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**Flexible pavement damage during spring thaw: a field study
 using the Falling Weight Deflectometer**

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Complete List of Authors:	Bilodeau, Jean-Pascal; Université de Laval, Department of civil engineering, Laval University; Doré, Guy; Université Laval,
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 Manuscripts

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2 **Falling Weight Deflectometer**

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4 **Corresponding author**

5 Jean-Pascal Bilodeau
6 Department of civil engineering
7 Laval University
8 1065 avenue de la Médecine
9 Québec, Canada
10 G1V 0A6
11 418-656-2131 #7242
12 jean-pascal.bilodeau@gci.ulaval.ca

13

14

15

16 Guy Doré
17 Department of civil engineering
18 Laval University
19 1065 avenue de la Médecine
20 Québec, Canada
21 G1V 0A6
22 418-656-2203
23 guy.dore@gci.ulaval.ca

24

25

26 Spring thaw creates critical performance conditions for pavement networks. The increase of
27 water content in the pavement environment is significant during spring thaw. Combined with
28 poor drainage conditions, materials condition variations are triggering factors that accentuate
29 the effect of heavy vehicle loading on pavement response and damage. Two experimental
30 pavement sections were monitored in 2014 and 2015 for temperature and deflections. the
31 section with the lowest structural capacity was found to be more sensitive to thaw weakening.
32 Fatigue damage calculated for this section was found to be 31 % higher than the section with
33 the highest structural capacity. Moreover, it was shown that a load reduction in the range of 20
34 % can decrease the total yearly damage by about 7 to 10 % for the considered test sections. In
35 general, fatigue damage was found to increase from spring onset to the warmest conditions of
36 the yearly cycle, in July.

37

38 Keywords: Spring thaw, flexible pavements, falling weight deflectometer, pavement damage

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Flexible pavement damage during spring thaw: a field study using the Falling Weight Deflectometer

1. Introduction and background

In areas subjected to seasonal freeze, the structural capacity of flexible pavements significantly varies throughout the yearly cycle. This variation is associated with, among other things, the effect of freezing and low temperatures on the stiffness of soils, unbound granular materials and asphalt concrete, and with the thaw weakening mechanisms. As a matter of fact, it is reported in the literature that a significant part of yearly pavement damage occurs during the spring thaw period (Janoo and Shoop 2004; Barksdale 1991; Saint-Laurent 2003). The weakening of flexible pavement during spring thaw can be divided into two distinct periods (Doré and Zubeck 2009; Konrad and Roy 2000; Doré 1997). In early spring, the thaw front penetrates the pavement from the surface, typically with a deeper thaw penetration underneath the center of the pavement. The melting of snow in pavement environments, as well as frequent liquid precipitation, will typically cause an important increase of the degree of saturation within the granular layers underneath the asphalt concrete layer due to poor drainage conditions in the thawed zone (Côté 1997;). This increase in the degree of saturation combined with poorly drained layers causes a decrease of material stiffness and resistance to permanent deformation (Bilodeau 2009; Lekarp et al. 2000a; Lekarp et al. 2000b; Theyse 2002; Bilodeau and Doré 2012; Suarez et al. 2010; Doucet and Doré 2004; Richter 2006; Ekblad 2007; Uthus 2007). Positive water

pressures were also reported during early spring periods (Thom and Brown 1987), leading to a decrease in effective stress.

During the period referred to as late spring, the upper thaw front reaches subgrade soils that are typically more thaw sensitive, while the lower thaw front slowly migrates upward (Konrad and Roy 2000). The melting of ice lenses that formed in the subgrade soil during winter combined with water from melting snow and ice and frequent rain events in spring contribute to high degree of saturation in the soils. The vulnerable soil state and the consolidation process in thawing soils are the main factors contributing to the decrease of the pavement bearing capacity and the increase of its sensitivity to damage accumulation (Doré 2004). It is reported in the literature that weakening in flexible pavements can be associated with the rate of thawing, the rate of consolidation, the thickness of the subgrade soil affected by frost action and the magnitude of frost heave (Doré 2004; Andersland and Ladanyi 2004; Nixon and Morgenstern 1971). Because of their typically higher segregation potential, water sensitivity and lower drainage capacity, fine grained subgrade soils typically show higher bearing capacity loss during springtime. A 50% loss is expected in fine-grained soils, while 10-35 % loss is expected in clean sand and gravels to silty-clayey gravel-sands (Doré and Zubeck 2009). Pavement damage, such as fatigue and alligator cracking, permanent deformation, and roughness deterioration, are generally found to significantly increase during spring thaw (Doré and Savard 1998).

