New England Transportation Consortium

NETC 18-2: Framework of Asphalt Balanced Mix Design (BMD) for New England Transportation Agencies

Technical Memorandum

Task 3: Collection of Existing Performance Data
Task 4: Development of Preliminary Recommendations for BMD Approach

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Prepared For:

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Project 18-2 Technical Committee

Background

In Tasks 3 and 4 of the study, state agencies were requested to provide laboratory performance data and Pavement Management data regarding the rutting and fatigue cracking field measurements in their respective state. The main premise of these tasks was to help develop performance criteria that the respective state agency could utilize that would relate a property measured in the laboratory to an expect field performance.

At the time of this memorandum, only three (3) state agencies provided information – Connecticut, Maine and Vermont. It should be noted that each of the three states provided different levels of information for the analysis;

- Connecticut – Connecticut DOT provided Pavement Management information but did not have asphalt mixture performance testing data to accompany their pavement sections;
- Maine – Maine DOT provided asphalt mixture performance data for the Hamburg Wheel Tracking test (rutting) and IDEAL-CT Index (fatigue cracking), as well as Pavement Management information. However, the Pavement Management data only pertained to those specific pavement areas where the laboratory evaluated asphalt mixtures were placed; and
- Vermont – Vermont AOT provided asphalt mixture performance data for the Hamburg Wheel Tracking test (rutting) and SCB Flexibility Index (fatigue cracking). The entire Vermont Pavement Management database was accessible to download for evaluation.

The differences in available information provides a good insight on how state agencies can look to develop asphalt mixture performance testing criteria with variable levels of data. Factors such as pavement structure (i.e. – thickness, flexible/composite) and traffic should also be considered as significant factors and may need to be addressed in the performance specifications.

It should be noted that to truly develop confidence in the performance criteria, a strong communicative relationship needs to exist between the respective state agency's Materials Bureau and Pavement Management Division. Both groups need to work with one another to catalog and monitor the materials in the field. Initial criteria may need continual modification as materials, production, and construction practices continue to evolve.

Connecticut

The Connecticut Department of Transportation (CDOT) provided the Research Team with access to their Pavement Management data. However, CDOT has not yet initiated asphalt mixture performance testing. Therefore, attempts to directly compare lab to field performance could not be accomplished. In turn, the Pavement Management information was utilized to identify "Good" vs "Poor" pavement surface performance where CDOT could recover field cores and evaluate the relative performance of the asphalt materials.

Rutting

Pavement sections from CDOT's Pavement Management System (PMS) were extracted to help define areas for future coring and laboratory evaluation. For this study, "Good" rutting performance was defined as wheelpath rutting less than 0.15 inches, while "Poor" rutting performance was defined as wheelpath rutting greater than 0.3 inches.

Tables 1 and 2 show the different Connecticut pavement sections noted as having Good and Poor rutting performance, respectively. The table is broken out by traffic level, which was defined as AADTT (ADT x % Trucks). To establish performance criteria, it is important to incorporate traffic level in the preliminary analysis to evaluate whether or not the field performance is dependent on the applied traffic levels. Additionally, the tables also contain the year the surface material was placed. Aging plays a critical role in the development of field distress and should also be considered when comparing test data. For example, rutting is generally an "early life" pavement distress – meaning that typically rutting occurs within the first few years after placement. Meanwhile, fatigue cracking is often a function of the amount of aging that occurs in the field, and therefore, is a "later life" pavement distress. With respect to rutting, one may not want to core field projects for lab characterization that are older than four to five years as the existing stiffness of those materials may be significantly higher than at the time when a majority of the permanent deformation was occurring (i.e. - < 3 years).

Table 1 – Connecticut Asphalt Pavements with Good Field Rutting Performance

Good Rutting Performance (< 0.15 Inches)						
Traffic Level	**RoadName**	**From**	**To**	**ADT x % Trk**	**RUT_AVG**	**SURFACE_ YEAR**
Low	042 L	9.6	11.8	138	0.10	2016
	082 L	17.6	20	198	0.08	2016
	066 L	27.6	29.6	398	0.07	2018
	011 L	10	13.4	441	0.10	2015
	006 L	84	88	630	0.10	2016
	0.58 L	0.2	3.4	716	0.10	2015
Moderate	008 L&R	7.9	8.9	2944	0.11	2011/15
	072 L	3.2	3.9	2463	0.13	2017
	095 L	101.7	103.9	4335	0.07	2018
	291 L&R	3.2	5.1	4357	0.13	2010
	384 L&R	1.6	2.8	5345	0.09	2014/15
	084 L&R	16.7	18.7	6500	0.13	2008
High[1]	091 L&R	3.4	4.5	10479	0.13	2012
	084 L&R	61.1	62.4	12110	0.12	2014
	095 L	10	15	12848	0.10	2015/16
	084 L&R	65	66.3	14540	0.09	2012

[1] - Almost anywhere on 084, 091, and 095

Table 2 – Connecticut Asphalt Pavements with Good Field Rutting Performance

Poor Rutting Performance (> 0.3 Inches)						
Traffic Level	**RoadName**	**From**	**To**	**ADT x % Trk**	**RUT_AVG**	**SURFACE YEAR[1]**
Low	201 L	15.5	17.7	68	0.43	2013
	695 L	2.1	2.7	214	0.4	2018
	167L	5.1	6.6	466	0.29	2018
	004L	36.5	38	550	0.29	2013
	044L	51.5	52.6	740	0.29	2014
Moderate	072L	2	2.8	2843	0.28	2017
High	084L	56.3	57.4	10688	0.33	2016

[1] - Older the resurface, more aged asphalt binder
(rutting may have occurred much earlier than present condition)

Fatigue Cracking

Similar to the Rutting analysis shown earlier, the CDOT PMS was mined to determine locations of "Good" and "Poor" fatigue cracking pavement sections. Table 3 contains recommended pavements sections that show relatively low levels of fatigue cracking, while Table 4 contains the "Poor" fatigue cracking pavement sections. For the analysis, the CDOT HPMS Crack Percentage and Wheelpath + Non-wheel Load Associated cracking parameters were used as field fatigue cracking indicators. Once again, AADTT and age of pavement surface are included in the tables for analysis purposes.

Table 3 – Connecticut Asphalt Pavements with Good Field Fatigue Cracking Performance

Good Cracking Performance (HPMS Crk Pct & WP+NWL Crk)						
Road Name	From	To	ADT x % Trk	HPMS Crk Pct	WP + NWL Crk	SURFACE YEAR
001 L	14.2	15.8	618	0	4.9	2017
001 L	28.2	20.4	633	0	5.2	2017
009 L	0.6	4	1284	0	5.4	2011
030 L	17.9	20.9	189	0	0.4	2017
058 L	1	3.5	797	0	3.2	2015
084 L	19.1	22.1	5956	0	22.6	2012
091 L	39	42	12415	0	0.9	2015
095 L	95	98	5700	0	11	2014
198 L	8.2	11.9	88.2	0	2	2009
244L	0	3	74.8	0	4.7	2014
395 L	0.6	3.6	2398	0	8.3	2009

Table 4 – Connecticut Asphalt Pavements with Poor Field Fatigue Cracking Performance

Poor Cracking Performance (HPMS Crk Pct & WP+NWL Crk)						
Road Name	**From**	**To**	**ADT x % Trk**	**HPMS Crk Pct**	**WP + NWL Crk**	**SURFACE YEAR**
030 L	2.0	3.5	374	32.5	213	2000
045 L	2.2	3.7	78	32.8	180	1995
083 L	19.5	21.9	219	33.5	218	1998
179 L	5.0	8.0	118	25.6	145	1999
201 L	7.6	9.2	50.4	22.8	99	1993
305 L	1.0	3.0	566	19.0	132	1999
534 L	0.5	3.5	182	29.7	193	1996

Maine

The Maine Department of Transportation (MaineDOT) provided the research team with asphalt mixture performance test results and PMS data for the pavement sections the respective asphalt material was placed. The complete PMS data was not provided, however, by utilizing the distress information for the exact pavement sections where the asphalt mixtures were placed, it is hopeful that a lab to field relationship can be established. For this study, MaineDOT utilized the Hamburg Wheel Tracking test for rutting evaluation and the IDEAL-CT Index test for fatigue cracking evaluation of asphalt mixtures.

General Mixture Performance – Fatigue Cracking

The asphalt mixture performance data was reviewed to see the general asphalt mixture performance for the IDEAL-CT Index testing as MaineDOT had previously identified fatigue cracking as the most prominent pavement distress. Figure 1 presents the different asphalt mixtures based on their relative RAP content. The general trend in the data shows that as RAP content increases, the IDEAL-CT Index also decreases, which would be an indication of poorer fatigue cracking performance. It should be noted that the 0% RAP only had two mixtures with greatly varying IDEAL-CT Index performance (491.0 and 183.5). This is also highlighted by the large error bars, which indicate the standard deviation above and below the average value.

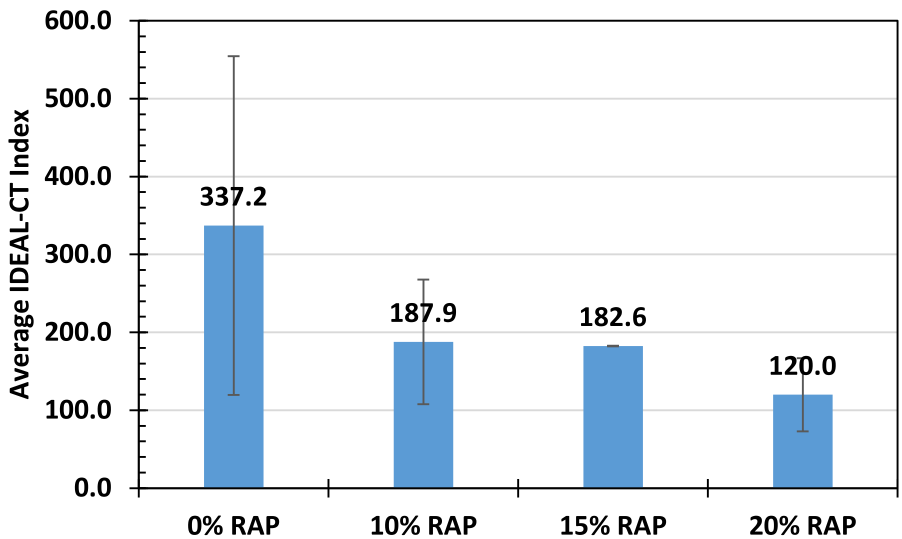


Figure 1 – MaineDOT IDEAL-CT Index Values for Different RAP Contents

Further breaking out the asphalt mixtures by asphalt binder grade and RAP content shows that (Figure 2);

- For the PG64-28 asphalt binder, 0% to 15% RAP content results in very similar IDEAL-CT Index results. However, as the RAP content increased to 20%, there was a significant decrease in the values, similar to the results in Figure 1;

- The use of polymer-modified asphalt binders (PG64E-28 and PG70E-28) resulted in a significant increase in IDEAL-CT performance at 0% RAP. However, once RAP was added to the polymer-modified asphalt binders, the results were extremely similar to the unmodified PG64-28 asphalt binder results.

