Evaluation of Environmental Impact on Perpetual and Conventional Pavement Designs: A Canadian Case Study

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Abstract

The Ministry of Transportation of Ontario (MTO) constructed a test section in partnership with University of Waterloo, the Ontario Hot Mix Producers Association (OHMPA) and various other partners to evaluate the use of perpetual flexible pavement design on Highway 401. Samples from different asphalt mixes used in the construction of the test sections were structurally evaluated by implementing dynamic modulus testing on the construction year. The samples were then stored and subjected to all seasonal effects, environmental impacts and aging for a year. The same asphalt samples were used to evaluate the dynamic modulus of the asphalt mixes after being subjected to freeze-thaw cycles representing one Canadian winter.

The dynamic modulus results showed strong statistical evidence that a significant deterioration in average |E*| results occurred in the SuperPave (SP) 12.5, SP 25 and SP 25 with Rich Bottom Mix (RBM). The deterioration mainly occurred at the results of dynamic modulus at low temperatures as -10ºC and 4ºC. However, the SP 19 mix showed weak statistical evidence of deterioration after one seasonal effect on the samples.

The dynamic modulus results were used to evaluate the benefits obtained by adding 0.8% of additional binder to the regular SP 25 mix to develop the SP 25 RBM. The dynamic modulus results did not show statistical significant difference between average |E*| of the SP 25 and SP 25 RBM on the construction year. Moreover, the benefits of additional binder content showed up clearly after one season of freeze-thaw cycles. The results of the dynamic modulus testing after one year of conditioning showed statistical evidence that the strain developed in SP 25 RBM is less than that developed in SP 25, when both mixes are subjected to the same load and loading frequencies especially at -10ºC and 4ºC.

INTRODUCTION

Pavement structural performance depends mainly on traffic load spectrum and environmental impact. Various pavement design theories and best practices consider traffic load, environmental impact and pavement life time in determining the appropriate design. In the last few decades, several research projects investigated the environmental impact on pavement performance. The development of sound correlation relating pavement structural performance to environmental impact has been one of the hot topics.

The Canadian climate is characterized by a wide range of temperature change resulting in a series of freeze-thaw cycles. The environmental impact results in severe damage to the Canadian highway network. The Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo designed and implemented a research project to study the climate impact on various designs of flexible pavements in Canada. The research project included the construction of three test sections on Highway 401. The three pavement designs include two perpetual pavement designs and one conventional flexible pavement design. This research will help designers, researchers, contractors and consultants, working in the pavement field, understand how perpetual pavement designs perform and deteriorate, taking into account the environmental and traffic impact, and by including the effect of freeze-thaw cycles on pavement performance and crack propagation.

The test sections constructed were all equipped with various sensors monitoring the tensile strain at the bottom of asphalt layers, moisture in the subgrade and temperature of pavement layers. The first stage of the project was to use asphalt samples from the different layers to execute several laboratory tests to structurally evaluate the mechanical performance of the different asphalt mixes.

SCOPE AND OBJECTIVE

This paper presents the results and analysis of dynamic modulus testing on the various asphalts mixes. The dynamic modulus testing was performed on four different asphalt mixes in the construction year. Subsequently, the dynamic modulus samples were stored at the CPATT Test Track and subjected to the normal air temperature for a year. The storage of these samples simulated the freeze-thaw cycles that impacted the asphalt installed on-site. In addition, the storage of samples simulated the aging effect of asphalt. The asphalt samples were re-tested again after being subjected to various environmental and aging impacts to investigate the freeze-thaw and aging effect on dynamic modulus results.

PROJECT DESCRIPTION

The project test site is located on the eastbound lanes of Highway 401 in southwestern Ontario. This highway connects Ontario from the US boarders with Windsor to Toronto, Kingston, Ottawa and leading to Quebec. This section of the highway is located between Waterloo and Woodstock Ontario. CPATT, Ministry of Transportation in Ontario (MTO), Ontario Hot Mix Producers Association (OHMPA), Natural Science Engineering Research Council (NSERC), Stantec Consultants and McAsphalt Industries Ltd. are partnering to evaluate the pavement performance of three flexible pavement designs. The three pavement designs include two perpetual pavement designs and one conventional flexible pavement design. The two perpetual pavement sections have 420 mm (16.5”) thickness of asphalt layers, while the conventional design includes 300 mm (11.8”) thickness of asphalt layers. The two perpetual pavement sections were constructed using the same asphalt mixes and having the same thickness with exception of the bituminous (binder) content in the bottom asphalt layer. One of the two perpetual designs is having additional 0.8% of binder content above the optimum value. The additional binder content increases the flexibility of the layer and enhances the structural capacity to resist fatigue bottom-up cracking at low temperatures. The perpetual section with RBM, perpetual without RBM and the conventional pavement cross sections are presented in Table 1.