The effect of variation in the degree of saturation of flexible pavement performance, has been investigated by Saevarsdottir and Erlingsson (2013). Using accelerated pavement testing of an experimental pavement structure, for conditions reported as moist and wet (silty sand subgrade

water content of 8 and 16%, respectively), the authors found a significant increase in tensile strain at the bottom of the asphalt concrete layer (between 20- 45%, for two different flexible pavement structures). Moreover, in a recent FWD study on an experimental site in Sweden, Salour and Erlingsson (2013) showed that the stiffness decrease during spring thaw was about 48% for pavement granular layers and 60% for a sandy silt subgrade. During this study, the thaw-weakening period lasted about three weeks, and the recovery period lasted more than two months. Deflection indices—such as Maximum Deflection (D_{max}), the Surface Curvature Index (SCI), the Base Deflection index (BDI) and the Base Curvature Index (BCI)—all reached a yearly maximum during the spring thaw. All indices increased by approximately 30% during thaw weakening (relative to recovered values). The dissipated energy measured in the hysteresis loop of FWD deflection sensors also confirmed that a significant amount of damage occurs in the pavement system during spring thaw. In a weakened state, the measured dissipated energy can be approximately 3.6 times larger than the dissipated energy measured in recovered state. Finally, the back calculated moduli for a given layer during spring thaw were highly correlated with the water content in the layer.

Due to bearing capacity loss during the spring period, many transportation agencies enforce spring load restrictions (SLR) for commercial vehicles in order to protect and maintain the

107 pavement network. This is the case in Canada, in some US states, and in some European
108 countries. According to Ovik et. al. (2000), SLR are typically enforced at the end of February or
109 beginning of March, and remain in effect until April or May (an eight-week period is standard).
110 According to Doré and Zubeck (2009), regional and local networks are most sensitive to thaw
111 weakening and many agencies use these regulations primarily to protect the secondary
112 network. These regulations are likely to have an important impact on pavement life. Indeed,
113 load reductions of 20-50% during spring thaw may extend pavement life by 62-95%. According
114 to Saint-Laurent (2003), actual SLR practices in Quebec province (Canada) help reduce the truck
115 factor of heavy vehicle by about 22%, but cause an increase of about 7% in number of trucks
116 using the network. Load-associated damage during spring thaw makes up 18-72% of the total
117 yearly damage. According to Saarenteko and Aho (2005) weight restrictions that are
118 implemented seven days too late can decrease pavement service life by 4-8%. These
119 percentages can increase to 5-12% if winter loading premium loads are removed seven days too
120 late. The expected savings of an SLR program was documented by C-SHRP (2000) for
121 administrations around the world. The study took various factors (such as climate and traffic)
122 into account and found that the average savings associated with SLR is approximately 79%
123 (when an economic analysis is performed for a severe winter with and without SLR).

124

125 As shown Table 1, spring load restriction practices vary across Canada (Bullock et al. 2006; C-
126 SHRP 2000). According to Doré and Zubeck (2009), there are as many criteria and regulations as
127 there are northern transportation agencies. MacLeod et al. (2002) and Saarenteko and Aho
128 (2005) state that these regulations change with the need, size, and duration of the restriction
129 period. The criteria used to decide when to begin SLR is usually based on direct or indirect

methods. Ovik et al. (2000) summarized the criteria used to decide when SLR procedures should be in effect. The methods include:

- Fixed date;
- Engineering judgement;
- Pavement history;
- Pavement design;
- Visual observations (such as water seeping from the pavement);
- Daily air and pavement temperature measurements;
- Frost depth measurements (drive rods, frost tubes, or other sensors);
- Deflection tests.