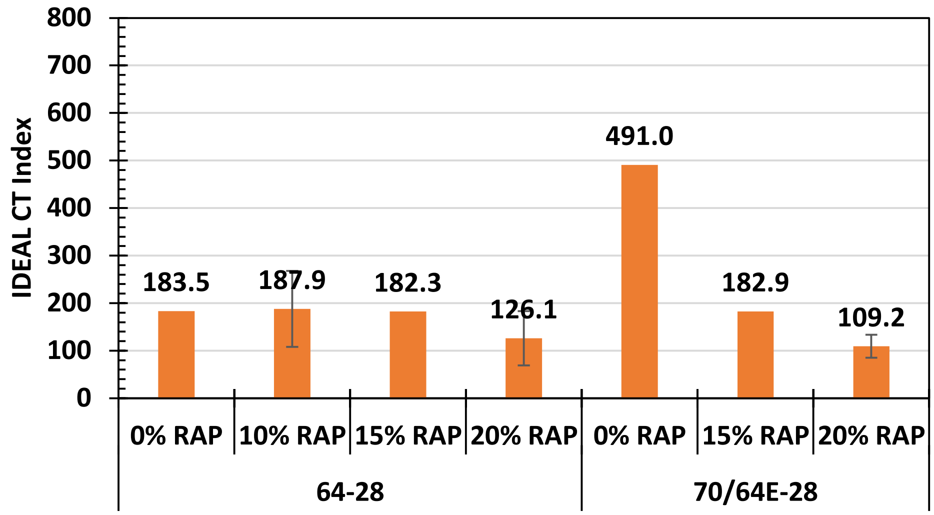


Figure 2 – MaineDOT IDEAL-CT Index Values for Different RAP Contents and Polymer Modified Asphalt Binders

Fatigue Cracking – Lab vs Field

One of the difficulties in developing lab vs field relationships for fatigue cracking is that often fatigue cracking takes time to develop in the field. The aging the asphalt material undergoes plays a significant role in the cracking response. Further creating calibration issues are the different modes of cracking; Load Associated Wheelpath, Non-Load Associated (Outside of Wheelpath), Transverse Cracking (Non-Load Associated/Thermal Cracking), and Reflective Cracking (commonly found with composite pavements). Therefore, it is important that state agencies understand the need to allow for additional time for fatigue cracking analysis, as well as the need to categorize and separate different pavement types (i.e. – flexible vs composite) and possibly even different cracking modes (i.e. – load associated cracking vs thermal cracking) when possible.

For the MaineDOT analysis, a few cracking parameters from the PMS were used to compare to the IDEAL-CT Index; 1) % Cracking, 2) CRACK FUNC, and 3) CRACK STRC. Additionally, only the latest cracking measurements (2019) were used in the analysis to allow as much time to have passed after construction. Information was not provided on how the indices were calculated, however, the general relationship between the parameters are shown in Figure 3. Additionally, it is not known what the threshold values are for these indices before MaineDOT takes some type of maintenance action.

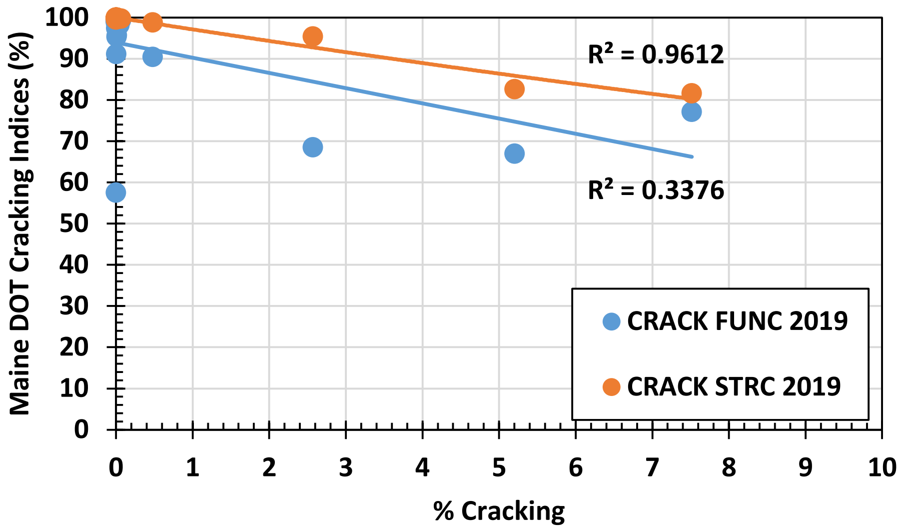
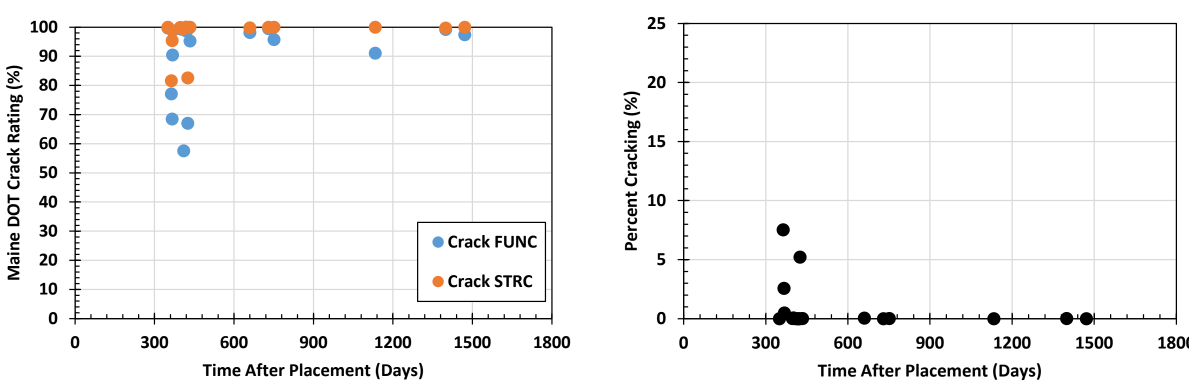


Figure 3 – Relationship Between MaineDOT PMS Cracking Indices with 2019 Field Data

As mentioned earlier, cracking performance is significantly influenced by aging, and therefore, the amount of time after the asphalt mixture has been placed. An attempt to compare the MaineDOT Cracking Indices and the time after the material was placed is shown in Figure 4. Unusual to notice that there appeared to be cracking occurring in earlier stages after placement. This may be an indication that the cracking observed on these particular projects may not necessarily be due to aged induced factors, but most likely from load associated factors.



(a) (b)

Figure 4 – Comparison of MaineDOT IDEAL-CT Pavement Sections; (a) Crack Ratings (CRACK FUNC and CRACK STRC) vs Time After Placement; (b) Crack Ratings (% Cracking) vs Time After Placement

Figure 5 shows the same cracking indices from Figure 4, but this time, compared to the Average Annual Daily Traffic (AADT). Pavement designs are commonly done with Average Annual Daily Truck Traffic (AADTT) as it is well known that truck traffic generates a significantly larger amount of distress on a pavement than car traffic. However, Percent Trucks was not provided in the data, so the traffic information is represented by AADT. In Figure 5, there does appear to be a moderate relationship between the MaineDOT PMS cracking indices and AADT. This would suggest that within the timeframe these particular asphalt mixtures were placed, the primary factor creating the measured cracking distress was the traffic. Therefore, traffic levels may need to be included in future performance criteria for Maine's Balanced Mixture Design.

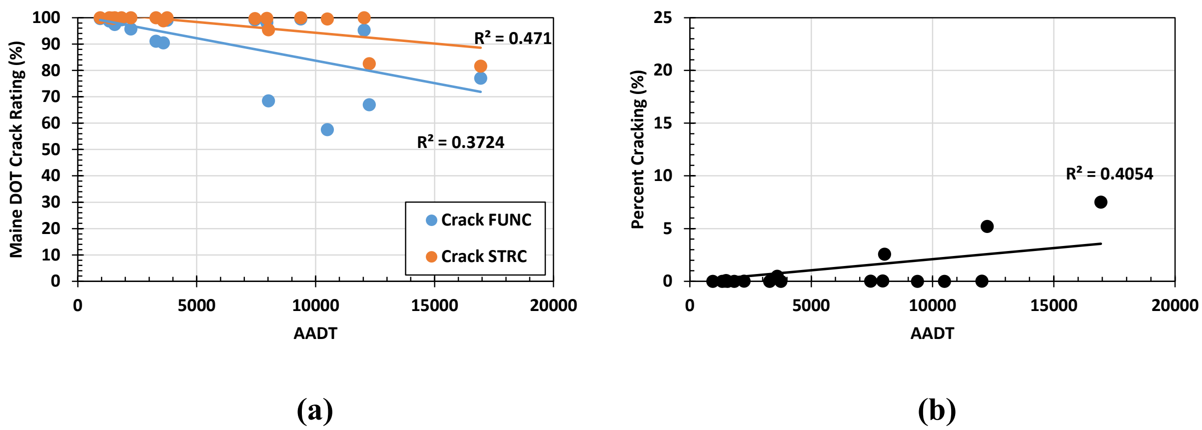
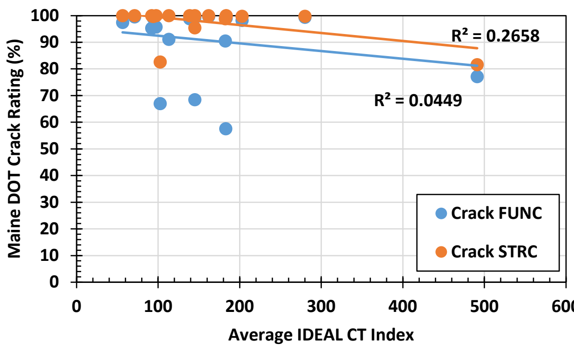
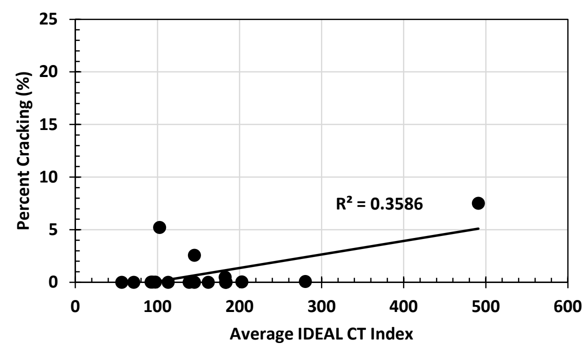


Figure 5 – Comparison of MaineDOT IDEAL-CT Pavement Sections; (a) Crack Ratings (CRACK FUNC and CRACK STRC) vs AADT; (b) Crack Ratings (% Cracking) vs AADT

An initial attempt was made to directly compare the IDEAL-CT Index values to the MaineDOT PMS cracking indices. Figures 6a and 6b show the results of the analysis. The overall trends are counter-intuitive than one would expect or hope. The results in Figures 6a and 6b show that as the IDEAL-CT Index value increases, the measured cracking in the field also increases. One would expect that good performing asphalt mixtures (i.e. – higher IDEAL-CT Index values) should result in better field cracking performance.





(a) **(b)**

Figure 6 – MaineDOT Cracking Indices Compared to IDEAL-CT Index Values

Additional analysis was conducted to determine why the relationship between lab and field cracking did not follow an expected trend. Figures 7 through 9 show the asphalt mixtures broken out by asphalt binder grade and RAP percentage while compared to the MaineDOT cracking indices. Overall, the PG64-28 asphalt binder mixtures appeared to have lower field cracking when compared to the polymer modified PG64E-28 and PG70E-28. In fact, the pavement section with the worst cracking performance also happened to be the section with the PG70E-28 asphalt binder. Once again, this is counter intuitive to what one would expect as this would have to be a highly polymer modified asphalt binder.

