ASSESSING SPRING LOAD RESTRICTIONS USING CLIMATE CHANGE AND MECHANISTIC-EMPIRICAL DISTRESS MODELS

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ABSTRACT

During the spring, as the average daily air temperature and declination of the sun increases, so does the temperature of the asphalt surface layer. As the increase in temperature travels through the surface layer and reaches the unbound aggregate base layer, the saturated base layer begins to thaw. For a flexible pavement with a fine-grained subgrade, the moisture in the base becomes trapped between the impermeable asphalt concrete layer and the frozen, fine-grained subgrade.

The excess water requires a long period of time to properly drain. During this period, the subgrade becomes considerably softer and decreases in stiffness, and the roadway experiences an increase in damage and reduction in service life. The duration of thaw for a typical pavement structure depends on soil type, moisture and thermal properties, air temperature, solar radiation, drainage and the location of the site. Spring load restrictions are applied to protect weak pavement structures that would otherwise experience excessive deflections. Nearly 58% of the Manitoba Provincial pavement network is subjected to spring load restrictions, and most of these roads consist of a thin flexible pavement or an asphalt surface treatment.

This paper relates pavement deflection data from FWD testing to environmental indices such as the thaw index. Deflection data collected since 1990 on pavement sections and the LTPP site in Manitoba are used to establish network-level and statistically representative values for pavement properties during the thaw weakening and recovery period.

The base and subgrade moduli during spring thaw are computed using a back-calculation algorithm and categorized in terms of ranges of the thaw index. The data is used with the prediction models of the AASHTO Mechanistic Empirical Design Guide to assess the impact of spring load restrictions on pavement service life.

Five scenarios are considered and these accounted for base conditions on an unrestricted road and for the cases of reducing axle loads, with and without an increase in the number of trips, required to transport a certain payload. An economic analysis on a typical pavement structure reveals that the cost of removing the restrictions can be assessed using the service life predicted by the models.

The results indicate that, in some cases, spring load restrictions may in fact adversely impact pavement life. An understanding of the relationship between axle loads and generated traffic volumes is necessary in order to optimize the utilization of the network and protect investment in infrastructure.

1.0 INTRODUCTION

1.1 Dependence of Infrastructure on Climate

Movement of passengers and freight by roads, railways, airports, seaways and bridges is a vital part of our economy and society. Transportation industries account for approximately 4% of Canada's gross domestic product and employ more than 800,000 people [1]. Planning, design, construction, maintenance and performance of this infrastructure is affected by weather and climate change throughout its service life.

By the end of this century, Manitoba will be 4 to 6 degrees Celsius warmer, on average, than it is today [2]. This change in temperature will have impacts on our society, economy and health. The extensive system of 18,000 km of roads and 2,400 bridges and structures in the province have an estimated replacement cost of $6.6 billion. As well, this transportation network is a vital component of Manitoba's economy, having exported an average of $8.8 billion annually from 1990 to 1999 [3].

Deterioration rates for asphalt pavements will increase with warmer temperatures and more frequent thermal cycles. Increased severity and frequency of higher temperatures could lead to more rutting and structural problems on roads. Further, an increase in thermal cycles could lead to more freeze-thaw pavement cracking.

During thaw periods, excess water in the pavement structure cannot properly drain and thus the base layer becomes considerably softer and decreases in stiffness. It is during this period of saturation that the roadway experiences an increase in damage and reduction in service life. The duration of thaw for a typical pavement structure depends on soil type, moisture and thermal properties, air temperature, solar radiation, drainage and the location of the site.

1.2 Manitoba Road Network and LTPP Monitoring Sites

Manitoba Infrastructure and Transportation (MIT) oversees a provincial road network consisting of over 18,000 kilometres of various pavement and road types. The majority of the network is bituminous pavement overlaying granular subbase. For planning purposes, the provincial highway network is divided into approximately 1,500 control sections that are homogenous with respect to traffic volumes. The provincial network breakdown is shown in Table 1. It should be noted that the term AST refers to an Asphalt Surface Treated pavement where a thin layer of stone chips is placed upon a prepared granular surface.

