 5.7 and 5.8, and proceed to the PIM module by clicking the "Next" button.

## 5.7 PIM

The sixth step is viewing the conceptual pavement design in Autodesk Revit. Based on the material and thickness design, the 3D model of the conceptual design is generated in Autodesk Revit and shows the design details on the front side view. Figure 5.9 and Figure 5.10 illustrate the outcomes of the PIM module for flexible and rigid pavements, respectively.




8

**Figure 0.9 – Revit Model Presentation for Flexible Pavement**

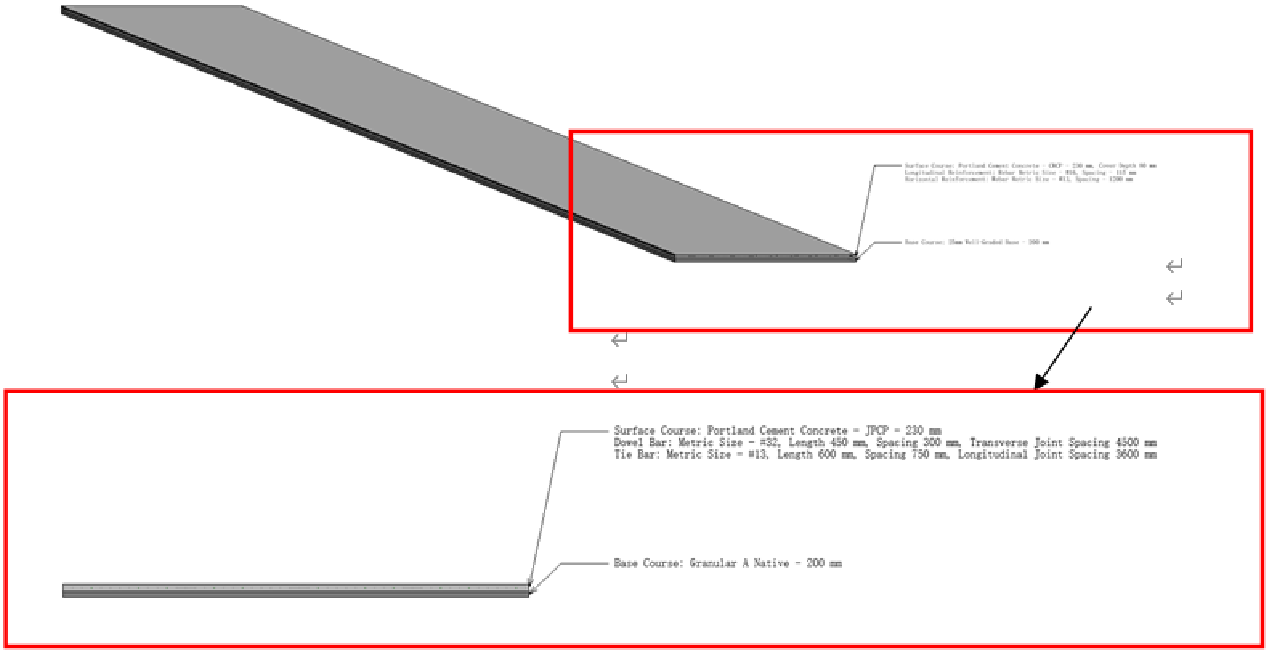



**Figure 0.10 – Revit Model Presentation for Rigid Pavement**

If the current design is not satisfied and a new design is required, users need to close the design page and start a new design by clicking the "3D Modelling" button inserted below the "Parametric Modelling" add-in button in Revit, as shown in Figure 5.11.

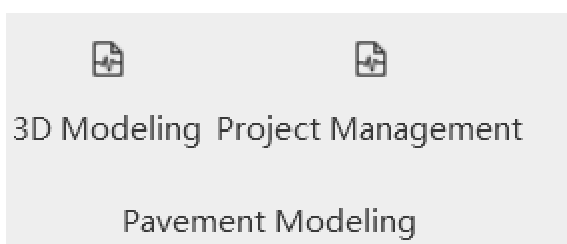



**Figure 0.11 – Model's Initial Interface**

## 5.8 Conceptual Cost Estimation

The seventh step is to use the conceptual cost estimation module to generate the conceptual cost estimates of the pavement project at both the construction stage and the rehabilitation stage. Cost estimates at the construction stage include two components: material costs and labor and equipment costs. The material costs are collected from historical pavement bidding results. To generate the cost estimate for labor and equipment during the construction stage, the working hours per day and working type are required to be determined first, as illustrated in Figure 5.12.

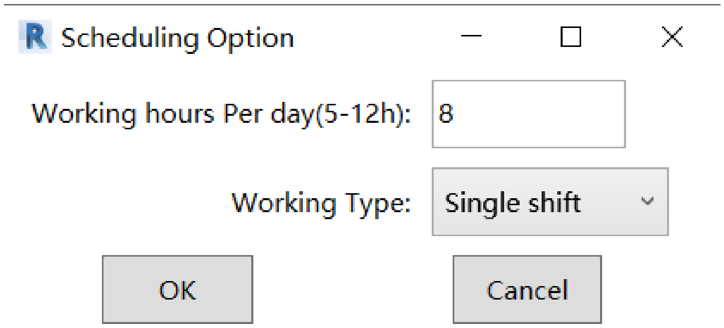



**Figure 0.12 – Scheduling Option Data Entry**

Since the scheduling information is not provided by this case project, the default value of 8 hours and single shift are applied for the working hours per day and working type, respectively.

After clicking the "OK" button, the project is assumed to be constructed at 8 hours per day, single shift, and with previously determined designs. Certain input data is required by the module to estimate the costs. The required data is as follows: 1) Inflation rate; 2) Estimated Pavement Life; 3) Deterioration type; and 4) Early percentage. The inflation rate is assumed to be 3% since it is commonly used in the Canadian industry. For pavement life, as indicated by TAC in Table 5.1, 50 years is applied to the case project. Based on literature review, the pavement design manual for Ottawa overestimates the traffic, which leads to the overdesign of the pavements. With overdesigned pavement, the roads have better performance when facing future climate change. Therefore, a normal deterioration condition is assumed for pavement designs in Ottawa, with a 3% early rehabilitation period. Figure 5.13 illustrates the data entry for the cost estimation module.