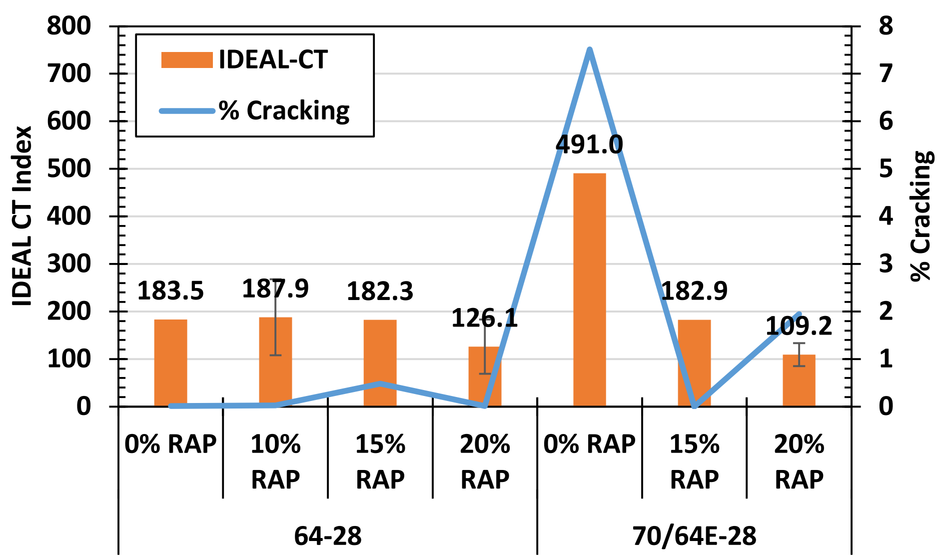


Figure 7 – IDEAL-CT Index and MaineDOT % Cracking for Different Asphalt Binder Grades and RAP Contents

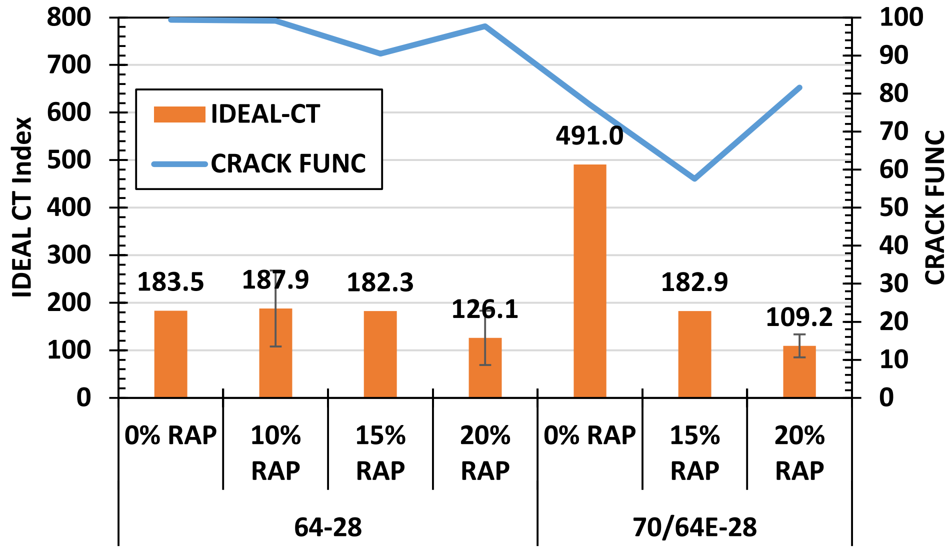


Figure 8 – IDEAL-CT Index and MaineDOT CRACK FUNC for Different Asphalt Binder Grades and RAP Contents

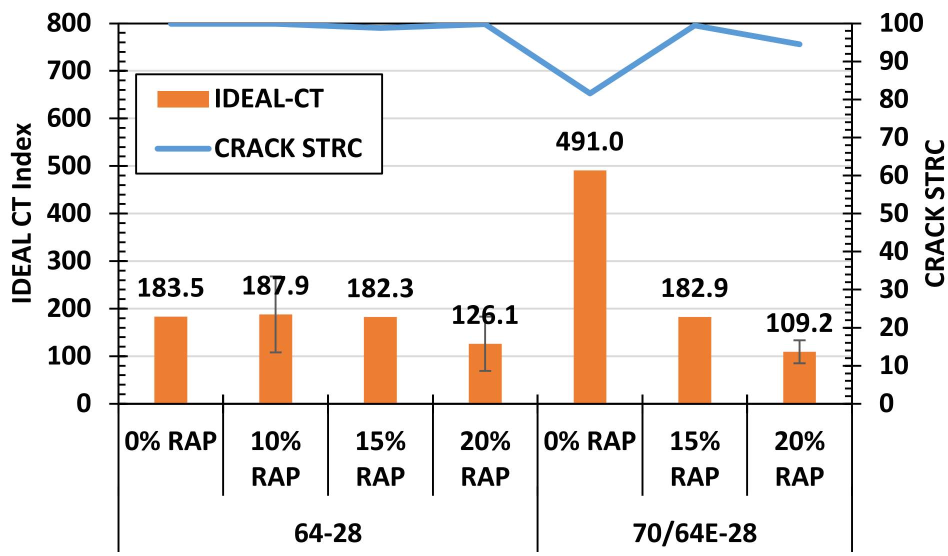


Figure 9 – IDEAL-CT Index and MaineDOT CRACL STRC for Different Asphalt Binder Grades and RAP Contents

The previous figures showed a troubling situation occurring where the polymer-modified asphalt mixtures were under-going higher levels of field cracking within the first five years of service life. However, as shown earlier, the field cracking was found to be related to the traffic levels (AADT). AADT and the IDEAL-CT Index values were plotted against the different asphalt mixtures (PG grade and RAP content). Figure 10 shows the results of the analysis. It is clear from the graph that higher traffic levels are associated for the polymer-modified asphalt mixtures. In fact, the PG70E-28 pavement section that achieved the IDEAL-CT Index had the highest level of traffic. Figure 10 clearly demonstrates high significant the impact of traffic volume was in the analysis, and that traffic levels should be included in the performance criteria.

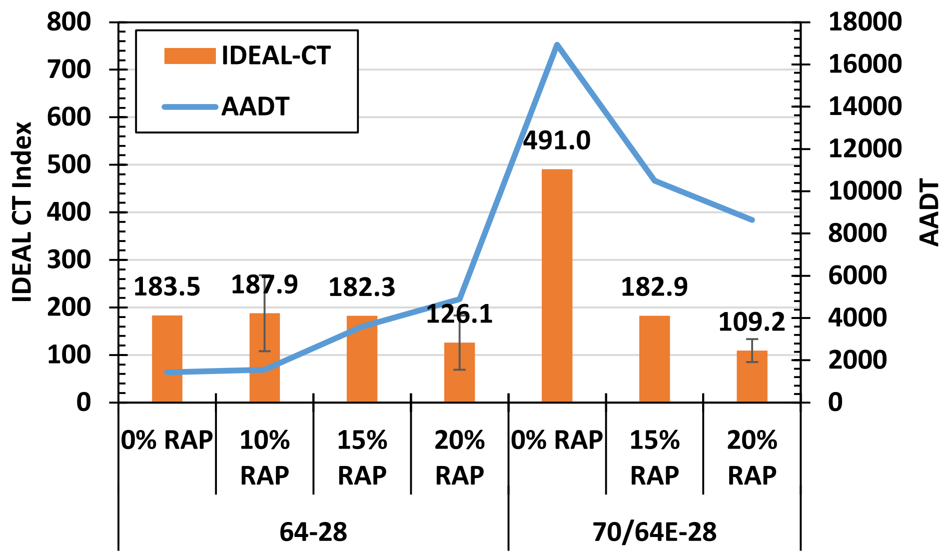


Figure 10 – Resultant IDEAL-CT Index Values for Different Asphalt Mixtures with the Pavement Sections' Traffic Volume (AADT)

To help move forward in establishing some recommendations for IDEAL-CT Index criteria for Balanced Mixture Design, the data provided was broken out into three different traffic categories; 1) Less than 5,000 AADT; 2) 5,000 to 10,000 AADT; and 3) Greater than 10,000 AADT. Table 5 contains the data used and divided into these divisions.

Using the IDEAL-CT Index values divided by the AADT reported for the pavement section constructed, Figure 11 was generated. It should be known that the four asphalt mixtures evaluated for AADT > 10,000 contained the PG70E-28 asphalt mixture that achieved a 491.0 IDEAL-CT Index. This greatly influenced the average test results so two different data are shown to represent the average IDEAL-CT Index at > 10,000 AADT – one with and one without the PG70E-28. Assuming that the PG70E-28 is very uncommon, the average results without the PG70E-28 is used for comparison among the other mixtures. The MaineDOT test results show that as the AADT increases, the average IDEAL-CT Index values decrease. Further review of the individual mixture designs would be required to help determine the exact reasoning, however, one of the most likely reasons is the increase in gyration level as traffic level increases.

Typically, as gyration level increases, the aggregate skeleton is pushed tighter together, essentially squeezing out asphalt and thereby lowering effective asphalt contents. Lower effective asphalt contents, in conjunction with higher traffic levels, could have led to the higher levels of cracking observed in the MaineDOT fatigue cracking indices.

Table 5 – Asphalt Mixture Fatigue Cracking Data for Different AADT Divisions

AADT	Age (Days)	PG Grade	RAP %	Ave CTI	CRACK FUNC 2019	CRACK STRC 2019	Percent Crack
942	350	64-28	20	161.8	99.683	100	0.0001
1327	418	64-28	10	138.5	98.775	100	0.0001
1440	397	64-28	0	183.5	99.392	99.906	0.0105
1488	401	64-28	10	280.1	99.341	99.721	0.068
1546	1471	64-28	20	56.5	97.433	99.992	0.003
1830	418	64-28	10	145.0	99.224	100	0.0001
2230	751	64E-28	20	97.3	95.709	99.967	0.011
3286	1133	64-28	20	113.1	91.107	99.977	0.002
3596	368	64-28	15	182.3	90.431	98.813	0.48
3747	424	64-28	20	183.5	99.044	100	0.0001
7444	1399	64-28	20	93.9	99.188	99.686	0.018
7944	660	64-28	20	202.8	98.166	99.754	0.044
8017	366	64E-28	20	145.0	68.484	95.411	2.571
9375	730	64-28	20	71.2	99.487	100	0.001
10493	410	64E-28	15	182.9	57.531	99.478	0.0001
12035	434	64E-28	20	92.1	95.277	99.962	0.009
12248	425	64E-28	20	102.6	66.992	82.589	5.205
16939	363	70E-28	0	491.0	77.078	81.617	7.514

Figures 12 and 13 show the MaineDOT PMS cracking indices also broken out into the three AADT divisions. It is very clear from the graphs that as the AADT increases, greater magnitudes of cracking are observed.

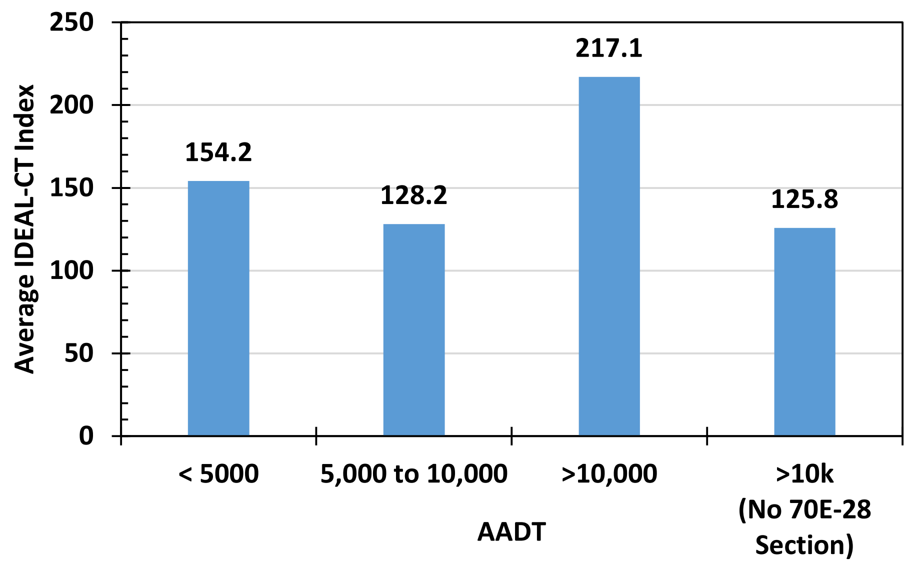


Figure 11 – MaineDOT AADT Divisions and Resultant IDEAL-CT Index Values

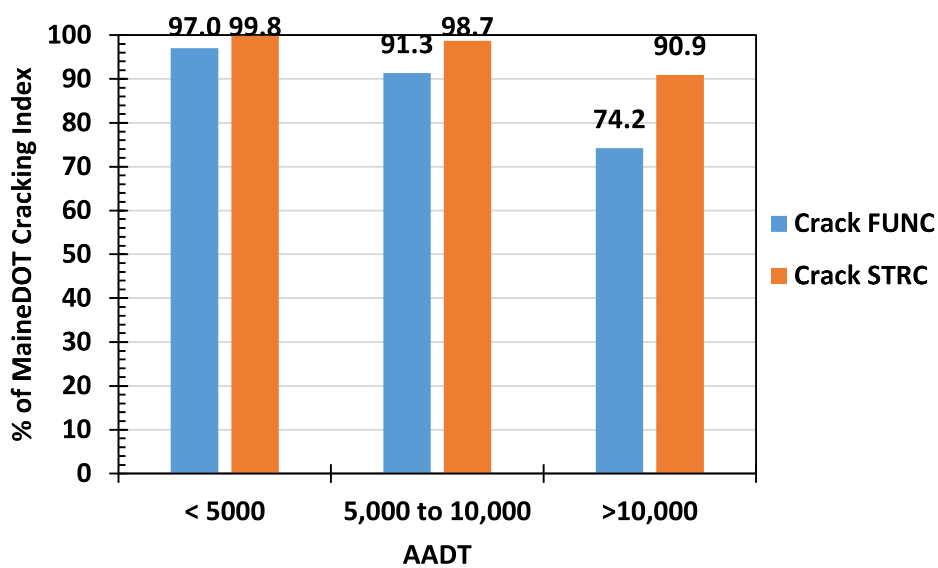


Figure 12 – MaineDOT AADT Divisions and Resultant PMS Cracking Indices (Crack FUNC and Crack STRC)