Table 1 – Pavement Type & Distance (2-lane km) vs. Classification [4]

Classification	Gravel	AST	Bituminous + Road Mix	Concrete + Composite	Total Distance
B1	5295.4	2745.9	1397.8	13.2	9452.3
A1	580.7	1550.0	1543.1	17.4	3691.2
RTAC	8.2	407.3	4427.8	943.6	5786.9
Network	5884.3	4703.2	7368.7	974.2	18930.4

In Manitoba there are two Long Term Pavement Performance (LTPP) Seasonal Monitoring Program (SMP) sites. The SMP program is an attempt to understand the impact of daily and yearly, temperature and moisture changes on pavement structure and its response to loads. The extensive data collection effort includes inventory, material testing, pavement performance monitoring, climatic, traffic, maintenance, rehabilitation, and seasonal testing modules.

1.3 Motivation and Objectives

According to MIT, in 2004, 57.5% of the road network was under Spring Load Restrictions (SLR). A summary of this information can be seen in Table 2. This information excludes access roads and reflects the SLR policy, involving thawing index, implemented in 2003. The majority (52.7%) of the AST and flexible pavement road network was warranted for restriction under the policy.

Table 2 – Warranted and Actual Lengths of Pavements Restricted [5]

Class	Level	2004 Distance in 2-lane km Warranted	2004 Distance in 2-lane km Actual
RTAC A1 B1 Total	1	798 731 882 2412	158 806 921 1885
RTAC A1 B1 Total	2	86 1305 2302 3693	6 1067 2295 3367
Total Level 1 & 2	1 & 2	5854	5253
Total Network	1 & 2	12370	12370
% Restricted	1 & 2	47.3%	42.5%
% Unrestricted	1 & 2	52.7%	57.5%

The purpose of this research is to (1) evaluate and analyze, from an engineering standpoint, the current Manitoban spring load restriction policy as well as (2) address the vulnerability of the province's transportation infrastructure to climate change. Under these main objectives there is also the intent to determine adaptation strategies and possible recommendations for the improvement of spring load restrictions with respect to current practices in addition to climate change.

2.0 DATA COLLECTION

2.1 Environment Canada Climate Data

Environment Canada weather stations with climate normals in Manitoba were used in the analysis of this study. Climate normals or averages are used to summarize or describe the average climatic conditions of a particular location. Environment Canada defines a climate normal weather station as one that has a minimum of 15 years of average climate data between 1971 and 2000. There are currently 110 weather stations in Manitoba that meet these climate criteria.

2.2 Falling Weight Deflectometer Data from MIT and LTPP

Falling weight deflectometer (FWD) data from various testing programs was provided by MIT. The data covers a period of time from 1993 to 2005. This results in a varied cross section of pavement types, structures and dates. Based on the FWD data files, it was determined that 27 Provincial Trunk Highways (PTH) and Provincial Roads (PR) were tested. Of these 27 PTHs and PRs, there were tests conducted on 90 MIT control sections in all five highway regions in the province. These tests result in a total of 446 FWD data files.

Of these 90 control sections tested, 17 have a Portland Cement Concrete (PCC) surface layer or a PCC layer within their structure. Also, there are a total of six Asphalt Surface Treated (AST) tested roads. The remaining 67 control sections are bituminous pavement structures and will be the focus of this analysis.

Between 1990 and 2004, there were 60 and 33 FWD tests for LTPP sites 83-1801 and 83-3802, respectively. Unfortunately, after many attempts to obtain it, the raw data files were unavailable to the author. All testing previously carried out at both SMP sites was done using a Dynatest 8000 FWD, generating loads between 26.7 kN and 71.2 kN. Both site 83-1801 and 83-3802 were tested under three load levels. These load levels being 40.0 kN, 53.4 kN and 71.2 kN for three separate drops. An additional drop is performed for flexible pavements at a load level of 26.7 kN.