As stated by Ovik et al. (2000), deflection testing is less practical than other techniques, due to the relatively rapid changes in conditions during early spring thaw, combined with the large territories and distances that need to be covered. It can be noted that, from the work of Ovik's (2000) on US states that enforce SLR policies using engineering judgement, experience and analytical approaches, qualitative approaches are more used in general practice than quantitative approaches.

Considerable research efforts have been conducted in the past few decades on thaw weakening. However, research on the complex interaction between heavy loads and pavement response during spring thaw is still necessary due to the important impact of SLR on the industry, the need to preserve the pavement networks. The research project presented in this paper is part of

a larger research initiative on the implementation of automated road weather stations in Quebec for continuous monitoring of pavement temperatures. As part of the project, two representative experimental pavement sections, , were intensively monitored during spring thaw. The objective of this paper is to quantify the amount of theoretical fatigue damage that occurs in typical pavement structures built in Quebec province using structural response measurements obtained from falling weight deflectometer tests.

Table 1. Criteria for imposing and lifting spring load restrictions across Canada (after Bullock et al. 2006)

Province	SLR imposition method	SLR lifting method
Quebec	Frost tubes and automated road weather stations (300 mm thaw depth)	5 weeks after thaw depth reaches 900 mm
Ontario	Temperatures and road condition	Fixed dates
Manitoba	Thawing index	Fixed dates
Saskatchewan	Thermistors, climate and road conditions	Road inspection or maximum length of 6 weeks
Alberta	Frost tubes (300 mm thaw depth), climatic conditions	FWD and frost tubes
British Columbia	Frost tubes, engineering judgment, climate historical data	Benkelman beam survey
New Brunswick	Temperatures	FWD, Benkelman beam and road conditions
Nova Scotia	Climate and road conditions	Dynaflect and road conditions
Prince Edward Island	Frost tubes and Benkelman beam survey	Frost tubes, Benkelman beam survey and climate
Yukon	Thermistors	Thermistors
Northern Territories	Pavement inspection	Benkelman beam and pavement inspection

2. Experimental site and methods

Two experimental pavement structures located at the Laval University experimental site of Forêt Montmorency were intensively monitored for two years (2014 and 2015). Figure 1 presents the average daily temperature for the two years of monitoring. The air freezing index FI_a values were 1889 and 1938 °C*d for 2015 and 2014, respectively. The experimental site is located near kilometer 103 along national highway 175, on local road 33. The two experimental pavement structures are built over a silty sand subgrade (SM) with 20 % of fine particles and except for the asphalt concrete thickness, they are identical. As illustrated in figure 2, the pavement layered systems tested consist of 450 mm of granular subbase (SB), 200 mm of granular base (B) and 100 mm or 200 mm of asphalt concrete (AC). It should be noted that the base and subbase materials at the test site are poorly differentiated and very similar.

Figure 1. Average daily temperatures for 2014 (a) and 2015 (b) at the Forêt Montmorency experimental site

Figure 2. Tested experimental sections

For the 100 mm AC section, the AC layer consists of hot-mix asphalt with a maximum aggregate size of 10 mm (ESG-10). For the 200 mm AC section, two different hot-mix asphalts were used. The top layer consists of an ESG-10 mix, while the bottom layer consists of a hot-mix asphalt with a maximum aggregate size of 20 mm (GB-20). Some of the volumetric properties of the AC layers, such as the effective bitumen V_{be} and void ratio V_v , are presented in Table 2. The fatigue damage will be quantified in this study. Therefore, Table 2 also introduces the KF1 parameter, which is obtained from the mix properties and which is used in the Asphalt Institute's empirical transfer function for fatigue cracking (Asphalt Institute 1991; Bilodeau et al. 2017). This function is defined by

$$N_F = 0.001135 \times K_{F1} |\epsilon|^{(-3,291)} \times |E^*|^{(-0,854)}$$

$$[2] \quad K_{F1} = 10^{4.84 \left(\frac{V_{be}}{V_v + V_{be}} - 0.69 \right)}$$