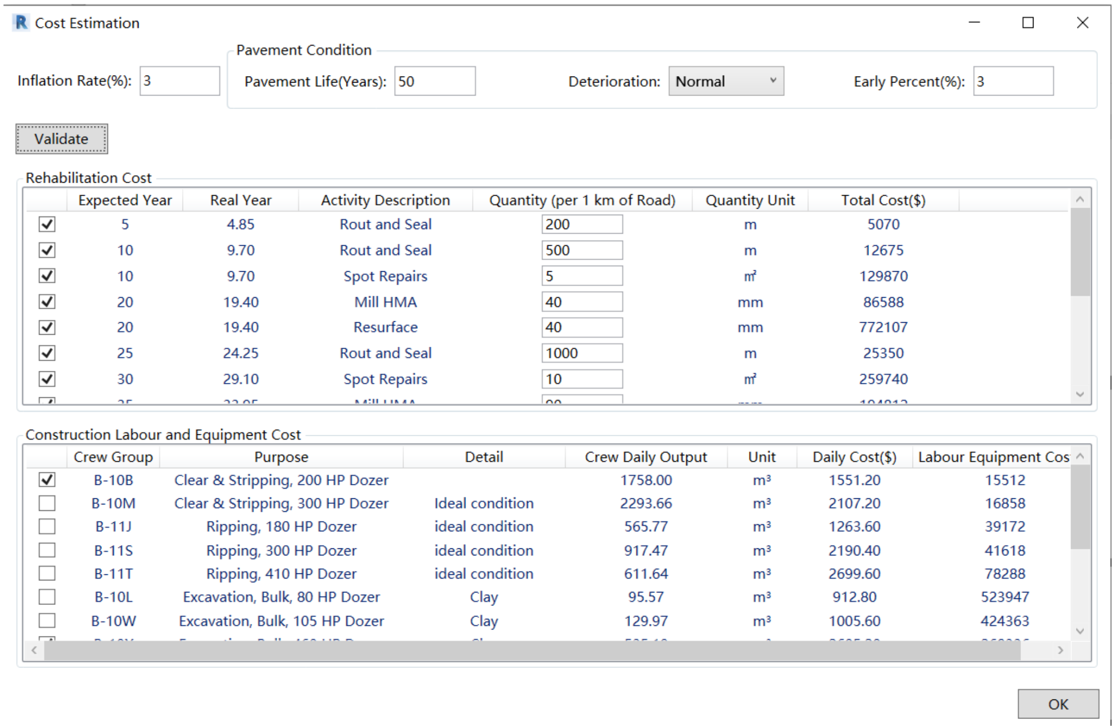



**Figure 0.13 – Cost Estimation Module Data Entry (Flexible Pavement)**

The conceptual cost estimation for rehabilitation and construction is shown in Figure 5.13. For the rehabilitation costs, the total cost ($) column represents the rehabilitation cost for that action. All the rehabilitation activities are assumed to be taken at the corresponding time to preserve the pavement performance. If the results are not accepted, users may modify the deterioration type, early percentage, and rehabilitation quantity per kilometer of road and click the “Validation” button to reach satisfaction. For both cost estimations, users have the option to select whether to take specific action into consideration or not. On the other hand, cost estimation during the construction phase contains the labor and equipment costs for different activities, including clearing and stripping, ripping, excavation, grading, subbase construction, base construction, binder construction, and surface course construction. As illustrated in Figure 5.13, the costs for different equipment types and purposes are gathered based on RS Means and displayed, and the activity under the same category with the lowest cost is selected. For pavement construction in Ottawa, the activities for clearing and stripping, excavation, grading, subbase, base, binder, and surface construction are automatically selected since these are the necessary activities. Since the soil applied in this case project is fine-grained, ripping is not included in the construction. However, based on different requirements, users may select different crew types or change activities by clicking the check box on the left. The cost estimation module for rigid pavements follows the same input steps as for flexible pavements. The input values are accepted, and no modifications are desired, the next step is proceeding to the conceptual scheduling analysis.

## 5.9 Conceptual Scheduling Analysis

The eighth step is to use the conceptual scheduling module to generate the line of balance for the pavement project. The project schedule and LOB chart are generated based on the crew daily output information provided by RS means, as illustrated in Figure 5.13, and working load of the project. The duration of each construction activity is calculated by dividing the working load by the crew's daily output. The activities are assumed to follow certain prerequisites. Based on the prerequisites and activity durations, as illustrated in Figure 5.14, the project LOB chart is generated for the selected construction activities.

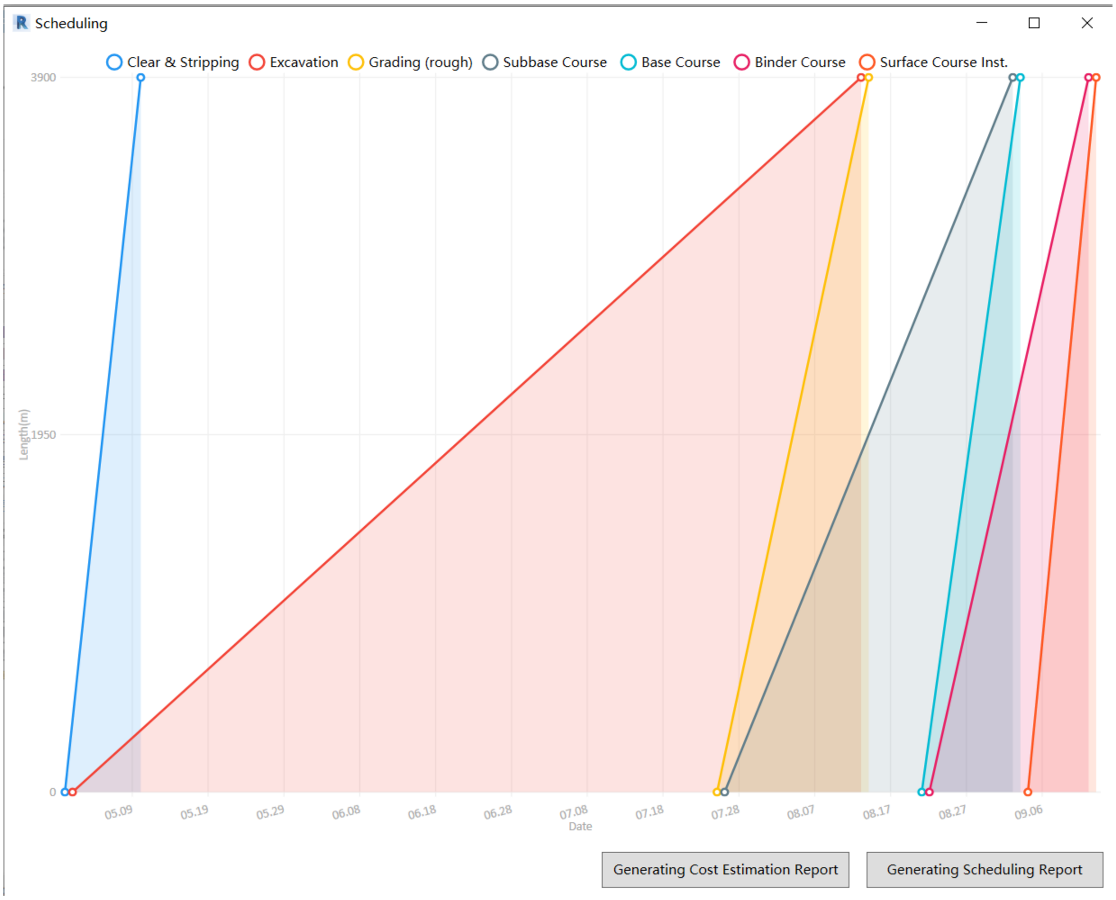
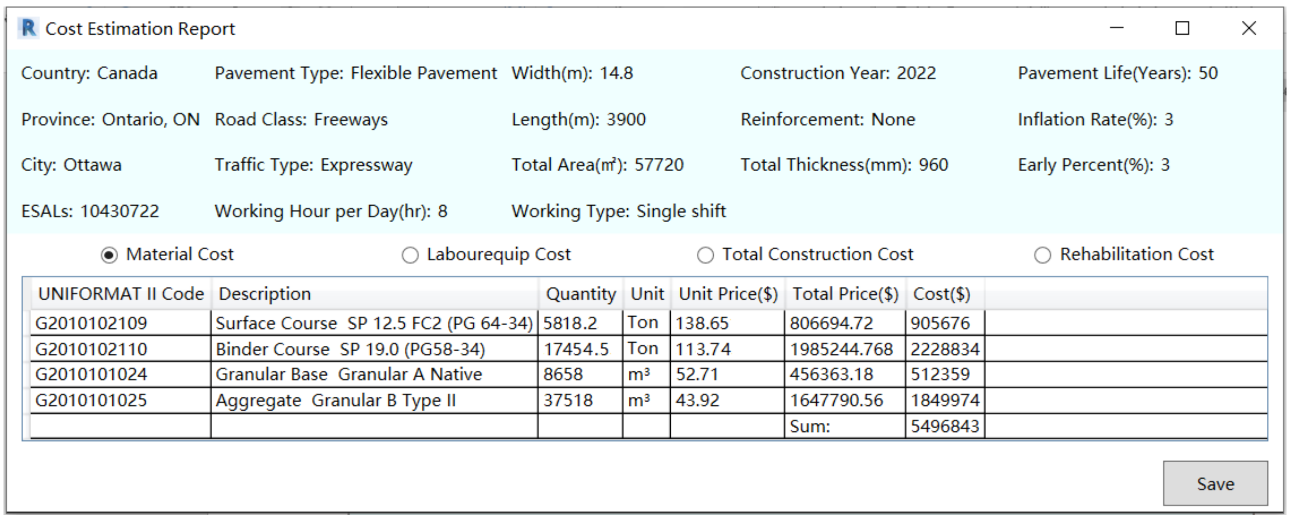