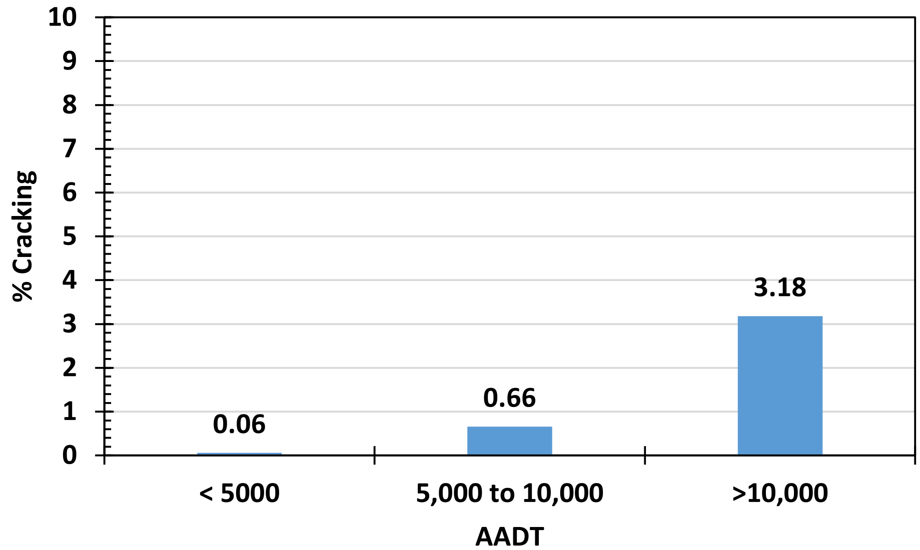


Figure 13 – MaineDOT AADT Divisions and Resultant PMS Cracking Index Percent Cracking

Fatigue Cracking – Final Recommendations

Understanding that the laboratory tested asphalt mixtures are less than five years old in the field, preliminary recommendations for IDEAL-CT Index values for MaineDOT are provided based on the following;

- Average value of 154.2 was measured for AADT < 5,000 and indicated good field performance after an average of 1.7 years for the ten field sections evaluated;
- Average value of 128.2 was measured for AADT 5,000 to 10,000 and indicated good field performance after an average of 2.2 years for the four field sections evaluated. However, it must be noted that greater magnitudes of field cracking was observed when increasing the AADT from < 5,000 to 5,000 to 10,000 AADT; and
- Average value of 125.8 (excluding the PG70E-28) was measured for AADT > 10,000 and indicated that field performance began to show greater magnitudes of cracking over the other two AADT divisions after an average of 1.1 years for the three field sections evaluated.
- Traffic levels had an impact on the MaineDOT PMS cracking distress indices.

Preliminary minimum IDEAL-CT value for MaineDOT should be set at 150 at a test temperature of 25°C. In addition, further research should be conducted to verify whether or not the IDEAL-CT value should be increased for higher levels of traffic.

MaineDOT should also look at whether or not the test temperature of 25°C is best to represent their climate conditions when utilizing the IDEAL-CT test. Work conducted under NCHRP Project 9-59 recommended that a better method to represent intermediate test temperature is to utilize the low temperature PG grade and the relationship shown in Figure 14 (Christensen et al., 2019). Using the LTPPBind3.1 software, the low temperature PG grade was determined at a 98% reliability. This shows Maine has two different low temperature grades; -28°C along the coastal area and -34°C inland (Figure 15). The resultant intermediate test temperatures, based on the recommendation from NCHRP 9-59, would then be 22°C and 19°C, respectively.

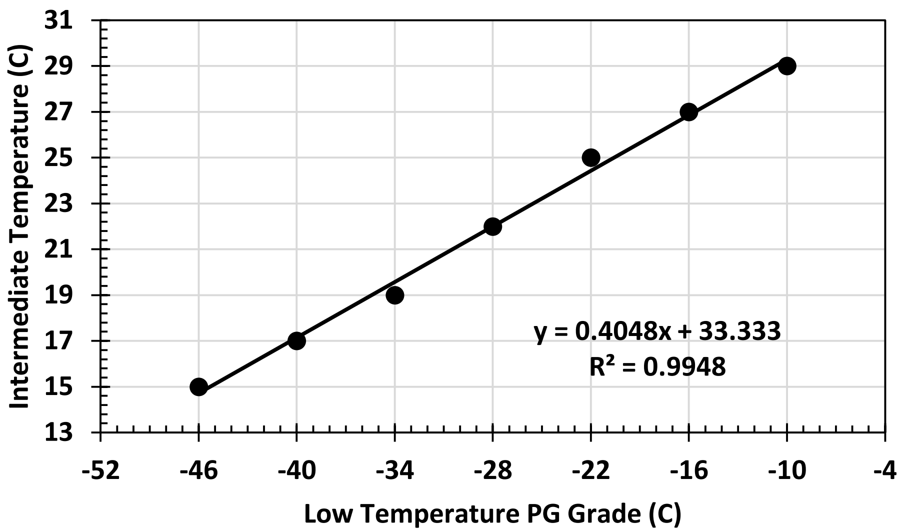


Figure 14 – Recommended Intermediate Temperature for Fatigue Cracking Analysis Based on Representative Low Temperature PG Grade (Christensen et al., 2019)

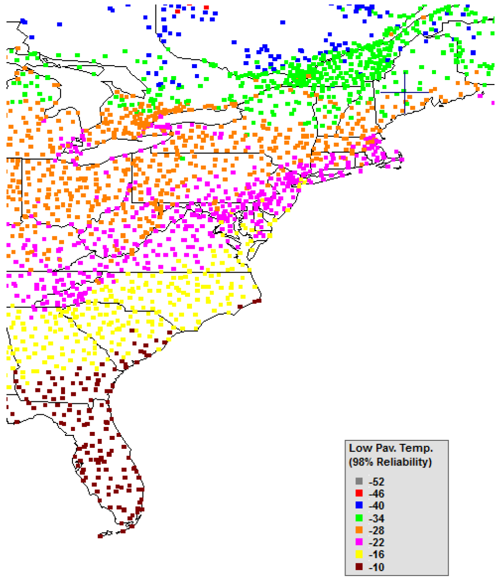


Figure 15 – Low Temperature PG Grade Determined at 98% Reliability Using LTPPBind 3.1

Rutting – Lab vs Field

The MaineDOT has proposed to use the Hamburg Wheel Tracking Test (AASTO T324) as the test procedure to evaluate the rutting potential of asphalt mixtures. At this time, it appears that a test temperature of 45ºC is being selected for use, although there were occasions where the laboratory technicians utilized temperatures of 42ºC and 48ºC. However, the majority of the test data collected, and used in the analysis, was 45ºC.

A first attempt at comparing the field measured rutting and the Hamburg rutting is shown as Figure 16. As the figure clearly shows, no direct relationship existed between the test data and field rutting based on their raw measurements.

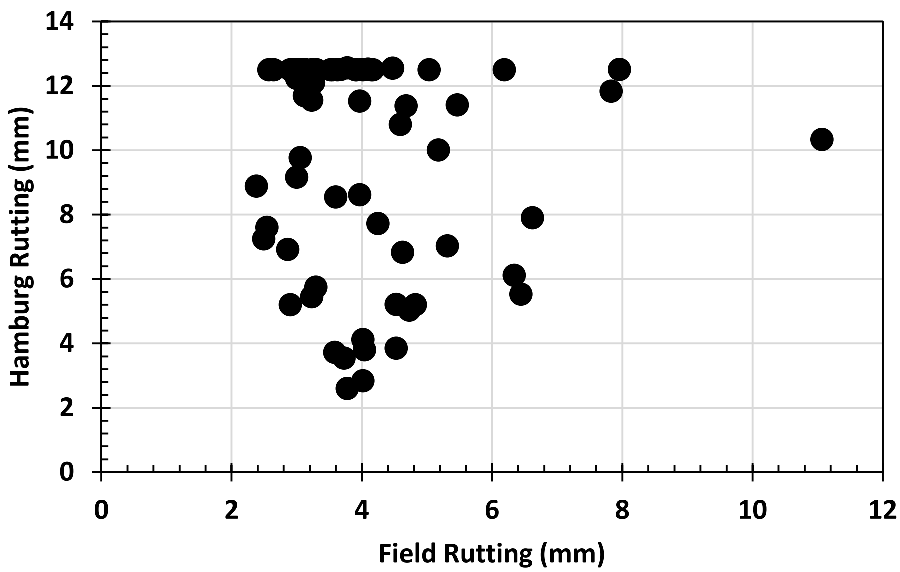


Figure 16 – Measured Field Rutting vs Hamburg Wheel Tracking Rutting of Asphalt Mixture Placed on Pavement Sections

Additional analysis was conducted to help determine what factors may be critical to consider when trying to develop a relationship between field rutting and the Hamburg rutting. First, the AADT for the pavement sections were compared to the measure field rutting (Figure 17). As the figure shows, there is a logical relationship (i.e. – more traffic equals more rutting). However, there is quite a bit of scatter in the data and a relatively poor statistical relationship exists.

In a correspondence with Mr. Dale Peabody, Mr. Peabody mentioned that historically, asphalt pavements in Maine have a tendency to continue to show rutting past the typical 1 to 2 years most state agencies observe on their pavements. Although more research is necessary to determine the exact reasons, one can make the initial assumption that due to Maine's moderate temperatures, the asphalt materials age/stiffen at lower rates than observed in central and southern states.