3.0 MODELING SEASONAL VARIATIONS

3.1 Backcalculation of Base and Subgrade Moduli

The "Deflection Basin Fit" method of calculation was employed to backcalculate the effective moduli of the pavement structure. This method utilizes the Odemark-Boussinesq methods, but the convergence criteria is based on degree of fit between the measured and calculated deflection basins. The basin fit option methodology starts with a set of estimated moduli for the pavement structure. The theoretical deflection bowl for this pavement structure is calculated and the error between the measured deflections and calculated deflections is then assessed. The moduli in the structure are then increased or decreased by a small amount (typically 10%), and if the error in either of these deflection bowls is less than the original deflection bowl this is taken to be a better solution. This process is iterated until a minimum in error between the calculated and measure deflection bowls are found [6].

Using this method does have a drawback in that several pavement structures may result in the same deflection bowl and the program may output any one of these solutions. Therefore, to aid the programs precision, the options within ELMOD to reduce the percent difference between solutions and to use the old moduli as seed values were chosen. As well, the following limits for layer stiffness were inputted based on the date of the various FWD tests (i.e. the tests were conducted between March and August):

- Asphalt Concrete / Bituminous layers: E = 1,000 to 8,000 MPa
- Granular / Subbase layers: E = 100 to 350 MPa
- Subgrade / Soil layers: E = 25 to 125 MPa
- Portland Cement Concrete layers: E = 10,000 to 40,000 MPa

With regard to the pavement structures themselves, all adjacent bituminous and bituminous pavement mixture layers were combined to form one layer. This was also done for granular and base layers.

3.2 Analysis of LTPP FWD Files

The pavement structure at LTPP site 83-1801 consists of 111 mm of asphalt concrete and 478 mm of granular base material overlain native subgrade material. No rigid layer exists within 6000 mm [7]. Site 83-1801 is believed to be a 4.8 kilometre section of road that underwent grading in 1984, asphalt surfacing in 1986 and received an asphalt sealcoat in 1998. Due to the fact that the reported moduli from LTPP were unrealistically high and that the details of the backcalculation methods are unknown, the raw deflection database files were obtained from the LTPP online database for analysis using ELMOD5. A focus was placed on the FWD tests performed in the same time frame as the MIT tests. Therefore, of the 60 LTPP FWD monitoring tests, a total of 39 dates were analyzed. The remaining 21 dates were outside of the desired date range of February through August. Of the 39 analyzed files, four files returned exceedingly high moduli values most likely due to the fact that structure was still frozen during testing (the test dates were in either February or early March). Thus, these calculated values were not plotted with the remaining 35 test dates. The resulting moduli values are shown in Figures 1 and 2. The curves in Figures 1 and 2 were created using Loess statistical modeling.