195

196 in which N_F is the allowable number of load repetitions prior fatigue cracking failure, ϵ is the
 197 tensile strain developing at the bottom of the asphalt concrete layer and $|E^*|$ is the dynamic
 198 modulus. Both mixes have been characterized using complex modulus testing and the results
 199 have been used to quantify the parameters of the Witczak model. This model is used in
 200 Quebec's standards (Doucet and Auger, 2012) and is defined by

$$[3] \quad \log |E^*| = \delta + \frac{\alpha}{1 + e^{(\beta + \gamma \log f_r)}}$$

202

$$[4] \quad f_r = a_T * f$$

204

$$[5] \quad \log a_T = a_1 * (T - T_r) + a_2 * (T - T_r)^2$$

206

207 Table 3 presents the results of the complex modulus characterization. Complex modulus tests
 208 have been performed in order to define the range of dynamic modulus values to be expected
 209 for the backcalculation process.

210

211 **Table 2. Asphalt concrete mix characteristics**

Parameters	AC 100 mm (ESG 10)	AC 200 mm (GB-20)
V_{be} (%)	5.3	4.5
V_v (%)	5.2	5.8
K_{F1}	0.127	0.060

212

Table 3. Witczak model parameters for the asphalt concrete

	ESG-10 (100 mm AC)	ESG-10 (200 mm AC)	GB-20 (200 mm AC)
δ	0.261	0.675	-0.543
α	4.196	3.811	4.983
β	-1.404	-1.362	-1.934
γ	-0.402	-0.422	-0.395
a_1	-0.145	-0.147	-0.144
a_2	5.01×10^{-4}	6.03×10^{-4}	3.61×10^{-4}

Falling weight deflectometer (FWD) tests have been carried out periodically over the two years of monitoring, with more intensive measurements during spring thaw. In 2014, 11 deflection tests were carried out between April 13th and September 24th. In 2015, a total of 10 deflection tests were performed between April 17th and September 22nd. Table 4 summarizes the dates at which the tests have been done along with the corresponding asphalt concrete temperature. FWD deflection basins have been used to backcalculate the modulus of each layer. For the AC layer, in addition to backcalculation, the dynamic modulus has also been determined using Witczak model using the temperature measured manually within the asphalt concrete layer and the load frequency f determined using the following relationship:

[6]
$$f = \frac{1}{2\pi t}$$

in which t is the duration of the half-sine loading pulse.

230

231 **Table 4. Dates of testing in 2014 and 2015 and asphalt concrete temperature**

2014		2015	
Date	T_{AC} (°C)	Date	T_{AC} (°C)
April 11th	12.3	April 13th	15.8
April 17th	4.3	April 16th	12.4
April 24th	3	April 20th	9.6
April 30th	7	April 22nd	8.9
May 5th	7.5	April 28th	11.2
May 12th	9.8	May 29th	26.6
May 20th	15.5	June 3rd	19.6
May 26th	17	June 11th	18
June 9th	25.3	August 24th	28.4
August 20th	22.9	September 24th	18.7
September 22nd	18	-	-

232 The deflection basins have been collected with the following geophone distances: 0, 200, 300,
 233 450, 600, 900, 1200, 1500 and 1800 mm. Loads of 32, 40, 53 and 71 kN were used on a 300 mm
 234 diameter plate, for surface contact stresses σ_0 of 453, 566, 750 and 1004 kPa. The 300 mm plate
 235 diameter was selected because it is the standard plate, which provides equivalent surface
 236 contact similar to the one obtained by standard dual wheel. Figure 3 presents examples of
 237 deflection basins collected at 40 kN for the 100 and 200 mm AC sections in 2014 and 2015.
 238 Variations in the deflection basins can be observed between the spring and summer period. The
 239 pavement behaviour is influenced by spring thaw weakening, but also by AC temperature. In
 240 order to quantify the pavement structural capacity, backcalculation of the layers modulus was
 241 performed. The backcalculation software ELMOD was used to fit the measured deflection at a
 242 distance i from load center d_i^m with the calculated deflection d_i^c using iterative layer modulus
 243 variations to minimize the residual mean square error (RMSE). This backcalculation optimization
 244 was performed using