SAMPLE PREPARATION

The asphalt mixes were prepared by the contractor supplying the asphalt material to construct the test sections. Capital Paving Inc. provided 500 kg of each mix to conduct laboratory testing in the CPATT laboratory. Asphalt samples were prepared to test dynamic modulus of the four asphalt mixes. Samples were prepared using the gyratory compactor and by following the ASTM D4013-09 (1) and AASHTO T312 (2) for preparing the gyratory samples and several trials were conducted for each mix prior to preparation of the samples that were tested. Air voids testing was conducted in accordance to AASHTO T269 (3) and the target air void ratio for laboratory prepared samples used for dynamic modulus testing was 7±1%. The number of gyrations was changed several times for all pavement mixes until it reached the appropriate number of gyrations needed to achieve the required air void ratio.

TABLE 1 Thickness and Binder Grade of Different Layers of the Test Sections

	Perpetual with RBM	Perpetual without RBM	Conventional Pavement
SP 12.5 FC2 – PG 64-28**	40 (1.5”)	40 (1.5”)	40 (1.5”)
SP 19 – PG 64-28	180 (7.0”)	180 (7.0”)	170 (6.7”)
SP25 – PG 58-28	100 (4.0”)	200 (8.0”)	90 (3.5”)
SP25 RBM – PG 58-28	100 (4.0”)	-	-
Granular A	200 (8.0”)	200 (8.0”)	200 (8.0”)
Granular B	550 (21.5”)	550 (21.5”)	200 (8.0”)

Note: Dimensions are in mm and inches are indicated between brackets.

** SP represents the Superpave mix design methodology which was followed in designing all asphalt mixes used in this project. FC2 is a representation for Friction Course two. This is an indication showing that aggregates used in this asphalt mix are 100% crushed aggregates.

Preparation of samples included four replicates of each mix per test although the minimum number of samples recommended in specifications of dynamic modulus testing was only three samples. An additional sample for each test was also carried out. All dynamic modulus samples were cored, cut and surface grinded according to the specimen preparation section of AASHTO TP62-07 (4).

DYNAMIC MODULUS RESULTS

The dynamic modulus test was performed according to the AASHTO TP62-07 specifications (4) and four replicates were tested for every asphalt mix. The specimens were tested at six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and five different temperatures (-10, 4, 21, 37 and 54 degree Celsius). The dynamic modulus results were analyzed to obtain master curves for the different asphalt mixes. Master curve development was implemented using the AASHTO PP62-09 specification (5). Table 2 presents the average dynamic modulus results for different asphalt mixes on the construction year. At this point, the samples were not subjected to environmental impact, freeze-thaw cycles and aging. Samples were tested again after being subjected to one season of environmental impact and the results after conditioning will be presented in Table 3.

The methodology of developing master curves relies on transferring the results obtained at different temperatures to a certain average temperature. This transferred temperature is usually assumed to be 34ºC (70ºF) (5). The shifting process will result in calculation of a reduced frequency leading to shifting the |E*| results (5).

The master curve is developed using shift factors. In the master curve, low reduced frequency presents the high temperature testing. Whereas, the high modified frequency on master curve represents the low temperature testing of dynamic modulus samples. Figure 1 demonstrates the master curve development theory.

Previous research projects concluded that rutting of asphalt pavement is expected to occur at high temperatures and low loading rates. This case can be simulated by the dynamic modulus testing at 54ºC and 0.1 Hz loading frequency (6, 7). In addition, fatigue cracking is also expected to take place at low temperatures and high loading rates as simulated by -10ºC and 25 Hz frequency.