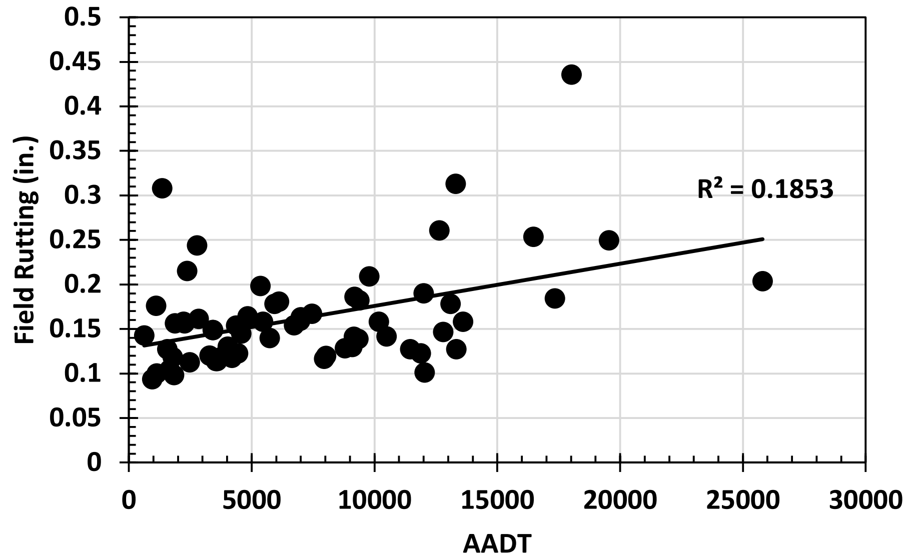


Figure 17 – Field Rutting vs AADT Measured by MaineDOT

To determine if the field rutting did increase with time, the field rutting from the laboratory test data pavement sections were compared against the time after construction. The data set was also filtered to show the AADT (Figure 18). The data in Figure 18 does actually show the magnitude of field rutting increases over time, at least within the four year period of the provided test data. It should be noted that these are not the same sections evaluated each year, but different pavement sections of similar asphalt mixtures with similar Hamburg Wheel Tracking properties.

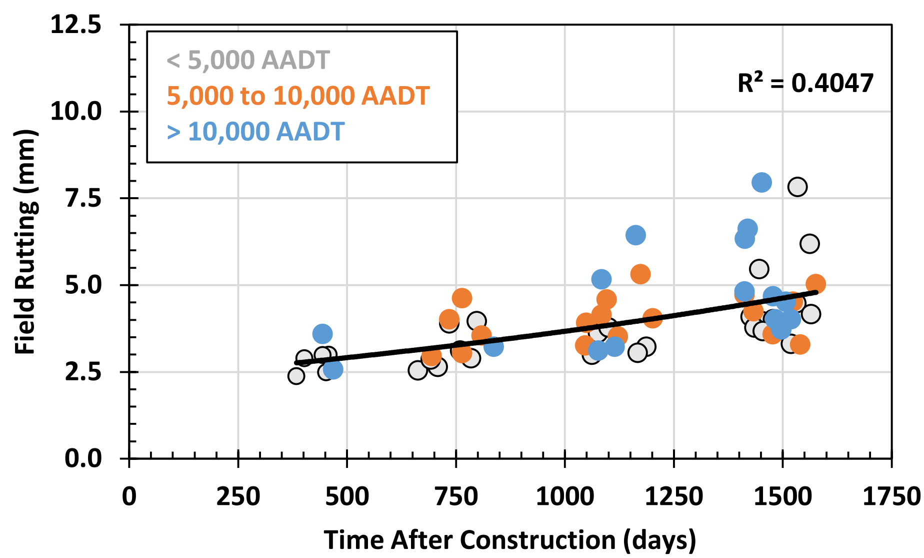


Figure 18 – Field Rutting vs Time After Construction in Maine

The trend in Figure 18 causes a dilemma with generating performance criteria since it is unknown at this time exactly when the rutting stops on the Maine asphalt pavements. Therefore, to help with developing a preliminary criteria, only the laboratory and corresponding field data that are approximately four years or older was used in the analysis. The final data used to generate a preliminary criteria is shown in Table 6.

Table 6 – MaineDOT Field Rutting and Hamburg Wheel Tracking Performance Results for Pavement Surfaces at Least Four Years Old

AADT	Age	Temp	Field Rutting (inches)	Field Rutting (mm)	HWT Rutting (mm)	HWT Passes	Stripping Slope	Creep Slope	Stripping	LC12 5S	LC12 5C
1106	1488	45	0.176	4.47	12.50	4626	4.27	1.45	2.94	4614	4592
1346	1491	45	0.308	7.82	11.83	16823	1.34	0.35	3.81	17508	20142
1875	1409	45	0.156	3.96	11.53	10022	2.34	0.64	3.65	17425	17390
2225	1435	45	0.158	4.01	12.50	9702	3.05	0.38	8.06	9699	9676
2365	1403	45	0.215	5.46	11.41	11633	1.61	0.85	1.88	14000	15856
2766	1519	45	0.244	6.18	12.50	12487	1.82	0.44	4.11	12487	12487
2834	1383	45	0.161	4.09	12.50	4452	5.44	1.08	5.03	4448	4434
3427	1392	45	0.149	3.77	2.60	19891	0.08	0.12	0.65	99751	99751
4022	1476	45	0.130	3.30	12.50	4680	6.01	1.12	5.35	4680	4680
4567	1411	45	0.145	3.67	12.50	6252	3.17	0.82	3.88	6249	6240
4825	1522	45	0.164	4.17	12.50	7346	3.87	0.72	5.39	7344.5	7332
5348	1533	45	0.198	5.03	12.50	7141	3.99	0.49	8.18	7140	7130
5924	1480	45	0.178	4.52	5.21	19994	0.34	0.17	1.96	41666	62501
7444	1390	45	0.167	4.24	7.73	20000	0.47	0.18	2.55	60023	62908
9094	1497	45	0.130	3.29	5.75	19996	0.41	0.16	2.61	36267	62528
9160	1434	45	0.141	3.58	3.72	19967	0.15	0.14	1.07	77220	83335
9186	1369	45	0.186	4.72	5.03	19932	0.15	0.26	0.58	48420	48420
10175	1440	45	0.158	4.01	4.12	19984	0.17	0.09	1.83	69131	110055
11999	1369	45	0.190	4.82	5.20	18620	0.72	0.19	3.87	52703.6	75457.3
12632	1377	45	0.261	6.62	7.91	16726	1.04	0.23	4.52	51310	51291
12793	1454	45	0.147	3.72	3.55	19943	0.13	0.15	0.87	79968.67	81595
13088	1464	45	0.178	4.52	3.85	19954	0.13	0.16	0.83	73570	73570
13298	1409	45	0.313	7.95	12.50	3074	8.00	1.20	6.65	3073	3066
13604	1476	45	0.158	4.01	2.84	19909	0.10	0.10	0.97	105189	115616
17347	1435	45	0.184	4.67	11.38	14791	1.63	0.42	3.86	18007	21051.2
19549	1370	45	0.250	6.34	6.12	19988	0.44	0.17	2.58	34352	57008

The final test data to use in the preliminary Hamburg rutting criteria is shown in Figure 19. The data does follow an increasing linear relationship whereas Hamburg rutting increases, so does the field rutting. More scatter in the data can be witnessed for the lower volume traffic as opposed to the higher volume traffic. This is most likely due to the fact that the same Hamburg Wheel Tracking test protocols are used regardless of traffic level in the field. Simply put, there is no attempt to modify the loading magnitude in the Hamburg to better represent the loading conditions in the field, even though asphalt mixture selection is modified to consider traffic volume (i.e. – differences in gyration level, modified asphalt binders, etc.). Therefore, traffic level should be included within the final criteria to help distinguish between the needs in the field.

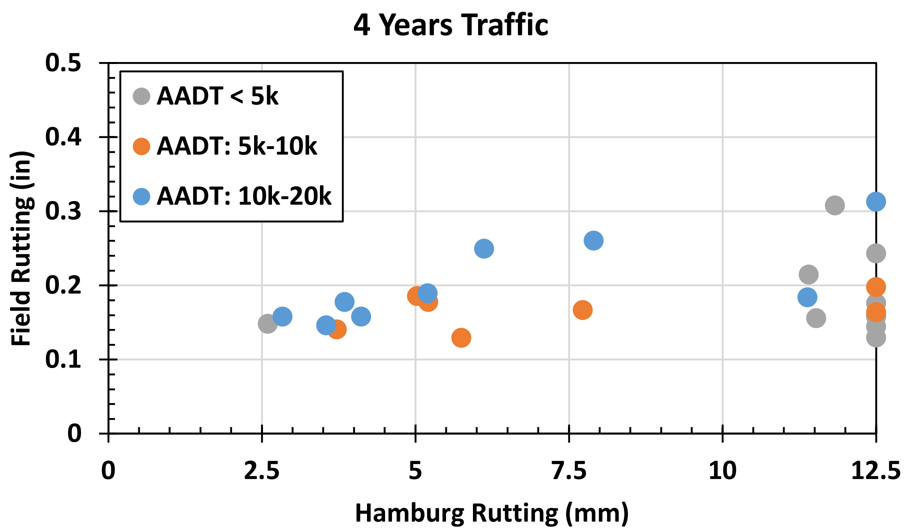


Figure 19 – Hamburg Wheel Tracking Rutting vs Field Rutting Collected by MaineDOT

Rutting – Final Recommendations

The Hamburg rutting and field rutting from Table 6 was broken out and averaged for every 5000 AADT range. This is shown in Figure 20 with error bars that represent the standard deviation above and below the average. The data shows that the average field rutting for the different levels of AADT range between 0.16 inches and 0.21 inches. If using the nomenclature of "Good" and "Poor" rutting performance as denoted during the Connecticut analysis, the Maine's rutting field performance would fall in the "average" area, somewhere between 0.15 inches and 0.30 inches. This would indicate that a selection for the criteria of Hamburg rutting should not deviate much more than what the average values currently fall on. By selecting a lower Hamburg rutting magnitude would be very conservative since the pavements are not showing severe rutting issues, while selecting a higher value may actually lead to field rutting higher than the 0.15 - 0.20 inches currently witnessed. Utilizing this rationale, Figure 21 was developed and represents the "PASS-FAIL" Hamburg Wheel Tracking rutting criteria for different AADT levels on Maine's asphalt pavements. The rutting criteria is based on testing the asphalt mixtures to 20,000 loading cycles at a test temperature of 45°C.

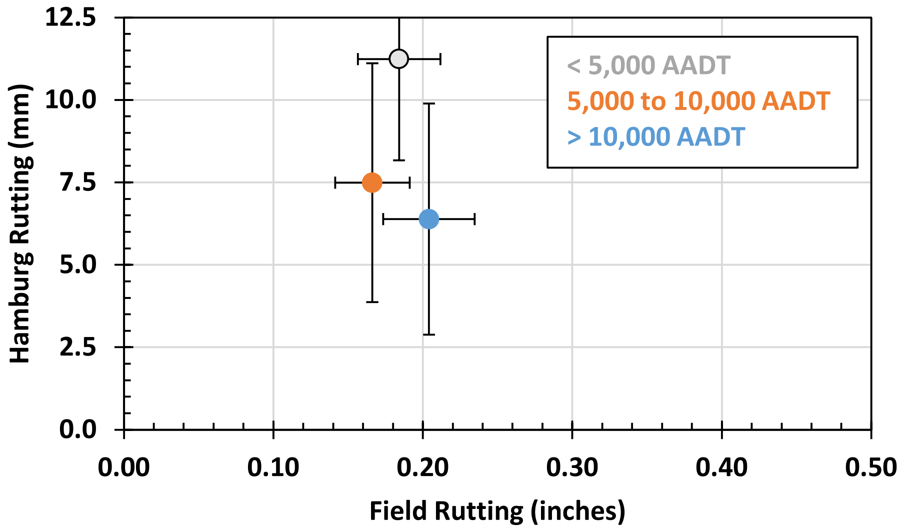


Figure 20 – Average Field Rutting vs Average Hamburg Rutting (Error Bars Represent Standard Deviation Above and Below Average)

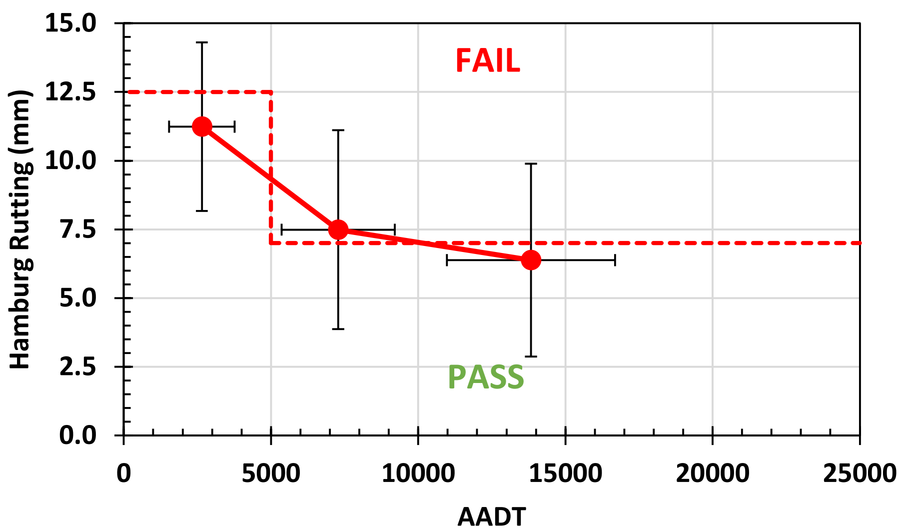


Figure 21 – Proposed Hamburg Wheel Tracking Criteria for MaineDOT Asphalt Mixtures

Vermont

The Vermont Agency of Transportation (VAOT) provided the research team with asphalt mixture performance test results and a link to download the complete set of Pavement Management distress information. In the study, VAOT is utilizing the Hamburg Wheel Tracking test to measure the rutting resistance of asphalt mixtures while using the SCB Flexibility Index to characterize the fatigue cracking potential of their asphalt mixtures. The rutting analysis was conducted with lab and field data for surface course asphalt mixtures with up to three years of performance history. The fatigue cracking analysis was conducted with lab and field data with approximately only two years of performance history.