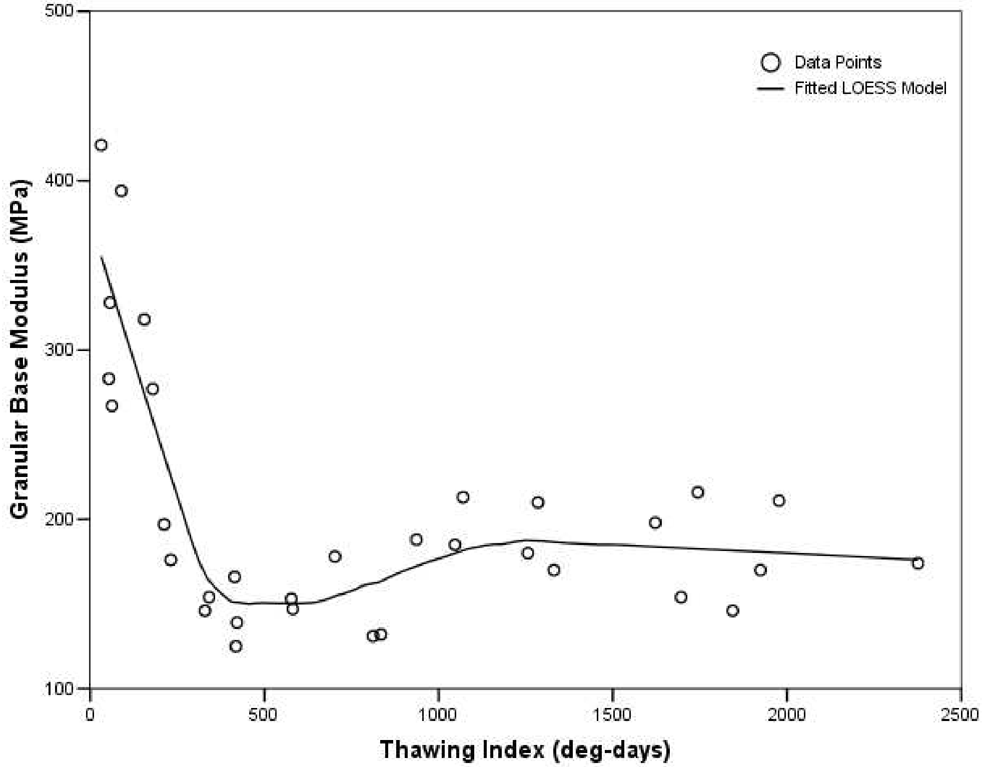

Figure 1 – LTPP Granular Base Modulus vs. Thawing Index

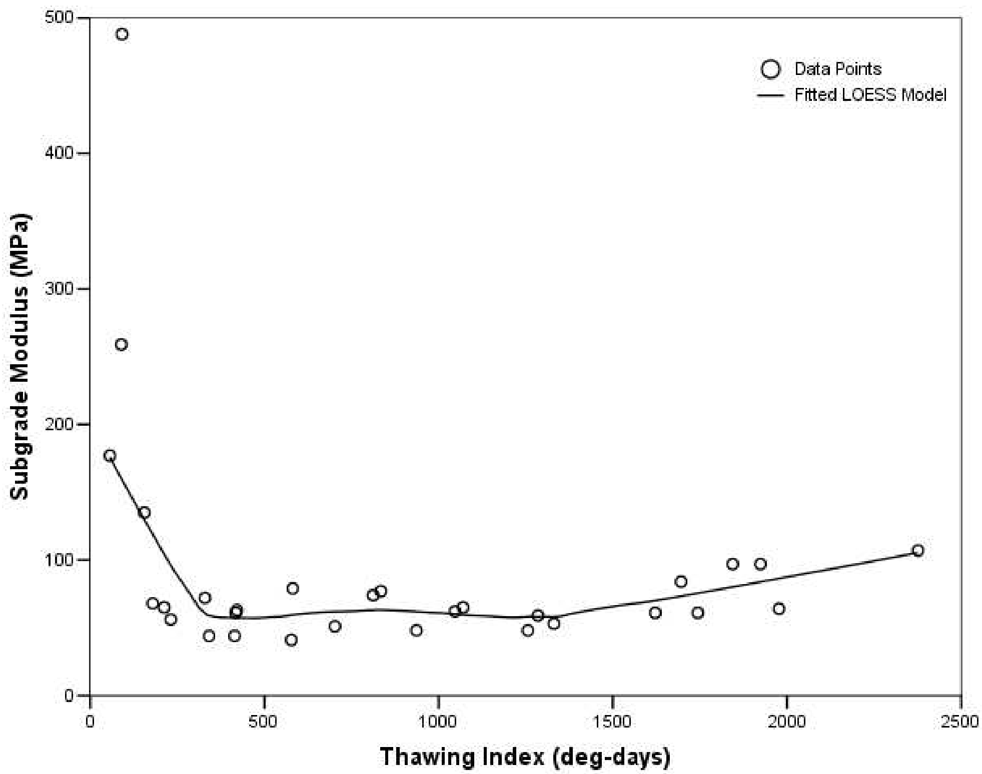

Figure 2 – LTPP Subgrade Modulus vs. Thawing Index

The data from LTPP follows the theoretical relationship expected in terms of modulus and thawing index. The values are within expected ranges for strength and differentiate between base and subgrade from frozen to thaw to recovered conditions. The reason for this is likely due to the fact that the LTPP testing is a more consistent and detailed location with ample information regarding layer properties. The backcalculated information demonstrates the seasonal variation of subgrade and granular base moduli in Manitoba. This reinforces the need for a mechanistic method of SLR implementation in order to account for the variability of the materials throughout the province and spring season.