**Figure 0.14 – Scheduling Analysis Module LOB Chart**

The starting time of the construction is assumed to be on May.1st to have the ideal working condition. According to Figure 5.14, for the pavement construction project taken in Ottawa, it takes 10 days to finish the clearing and stripping from May.1st to May. 11th, 104 days to finish the excavation from May.2nd to Aug.14th, 20 days to finish the rough grading from July 26th to Aug.15th, 38 days to finish the subbase installation from July 27th to Sept.3rd, 13 days to finish the base installation from Aug. 22th to Sept. 04th, 21 days to finish the binder course construction from Aug. 23th to Sept. 13th, and 9 days to finish the surface course construction from Sept. 5th to Sept. 14th. The scheduling module for rigid pavements follows the same steps as the flexible pavements, except that the rigid pavement construction does not contain the subbase course and binder course installation.

## 5.10 Conceptual Cost Estimation and Scheduling Reports

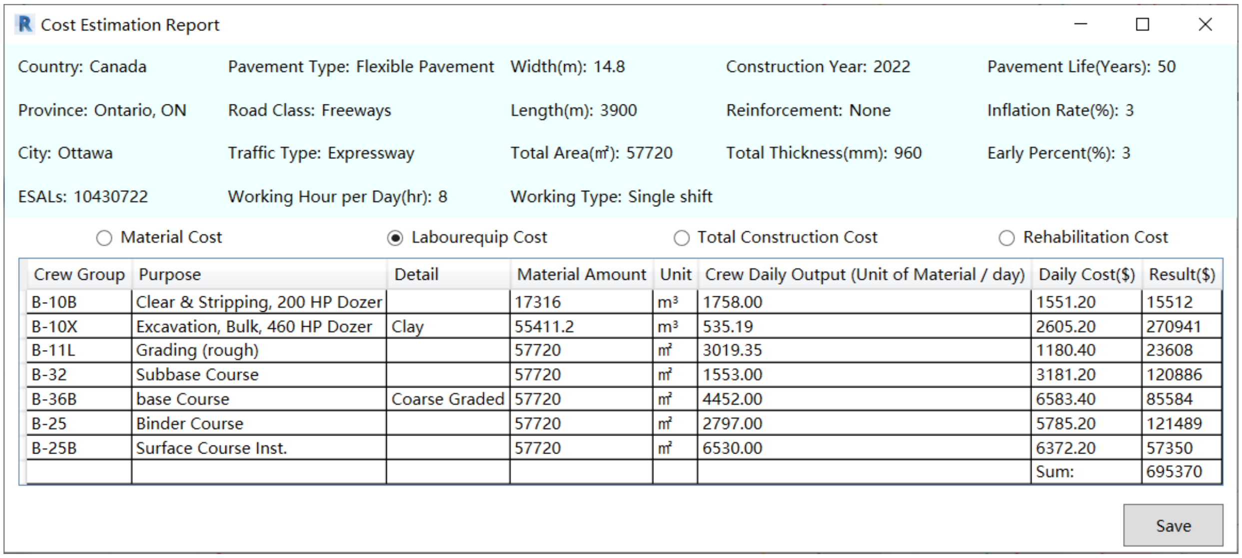
The final step is to generate the conceptual cost estimation and scheduling reports. As shown in Figure 5.14, the user may choose to generate the cost or scheduling reports by clicking the corresponding button. Figure 5.15 shows the conceptual cost estimation for flexible pavement.




| UNIFORMAT II Code | Description | Quantity | Unit | Unit Price($) | Total Price($) | Cost($) |
|---|---|---|---|---|---|---|
| G2010102109 | Surface Course SP 12.5 FC2 (PG 64-34) | 5818.2 | Ton | 138.65 | 806694.72 | 905676 |
| G2010102110 | Binder Course SP 19.0 (PG58-34) | 17454.5 | Ton | 113.74 | 1985244.768 | 2228834 |
| G2010101024 | Granular Base Granular A Native | 8658 | m³ | 52.71 | 456363.18 | 512359 |
| G2010101025 | Aggregate Granular B Type II | 37518 | m³ | 43.92 | 1647790.56 | 1849974 |
| | | | | | Sum: | 5496843 |

**Figure 5.15 – Cost Estimation Report (Material Cost – Flexible Pavement)**

There are four breakdown forms of the cost estimate report that the module generated, along with the project's inputs. Users can switch among four forms by clicking the button. The first form is a material cost estimation report as shown in Figure 5.15, the second form is a labor and equipment cost estimation report as shown in Figure 5.16, the third form is a total construction cost estimation report as shown in Figure 5.17, and the fourth is a rehabilitation cost estimation report as shown in Figure 5.18.



Cost Estimation Report

Country: Canada | Pavement Type: Flexible Pavement | Width(m): 14.8 | Construction Year: 2022 | Pavement Life(Years): 50

Province: Ontario, ON | Road Class: Freeways | Length(m): 3900 | Reinforcement: None | Inflation Rate(%): 3

City: Ottawa | Traffic Type: Expressway | Total Area(㎡): 57720 | Total Thickness(mm): 960 | Early Percent(%): 3

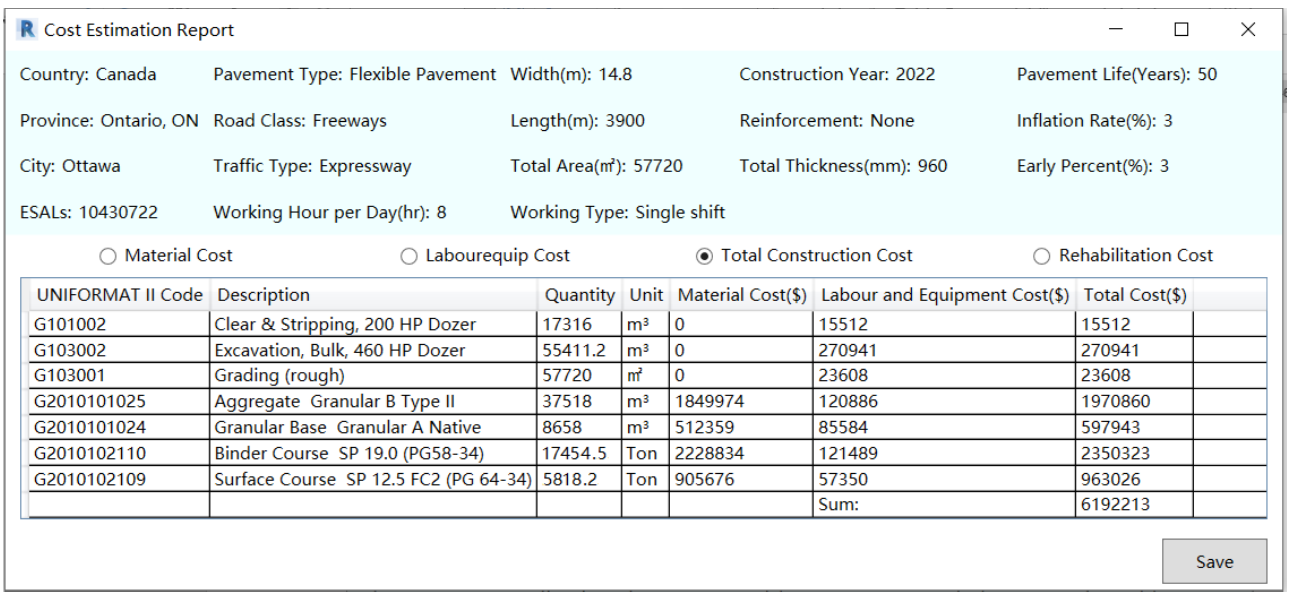
ESALs: 10430722 | Working Hour per Day(hr): 8 | Working Type: Single shift