TABLE 2 Average Dynamic Modulus Prior to Environmental Conditioning (MPa)

Temp (°C)	Frequency (Hz)	Superpave 12.5	Superpave 19	Superpave 25	Superpave 25 RBM
-10	25	28472	26582	34705	33985
	10	26903	25600	33230	32382
	5	25778	24568	31504	31685
	1	22178	21367	27732	28454
	0.5	20746	19897	26412	26897
	0.1	17813	16920	22506	23160
4	25	18235	16880	22762	23188
	10	16726	16117	21342	21676
	5	15429	15131	19603	20377
	1	12333	12156	15694	16650
	0.5	11189	11069	14001	15219
	0.1	8900	8795	10835	12566
21	25	9014	9287	11351	13264
	10	7757	8085	9625	11473
	5	7017	7088	8497	10391
	1	5317	5292	6098	8040
	0.5	4706	4799	5417	7217
	0.1	3688	3697	4149	5688
37	25	3711	3491	5462	6944
	10	3147	2857	4388	5803
	5	2750	2466	3784	5201
	1	2008	1833	2745	3789
	0.5	1812	1642	2424	3376
	0.1	1440	1330	2052	2597
54	25	1310	988	1543	2275
	10	1187	775	1344	1834
	5	1078	670	1271	1647
	1	875	513	1156	1257
	0.5	830	473	1105	1132
	0.1	681	403	1028	940

TABLE 3 Average Dynamic Modulus After Environmental Conditioning (MPa)

Temp (°C)	Frequency (Hz)	Superpave 12.5	Superpave 19	Superpave 25	Superpave 25 RBM
-10	25	23073	25367	23490	25842
	10	22386	24749	23246	25318
	5	21603	23985	22769	24512
	1	19089	20950	20578	21700
	0.5	18109	19850	19904	20622
	0.1	16691	17321	17726	18239
4	25	14555	16004	19093	17834
	10	13922	15188	18341	17981
	5	13067	13961	16702	16835
	1	10774	11457	14224	14193
	0.5	9984	10418	13029	13116
	0.1	8039	8619	10681	10962
21	25	9324	9835	10986	10944
	10	8370	8585	9687	9840
	5	7603	7547	8558	9402
	1	5797	5838	6283	7401
	0.5	5167	5254	5601	6666
	0.1	4057	4169	4375	5299
37	25	4034	3696	3485	5262
	10	3348	3117	2853	4563
	5	2920	2714	2449	3989
	1	2174	2064	1791	3023
	0.5	1961	1861	1610	2706
	0.1	1556	1527	1344	2178
54	25	1179	1061	1007	1523
	10	959	863	787	1269
	5	826	757	666	1091
	1	614	605	521	829
	0.5	545	565	486	752
	0.1	448	495	429	634

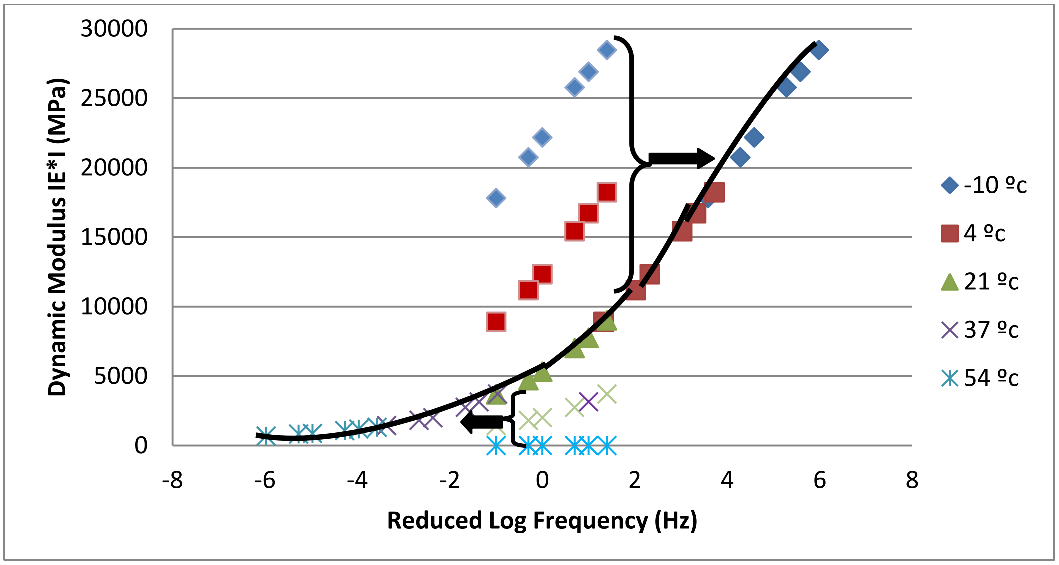

FIGURE 1 Master Curve Development Theory

ANALYSIS

The development of master curves can clearly show the change in dynamic modulus results along all temperatures and frequencies due to environmental conditioning and pavement aging (8). The master curves showing the initial dynamic modulus compared to the condition after freeze-thaw conditioning are presented in Figures 2-a, 2-b, 2-c and 2-d.