3.3 Normalized Loss of Bearing Capacity due to Spring Thaw

It was possible to gather spring and summer backcalculated moduli values for a small number of MIT FWD test sites on PTH 1, 2 and 3 for the same testing year. The LTPP test site 83-1801 was also tested at various times during a typical year and thus seasonal moduli values were readily available for that location.

The average loss of bearing capacity for the granular base and subgrade layers for the MIT sections were found to be 0.07 and 0.21, respectively. In other words, the average MIT section was 7% and 21% weaker from spring to summer for the granular base and subgrade, respectively. The LTPP test site experienced higher loss of bearing capacity for its granular and subgrade layers. The normalized losses of bearing capacity for this section were found to be 0.19 and 0.26 for the granular base and subgrade layer, respectively. The subgrade layers for both the MIT sites and the LTPP site were found to be comparable, having losses of 21% and 26% of strength. Conversely, the average loss of strength experienced by subgrade layer of the LTPP site was almost triple that of the MIT sites (19% to 7%). This may be attributed to the fact that three of the seven MIT sites showed an increase in granular base strength from spring to summer testing.

4.0 CLIMATE CHANGE IMPACTS ON PAVEMENTS

4.1 Climate Change Model Development

A regional downscaled climate change scenario was used in this research to address impacts on transportation infrastructure due to climate change. Dynamic downscaling techniques using a meso-scale weather prediction model were used, where finer resolution climate information is derived from coarser resolution Global Climate Model (GCM) output. Canadian Centre for Climate Modeling and Analysis (CCCma) data, after necessary transformation, was used as the boundary condition driving the downscaling process for years representing 1978 (1 x CO_2) and 2044 (2 x CO_2). Output from the two representative years was used to develop a climate change scenario. Mean annual temperature increase resulting from doubling of CO_2 is in the order of 3.5°C to 4.0°C. Much of these increases result from warming in the spring and early summer. There is a cold bias in the model compared to the climate normal. Also, the number of frost free days increased by 15 to 20 days due to the increase in CO_2. Results obtained in the study are in general agreement with other climate scenarios, although there are a number of known deficiencies in this analysis. Namely, a short period of integration relative to climate, and initialization of surface fields, such as snow cover and soil moisture may not be representative. Therefore, results should not be taken directly to reflect average future climate change [8].

The climate change model found that due to the increase in CO_2, warming is seen to be between 2°C to 5°C year round except for the month of February. Warming during the winter (January) varies over the model domain. Winter warming of 1°C to 6°C is seen over the model domain with an average increase of 3.5°C. Warming in the month of July is relatively evenly distributed over the domain. An average warming of 4°C is seen during the mid-summer month [8].

4.2 Impacts on Spring Load Restrictions

Using the output of the downscaled regional model for the average daily air temperature at Winnipeg International Airport, the impacts on SLR were estimated. The most common method used to transfer the signal of climate change from climate models to hydrological models has been the delta approach. In this approach, only differences in the most relevant climate variables—typically precipitation, temperature and evapotranspiration—are extracted from the control and scenario simulations of the climate model [9]. In this research a comparison between the model temperature trends for the year 2044 and observed data from 1990-2005 for Winnipeg International Airport was made. Figure 3 shows the observed average daily temperature for 1990 to 2005 and the climate change model trends applied to it. The average temperature increase for the model year is 3.28°C. Further, the average temperature increases for the seasonal periods of winter (December-January-February), spring (March-April-May), summer (June-July-August) and fall (September-October-November) are 1.68°C, 5.78°C, 4.89°C and 0.78°C, respectively. Both data sets were fit with a sinusoidal model relationship with the parameters shown in the figure. The difference in winter temperatures is explained as possibly being due to snow cover data used in the development of the model [8].