245

246 [7] $|\varepsilon_i| = \left| \frac{d_i^c - d_i^m}{d_i^m} \right| = \left| 1 - \frac{d_i^c}{d_i^m} \right|$

247

248 [8] $RMSE(\%) = \sqrt{\frac{1}{n} \sum (100 * \varepsilon_i)^2}$

249

250 in which ε_i is the relative difference between d_i^m and d_i^c for a given geophone distance i from
251 the load center and n is the number of geophones. The deflection basins have not been
252 corrected for temperature in order to determine the appropriate modulus for each layer for
253 further analysis of pavement response using multilayer elastic theory. Moreover, no
254 temperature correction has been used since actual layer modulus were needed to assess the
255 allowable number of load repetitions with equation 1, and corresponding pavement damage.

256

Figure 3. Examples of deflection basins measured in 2014 and 2015 for the 200 and 100 mm AC test sections

3. Experimental results and discussion

Figure 4 presents the thaw penetration in the 100 and 200 mm AC sections for 2014 and 2015.

The frost penetration was larger in 2014, resulting in a larger thaw depth, as shown in the figure.

Figure 5 presents the results of the backcalculated elastic modulus of the pavement layers

266 during the monitoring period of 2014 and 2015. It can be observed that, as the thaw front
267 progresses downward within the pavement structure (Figure 4), the elastic modulus of the
268 layers changes significantly. It should be noted that the results presented are punctual
269 measurements during the monitored period and that they may be influenced by specific factors,
270 such as temperature, at the time of testing. For example, for the results presented, the
271 increasing air temperatures induces warming of the asphalt concrete layer, which can be
272 noticed from the spring to summer period. The highest dynamic modulus for the asphalt
273 concrete layer recorded during the monitoring period of April to November of each year,
274 reached 16000 MPa. The dynamic modulus decreased to 2700 MPa for the summer (100 mm AC
275 section). The important decrease of the stiffness of the unbound layers during the thawing of
276 the experimental pavement sections is noticed approximately from mid-April to May. The
277 decreased elastic modulus are about 30-40, 100-200 and 100-250 MPa for the subgrade,
278 subbase and base layer, respectively, depending on the year and pavement section considered.
279 For each year and experimental section, following the moment when the unbound layers and
280 soil elastic modulus were the lowest (thaw depth of approximately 900-1000 mm), the recovery
281 period is noticeable with respect to the experimental results collected. For the subgrade soil,
282 subbase and base, the recovered elastic modulus is in the range of 60-120, 300-450 and 300-450
283 MPa, respectively. The increase of the unbound granular layers elastic modulus during summer
284 is also partly attributed to the stress dependence of these materials. The progressive decrease
285 of the dynamic modulus of the asphalt concrete layer increases the stress transmitted to the
286 granular layers increasing thus their modulus.

287

288

Figure 4. Thaw penetration in 2014 and 2015 for the two experimental test sections

Figure 5. Backcalculated layers elastic modulus in 2014 and 2015 for the 200 and 100 mm AC test sections

A fatigue damage analysis was performed using the FWD data collected over the two years of experimental pavement response monitoring. This analysis is based on surface deflections obtained with the FWD tests. The deflection basins measured were used to backcalculate layers modulus, which were used to determine critical tensile strain at the bottom of the asphalt concrete using multilayer linear elastic calculations. From the tensile strain, the allowable number of load repetitions prior fatigue failure was determined using the empirical transfer

function introduced in equation 1. Finally, the damage D was calculated using the Miner's hypothesis of linear damage accumulation expressed by equation 9:

$$D = \frac{n}{N_{adm}} \quad [9]$$

in which n is the actual number of load repetitions and N_{adm} is the allowable number of load repetitions, calculated with Equation 1 for fatigue cracking (N_f). In order to quantify N_f using the empirical transfer function and then the damage, the tensile strain ϵ at the bottom of the asphalt concrete layer occurring under the FWD loading conditions of 40, 36 and 32 kN, was calculated using the software WinJULEA. This software uses the multilayer elastic calculation proposed by Burmister (1942). The backcalculated layers modulus (example in Figure 5), as determined with MODCOMP, for each load and testing date were used in WinJulea as elastic modulus. The layer thicknesses given in Figure 2 were used as inputs in the multilayer elastic software. A surface contact of 70686 mm² was used for the circular loading plate (diameter of 300 mm), and Poisson ratio of 0.35 were used for each layer, with the exception of the subgrade where a value of 0.45 was used. For the winter, typical elastic modulus values of 10000, 1000, 800 and 500 MPa were used for the AC, base, subbase and subgrade, respectively (Doré and Zubeck 2009). Since the actual number of load repetitions n is not a known parameter for this theoretical analysis, the damage D_i experienced for 1 load repetition at measurement i is used and is defined as

[10]
$$D_i = \frac{1}{N_{adm-i}}$$

in which n was set to 1. The value of D_i was calculated for each date where measurements were taken. The damage was normalized (D_{norm}) with respect to the damage measured in summer conditions with

[11]
$$D_{norm} = \frac{D_i}{D_{summer}}$$

where D_{summer} is the damage experienced in summer. The D_{norm} value allows a direct quantification of the effect of the variation of the environmental conditions in terms of pavement damage, which is inferred from fatigue empirical transfer function. Therefore, as the focus of this study is to document the effect of environmental condition on pavement response and performance, with focus on spring thaw, this data representation allows assessing the relative damage accumulation changes that occur from season to season. For each load considered, Figure 6 presents the AC thickness, the year and the D_{norm} with respect to time. For example, a D_{norm} value of 2 in Figure 6 would mean that a load applied at a specific date and for specific conditions would cause twice the damage induced by the same load applied in summer conditions. Considering the SLR policies used in Quebec, the SLR start and end dates for 2014 and 2015 are presented in Table 5. As observed in Figure 6, the D_{norm} value as a function of time shows that almost no damage occurs during the winter, but there is a rapid increase of damage when spring thaw begins until the pavement is at its warmest conditions around early July. The

results also show that the D_{norm} value decreases with a decrease of the load, emphasizing the effect of SLR.

Figure 6. Calculated normalized damage in 2014 and 2015 for the 200 and 100 mm AC test sections

Table 5. Start and end date for SLR in 2014 and 2015

AC thickness	Year	Start	End
200 mm	2014	April 15th	June 4th
200 mm	2015	April 15th	June 3rd
100 mm	2014	April 11th	May 30th
100 mm	2015	April 13th	May 26th

Considering the area under the curve for each D_{norm} as a function of time plot, it is possible to calculate the total relative damage occurring throughout the yearly cycle, as presented in Table 6. When looking at the data presented in Table 6, it is observed that the relative damage for the two experimental sections reduced by about 7% (200 mm AC, 0.82 to 0.76 in 2014) to 10 % (100 mm AC, 1.15 to 1.03 in 2015) when the load is reduced from 40 to 32 kN during spring time. The 100 mm AC test section shows the highest relative damage values, for both 2014 and 2015, with values higher than 1 in 2015 (1.15). This means that the total damage over the yearly cycle, considering a variable pavement response, is higher than if the pavement response was kept constant at summer-like conditions.

