Roughness degradation models of flexible pavements subjected to seasonal frost action

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Abstract

In cold regions, flexible pavements are constantly submitted to the effects of repeated traffic loads combined with the climate effect. The frost heave of the subgrade soils due to formation of ice segregation is among the main mechanism involved in the high deterioration rate of flexible pavement. This paper presents developments of flexible pavement damage models, developed through a multiple linear regression (MLR) analysis, associating long-term roughness performance to degradation mechanisms, such as, among others, frost heave. Those models would be essential to assess the advantages or consequences to have a frost heave lower, equal or higher than the allowable threshold values established by the Ministry of Transportation of Quebec (MTQ) according to the roads functional classification. One of the models developed uses the cracking performance index as a direct quantification of cracking and the other one uses indirect cracking quantification using thickness and age of pavement. This research illustrates that a notable increase in long-term IRI deterioration rate of pavement is usually caused by frost heave, variable subgrade soil and traffic. Obtaining the flexible pavement damage models with the various degradation mechanisms will help to predict and reduce the residual distortions that affect the structural and functional capacities of cold region's road network.

Introduction

In northern regions, the performances of flexible pavements are considerably affected by repeated traffic loads and frost action. In order to achieve a good pavement performance, road designers must inevitably consider frost action and the related phenomena. Damage mechanisms related to frost heaving of soils due to ice lens formation can significantly increase the long-term IRI degradation rate of pavement. Based on Doré et al. (2005), those damage mechanisms subjected to seasonal freezing can contribute up to 75% of flexible pavements degradation. In specific area of the Province of Quebec, frost heave of up to 200 mm can be the result of a poorly designed pavement (Doré 1997) due to volume change of freezing water in the pavement structure and, even more important, to uneven segregation ice growth within frost sensitive subgrade soils. Despite the more than obvious evidence of the existence of a cause and effect relationship between frost heave and pavement degradation, there is no deterioration model that clearly establishes a link between frost heave and flexible pavement service life. Indeed, at the design stage, it is difficult to evaluate the advantages or the consequences of theoretical frost heave on pavement service life when the values calculated are lower, equal or greater than the allowable thresholds, such as the empirical ones established by Ministry of Transportation of Quebec according to the functional classification of roads.

In order to assess the impact on flexible pavement service life when it comes to frost heave, the main goal of this paper is to introduce a flexible pavement damage model linking long-term roughness performance to frost heave and the associated degradation mechanisms. At a design stage, the frost protection pavement design method proposed by the Ministry of Transportation of Québec is mechanistic-empirical. This method calculates a theoretical frost heave using the combined Saarelainen-Konrad approach based on the iterative solution of the heat balance equation at the frost front (St-Laurent 2006, 2012), and then compares the calculated value to the allowable thresholds established empirically. Using a theoretical frost heave, the models developed in this research could be beneficial to assess the repercussions of the choices made at a design stage. The well-documented databases from the Ministry of Transportation of Quebec, the LTPP program and field surveys are used to study the relationship between frost heave and the pavement deterioration rate.

Background

In northern regions, flexible pavements are continually submitted to the complex interaction between the effects of climate and repeated traffic loads, which are the main pavement damage mechanisms. Based on specific Canadian study, the percentage of total annual damage associated to this typical interaction concluded that traffic damage indices vary from 0.45 to 0.8 for a local road and major highways, respectively (Doré et al. 2005). Moreover, other research stated from their study result that climate is causing for 60% to 75% and for 50% to 80% of road degradation based on climate, traffic intensity and pavement construction (Tighe 2002, Nix 2001).

When flexible pavements are subjected to frost action, the pavement structure inevitably suffers from the negative effects of frost heave on his long-term performance. The effects of frost heave are encountered by the pavement structure swelling as the volume change from water to ice and by segregation ice formation in the subgrade soil. Ice lens formation occurs when 3 main conditions are encountered: frost-sensitive soils, high water table and subfreezing temperatures (Konrad and Roy 2000). When temperature at the roadway surface decreases below zero Celsius degrees, the heat transfer extraction process is initiated in the pavement structure and frost penetration occurs from the pavement surface (Doré and Zubeck 2009, Coussy 2005). While this process is freezing the pore water contained in the pavement causing a volume increase by 9% due to phase change if saturated pores are encountered, unfrozen water existing below subfreezing temperature engenders thermodynamic instability. The percentage of unfrozen water is controlled by soil type and temperature. (Andersland and Ladanyi 2004, Hermansson and Guthrie 2005). This coexistence of ice and water causes important thermodynamic instability and suction stresses exists on the remaining unfrozen water. This creates favorable conditions for upward migration of available water in the pavement system, given rise to ice lenses (Konrad 1999, Doré and Zubeck 2009, Charlier et al. 2009) at the segregation front. In cold regions, the segregation potential (SP) remains one of the most important engineering parameters involved in the regulation of heat and mass flow in a freezing soil. The overall hydraulic conductivity and the relationship between the water flux and the thermal gradient underneath the last ice lens in described by the segregation potential theory, which is a mechanistic frost sensitivity index (Konrad 1987, 2005, Konrad and Morgenstern 1980).