○ Material Cost ◉ Labourequip Cost ○ Total Construction Cost ○ Rehabilitation Cost

| Crew Group | Purpose | Detail | Material Amount | Unit | Crew Daily Output (Unit of Material / day) | Daily Cost($) | Result($) |
|---|---|---|---|---|---|---|---|
| B-10B | Clear & Stripping, 200 HP Dozer | | 17316 | m³ | 1758.00 | 1551.20 | 15512 |
| B-10X | Excavation, Bulk, 460 HP Dozer | Clay | 55411.2 | m³ | 535.19 | 2605.20 | 270941 |
| B-11L | Grading (rough) | | 57720 | ㎡ | 3019.35 | 1180.40 | 23608 |
| B-32 | Subbase Course | | 57720 | ㎡ | 1553.00 | 3181.20 | 120886 |
| B-36B | base Course | Coarse Graded | 57720 | ㎡ | 4452.00 | 6583.40 | 85584 |
| B-25 | Binder Course | | 57720 | ㎡ | 2797.00 | 5785.20 | 121489 |
| B-25B | Surface Course Inst. | | 57720 | ㎡ | 6530.00 | 6372.20 | 57350 |
| | | | | | | Sum: | 695370 |

Save

**Figure 0.16 – Cost Estimation Report (Labor and Equipment Cost – Flexible Pavement)**



Cost Estimation Report

Country: Canada | Pavement Type: Flexible Pavement | Width(m): 14.8 | Construction Year: 2022 | Pavement Life(Years): 50

Province: Ontario, ON | Road Class: Freeways | Length(m): 3900 | Reinforcement: None | Inflation Rate(%): 3

City: Ottawa | Traffic Type: Expressway | Total Area(㎡): 57720 | Total Thickness(mm): 960 | Early Percent(%): 3

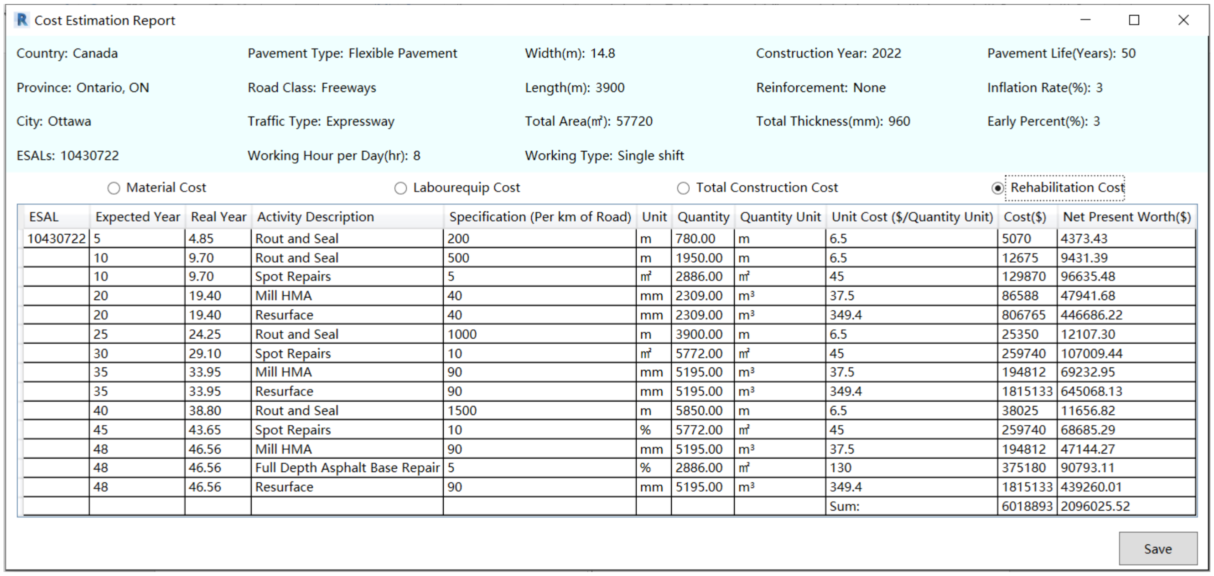
ESALs: 10430722 | Working Hour per Day(hr): 8 | Working Type: Single shift

○ Material Cost ○ Labourequip Cost ◉ Total Construction Cost ○ Rehabilitation Cost

| UNIFORMAT II Code | Description | Quantity | Unit | Material Cost($) | Labour and Equipment Cost($) | Total Cost($) |
|---|---|---|---|---|---|---|
| G101002 | Clear & Stripping, 200 HP Dozer | 17316 | m³ | 0 | 15512 | 15512 |
| G103002 | Excavation, Bulk, 460 HP Dozer | 55411.2 | m³ | 0 | 270941 | 270941 |
| G103001 | Grading (rough) | 57720 | ㎡ | 0 | 23608 | 23608 |
| G2010101025 | Aggregate Granular B Type II | 37518 | m³ | 1849974 | 120886 | 1970860 |
| G2010101024 | Granular Base Granular A Native | 8658 | m³ | 512359 | 85584 | 597943 |
| G2010102110 | Binder Course SP 19.0 (PG58-34) | 17454.5 | Ton | 2228834 | 121489 | 2350323 |
| G2010102109 | Surface Course SP 12.5 FC2 (PG 64-34) | 5818.2 | Ton | 905676 | 57350 | 963026 |
| | | | | | Sum: | 6192213 |

Save

**Figure 0.17 – Cost Estimation Report (Total Construction Cost – Flexible Pavement)**




| ESAL | Expected Year | Real Year | Activity Description | Specification (Per km of Road) | Unit | Quantity | Quantity Unit | Unit Cost ($/Quantity Unit) | Cost($) | Net Present Worth($) |
|---|---|---|---|---|---|---|---|---|---|---|
| 10430722 | 5 | 4.85 | Rout and Seal | 200 | m | 780.00 | m | 6.5 | 5070 | 4373.43 |
| | 10 | 9.70 | Rout and Seal | 500 | m | 1950.00 | m | 6.5 | 12675 | 9431.39 |
| | 10 | 9.70 | Spot Repairs | 5 | $m^2$ | 2886.00 | $m^2$ | 45 | 129870 | 96635.48 |
| | 20 | 19.40 | Mill HMA | 40 | mm | 2309.00 | $m^3$ | 37.5 | 86588 | 47941.68 |
| | 20 | 19.40 | Resurface | 40 | mm | 2309.00 | $m^3$ | 349.4 | 806765 | 446686.22 |
| | 25 | 24.25 | Rout and Seal | 1000 | m | 3900.00 | m | 6.5 | 25350 | 12107.30 |
| | 30 | 29.10 | Spot Repairs | 10 | $m^2$ | 5772.00 | $m^2$ | 45 | 259740 | 107009.44 |
| | 35 | 33.95 | Mill HMA | 90 | mm | 5195.00 | $m^3$ | 37.5 | 194812 | 69232.95 |
| | 35 | 33.95 | Resurface | 90 | mm | 5195.00 | $m^3$ | 349.4 | 1815133 | 645068.13 |
| | 40 | 38.80 | Rout and Seal | 1500 | m | 5850.00 | m | 6.5 | 38025 | 11656.82 |
| | 45 | 43.65 | Spot Repairs | 10 | % | 5772.00 | $m^2$ | 45 | 259740 | 68685.29 |
| | 48 | 46.56 | Mill HMA | 90 | mm | 5195.00 | $m^3$ | 37.5 | 194812 | 47144.27 |
| | 48 | 46.56 | Full Depth Asphalt Base Repair | 5 | % | 2886.00 | $m^2$ | 130 | 375180 | 90793.11 |
| | 48 | 46.56 | Resurface | 90 | mm | 5195.00 | $m^3$ | 349.4 | 1815133 | 439260.01 |
| | | | | | | | | Sum: | 6018893 | 2096025.52 |