The analysis of dynamic modulus master curve should be demonstrated taking into account the physical interpretation of dynamic modulus results. The E* calculation is executed by a series of advanced calculations. The average dynamic modulus at a given temperature and frequency is a function of average stresses and strains measured throughout different cycles at the specified temperature and frequency. Average |E*| can be estimated as (4):

$$|E^*(\omega)| = \frac{|\sigma^*|}{|\overline{\varepsilon^*}|} \qquad \text{eq. (1)}$$

Where $|E^*(\omega)|$ is the dynamic modulus for a frequency ω and result would be in KPa

$|\sigma^*|$ is the stress magnitude in KPa

$|\overline{\varepsilon^*}|$ is the average strain magnitude. This is a function of strain magnitude for strain transducer and number of strain transducers.

The stress applied on the samples during testing in the construction year was identical to that applied in the second round of testing after the freeze-thaw conditioning. Thus, the change in |E*| results is inversely proportional to the strain occurring in asphalt sample, due to the same stress. If the |E*| is noted to decrease after the sample was subjected to environmental impact, this would indicate that strain resulting from the same stress has increased. Thus, the samples ability for deformation has increased and this would be a clear sign of deterioration due to environmental conditioning. It should be noted that this experimental matrix simulates the environmental impact and aging without considering the traffic loading applying on the pavement throughout different seasons.

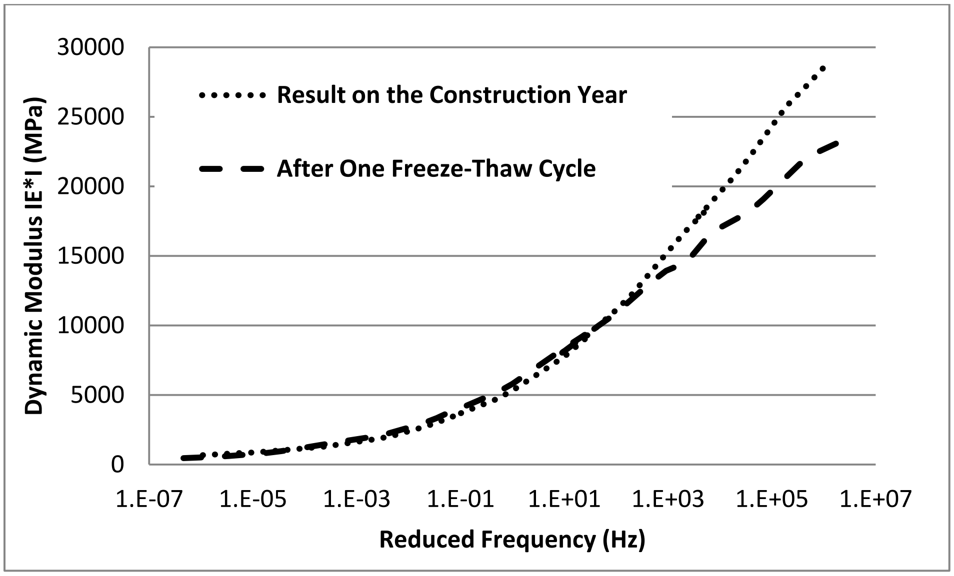

FIGURE 2-a Master Curve for SP 12.5

The environmental condition impact is noticeable in the SP 12.5 evaluation. The freeze-thaw cycles resulted in the deterioration of mix performance particularly at low temperatures regardless of the loading frequency. It is noted that the deterioration in |E*| results occurs mainly at temperatures -10ºC and 4ºC. The variation in |E*| result decreased in higher temperatures as 21, 37 and 54ºC. The surface layer of perpetual pavement is mainly design as rutting-resistant layer. This layer regularly milled and patched every few years due to the surface-down cracking (9). The performance of this layer in high temperatures did not demonstrate rapid deterioration. Hence, it maintained its structural capacity in rutting resistance after one season of freeze-thaw.