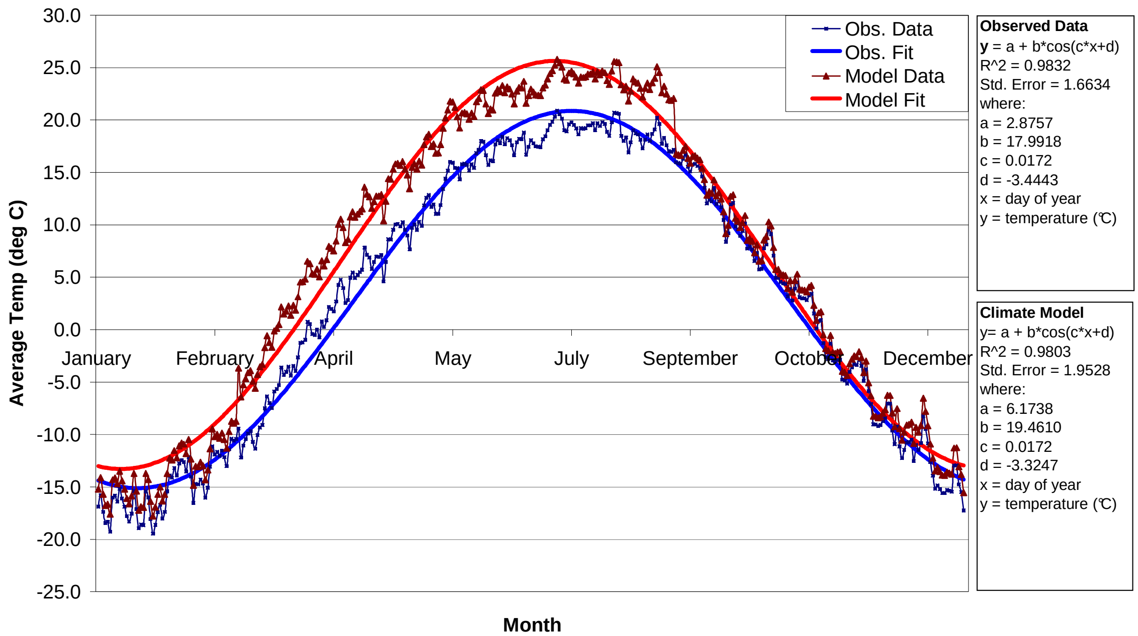

Figure 3 – Observed and Climate Model Temperature Data

First, a comparison of historical start date and the climate model start date were completed. It was determined that the average historical start date for the period between 1990 and 2005 is March 18th, whereas the projected climate model data translates to a start date of March 6th. Therefore, comparing 1990-2005 with 2044, along with the increase of 12 days at the start of SLR, this would likely lead to a decrease in the duration of winter weight premiums for Manitoba. As well, the difference between the hypothetical historical end dates for the granular base and subgrade layers is 20 and 30 days, respectively. As well, a decrease of the cumulative freezing index in the order of 529°C-days was found for the projected year 2044.

An increase of 12 days in the start date of SLR is significant in terms of the strength of the pavement structure and the current policy, since for a warm spring the date may actually theoretically fall in February leading to a major change in the SLR TI calculation guidelines. Further, even though this research finds a net increase in SLR start date of 12 days for 2044, it is unknown what the year-to-year influences of climate change actually are. This fluctuation appear that the impact of climate change on SLR policy will be a fundamental shift in the start date commencing in early March or late February. As well, a significant change in the length of SLR may be required to accommodate the change in temperature in the pavement structure. The benefits of a flexible start date may also be seen by the addition of a flexible end date to minimize damage to the road structure and impact to trucking industry. Ultimately, SLR policy should be condition or climate based, rather than fixed date, in order to provide the necessary flexibility to compensate for climate change impacts. Whether or not SLR policy can be modified remains to be seen as such decisions are influenced by economics and politics as well as the condition of the road network.

5.0 Mechanistic-Empirical Damage Comparisons

Mechanistic-empirical damage comparisons were carried using the American Association of State Highway Traffic Officials (AASHTO) Mechanistic-Empirical Design Guide (MEDG). The MEDG was developed in part by numerical models provided through the LTPP program. As well, the LTPP dataset is the most consistent and reliable data available to this research. As such, a damage analysis using the MEDG was carried out, using the LTPP site 83-1801, to determine the accumulated damage consequences and costs associated with spring thaw weakening. Unfortunately, consistent testing on other highway sections for Manitoba is unavailable at this time. As much as possible, site specific inputs for traffic, climate, and structural information were used to accurately depict the existing conditions at the LTPP site.