**Figure 0.18 – Cost Estimation Report (Rehabilitation Cost – Flexible Pavement)**

As the cost estimate reports exhibit in Figure 5.17, the conceptual cost estimate of constructing the flexible pavement project in year 2022 in Ottawa is $6,192,213. The cost of constructing a flexible pavement for this case project was estimated by the TAC in 2012 to be $5,150,000. A time adjustment is required to obtain the estimated cost in the year 2022, and the time adjustment equation is shown in **Eq. 7** with an assumed 3% inflation rate based on the MTO Parametric Cost Estimating guide. Hence, the TAC estimated cost for the OR-174 flexible pavement in the year 2022 is calculated as follows:

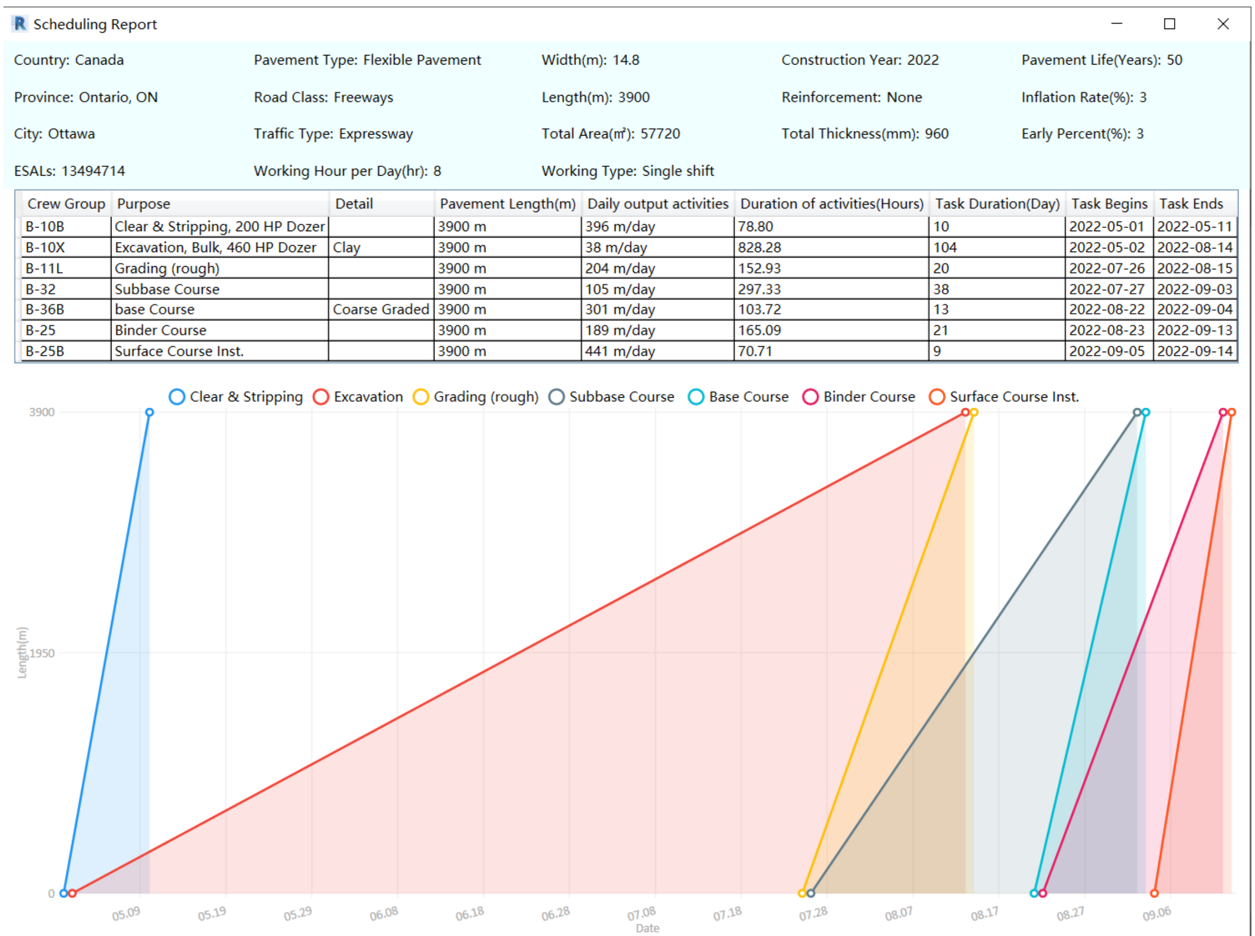
$$\$5,150,000\ (1+0.03)^{10} = \$6,921,169.$$

The project's conceptual cost for flexible pavement in the year 2022 was estimated by the model at $6,192,213. Thus, a comparison of both values indicates that there is a percentage error of 10.5%, which is acceptable for conceptual cost estimation. The difference in total construction cost can be due to the following reasons: 1) the cost estimates performed in 2013 considered additional construction activities that were not included in the model; 2) the difference in the materials' unit cost; 3) the design thickness proposed by the model is thinner than the thickness used in the real case project, which leads to a difference in the quantity of materials.

As shown in Figure 5.18, the net present worth of the conceptual rehabilitation cost for the flexible pavement project in the year 2022 is $2,096,026, whereas the rehabilitation cost estimated by the TAC in 2012 was $1,500,000. By applying the same time adjustment method as performed for flexible pavement cost estimation, the rehabilitation cost estimated by TAC in 2022 is calculated as follows:

$$1{,}500{,}000\ (1+0.03)^{10} = \$2{,}015{,}875.$$

The project's conceptual rehabilitation cost for flexible pavement in the year 2022 was estimated by the model at $2,096,026. Thus, a comparison of both values indicates that the percentage of error is 4%, which is ideal for conceptual rehabilitation cost estimation. This model's rehabilitation cost estimation is still viable for use in other projects with high reliability.