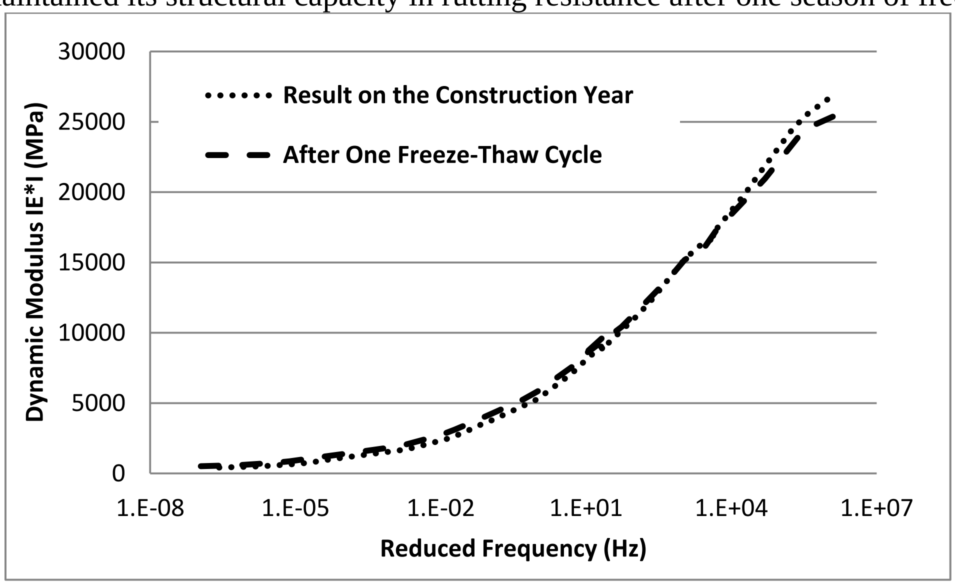

FIGURE 3-b Master Curve for SP 19

Limited deterioration in |E*| was noticed due to environmental conditioning. The partial deterioration mainly occurred in the -10ºC and high loading frequency of 25 Hz. The little deterioration is a reflection for a fatigue-resistant intermediate asphalt layer. This layer is fundamental for increasing the pavement lifetime resulting in a perpetual pavement design.

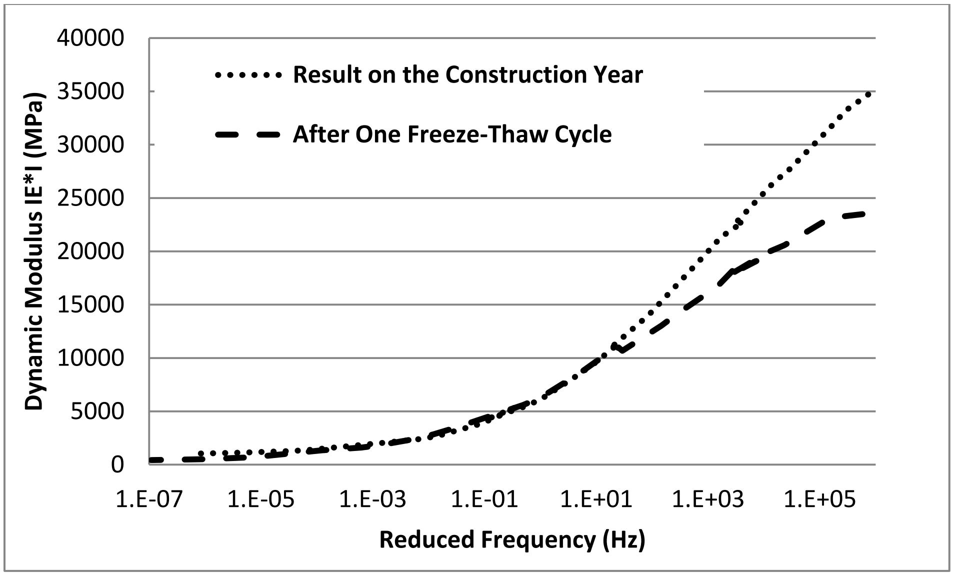

FIGURE 4-c Master Curve for SP 25

The lower asphalt layer is important in resisting bottom-up fatigue cracking. The success of this layer in resisting bottom-up fatigue cracking will have a significant impact on increasing the pavement life. The main purpose of this layer is to have sufficient structural capacity to resist traffic loads. Moreover, this layer should be flexible enough to withstand the expansion and contraction due to temperature variation in different seasons. The two alternatives of regular SP 25 and SP 25 with RBM were used as bottom asphalt layers in the perpetual pavement section with and without RBM, respectively.