General Mixture Performance – Fatigue Cracking

As mentioned earlier, the Vermont Agency of Transportation (VAOT) uses the SCB Flexibility Index (AASHTO TP124) at a test temperature of 25°C to evaluate fatigue cracking properties of their asphalt mixtures. Figures 22 and 23 show a summary of the different asphalt mixtures tested and placed on Vermont asphalt pavements. Overall, the average SCB FI results appear to very good with the lowest average value measured of 6.7 for the 2019 Type IIS with 15% RAP. Preliminary research has shown that an SCB FI value greater than 8.0 generally show good field performance.

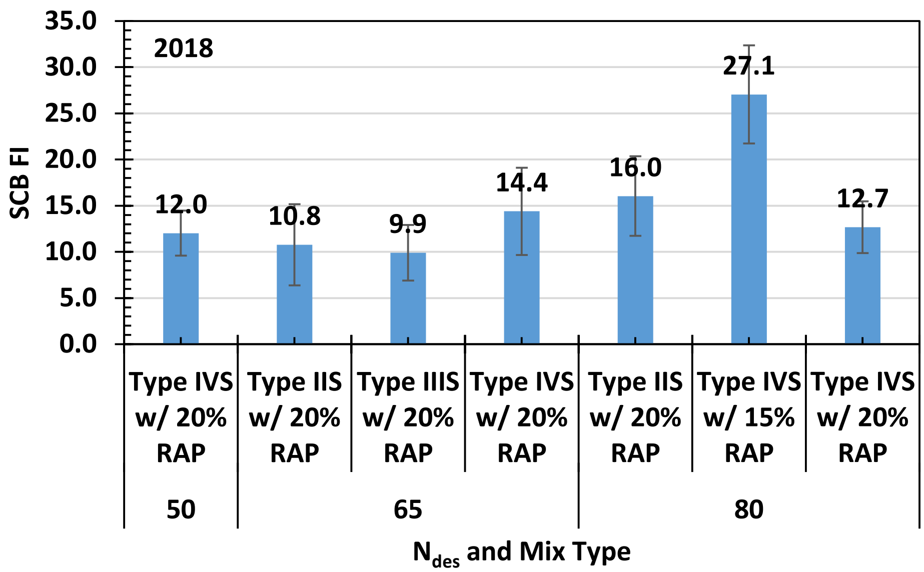


Figure 22 – SCB Flexibility Index Results for 2018 VAOT Tested Asphalt Mixtures

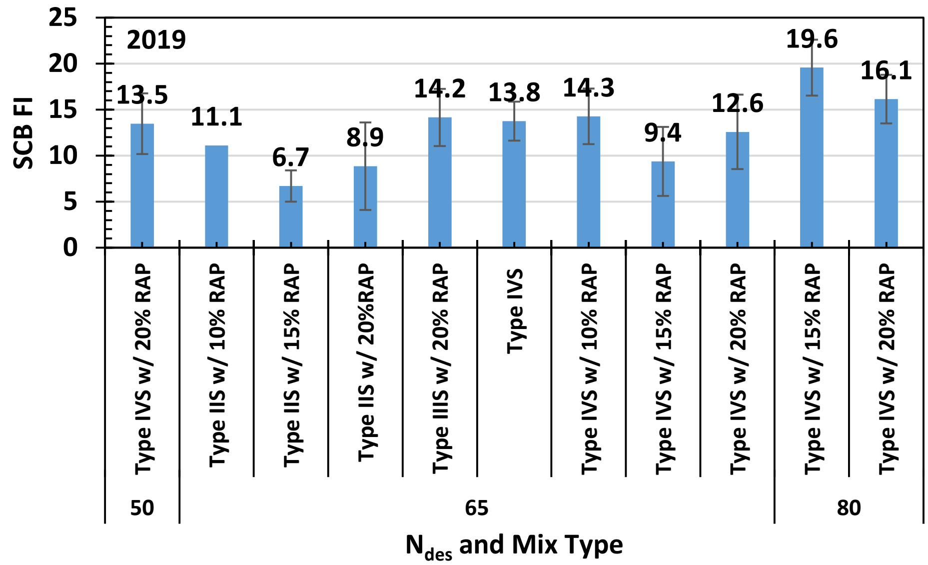


Figure 23 – SCB Flexibility Index Results for 2019 VAOT Tested Asphalt Mixtures

An important part of developing a specification and monitoring mixture performance is the consistency of mixture performance from year to year. Table 7 and Figure 24 show asphalt mixtures produced in both 2018 and 2019 and their respective SCB Flexibility Index values. For the most part, there is general agreement between the two different years respective fatigue cracking performance showing relatively good consistency in the mix types in Vermont.

Table 7 – Comparison of 2018 and 2019 SCB Flexibility Index Performance for Same Mixture Type in Vermont

N_{des}	Mix Type	Year Produced	SCB FI Average	SCB FI Std Dev
50	Type IVS w/ 20% RAP	2018	12.0	2.4
		2019	13.5	3.3
65	Type IIS w/ 20% RAP	2018	10.8	4.4
		2019	8.9	4.8
	Type IIIS w/ 20% RAP	2018	9.9	3.0
		2019	14.2	3.1
	Type IVS w/ 20% RAP	2018	14.4	4.7
		2019	12.6	4.0
80	Type IVS w/ 15% RAP	2018	27.1	5.3
		2019	19.6	3.0
	Type IVS w/ 20% RAP	2018	12.7	2.8
		2019	16.1	2.6

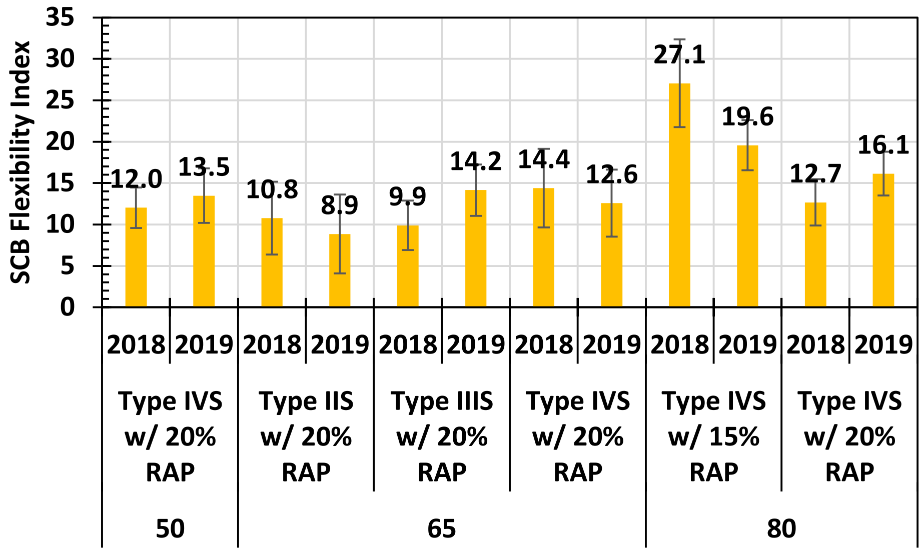


Figure 24 - Comparison of 2018 and 2019 SCB Flexibility Index Performance for Same Mixture Type in Vermont

Fatigue Cracking – Lab vs Field

Due to the relatively "young" nature of the asphalt materials placed, only the 2018 asphalt mixtures were used to attempt to develop a preliminary SCB Flexibility Index Criteria. Two VAOT PMS parameters were used to compared to the laboratory fatigue cracking tests; 1) Trans Index and 2) NPRM WP Crack. Although definitions were not provided, it is assumed that the "Trans Index" is related to the extent of transverse cracking while the NPRM WP Crack is a measure of load associated wheel path cracking. The final data set used to evaluate a tentative criteria is shown in Table 8.

An initial comparison between the measured SCB Flexibility Index values and the VAOT fatigue cracking indices are shown in Figure 25. The figure indicates that most of the pavement sections with SCB Flexibility Index data are available are in relatively good condition. There are some pavement sections that are showing levels of cracking. An interesting trend in the data does seem to indicate that as the cracking distress level increases, the SCB Flexibility Index also increases. This is counter-intuitive to what one would expect and hope for.

Table 8 – 2018 Vermont AOT Fatigue Cracking Data

Project	Mix Type	Producer	Site Manager Sample ID	Average NPRM WP Crack Over Project Length (per 0.1 mile)	Tran Index	Ave. AADT	Ndes	Ave SCB FI	FI COV (%)	Avg. Fracture Energy	Fract Energy COV (%)	Avg. Strength	Strength COV (%)
Bennington NH 2966(1)	Type IVS w/ 20% RAP	WE Dailey/Peckham (45) - S Shaftsbury, VT	tarel188D061826	19.92	87.3	6800	65	27.1	8	2276.8	12	297.1	6
			scrowley1896145508					18.5	21	2736.7	9	401.1	9
			dconnoll189R084245					18.2	13	2614.7	10	376.1	10
			tarel18A2093504					18	13	2357.6	13	358.9	14
Bennington - Wilmington NH SURF(51)	Type IVS w/ 20% RAP	WE Dailey/Peckham (45) - S Shaftsbury, VT	tcoletta1865055842	0.40	99.43	4325	65	16.3	16	1932.6	4	320.4	5
Montpelier STP 2950(1)	Type IVS w/ 20% RAP	Pike IND (901) - Berlin, VT	jjacobso186Q075145	0.28	96.54	4900	65	8.7	15	2239.5	5	461.4	5
St. Albans City STP 2957(1)	Type IVS w/ 20% RAP	VT Blacktop - Colchester, VT	scrowley185D180532	1.00	89.83	9200	80	9.5	23	2017.6	11	376.6	6
			tarel186J165054					14.8	10	1901.2	7	303.2	5
			jbreton187G173419					13.7	17	2187.7	5	353.8	5
Reading - Windsor STP FPAV(11)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	gporter189D105848	0.03	99.94	1450	50	8.9	5	2349.4	1	470.9	3
			tarel189J145920					13.1	27	2429.3	9	435.1	6
			tarel189O094332					10	11	2242	6	433.1	5
Weathersfield - Reading STP FPAV(12)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	jjacobso187R061056	0.00	100.00	1250	50	16.9	18	2572.2	5	414	7
			ldonavan187R080905					14.3	17	2295.3	25	397	26
			gporter1888073953					12.8	29	2528	9	433.2	3
			tarel189Q133152					12.2	18	2555.7	10	438	5
			jbreton18A4054111					13	20	2438.8	4	430.2	8
			etavares18AC055915					12	20	2612	6	446.8	6
			etavares18AI145911					9.8	16	2273.9	6	429.4	9
			etavares18AG061421					13	23	2478.7	5	424.1	7
Weathersfield - Windsor STP FPAV(13)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	gporter188D075139	0.26	99.84	975	50	8.3	8	2159.7	5	445.4	4