A frost-sensitive soil refers to a soil that promotes the upward migration of unfrozen water towards a potential ice liens. One of the main factor which influences the soil sensitivity to frost action is the grain-size distribution, especially the percentage of fine particles. This element contributes to the development of essential capillary suction stresses within the soil matrix thus promoting the upward migration of pore water towards the growing of ice lenses (Konrad and Lemieux 2005). Moreover, the mineralogy of the soil particles, the soil fabric and the degree of consolidation are most of the parameters influencing frost sensitivity (Konrad 1999, Rieke et al. 1983, Nixon 1991, Konrad 1989b).

Along a given pavement longitudinal profile, soil characteristics and properties variations, such as, amongst others, frost sensitivity, may give rise to uneven frost heave at the pavement surface leading to differential heaving (Doré et al. 2001, Bilodeau and Doré 2013). This phenomenon may lead to premature reduction of the ride quality and the smoothness of pavement profiles. The damage associated to the early reduction of the smoothness pavement profiles can be described by long and short wavelengths deformations, ranged from 8 to 12 meters and 1 to 3 meters, respectively (Sayers and Karamihas 1998, Doré 1997, Vaillancourt et al. 2003). Long wavelengths deformations are associated to subgrade soil differential frost heave, which can be called differential longitudinal deformation. Short wavelengths deformations are related to heaving located near cracks in the presence of brine from de-icing salt. This phenomenon occurs when a salinity gradient is formed in the granular material underneath cracks during warm periods in the winter and early spring due to the use of de-icing salt, leading to located frost heave (Doré et al. 1997). Another phenomenon that is called differential transversal heaving is also encountered from the pavement surface centerline to pavement edges due to the insulation of pavement edges induced by plowed snow layers during winter maintenance. This layer of snow leads to deeper frost penetration and greater frost heave magnitude at the center of the roads, while the snow on pavement edges acts as a thermal insulator to the geothermal heat flow (Doré et al. 1999).

Over time, the International Roughness Index (IRI, m/km) became the most popular pavement condition index to quantify overall pavement smoothness and users comfort with the expansion of transportion network (Sayers and Karamihas 1998, Vaillancourt et al. 2003). This index, which is calculated with the quarter-car mechanical model, expresses the response of a vehicle stimulated by elevation variations of the pavement profile at a speed of 80 km/h. During commissioning of the road, values of IRI are approximately ranged from 0.8 to 1.2 m/km (TAC 1997). This parameter has several uses, such as deficiency threshold and performance indicator against frost action for road administrators.

In order to preserve good ride quality along the road network in the Province of Quebec, the Ministry of Transportation uses two complementary methods to determine the optimal design against frost action. A conventional pavement design method based on partial protection through empirical criteria is used to evaluate the total pavement structure thickness needed to achieve optimal behavior of the road against frost. The performance of pavement structures is considered acceptable when the total pavement thickness is about 50 % of the average frost penetration for given climatic conditions (Bilodeau et al. 2016, St-Laurent 2012). This method is adopted from a basic design curve developed by the Federal Department of Transport of Canada in the 1950's. This curve estimated the minimum thickness of pavement structure using the normal freezing index (FI_a), with correction factors developed for Quebec conditions for the type of subgrade and the functional class of the road (St-Laurent 2012). The second method uses a mechanistic-empirical design procedure against frost action using the Saarelainen-Konrad method based on the segregation potential theory (Saarelainen 1992). Using an iterative calculation, this method calculates the daily frost penetration with the heat balance equation at the frost front, which is expressed by equation 1.

$$q_- = q_+ + q_f + q_s \tag{1}$$

where q_- is the heat flux from the frost front through the frozen layer, q_+ is the geothermal heat flux, q_f is the heat flux from the phase change of the water to turning to ice and q_s is the heat flux from freezing water that migrated from the unfrozen soil to the segregation front. With each iteration, the heat balance at the frost front is calculated leading to the determination of frost depth. The frost heave encountered in each layer of the pavement structure for each time step is calculated simultaneously using the equation 2.

$$dh = 1.09 \cdot SP \cdot GradT \cdot dt \qquad (2)$$

where dh is the frost heave, SP is the segregation potential, GradT is the thermal gradient of the frozen soil at the segregation front and dt is the time interval. The multiplication factor of 1.09 represents the volume increase of water when turning into ice. The theoretical frost heave for the trial pavement structure is then compared to the allowable thresholds established with field performance observations by the Ministry of Transportation of Quebec. As presented in table 1, the thresholds defined by the MTQ and the Finnish Road Administration are ranged between 30 and 100 mm based on road functional class (Doré and Zubeck 2009, St-Laurent 2006, 2012, Bilodeau and Doré 2013, Tammirinne et al. 2002).