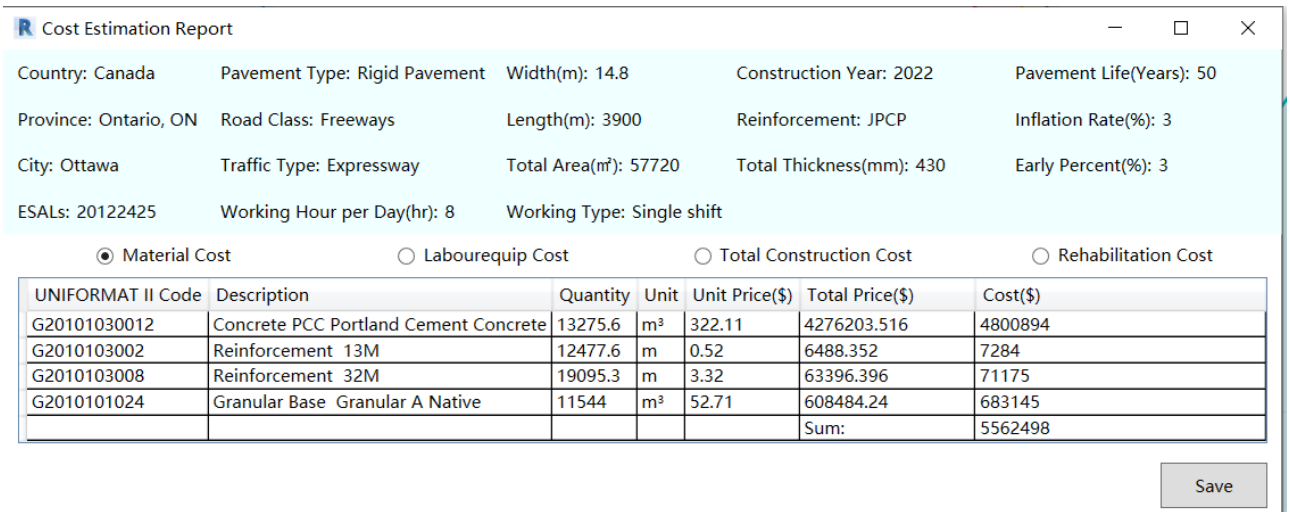



| Crew Group | Purpose | Detail | Pavement Length(m) | Daily output activities | Duration of activities(Hours) | Task Duration(Day) | Task Begins | Task Ends |
|---|---|---|---|---|---|---|---|---|
| B-10B | Clear & Stripping, 200 HP Dozer | | 3900 m | 396 m/day | 78.80 | 10 | 2022-05-01 | 2022-05-11 |
| B-10X | Excavation, Bulk, 460 HP Dozer | Clay | 3900 m | 38 m/day | 828.28 | 104 | 2022-05-02 | 2022-08-14 |
| B-11L | Grading (rough) | | 3900 m | 204 m/day | 152.93 | 20 | 2022-07-26 | 2022-08-15 |
| B-32 | Subbase Course | | 3900 m | 105 m/day | 297.33 | 38 | 2022-07-27 | 2022-09-03 |
| B-36B | base Course | Coarse Graded | 3900 m | 301 m/day | 103.72 | 13 | 2022-08-22 | 2022-09-04 |
| B-25 | Binder Course | | 3900 m | 189 m/day | 165.09 | 21 | 2022-08-23 | 2022-09-13 |
| B-25B | Surface Course Inst. | | 3900 m | 441 m/day | 70.71 | 9 | 2022-09-05 | 2022-09-14 |

**Figure 0.19 – Scheduling Report (Flexible Pavement)**

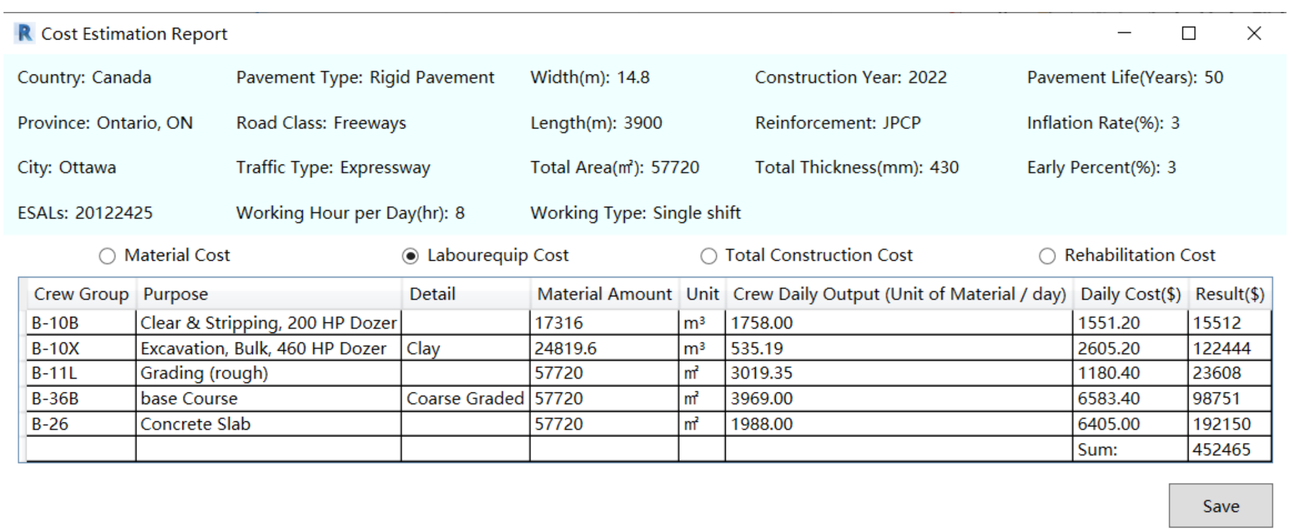
As illustrated in Figure 5.19, the duration of each activity and the schedules are presented, and a LOB chart is formed based on the scheduling information. For the OR-174 flexible pavement construction, the construction start date is set for May.1st, 2022, since it is the ideal time for construction to avoid the impact of the Canadian climate. The total construction duration is 215 days, and the ending time of the construction is estimated to be on Sept.14th, 2022. The construction duration is estimated based on crew daily output information provided by the RS means database. However, the real construction duration is confidential, and due to weather, traffic, and other force majeure factors, the real construction duration is longer than the estimated schedule.

On the other hand, by using the same modelling method, the cost estimation and scheduling reports for OR-174 rigid pavement are generated as illustrated from Figure 5.20 to Figure 5.23.




| UNIFORMAT II Code | Description | Quantity | Unit | Unit Price($) | Total Price($) | Cost($) |
|---|---|---|---|---|---|---|
| G20101030012 | Concrete PCC Portland Cement Concrete | 13275.6 | m³ | 322.11 | 4276203.516 | 4800894 |
| G2010103002 | Reinforcement 13M | 12477.6 | m | 0.52 | 6488.352 | 7284 |
| G2010103008 | Reinforcement 32M | 19095.3 | m | 3.32 | 63396.396 | 71175 |
| G2010101024 | Granular Base Granular A Native | 11544 | m³ | 52.71 | 608484.24 | 683145 |
| | | | | | Sum: | 5562498 |

**Figure 0.20 – Cost Estimation Report (Material Cost – Rigid Pavement)**




| Crew Group | Purpose | Detail | Material Amount | Unit | Crew Daily Output (Unit of Material / day) | Daily Cost($) | Result($) |
|---|---|---|---|---|---|---|---|
| B-10B | Clear & Stripping, 200 HP Dozer | | 17316 | m³ | 1758.00 | 1551.20 | 15512 |
| B-10X | Excavation, Bulk, 460 HP Dozer | Clay | 24819.6 | m³ | 535.19 | 2605.20 | 122444 |
| B-11L | Grading (rough) | | 57720 | ㎡ | 3019.35 | 1180.40 | 23608 |
| B-36B | base Course | Coarse Graded | 57720 | ㎡ | 3969.00 | 6583.40 | 98751 |
| B-26 | Concrete Slab | | 57720 | ㎡ | 1988.00 | 6405.00 | 192150 |
| | | | | | | Sum: | 452465 |

**Figure 0.21 – Cost Estimation Report (Labor Cost – Rigid Pavement)**

Cost Estimation Report

Country: Canada | Pavement Type: Rigid Pavement | Width(m): 14.8 | Construction Year: 2022 | Pavement Life(Years): 50

Province: Ontario, ON | Road Class: Freeways | Length(m): 3900 | Reinforcement: JPCP | Inflation Rate(%): 3

City: Ottawa | Traffic Type: Expressway | Total Area(m²): 57720 | Total Thickness(mm): 430 | Early Percent(%): 3