Five scenarios were tested as follows:

- Scenario 1 is a pavement structure that undergoes spring thaw weakening in March, April and May, experienced partial strength recovery in June and full recovery in July. This scenario simulates current conditions occurring during spring thaw without accommodation for SLR (100% axle loads).
- Scenario 2 is a pavement structure that undergoes spring thaw weakening identical to scenario 1 but axle load spectra was reduced to 90% of normal axle loads in March, April and May. This scenario simulates a pavement structure that subjected to Level 1 of SLR load limits by MIT. This scenario assumes that no additional trips are generated by the enforcement of SLR.
- Scenario 3 is a pavement structure that undergoes spring thaw weakening identical to scenario 1, but axle load spectra was reduced to 65% of normal axle loads in March, April and May. This scenario simulates a pavement structure that subjected to Level 2 of SLR load limits by MIT. This scenario assumes that no additional trips are generated by the enforcement of SLR.
- Scenario 4 is a pavement structure that undergoes spring thaw weakening identical to scenario 1, but axle load spectra was reduced to 90% of normal axle loads in March, April and May. Scenario 3 assumes that additional trips are generated by the loss of payload created by the enforcement of SLR.
- Scenario 5 is a pavement structure that undergoes spring thaw weakening identical to scenario 1, but axle load spectra was reduced to 65% of normal axle loads in March, April and May. Scenario 5 assumes that additional trips are generated by the loss of payload created by the enforcement of SLR.

With respect to scenarios 4 and 5, the enforcement of SLR was assumed to generate additional trips during the months of March, April and May due to the reduction in payload experienced by the shipping industry. From the current traffic collection information, it was determined that 90.5% of the truck traffic on this highway section was from class 9 (59%), class 10 (18%) and class 13 (13.5%) trucks. Based on tare weight information [10] and the current MIT gross vehicle weight restrictions, the additional trips theoretically generated by the enforcement of 90% and 65% SLR levels were determined.

In order to transport the same fully loaded mass of freight under Level 1 SLR as during normal conditions, class 9 trucks must utilize 1.15 trucks, class 10 trucks must utilize 1.17 trucks and class 13 trucks must utilize 1.18 trucks. This translates into a weighted average of 16 additional trips, per 100, theoretically generated during Level 1 SLR by class 9, 10 and 13 trucks. Further, In order to transport the same fully loaded mass of freight under Level 2 SLR as during normal conditions, class 9 trucks must utilize 2.23 trucks, class 10 trucks must utilize 2.46 trucks and class 13 trucks must utilize 2.67 trucks. This translates into a weighted average of 134 additional trips, per 100, theoretically generated during Level 2 SLR by class 9, 10 and 13 trucks.

Although the MEDG simulates failures in various categories, a focus was placed on fatigue (bottom-up) cracking, longitudinal (top-down) cracking and permanent deformation (rutting) as the three failure modes for the pavement structure. Typically, rutting and longitudinal cracking failure does not constitute failure of a roadway and can be sometimes repaired for a lower cost by crack sealing and/or sealcoating (as was performed in 1998).

Of the three failure modes, the bottom-up failure cracking was the governing failure mode over the 20-year design life. A comparison of the total rutting, top-down and bottom-up cracking failure modes can be seen for all simulations in shown in Table 3. It was found that the all simulations reached the failure limits.

Table 3 – Summary of MEDG Failure Modes

	Failure Mode		
Simulation (load)	**Top-down Cracking (ft/mi)**	**Bottom-up Cracking (%)**	**Total Rutting (in)**
Scenario 1 (100%)	1,000 at 17.42 years	25.0 at 9.42 years	0.750 at 17.58 years
Scenario 2 (90%)	1,000 at 17.58 years	25.0 at 9.50 years	0.750 at 17.58 years
Scenario 3 (65%)	1,000 at 18.67 years	25.0 at 9.83 years	0.750 at 18.67 years
Scenario 4 (90%)	1,000 at 16.83 years	25.0 at 9.33 years	0.750 at 17.50 years
Scenario 5 (65%)	1,000 at 15.42 years	25.0 at 8.42 years	0.750 at 15.50 years

When comparing the five scenarios, it can be seen that the expected behaviour of the pavement structure is seen when comparing scenarios 1, 2 and 3 where loads, and not trips, are restricted during spring thaw. The design life of scenario 1 is expected to be the shortest since it is under 100% axle loads when compared to the design lives of scenario 2 and 3 and limiting the loads should reduce spring damage caused to the roadway over its service life. It should also be recognized that the results of the simulation demonstrate small increases in service life for the Level 1 SLR when compared to the service lives from Level 2 SLR.