Methods

In order to achieve the desired model associating the IRI long-term degradation rate (ΔIRI_{LT}, m/km·yr) to degradation mechanisms, such as, among others, frost heave, the following steps need to be pursued in order to reach the goal previously established in the framework of this project.

The first step requires the formation of two separate databases to first develop and then validate the damage models. In order to build those databases, it is necessary to gather performance and other relevant information from the active pavement sections. Regarding the developed models that are representative of the degradations that occur on road networks in northern regions, a database containing specific information on 44 pavement sections in the province of Quebec was created using the well-documented database from the Ministry of Transportation of Quebec. Initially, more than 50 sections were included in this database, but specific statistical manipulations for multiple linear regression (MLR) have been used to remove outliers. The pavement sections selected consist of collector roads (19), regional roads (20), national roads (2) and highways (3). In order to build the validation database, 22 sections were selected from the ministry's database (11), the US LTPP program (8), as well as field surveys (3), for a total of 9 regional roads, 10 national roads and 3 highways. The data collected contain information on frost heave, summer longitudinal profiles to quantify the yearly IRI deterioration rate, traffic and pavement structure.

The second step consists in a statistical analysis to link the increase of ΔIRI_{LT} to frost heave and the different mechanisms of degradation with the first database of 44 sections. Independent complex variables that represent the degradation mechanisms involved on road subjected to frost action and traffic were linked in a multiple linear regression (MLR).

From the collected information in the available body of literature (Ullidtz 1987, Hoerner 1998, NCHRP 1-37A, 2004 and Djonkamla et al., 2015), the main degradation mechanisms involved on the long-term IRI deterioration (ΔIRI_{LT}) are represented by the climatic conditions associated to frost heave magnitude and soil variability, the effect of annual traffic and pavement structural capacity, as well as the effects of cracking. The latter can be associated to longitudinal and transversal cracking performance index as a direct quantification of cracking, and to asphalt concrete thickness and section age as indirect cracking quantification (Doré and Zubeck 2009). Based on the analysis performed, the developed models, expressed with the equation 3, will have the following form:

$$\Delta IRI_{LT} = x_1(Frost\ heave\ and\ Soil\ variability) + x_2(Traffic\ and\ Pavement\ structure) + x_3(Cracking) + x_4(Initial\ IRI) + x_5 \quad (3)$$

where x_i are regressions coefficients.

The last step of this research is to validate the damage models with the database built for this purpose. As a result of model development and validation, design support tools will assess the impact of design decisions on long-term IRI degradation (ΔIRI_{LT}).

Database

This section presents the actions performed on the raw data acquired from the different databases stated earlier in order to develop and validate the damage models.

In order to calculate the long-term IRI degradation on each section, summer IRI values are analyzed over the design period. The ΔIRI_{LT} stored in the database are the average values represented by the slope of the curve between IRI and time (m/km·yr). Figure 1 shows an example of the relationship between IRI and time on two sections used in the database. For the test sections of Adstock and Lac-Supérieur, the value of ΔIRI_{LT} are 0.289 and 0.198 m/km·yr, respectively. The average value of ΔIRI_{LT} gathered in the database is 0.212 m/km·yr with a standard deviation of 0.108 m/km·yr.

For this research, the analyzed periods are selected during the initial life cycle of the pavement section, after initial construction and before rehabilitation. The pavement ages gathered in the database are based on the length of the analyzed periods. The IRI_0 values are the initial IRI values at the beginning of the analyzed period. The average length of the selected section is 227 meters with a standard deviation of 83 meters.

The pavement surface cracking condition, such as the transversal and fatigue cracking, are listed in the database through their own performance index (ΔPI_T, ΔPI_F). A performance index is dimensionless and defined based on weighted cracking, which considers the severity of the cracks and its length, which is converted to percentage using a table specifically generated for this purpose by MTQ. As for ΔIRI_{LT}, the values listed in the database are the slope of the performance index rate with respect to time for the analyzed period. Figure 2 illustrates two examples of performance index rate for the fatigue and transversal cracking. For the test section of Adstock (Figure 2a), the transversal cracking and fatigue cracking performance index rate are -1.07 %/yr and -6.43%/yr, respectively. In the Figure 2b, the ΔPI_T and ΔPI_F are -4.45 % and -3.94 %, for the test section of St-Aurélie. The average values for these indices are, respectively, -3.42 % and -4.78 % for transversal and fatigue cracking with a standard deviation of -3.04 % and -3.81 %.

For the purpose of determining the average yearly frost heave on each analyzed section, a specific procedure was followed. The manual frost heave measurements were used to backcalculate the segregation potential of the subgrade soil using the climate data of the specific location considered, the year of the survey and the available soils and material characteristics of the pavement structure. An example of manual frost heave measurements is shown in Figure 3 for the test section of Chelsea in 2010. Manual measurements were taken using the rod and level technique every 10 meters on a 150-meter section illustrated by the dots, whilethe thin dotted line is the frost heave measurement calculated by the difference in elevation of winter and summer profiles obtained from road profiler. On this section, the average frost heave is 0.046 meters with a standard variation of 0.008 meters.