The results for the SP 25 encounter noticeable deterioration in mix performance and decrease in |E*| results mainly through the -10ºC and 4ºC.

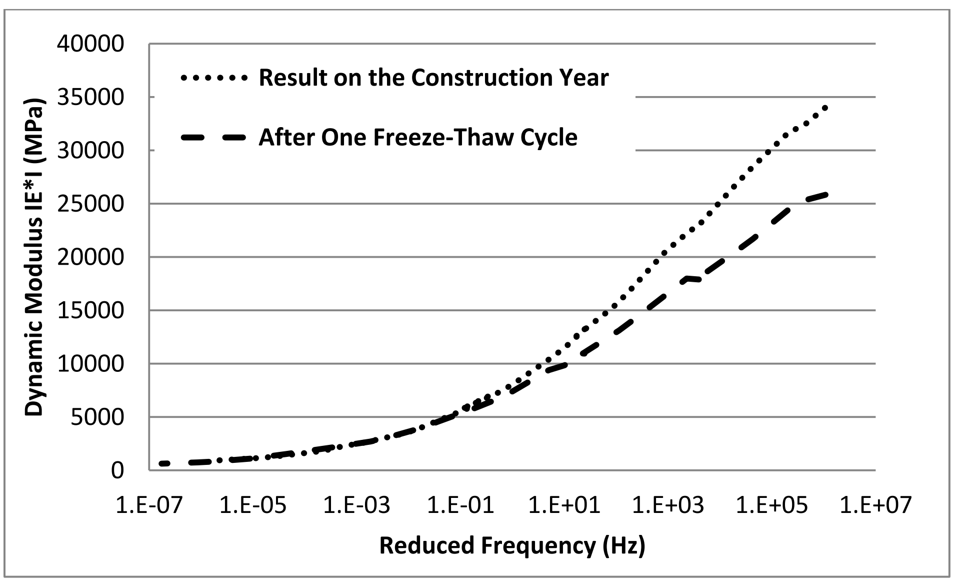

FIGURE 5-d Master Curve for SP 25 RBM

The deterioration in the SP 25 RBM mix was limited to performance in testing at -10ºC, 4ºC and 21ºC. The performance of this mix in the high temperatures maintained the high structural capacity to resist rutting. Moreover, the SP 25 RBM mix exhibited its superior performance at low temperatures compared to the regular SP 25 mix. This indicates that the additional 0.8% of binder content increased the mix flexibility and the ability to withstand freeze-thaw cycles with less deterioration. At -10ºC and 25Hz frequency, the average |E*| of the SP 25 RBM was noticed to be less than that in the SP 25 by 10% although both mixes were characterized by sparing difference in |E*| value in the construction year.

Statistical Analysis

The deterioration in different pavement mixes was statistically tested using t-test of paired two samples for means. The statistical t-test is used to verify that the decrease in |E*| results was statistically significant at low temperatures. The confidence level assumed in all t-test was 95%. Table 4 summarizes the results of various t-tests performed on the |E*| data at -10ºC for all asphalt mixes.

TABLE 4 Results of t-test Performed for Different Mixes

	SP 12.5		SP 19		SP 25		SP 25 RBM	
	Year 0*	Year 1	Year 0	Year 1	Year 0	Year 1	Year 0	Year 1
Null Hypothesis	H_o: μ_0- μ_1=0		H_o: μ_0- μ_1=0		H_o: μ_0- μ_1=0		H_o: μ_0- μ_1=0	
Alternate Hypothesis	H_1: μ_0- μ_1>0		H_1: μ_0- μ_1>0		H_1: μ_0- μ_1>0		H_1: μ_0- μ_1>0	
Mean	23648	20866	22489	22037	29348	21286	29427	22705
Variance	1.6E+7	7.0E+6	1.4E+7	1.1E+7	2.0E+7	5.2E+6	1.7E+7	8.3E+6
t_{Stat}	5.95		2.18		11.55		11.44	
P(T<=t) one-tail	4.82E-05		0.03		8.58E-08		9.5E-08	
$t_{Critical}$ one-tail	1.80		1.80		1.80		1.80	
Conclusion	Reject Null Hypothesis		Reject Null** Hypothesis		Reject Null Hypothesis		Reject Null Hypothesis	

* The initial |E*| in the construction year was presented as "Year 0" and the |E*| after one season of environmental conditioning was presented as "Year 1".
** The P-value in the case of SP 19 is less than the 0.05 (the value of assumed alpha based on 95% confidence interval). Moreover, the null hypothesis is rejected with weak evidence.