Where the MEDG results deviate from expected behaviour is when trips in scenarios 4 and 5 are increased to compensate for loss of payload due to SLR. When comparing the baseline scenario 1 to 4 and 5, it was found that the increased trips have a more substantial impact on the structures service life than the limitation of axle loads in the MEDG. This is seen by the consistently shorter service life values returned for scenarios 4 and 5 for all three failure modes. Current practice does not reflect that the number of trips experienced by the network during SLR periods will be the maximum values of 1.16 and 2.34. It would most likely be cost prohibitive for a trucking company to ship 2.34 trucks during Level 2 SLR to compensate for lost payloads. A case in between the baseline scenario and scenarios 4 and 5 would better reflect actual conditions. This case study is a comparison of two extremes with respect to load limitations and trip generation. Realistically, the impact of SLR is the deterrence of trucks using the network during spring thaw by creating a situation where it is not cost effective to haul, particularly at Level 2. Further research should be done to determine the exact impact of SLR on the number of trips created and their implications during spring thaw periods. The simulation of SLR impacts on a Manitoba highway does demonstrate the damage occurring the spring thaw and reinforces the need for an improved system with flexibility and climatic conditions to compensate and reduce the impact on the roadways. The differences between design lives returned by the MEDG may be considered minor but it should be remembered that this section is considered a strong pavement and the damage and effect on service would be greater on a weaker pavement structure such as the ones under SLR.

6.0 CONCLUSIONS

The backcalculated data from LTPP follows the theoretical relationship one would expect in terms of modulus and thawing index. The values are within expected ranges for strength and exhibit the change the layers undergo from frozen to thawed to recovered conditions. Based on the results, it appears that the pavement structure remains frozen at the SLR TI implementation value of 15 deg-days. Further, it would appear that both the granular and subgrade layers have not experienced strength recovery at the arbitrary end of SLR on May 31 for a given year and could take up to six further weeks of restrictions in order to recover. Using climate data for Winnipeg International Airport, average calendar end dates for SLR were determined for the critical values of thawing index for each layer. For the granular base, a TI of approximately 1000 deg-days is equal to an average date of June 9. For the subgrade layer, a TI of approximately 1500 deg-days is equal to an average date of July 8. This demonstrates the variability of pavement structures throughout the province and their strength behaviour during spring thaw. As well, the backcalculation of moduli does effectively estimate the loss of bearing capacity seen by the subgrade and granular base layers. This information enforces the need for a mechanistic method of SLR determination in Manitoba.

A damage analysis using the AASHTO Mechanistic-Empirical Design Guide was carried out, using LTPP site 83-1801, to determine the accumulated damage consequences and costs associated with spring thaw weakening. Realistically, the impact of SLR is the deterrence of trucks using the network during spring thaw by creating a situation where it is not cost effective to haul, particularly at Level 2.

Impacts on SLR policy were determined using the climate change model provided for this research project. As well as creating an earlier SLR start date, this would also likely lead to a decrease in the length of winter weight premiums for Manitoba. With respect to the layers of the pavement structure specifically, it was found that the largest difference lies in the determination of the subgrade end date.

Also, due to the warmer air temperatures, the start and end dates were determined to be earlier than the historical average from 1990 to 2005. An increase in the start date from the historical average would have significant impact on the strength of the pavement structure and the current SLR policy. This fluctuation appear that the impact of climate change on SLR policy will be a fundamental shift in the start date commencing in early March or late February. The benefits of a flexible start date may also be seen by the addition of a flexible end date to minimize damage to the road structure and impact to trucking industry. Ultimately, SLR policy should be condition or climate based, rather than fixed date, in order to provide the necessary flexibility to compensate for climate change impacts.

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