Afterwards, to determine the SP of the subgrade soil, the flexible pavement design software, CHAUSSÉE 2, developed by the MTQ which used to calculate the frost depth and frost heave using the combined Saarelainen-Konrad approach (St-Laurent 2012). Using the pavement structure and the air freezing index of the considered year, the SP of the subgrade soil was adjusted with the software in order to reach the average frost heave measured on the considered site. Thereafter, the air freezing index based on historical average temperatures over the period considered for the most representative weather station, climatic context and geographic location is used to determine the average frost heave for an average winter. The average normal air freezing index (FI_a) used is based on the average winter temperature and length of the freezing period collected for a weather station (St-Laurent 2012).

In order to characterize the effect of the traffic and the pavement structure on the deterioration rate of the IRI, the $ESAL_A$ and $ESAL_{AD}$ are used. The $ESAL_A$ describes the annual measured values of the equivalent single axle load on the section under study. The $ESAL_{AD}$ is based on the layer having the lowest bearing capacity expressed in ESALs. The latest parameter, which refers to the equivalent number of load repetitions for a reference single axle loaded at 80 kN, defines the layer the most likely to increase the deterioration of the pavement due to the effects of traffic loads.

Since it is known as a significant parameter contributing to the deterioration rate of the pavement surface, information about the subgrade soil variability was also gathered. As presented in the work of Doré (1997) and Bilodeau and Doré (2013) this parameter is expressed as the frost heave coefficient of variability CV_G. It is based on the variability of the frost sensitivity of the soil, quantified with the fines content in the subgrade soil along a given road profile for a specific geological context. As it is shown in Table 2, a value of 0.1 is associated to a soil with a low fines content variability. The CV_G for moderate and high variability are 0.3 and 0.5, respectively. Each geological environment for the test section were defined based on the location of the section and surface deposit maps.

This variable can also be determined using the procedure introduced in the research conducted by Doré (1997), with a minimum of 6 or 7 pairs of sample points collected at a distance of 4 meters each using the semivariogram analysis.

Multiple linear regressions process

A common mathematical method to estimate the relationship between one or several parameters by using a linear dependence calculation is a Multiple Linear Regression (MLR). Multiple linear regressions treat the quantitative data submitted for study and analysis according to the least squares criterion. The equation 4 expresses the estimated regression equation.

$$Y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k + \varepsilon \qquad (4)$$

where *Y* is the dependent variable, β_0, β_1, β_2, ..., β_k are regression coefficients, x_0, x_1, x_2, ..., x_k are explanatory or independent variables and ε is an unknown error.

Statistical concepts can be used to evaluate the quality and the predictive capacity of the resulting regression model. For this purpose, the coefficient of determination (R^2) and the root mean square error (RMSE) are used in most cases. Besides, the ANOVA (Analysis of Variance) table contains statistical parameters of the MLR in order to validate the assumptions.

All the parameters used in the models were selected according to their degree of influence on the independent variable, which is, in this research, the annual deterioration rate of the IRI. The degree of influence of each dependent parameter were obtained from a literature review and correlation studies between the different variables contained in the database. Based on these analysis, the frost heave shows the strongest association with the annual roughness deterioration rate (ΔIRI_{LT}).

Afterwards, complex independent variables were optimized in order to maximize the predictive capacity of the multiple linear regression models based on the form of the equation (3). Two models were proposed in order to characterize the degradation mechanism of cracking in two different forms, which will be beneficial for road engineers to analyze the effect of frost action on the pavement degradations. The first model uses the performance index for transverse and fatigue cracking as a direct quantification of surface condition, whereas the second one uses the asphalt concrete thickness and pavement age of the section as an indirect quantification of the effect of cracking (Doré and Zubeck 2009). A reduction in the thickness of the asphalt concrete layer increases the tensile stress at the bottom of the pavement layer and ageing reduces the tensile strength at the top of the asphalt concrete layer (Uhlmeyer et al. 2000, Svasdisant 2002).

Two models are proposed in this study. The first model, which uses performance index for cracking, would be essential to estimate the contribution of pavement cracking to the overall deterioration rate of pavements in cold regions and to estimate the service life of a pavement structure after construction. The second one would be used at a design stage in order to estimate the service life of a pavement structure using the design period and the trial pavement thickness.

The first model (model a), expressed in the equation 5, describes the annual degradation on the IRI using the performance index of cracking.

$$\Delta IRI_{LT} = 0.002 \cdot {IRI_0}^2 + 0.361 \cdot h^{0.164} \cdot CV_G^{0.055} + 0.200 \cdot Log(Age)$$
$$+1.931 \cdot 10^{-8} \left(\frac{{ESAL_A}^2}{ESAL_{AD}}\right) + 0.016 \cdot |\Delta PI_T|^{0.075} \cdot |\Delta PI_F|^{0.703} - 0.701 \qquad (5)$$

where h is the frost heave (mm), CV_G is the subgrade soil variability index, IRI_0 is the IRI value at the beginning of the study period (m/km), age is the duration of the study period (years), $ESAL_A$ is the annual number of load applications based on a standard axle, $ESAL_{AD}$ is the annual number of load allowed at a design stage on the critical layer of the pavement structure and the IP_F and IP_T indexes (%/yr) represent the absolute values of the yearly deterioration rate of the performance index associated with fatigue and transversal cracking, respectively.