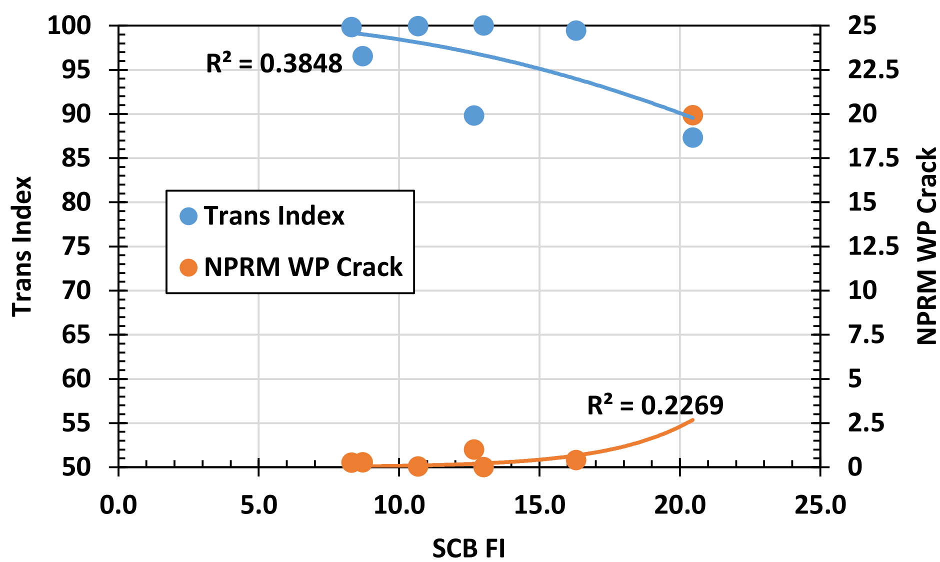


Figure 25 – SCB Flexibility Index Compare to VAOT Fatigue Cracking Indices; Trans Index and NPRM WP Crack

Additional analysis was conducted to evaluate the impact of traffic level on the cracking indices. In general, field aging and traffic are the two major factors regarding the deterioration of asphalt pavements due to cracking, assuming asphalt production and placement were conducted properly. However, it should be reiterated that all of pavement sections were only two years old at the time of the analysis. Figures 26 and 27 show the comparisons of the VAOT fatigue cracking indices with the AADT and SCB Flexibility Index of the respective pavement sections. The figures clearly indicate that as the AADT increases, the VAOT cracking indices' distress magnitudes also increase. However, with limited data, it would not be prudent to include AADT within the SCB Flexibility Index criteria yet, although it can be assumed that as traffic level increases, greater fatigue cracking performance would be required.

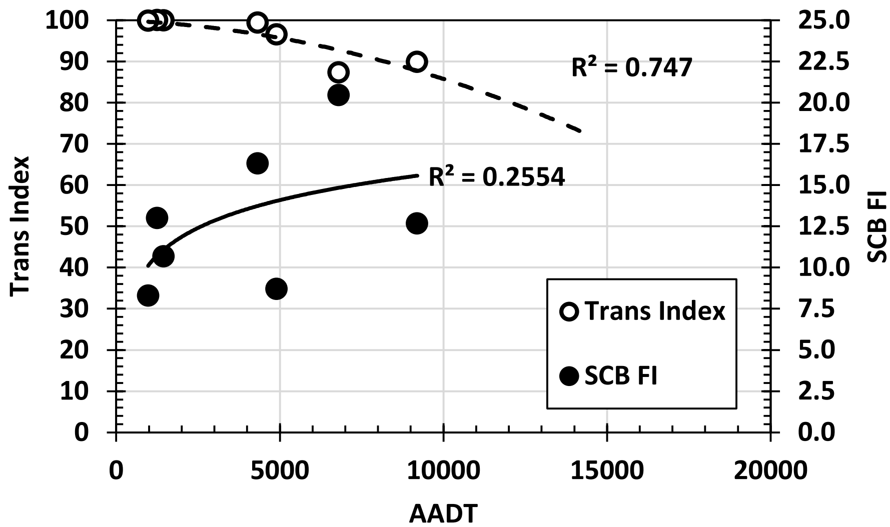


Figure 26 – Pavement Section AADT Compared to VAOT Trans Index Cracking Index and SCB Flexibility Index

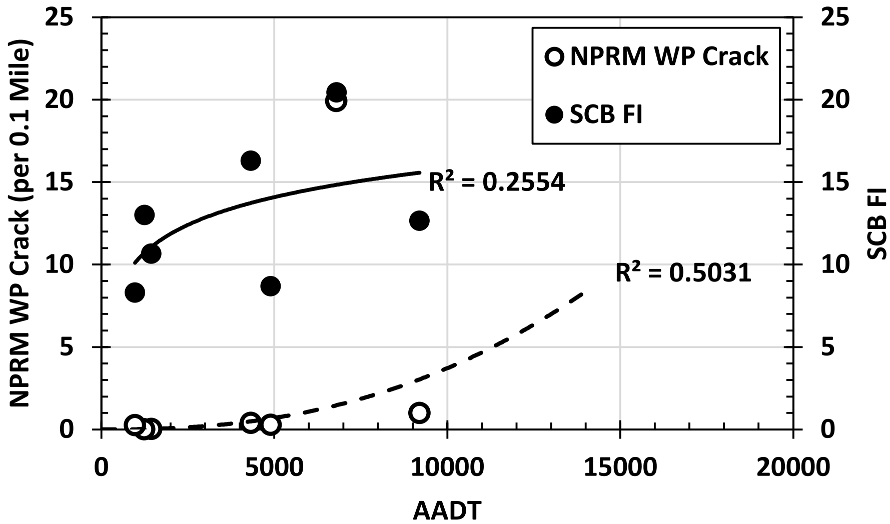


Figure 27 – Pavement Section AADT Compared to VAOT NPRM WP Crack Cracking Index and SCB Flexibility Index

Another factor greatly impacts that fatigue cracking resistance in asphalt mixtures is the asphalt content, which can be related to gyration level. The greater the gyration level, the closer the aggregate particles are pushed together. As the aggregates push together, the asphalt binder around the aggregate skeleton is squeezed out. This is why one generally sees the asphalt content decrease as the gyration level increases. Although lower asphalt contents may be good to help resist rutting, lower asphalt contents will in turn accelerate cracking. Volumetrics for the asphalt mixtures were not provided, but Figures 28 and 29 were developed by comparing the pavement sections AADT to the respective mixture design gyration level and VAOT cracking indices. The figures do show evidence that as design gyration level increases, so does the fatigue cracking index value.

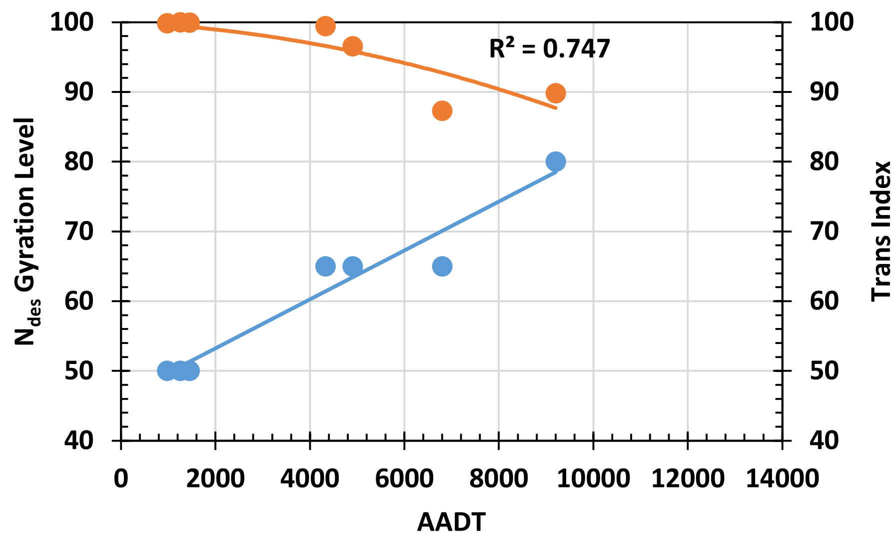


Figure 28 – Design Gyration Level Compared to VAOT Trans Index

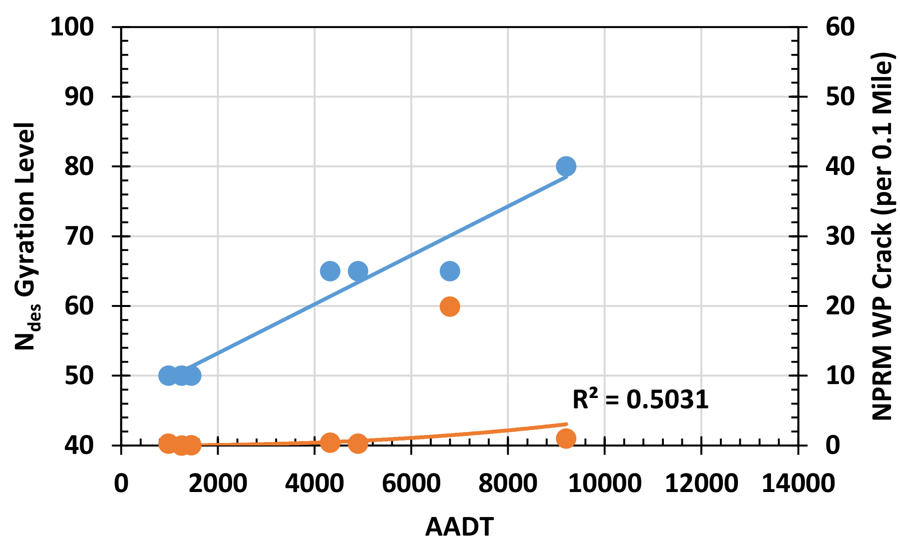


Figure 29 – Design Gyration Level Compared to VAOT NPRM WP Crack

The determination of a tentative criteria is difficult at this time as most of the pavement sections showed relatively good performance, except for the Bennington, NH 2966(1) project. At this time, it is not known whether this was an error with the data collection or not. This pavement section had the second highest AADT (6800) and the highest average SCB Flexibility Index value (20.5). **Based on the current data, it would appear that a preliminary SCB Flexibility Index Criteria would be a minimum of 8.0 at a test temperature of 25°C.** Figures 30 and 31 show where this criteria falls and how the pavement sections performed, respectively.