There is strong evidence based on the statistical testing that environmental conditioning resulted in a deterioration of the mean |E*| at -10ºC for mixes SP 12.5, SP 25 and SP 25 RBM. This conclusion was made based on a confidence level of 95%. Moreover, weak statistical evidence is showing that SP 19 mix was subjected to deterioration in |E*| values at -10ºC.

The statistical t-test was implemented to investigate the benefits gained by adding 0.8% of the binder content to develop the SP 25 RBM. The comparison was implemented at the construction year and after one season of freeze-thaw conditioning. The t-test of paired two samples for means was performed including the results of SP 25 and SP 25 RBM samples at -10ºC and 4ºC. Table 5 presents the summary of the t-test in the construction year and after one year of environmental conditioning.

The t-test results indicate that there was no significant statistical difference between the average |E*| of SP 25 and SP 25 RBM at temperatures -10ºC and 4 ºC in the construction year. Thus, the two mixes resulted in comparable strains due to applying the same stress at different frequencies. This result was disappointing in the construction year as it did not show the structural benefits from adding additional 0.8% of binder content to the regular SP 25 mix. However, the statistical t-test for the two mixes at temperatures -10ºC and 4ºC proved that there is weak evidence that the average |E*| of the SP 25 RBM is higher than that of the SP 25. This can be interpreted as the strains occurring in SP 25 RBM are significantly smaller than that occurring in regular SP 25 due to the same loading frequency. The t-test after one year of environmental conditioning showed promising results and the benefits of developing the RBM layer started to clarify after only one year of freeze-thaw cycles.

TABLE 5 Result of t-test Comparing SP 25 and SP 25 RBM

	Year 0		**Year 1**	
	SP 25	**SP 25 RBM**	**SP 25**	**SP 25 RBM**
Null Hypothesis	H_o: μ_{25}- $\mu_{25\ RBM}$=0		H_o: μ_{25}- $\mu_{25\ RBM}$=0	
Alternate Hypothesis	H_1: μ_{25}- $\mu_{25\ RBM}$>0		H_1: μ_{25}- $\mu_{25\ RBM}$<0	
Mean	23360	23853	18315	18930
Variance	5.6E+7	5.0E+7	1.7E+7	2.2E+7
t_{Stat}	-1.41		-1.94	
P(T<=t) one-tail	0.09		0.03	
$t_{Critical}$ one-tail	1.71		1.71	
Conclusion	Fail to Reject Null Hypothesis		Reject Null Hypothesis	

CONCLUSION

The dynamic modulus testing can provide a solid reference to compare strains developed in an asphalt mix due to loading at a certain temperature and frequency. The |E*| is inversely correlated to the strain in the asphalt sample. Through this project, samples representing four different asphalt mixes were tested in the construction year of the test section located on Highway 401 to determine their average dynamic modulus at five temperatures and six frequencies. The asphalt samples were stored and subjected to all environmental impacts and seasonal effects. After one year of sample environmental conditioning, the samples were tested again for dynamic modulus.

The dynamic modulus results proved that a significant deterioration in |E*|, especially at -10ºC and 4ºC. This conclusion was proven statistically by the t-test comparison between all the samples tested in the construction year and the results of the same samples after one year of conditioning. The statistical t-test showed strong evidence that deterioration in |E*| applied in mixes SP12.5, SP 25 and SP 25 RBM. Meanwhile, the SP 19 mix did not show statistical evidence of deterioration. However, further investigation of all mixes will be implemented as the samples are currently being subjected to freeze-thaw cycles simulating another seasonal effect and will be tested again. The deterioration trend will be further investigated once sufficient data representing several years of environmental impact are available.

The initially construction SP 25 RBM dynamic modulus sample results did not show statistical improvements with the addition of 0.8% binder to the SP 25. However, after one year of environmental impact, the benefits of the additional binder did indicate a statistical difference. The results of |E*| testing after one year proves that the average |E*| of the SP 25 is significantly lower than that of the SP 25 RBM. Thus, the strains developed in the SP 25 mix were significantly higher than those developed in the SP 25 RBM mix due to the same loading frequencies and at temperatures -10ºC and 4ºC. This difference will continue to be monitored over the life cycle of the pavement.

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