The second model (model b), represented in the equation 6, describes the annual deterioration of the IRI using the pavement age and thickness.

$$\Delta IRI_{LT} = 0.001 \cdot {IRI_0}^2 + 0.399 \cdot h^{0.164} \cdot CV_G^{0.055} + 0.070 \cdot Log\left(\frac{Age^3}{\sqrt{H_{AC}}}\right)$$
$$+ 1.813 \cdot 10^{-8} \cdot \left(\frac{{ESAL_A}^2}{ESAL_{AD}}\right) - 0.648 \qquad (6)$$

where H_{AC} is the thickness of the asphalt concrete layer (mm).

Figure 4 presents the measured versus predicted values for both models. The two graphs displayed in Figure 4a are associated to model *a* while the ones in Figure 4b show the comparison obtained with the model *b*. The multiple coefficient of determination, R^2, indicates that 64 % and 56 % of the annual degradation of the IRI is explained by the model *a* and *b*, respectively. In this research, the root mean square error (RMSE) is used to estimate the accuracy of both models. This index represents the average difference between the predicted values obtained from the multiple linear regression models and the measured values. The RMSE uses individual squared differences, referred as residuals, to measure and quantify the magnitude of the prediction errors into a single measure of predictive power. As shown in Figure 4, the RMSE for model *a* and *b* are 0.064 and 0.070 m/km·yr, respectively, which allows stating that the models of the annual degradation of the roughness seems reasonable with the admissible variations on the independent variables. The second graphs presented for both models represents the standardized residuals, used to show the strength of the difference between the observed and expected values. A good model should allow obtaining values greater than -2 and lower than 2, as it is the case with most of the values illustrated in both standardized residuals graphs. Also, when the values are randomly placed below and above the x axis, then a multiple linear regression model is appropriate.

Afterwards, the ANOVA table, illustrated in table 3, is used to express the significance value F, represented by the P-value, which should be less than 0.05 with both models to obtain a statistically significant development. As it is showed, both models are validated on this statistical perspective.

Multiple linear regressions validation

In order to validate the damage models developed in this research, an independent database is used to confirm the predictive capacity of the developed models. As showed previously, the validation is performed using 22 sections that come from three different databases. Figure 5 shows the validation process and the corresponding roughness deterioration values with different markers to distinguish the 3 databases used in the validation process. The 11 sections of the Ministry of Transportion of Quebec are divided into measured (5) and estimated (6) frost heave. The sections with measured frost heave follow more precisely the equality line. As it is possible to observe in Figure 5, most of the validation data points are located in the overestimation range and are basically all over the equality line. Thereby, a calibration process of the model was undertaken to get a better fit of the validation points using both damage models. The result of this process will add a shift parameter to both models, which multiplies both initial models in order to reduce the overestimation. From this process, a value of 0.6 is obtained, determined by reducing the root mean square error (RMSE) for both models.

By adding the shift parameter to both models, the final damage models *a* and *b*, expressed in equations 7 and 8 will become:

$$\Delta IRI_{LT} = 0.6\left[0.002 \cdot {IRI_0}^2 + 0.361 \cdot h^{0.164} \cdot CV_G^{0.055} + 0.200 \cdot Log(Age) + 1.93 \cdot 10^{-8}\left(\frac{{ESAL_A}^2}{ESAL_{AD}}\right) + 0.016 \cdot |\Delta PI_T|^{0.075} \cdot |\Delta PI_F|^{0.703} - 0.701\right] \quad (7)$$

$$\Delta IRI_{LT} = 0.6\left[0.001 \cdot {IRI_0}^2 + 0.399 \cdot h^{0.164} \cdot CV_G^{0.055} + 0.070 \cdot Log\left(\frac{Age^3}{\sqrt{H_{AC}}}\right) + 1.813 \cdot 10^{-8} \cdot \left(\frac{{ESAL_A}^2}{ESAL_{AD}}\right) - 0.648\right] \quad (8)$$

Figure 5 shows the validation section with the fitting factor. With this last process, the values are less overestimated and closer to the equality line.

In a near future, it would be beneficial to add sections in the validation database similar to the sections used in the development of the models that use measured frost heave in order to continually develop and increase the model predictive capacity.