Table 6. Total relative damage calculated for loads of 40, 36 and 32 kN for the two experimental sections in 2014 and 2015

		40 kN	36 kN	32 kN
200 mm AC	2014	0.82	0.79	0.76
200 mm AC	2015	0.48	0.47	0.45
100 mm AC	2014	0.93	0.9	0.86
100 mm AC	2015	1.15	1.08	1.03

The damage analysis also allowed assessing the percentage of damage specific to each season. This calculation was performed using the historical season lengths for the test site, as defined in Table 7. A simplified procedure, based on the work of Drolet (2015) and Bilodeau et al. (2013) that uses five seasons during the yearly cycle (summer, fall, winter, early spring and late spring), was used to define the length of a season. Using a yearly temperature sinus function, the winter length is defined by the number of days where the temperature is below 0 °C. The length of the

season combining early and late spring has been established based on temperature data previously collected and was found to be 57 days for the test location considered, starting at the end of the winter. The length of summer combined with fall seasons is the time between the end of spring and the beginning of winter.

Table 7. Historical seasons durations at the test site

Season	Winter	Spring	Summer and Fall
Number of days	153	57	155

The results of this analysis are presented in Table 8. Table 8 shows that a significant portion of pavement damage occurred during summer and spring (98 to 99 % of yearly total damage) for the experimental pavement tested. Pavement damage during summer represents a significant part of yearly fatigue cracking damage. However, considering the fact that the spring period is much shorter than the summer period, the amount of damage that occurs on a daily basis during spring is of greater importance. The results presented in Table 8 also emphasize the effect of the pavement structural capacity, as the amount of damage during spring is closer to, if not greater than (for the 2015 monitoring period), the summer damage for the 100 mm AC section. However, the spring damage for the 200 mm AC test section is significantly lower than the damage occurring during summer. Therefore, the 200 mm AC test section is less sensitive to spring thaw conditions than the 100 mm AC test section. As for the comparison of both experimental sections for the spring period, the amount of seasonal damage is about 31 % higher (2014 and 2015) for the test section with the lowest structural capacity.

Table 8. Seasonal damage for the two experimental sections in 2014 and 2015

		Seasonal damage (%)		
		Winter	Spring	Summer
200 mm AC	2014	2	32	67
200 mm AC	2015	1	38	61
100 mm AC	2014	2	42	57
100 mm AC	2015	2	50	48

4. Conclusion

Spring thaw induces challenging performance conditions to flexible pavements in cold regions. Among other factors, the rapid increase of the degree of saturation in soils and pavement materials due to more frequent liquid precipitations, the melting of ice and snow and the melting of segregation ice exacerbated by poor drainage conditions, are all factors involved in damage rate increase during the spring period. This paper summarizes the results of an experimental investigation on the evolution of pavement response during spring thaw. Two test sections located at the Laval University experimental road site were intensively monitored for temperature and deflections using a falling weight deflectometer during 2014 and 2015. The experimental test sections were built on a silty sand subgrade and had the same structure, except for the asphalt concrete thicknesses, which were 100 and 200 mm. Falling weight deflectometer tests were carried through the yearly cycle, but were more intensive during the spring thaw period. The main findings of this research work are the following:

- A rapid increase of fatigue damage is observed from the beginning of spring thaw until the warmest conditions of the year;
- The decrease of the applied load results in a significant decrease of fatigue damage. The relative damage for the two experimental sections is reduced by about 7 % (200 mm AC, 0.82 to 0.76 in 2014) to 10 % (100 mm AC, 1.15 to 1.03 in 2015) when the load is reduced from 40 to 32 kN during spring time; The section with the lowest structural capacity shows the highest yearly damage relative to summer conditions, reaching relative damage values of 1.15;
- As expected, very little fatigue damage (1 – 2%) occurs during the winter period. The fatigue damage is shared between spring and summer in a proportion of approximately 50% spring and 50% summer for the thin asphalt section and of about 33% spring 66% summer for the thick asphalt section;
- On a daily basis, fatigue damage evolves 50% faster on the thick asphalt section during spring than during summer (0,63%/day compared to 0,41%/day). On the thick asphalt section, fatigue damage evolves almost three times faster during spring (0,84%/day compared to 0,34%/day);
- The 100 mm asphalt concrete test section is more sensitive to spring thaw than the 200 mm, the fatigue cracking damage being 31% higher for the 100 mm AC test section when compared to the 200 mm AC test section;

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