ESALs: 20122425 | Working Hour per Day(hr): 8 | Working Type: Single shift

○ Material Cost ○ Labourequip Cost ◉ Total Construction Cost ○ Rehabilitation Cost

| UNIFORMAT II Code | Description | Quantity | Unit | Material Cost($) | Labour and Equipment Cost($) | Total Cost($) |
|---|---|---|---|---|---|---|
| G101002 | Clear & Stripping, 200 HP Dozer | 17316 | m³ | 0 | 15512 | 15512 |
| G103002 | Excavation, Bulk, 460 HP Dozer | 24819.6 | m³ | 0 | 122444 | 122444 |
| G103001 | Grading (rough) | 57720 | m² | 0 | 23608 | 23608 |
| G2010101024 | Granular Base Granular A Native | 11544 | m³ | 683145 | 98751 | 781896 |
| G20101030012 | Concrete PCC Portland Cement Concrete | 13275.6 | m³ | 4800894 | 192150 | 4993044 |
| | | | | | Sum: | 5936504 |

Save

**Figure 0.22 – Cost Estimation Report (Total Construction Cost – Rigid Pavement)**

Cost Estimation Report

Country: Canada | Pavement Type: Rigid Pavement | Width(m): 14.8 | Construction Year: 2022 | Pavement Life(Years): 50

Province: Ontario, ON | Road Class: Freeways | Length(m): 3900 | Reinforcement: JPCP | Inflation Rate(%): 3

City: Ottawa | Traffic Type: Expressway | Total Area(m²): 57720 | Total Thickness(mm): 430 | Early Percent(%): 3

ESALs: 20122425 | Working Hour per Day(hr): 8 | Working Type: Single shift

○ Material Cost ○ Labourequip Cost ○ Total Construction Cost ◉ Rehabilitation Cost

| ESAL | Expected Year | Real Year | Activity Description | Specification (Per km of Road) | Unit | Quantity | Quantity Unit | Unit Cost ($/Quantity Unit) | Cost($) | Net Present Worth($) |
|---|---|---|---|---|---|---|---|---|---|---|
| 20122425 | 12 | 11.64 | Reseal Joints | 25 | m | 7894.00 | m | 10 | 78940 | 55366.93 |
| | 12 | 11.64 | Partial Depth PCC repair | 2 | m² | 385.00 | m² | 175 | 67375 | 47255.47 |
| | 25 | 24.25 | Partial Depth PCC repair | 5 | m² | 962.00 | m² | 175 | 168350 | 80404.90 |
| | 25 | 24.25 | Full Depth PCC repair | 10 | m² | 5772.00 | m² | 150 | 865800 | 413510.90 |
| | 25 | 24.25 | Reseal Joints | 25 | m | 7894.00 | m | 10 | 78940 | 37702.18 |
| | 40 | 38.80 | Partial Depth PCC repair | 5 | % | 962.00 | m² | 175 | 168350 | 51608.84 |
| | 40 | 38.80 | Full Depth PCC repair | 15 | % | 8658.00 | m² | 150 | 1298700 | 398125.37 |
| | 40 | 38.80 | Reseal Joints | 25 | % | 7894.00 | m | 10 | 78940 | 24199.60 |
| | | | | | | | | Sum: | 2805395 | 1108174.19 |

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**Figure 5.23 – Cost Estimation Report (Rehabilitation Cost – Rigid Pavement)**

The conceptual cost estimate to construct the rigid pavement project at Ottawa in 2022 is $5,936,504, as shown in the cost estimate reports (Figure 5.22). For this case project, the TAC estimated the cost in 2012 of the rigid pavement would cost $4,970,000. To get the estimated cost in 2022, a time adjustment is needed, just like for flexible pavement estimation. Therefore, the following is the TAC anticipated cost for the OR-174 rigid pavement in 2022:

$$\$4,970,000\ (1+0.03)^{10} = \$6,679,264.$$

The model estimated that the project's conceptual construction cost for rigid pavement in 2022 would be $5,936,504. Given this, comparing the two outcomes shows that the percentage of error is 11.1%, which is acceptable for conceptual cost estimating. The difference in the cost can be reflective of various factors. The material unit cost is collected based on historical pavement bidding results. Based on the comparison between the material unit cost applied in the model and the unit cost proposed by TAC, there might be an underestimation for the unit cost of the base courses of the rigid pavement. Besides, the construction costs proposed by TAC in 2012 contain extra costs including the cost of cross over, traffic staging, and 10% increase in cost for nighttime work.

Figure 5.23 shows that the net present value of the notional rehabilitation cost for the rigid pavement project, in 2022, is $1,108,174, while the TAC's 2012 rehabilitation cost estimate was $1,000,000. Using the same temporal adjustment method, the TAC-estimated rehabilitation cost for 2022 is computed as:

$$1{,}000{,}000\ (1+0.03)^{10} = \$1{,}343{,}916.$$

The model indicated that the conceptual rehabilitation cost of the project for rigid pavement in 2022 would be $1,108,174. As a result, a comparison of the two numbers shows the percentage of error is 17.5%, which is appropriate for conceptual rehabilitation cost estimation. There are several reasons behind the differences in the cost. The rehabilitation amount and the maintenance duration suggested by TAC in 2012 can be changed, which results in a variation in cost, as the rehabilitation cost information is private.

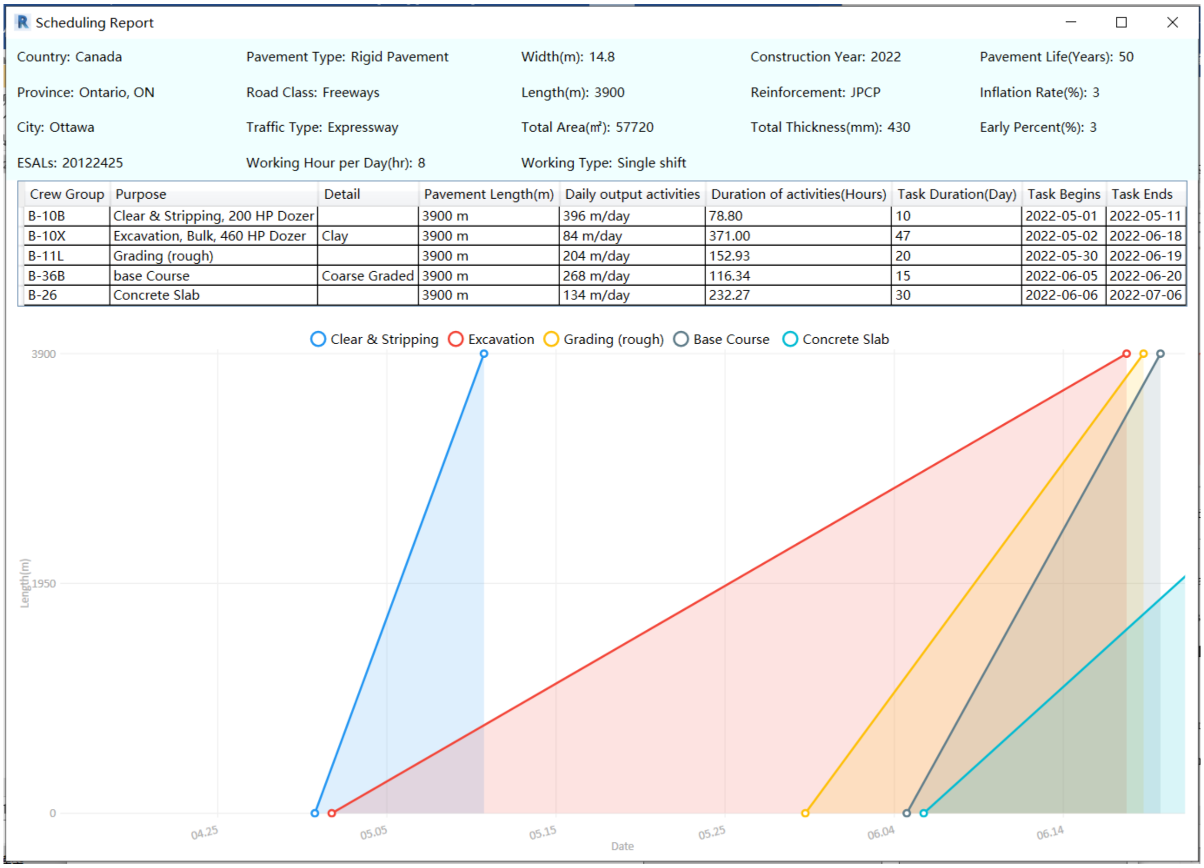


**Figure 5.24 – Scheduling Report (Rigid Pavement)**

As illustrated in Figure 5.24, like flexible pavement scheduling analysis, the construction start date of OR-174 rigid pavement is set for May.1st, 2022. The total construction duration is 122 days, and the ending time of the construction is estimated to be July.6th, 2022. However, due to weather, traffic, and other force majeure factors, the real construction duration can be longer than the estimation.