Analysis of the performance

In cold regions, the behavior of flexible pavements is a complex phenomenon that implicates several factors, making it difficult to model. Hence, with R^2 values of 64 % and 56 % for the model *a* and *b*, respectively, the results obtained for the estimation of the deterioration rate of the IRI based on instrumented sections are reasonable taking into account the complexity and the random nature of the degradation phenomena. In addition, a model developed in an uncontrolled environment is more likely to use more variable data as independent variables and this can significantly influence the R^2. Therefore, in order to validate the range of R^2 obtained in this research, a comparison with other MLR models that predict the IRI is significant. A well-documented model, which links several mechanisms of degradation of the IRI, developed in the MEPDG presents a R^2 of 50 %. As the model provides acceptable predictions of the IRI for original pavement, this model is qualified as reasonable by the authors (NCHRP 1-37A, 2004). Thus, a model proposed by Doré and Zubeck (2009) which only links the ΔIRI_{LT} and the frost heave on highway sections in Quebec is characterized by a R^2 of 55.3 %. The comparative analysis with these 2 models shows that the coefficient of determination obtained with both models are reasonable considering the mechanisms and phenomenon that are involved.

Since the estimation models developed in this research allow evaluating the contribution of cracking to pavement degradation differently, they will be used in a different way. The model *a*, which is using the performance index for cracking, will allow the evaluation of the pavement degradation in service. This model would be beneficial in determining or understanding the reason of accelerated pavement degradation when auscultation surveys are achieved. Thus, it would lead to the estimation of the remaining service life of a roadway in during its life cycle. At a design stage, the model *b* will allow estimating the benefits or consequences on the service life of pavements structure associated with having a frost heave lower, equal or higher than the allowable threshold values specified by the Ministry of Transportation of Quebec based on the functional classification of roads. The second model meets the needs of road engineers when considering the frost action in roads designing. For specific climatic conditions and soils, for instance, in order to meet the allowable frost heave thresholds, the Saarelainen-Konrad approach used by the Ministry of Transportation of Quebec outputs thick pavement structure, but for economic reasons or in urban context, the proposed design is sometimes not realistic. In this context, when other solutions are not possible, such as pavement insulation, design engineers face the challenge of counseling their clients about the impacts of respecting or not the thresholds and to determine the effects on pavement service life.

Conclusion

In cold regions, where the frost action is involved in the degradation of the pavement, a damage model quantifying the deterioration rate of flexible pavement with respect to frost heave is essential for roads engineers. This research described the relationship involving pavement service life and several pavement degradation mechanisms, such as frost heave, with a reasonable predictive capacity of pavement yearly IRI deterioration. The estimation models, developed through a MLR analysis, will allow road engineers to quantify the impacts on pavement service life when the theoretical frost heave calculated at the design stage is lower, equal or greater than allowable thresholds established by the Ministry of Transport of Quebec according to the functional classification of the roads. The model *a* assesses the degradation of pavement is service using the cracking performance index, which can be essential to estimate the remaining pavement service life during the life cycle. In order to estimate the performance of flexible pavements at a design stage, the model *b* links long-term roughness performance to several degradation mechanisms such as frost heave. The developed models are beneficial for roads engineers to adapt their pavements design taking into consideration frost action, and more specifically frost heave, to adequately estimate the roughness deterioration rate of flexible pavement in cold regions.

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Tables

Table 1 - Frost heave threshold values criteria established by MTQ and Finnra

Road functional class	Quebec (MTQ) (mm)	Finland (Finnra) (mm)
Highways	< 50	< 30
National roads	< 55	< 50
Regional and Collector roads	< 60	-
Municipal roads	70-80	< 100

Table 2 - CV_G de for the rural and urban context (Bilodeau and Doré 2013)

Variability	CV_G		
	Classification	Rural	Urban
Uniform (Alluvial terrace, Delta deposit, Marine or Lacustrine Flood Plain)	SP, SW, GP, GW, CL	< 0.1	< 0.5
Moderate (Deposition basin border, Alluvial deposit, Fluvio-glacial deposit)	SP-SM, GP-GM, CL-ML	> 0.1 and < 0.3	< 0.5
High (Glacial till, Varved clay, Soil with silt layers, Soil/Rock contact)	GM, GC, SM, SC, ML, CL	> 0.3	< 0.5

Table 3 - Results of the ANOVA analysis in the MLR

		df	Sum of Square	Mean Square	F	Significance F
Model *a*	Regression	5	0.318	0.064	13.511	1.36E-07
	Residual	38	0.179	0.005		
	Total	43	0.497			

Model *b*	Regression	4	0.279	0.069	12.430	1.31E-06
	Residual	39	0.219	0.006		
	Total	43	0.497			

Figures

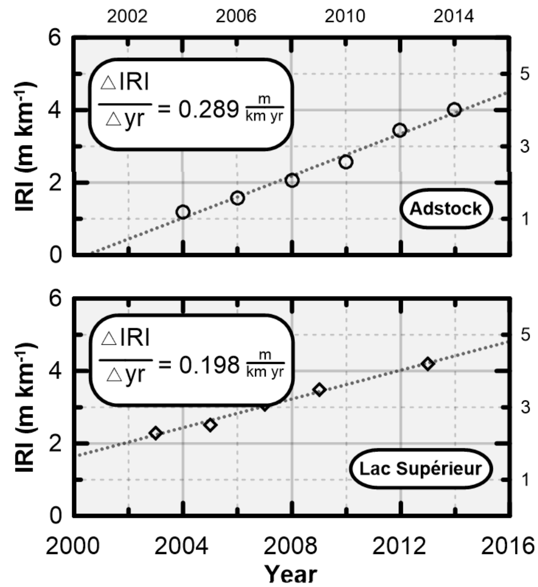

Figure 1 - Examples of the IRI deterioration rate stored in the database

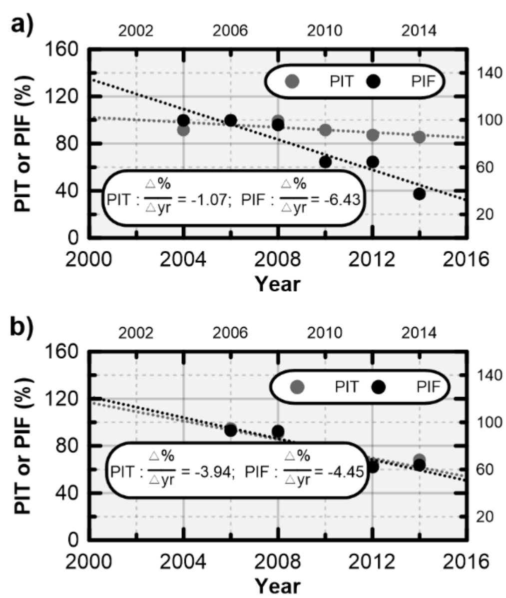

Figure 2 – Examples of the performance index deterioration rate for transversal and fatigue cracking stored in the database

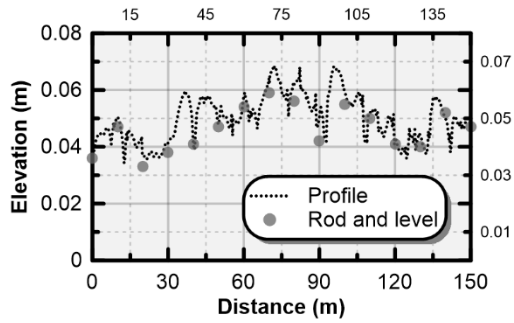

Figure 3 - Example of manual measurements of the frost heave for the section of Chelsea in 2010 *(Sylvestre et al., 2017)*