When verifying the cost and scheduling estimation, as shown in Figure 5.23, the cost estimation and scheduling results can be exported and summarized into an Excel spreadsheet by clicking the “Save” button for future analysis.

## 5.11 Summary

This chapter covered the steps taken to test the model. A pre-existing project was chosen, and the modules processed the input data for it. Each module's viability was examined in terms of producing the intended output, the data flow's logical sequence, and the analysis of the findings. Both flexible and rigid pavement designs were created using the traffic and structural data from the OR-174 pavement, which was chosen as the current pavement project. For both flexible and rigid pavements, the model offered up-to-date design methodologies together with sustainable design possibilities. Additionally, a user-friendly platform is suggested, enabling users to alter the suggested designs in accordance with their needs. The PIM module carried out the conceptual pavement design visualization and presented the design information in Autodesk Revit. Additionally, the conceptual cost estimating module creates reports for the material, labor, and equipment costs as well as the total construction and rehabilitation costs of the pavement project depending on the pavement design, rehabilitation timetable, and crew arrangement. In addition, a construction schedule was predicted for the pavement project based on the project scale and crew details. The scheduling data was combined with a LOB chart to create a scheduling report.

# Chapter Six

## Conclusion and Future Recommendations

### 6.1 Conclusion

The idea behind Pavement Information Modelling (PIM) is to use technological developments to improve the processes that are engaged in each stage of a pavement project's life cycle. Additionally, PIM can effectively and efficiently handle all the information associated with a pavement project and improve the communication among all the parties involved. This capacity is beneficial when incorporating sustainability concerns into a project's design, particularly when there is a greater flow of information. Concerning pavement projects, the literature is lacking detailed information about sustainability. The process of designing, visualizing, and managing a project is improved when sustainability design, LCCA concepts, and PIM concepts are integrated throughout the conceptual design phase as well as during the maintenance and rehabilitation phases. The methodology used in this thesis explained the creation and integration of an integrated model that incorporates three new modules to apply the PIM idea and offer a sustainable pavement project design approach. The model is comprised of six modules, which are: 1) an ESAL determination module; 2) a pavement design selection (PDS) module; 3) a sustainable and resilient pavement design (SRPD) module; 4) a pavement information modeling (PIM) module; 5) a conceptual cost estimation module; and 6) scheduling module. The ESAL determination module estimates the traffic conditions and ESALs using pre-established parameters and logic sequences based on different calculation formulas, guidelines, and traffic data over Canada. The PDS module makes recommendations for pavement designs based on the geographic information of the project and the result of the ESAL determination module, in accordance with different pavement design rules and design handbooks. Pavement designs produced by the PDS module can be modified, and climate adaptation solutions can be found in the SRPD methodology. The PIM module uses the dimension outputs and paving information from the PDS and SRPD modules to provide a 3D pavement model in Autodesk Revit, which is selected to enhance the project's visualization and design. The module for conceptual cost estimation computes the conceptual expenses of pavement projects by considering several factors such as paving materials, rehabilitation and maintenance, and equipment and personnel costs. Construction schedules and plans in the form of LOB charts are provided by the scheduling module. To enhance the project management procedure, the cost and schedule data are compiled into tables and presented to the users in different reporting formats.

The advantages of the developed model include:

1. Generating a conceptual pavement design for both rigid and flexible pavements with MEPDG consideration based on rudimentary inputs gives users the option to modify the given design within a certain level.
2. Integrating sustainable pavement design strategies for flexible pavements.
3. Visualizing the conceptual pavement in 3D for enhanced project design visualization and animation.
4. Providing routine maintenance plans and adjustment options for both types of pavements.
5. Generating conceptual cost estimates for construction and maintenance to provide comparison between various design options.
6. Generating a LOB schedule for the construction sequence to minimize delay.
7. Providing professional summary reports and documentation.

## 6.2 Research Contribution

The approach is meant to make the process of conceptually developing pavement projects simple and straightforward. The model can help stakeholders to speed up the process of generating conceptual designs, maintenance schedules, and cost estimates during the feasibility studies, as well as furnishing dependable results for informed decision-making. Consequently, the research contributions are:

1. The development of a simplified and complexity-understandability-balanced pavement design procedure that is implemented into an integrated model that provides users the options to conceptually design pavement projects in Canada. Also, the model integrates sustainable flexible pavement design approaches into the pavement design procedure to improve the pavement performance based on future climates.
2. The creation of a PIM module within BIM environment via Autodesk Revit through set of C# algorithms to accomplish the 3D visualization of the generated conceptual pavement parameters.
3. The execution of maintenance plans, LCCA, and construction schedules of the pavement projects and integrating them with the developed cost estimation and scheduling module then connecting them to the PIM module to facilitate the decision-making process.

## 6.3 Research Limitation

The developed model has the following limitations:

- The model provides only estimation and modeling for the projects initiated after the year 2021.

- The ESAL determination module is limited to truck factors and road information that used assumptions based on historical data and do not reflect the actual traffic information for specific projects; and ESALs of small vehicles are not included in the calculation procedures.
- The PDS module considers only conventional flexible pavement and rigid pavement designs for carriageways, other elements such as shoulders and drainage systems are not included in the model.
- The PDS module is limited to the construction locations. For most cases, different cities within a same province are assumed to follow the same design procedures.
- The SRPD module only provide adaptation strategies for flexible pavements.
- The SRPD module is integrated based on the RCP 8.5, where other climate scenarios are not considered.
- The PIM module is a one-way procedure that cannot modify the design parameters of the 3D model visualized in Autodesk Revit.
- The maintenance plans for rigid pavement only considers JPCP; and the model can only provide maintenance plans for pavements with a maximum design life of 50 years.
- The maintenance plans are provided based on studies performed by CAC and ARA, it only considered the pavement performance under certain conditions. The plans are recommended without calculating the stress and strain of the target pavement section, there is a limitation in determining the real service life of different pavement designs.
- The cost estimation module is based on compiled cost data of previous projects and RS Means cost data published in past years.
- The scheduling module only considers main construction steps required for the project, and the module provides limited crew types for each step; and the scheduling module is based on the RS Means construction cost data published in year 2014.

## 6.4 Future Expansion & Recommendation

Although the developed model has improved its capabilities, there is always potential for improvement as it is not flawless. The list of the suggested expansion and recommendations to enhance the model consists of:

1. Expand the algorithm of PDS to include more design guidelines and typical designs at various locations that covers designs under different traffic and climate conditions; and integrate the design specifications into a database format allowing the capability for future updates.
2. Consider additional sustainable structures and materials in SRPD module to provide extra design options and to integrate climate adaptation strategies for rigid pavements.
3. Improve the PIM module by creating a two-way communication between the model and Autodesk Revit for rapid exchange of information to accordingly ease the modification process.
4. Enhance Autodesk Revit templates to display greater levels of detail for superior visualization capabilities.
5. Update the database of the cost estimation and scheduling module to improve the reliability and dependency of the output reports.
6. Integrate MEPDG in pavement thickness design to increase the accuracy of design results and to increase the reliability of maintenance plans.

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