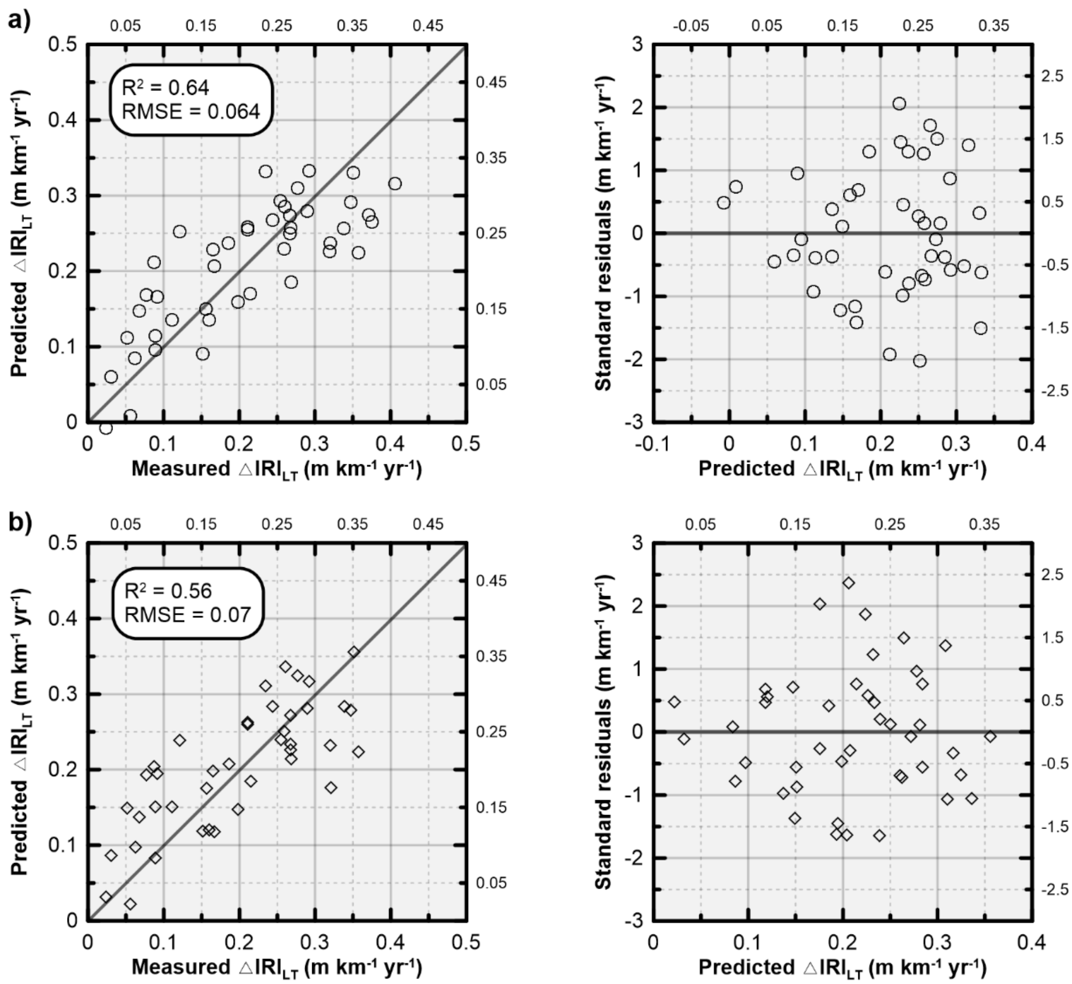

Figure 4 - Predicted vs measured values for model a and b (Sylvestre et al., 2017)

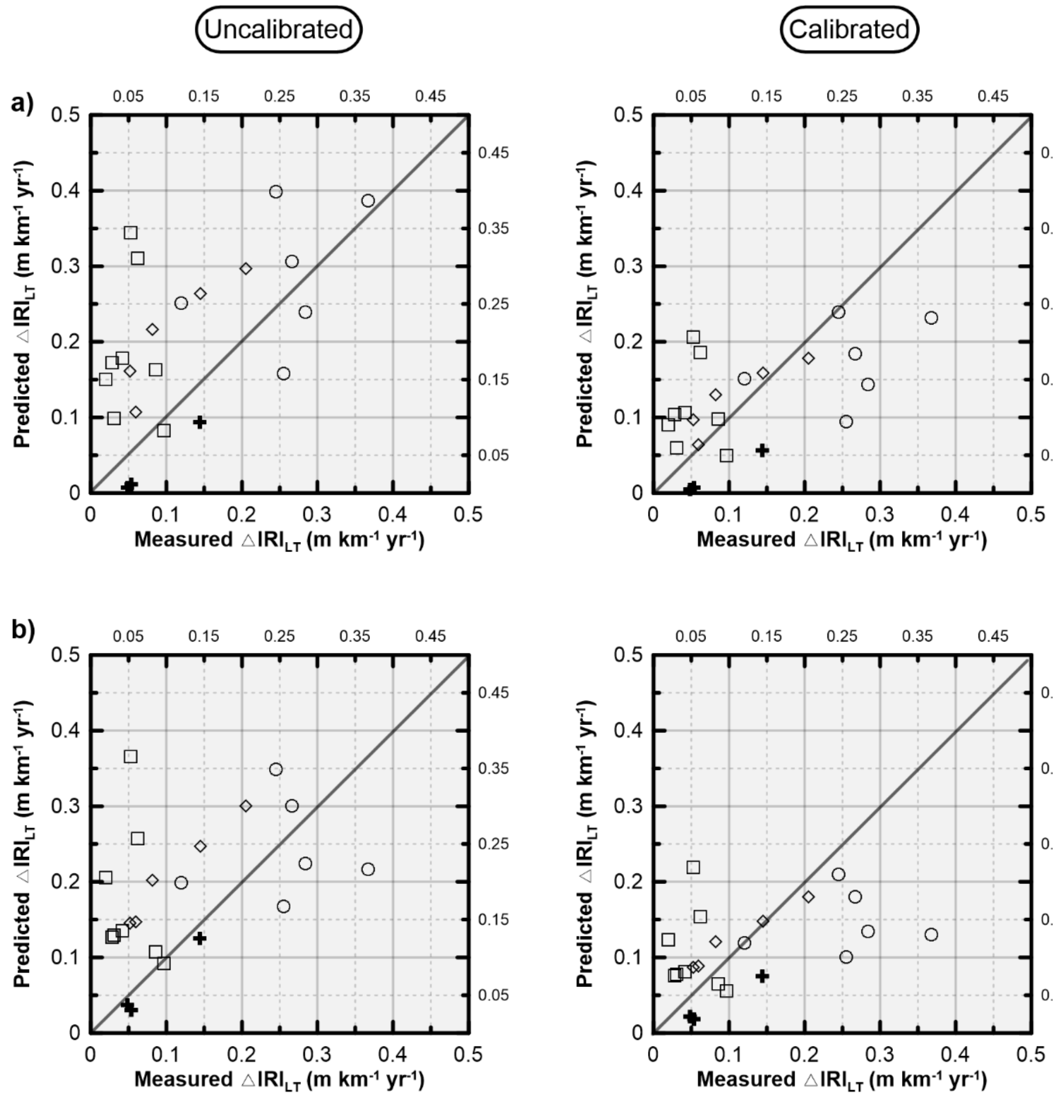

Figure 5 - Predicted vs measured values for the validation process with the uncalibrated and calibrated values for the model a and b