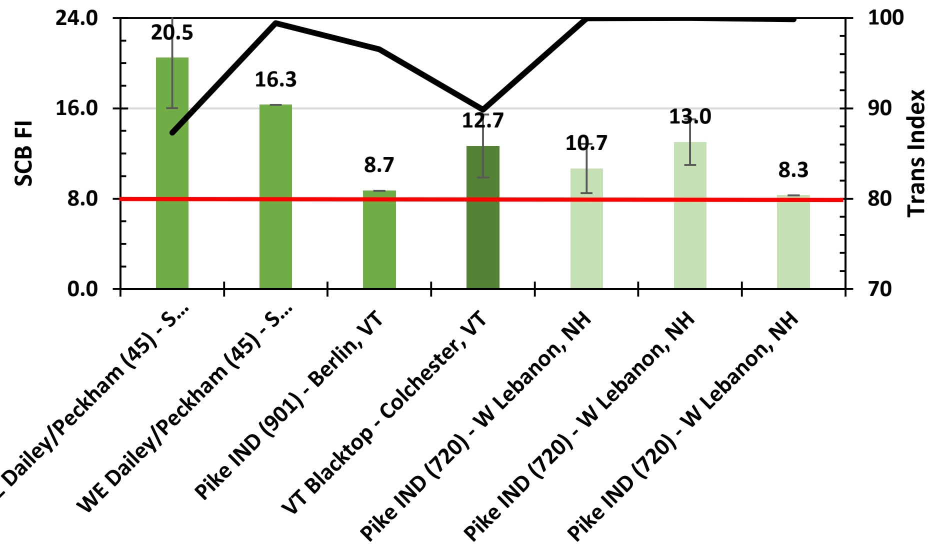


Figure 30 – SCB Flexibility Index and Trans Index with Preliminary Criteria for 2018 Pavement Sections

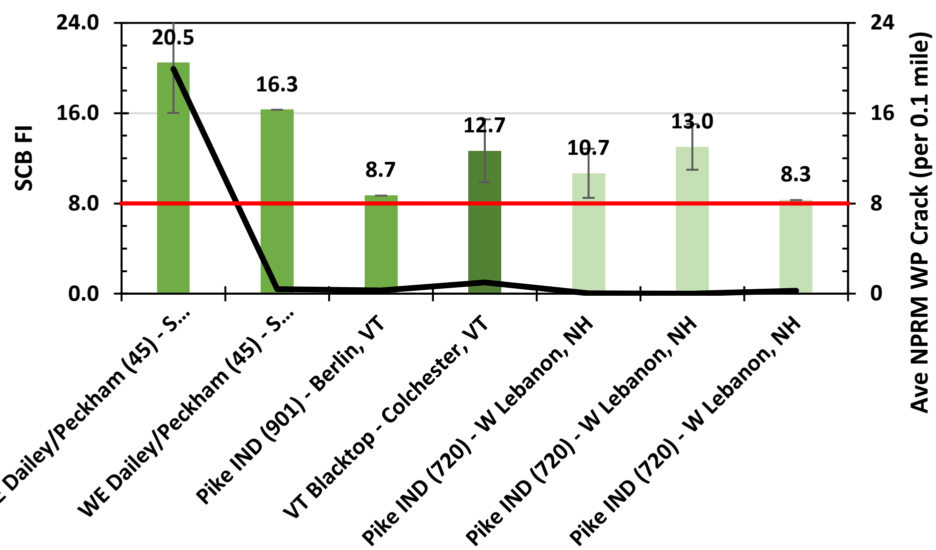


Figure 31 – SCB Flexibility Index and NPRM WP Crack with Preliminary Criteria for 2018 Pavement Section

VAOT Fatigue Cracking Recommendations

Similar to the climate conditions of Maine, Vermont consists of two different low temperature PG grades based on LTPPBind 3.1 at a 98% reliability, -28ºC and -34ºC. Using the relationship recommended in NCHRP project 9-59, intermediate testing temperatures would be 22ºC and 19ºC, respectively. Future research in Vermont may want to be directed to looking at reducing the SCB test temperature to an intermediate temperature more representative of the region.

Additionally, with the VAOT asphalt mixtures all showing relatively good SCB Flexibility performance, VAOT may want to evaluate the performance of field cores with poor field cracking performance. This method would provide a direct comparison between the SCB performance and poor field cracking performance to help validate a minimum SCB Flexibility Index value. Tables 9 shows pavement sections identified in the VAOT PMS that can be classified as poor cracking performance. Special care should be taken to determine the air voids of the field cores and include this information in the analysis as air void content has been found to directly influence the SCB Flexibility Index performance.

Table 9 – VAOT PMS Identification of Poor Cracking Performance

Poor Cracking Measurements (2018)									
Route Name	ETE_From	ETE_To	ETE_Road	BeginTown	NPRM Condition CRK	NPRM WP Crack	Last Work Project Name	Last Work Project Number	Last Work Year
US 2	101.2	101.3	U002	CABOT	POOR	34.15	District Paving	NE19PAV702	2018
	101.3	101.4	U002	DANVILLE	POOR	26	District Paving	NE19PAV702	2018
	101.6	101.7	U002	DANVILLE	POOR	33.25	District Paving	NE19PAV702	2018
	101.7	101.8	U002	DANVILLE	POOR	35.75	District Paving	NE19PAV702	2018
	101.8	101.9	U002	DANVILLE	POOR	39.75	District Paving	NE19PAV702	2018
US 5	186.8	186.9	U005	DERBY	POOR	30.38	District Paving	NE19PAV902	2018
	187.5	187.6	U005	DERBY	POOR	30.67	District Paving	NE19PAV902	2018
US 7	10.3	10.4	U007	BENNINGTON	POOR	21	Bennington	NH 2966(1)	2018
	10.4	10.5	U007	BENNINGTON	POOR	27.5	Bennington	NH 2966(1)	2018
	10.5	10.6	U007	BENNINGTON	POOR	21	Bennington	NH 2966(1)	2018
	11.4	11.5	U007	BENNINGTON	POOR	56.75	Bennington	NH 2966(1)	2018
	11.5	11.6	U007	BENNINGTON	POOR	47	Bennington	NH 2966(1)	2018
	11.6	11.7	U007	BENNINGTON	POOR	41	Bennington	NH 2966(1)	2018
	145.9	146	U007	MILTON	POOR	32	District Paving	NE19PAV501	2018
VT 9	3.1	3.2	V009	BENNINGTON	POOR	24.75	Bennington	NH 2966(1)	2018
	3.5	3.6	V009	BENNINGTON	POOR	22.75	Bennington	NH 2966(1)	2018
	3.6	3.7	V009	BENNINGTON	POOR	31.75	Bennington	NH 2966(1)	2018
	3.7	3.8	V009	BENNINGTON	POOR	36	Bennington	NH 2966(1)	2018
	3.8	3.9	V009	BENNINGTON	POOR	23	Bennington	NH 2966(1)	2018
	4	4.1	V009	BENNINGTON	POOR	26.5	Bennington	NH 2966(1)	2018
	4.7	4.8	V009	BENNINGTON	POOR	20.75	Bennington	NH 2966(1)	2018
	5	5.1	V009	BENNINGTON	POOR	35.5	Bennington	NH 2966(1)	2018
	5.5	5.6	V009	BENNINGTON	POOR	42.75	Bennington	NH 2966(1)	2018
VT 14	93.3	93.4	V014	ALBANY	POOR	57.25	District Paving	NE19PAV901	2018
	93.4	93.5	V014	ALBANY	POOR	59	District Paving	NE19PAV901	2018
VT 67A	2.2	2.3	V067A	BENNINGTON	POOR	36	Bennington	STP 2973(1)	2018
	2.6	2.7	V067A	BENNINGTON	POOR	29.25	Bennington	STP 2973(1)	2018
	2.7	2.8	V067A	BENNINGTON	POOR	31.75	Bennington	STP 2973(1)	2018
	2.8	2.9	V067A	BENNINGTON	POOR	22.25	Bennington	STP 2973(1)	2018
	3	3.1	V067A	BENNINGTON	POOR	22.75	Bennington	STP 2973(1)	2018
	3.2	3.3	V067A	BENNINGTON	POOR	25	Bennington	STP 2973(1)	2018
VT 131	1.1	1.2	V131	CAVENDISH	POOR	41.5	District Paving	NE19PAV201	2018
	1.3	1.4	V131	CAVENDISH	POOR	51	District Paving	NE19PAV201	2018
	1.7	1.8	V131	CAVENDISH	POOR	42.25	District Paving	NE19PAV201	2018
	1.8	1.9	V131	CAVENDISH	POOR	34.25	District Paving	NE19PAV201	2018
	1.9	2	V131	CAVENDISH	POOR	53.25	District Paving	NE19PAV201	2018
	2.1	2.2	V131	CAVENDISH	POOR	42.75	District Paving	NE19PAV201	2018

General Mixture Performance – Rutting

As mentioned earlier, VAOT uses the Hamburg Wheel Tracking test method at a temperature of 45ºC to evaluate the rutting performance of asphalt mixtures. Figures 32 and 33 show the Hamburg Rutting results for asphalt mixtures tested in 2017 and 2018, respectively. The test results show relatively good performance with generally better performance occurring as the design gyration level increases.

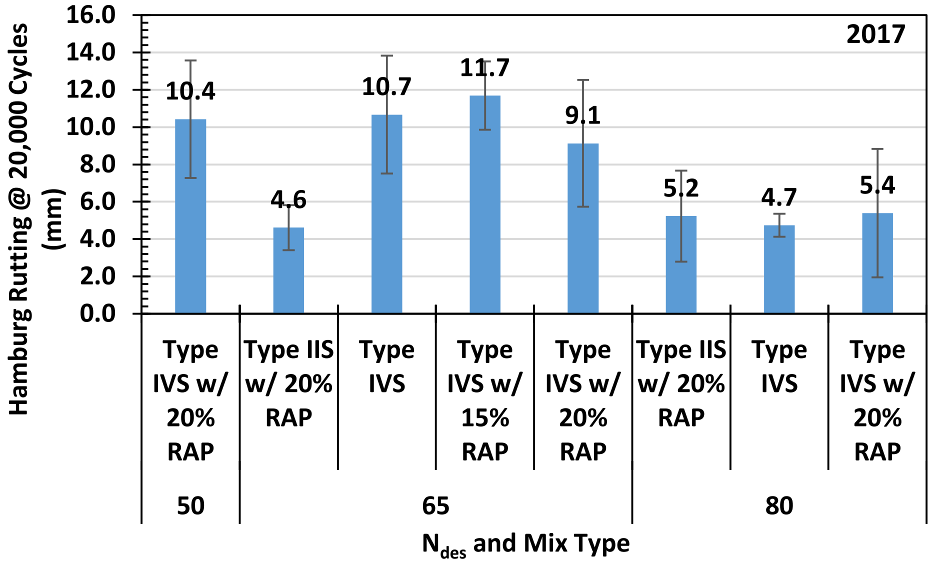


Figure 32 – Hamburg Wheel Tracking Rutting for Different Mixture Types Tested in 2017

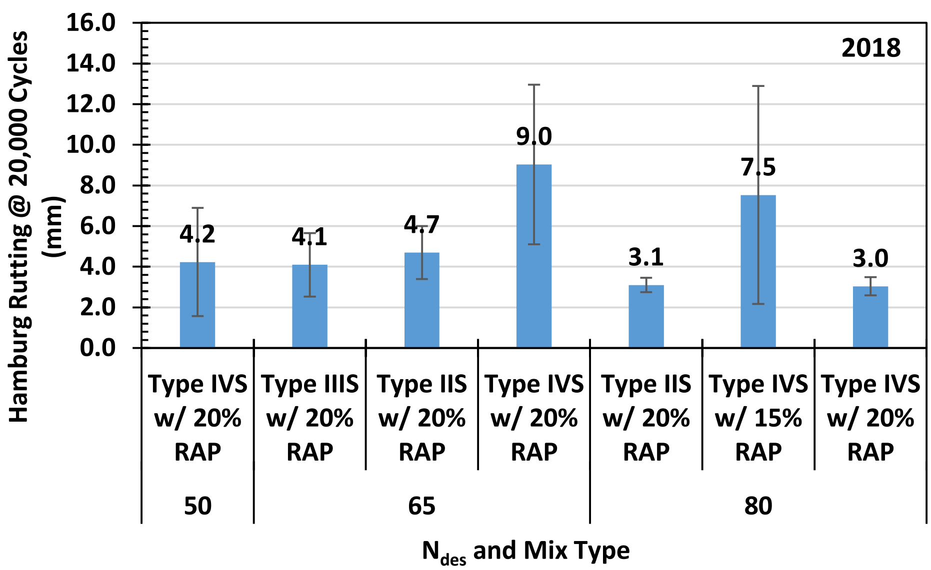


Figure 33 – Hamburg Wheel Tracking Rutting for Different Mixture Types Tested in 2018

A significant number of pavement sections with corresponding Hamburg Wheel Tracking test results were available for VAOT. Unfortunately, all of the pavement sections with Hamburg data all showed very low field rutting results. Table 10 and 11 shows the results from 2017 and 2018 laboratory testing, respectively. The greatest amount of field rutting observed was 0.19 inches.

Table 10 – 2017 VAOT Pavement Sections Containing Laboratory Wheel Tracking Test Results

Hamburg Rutting vs Measured Field Performance (2017 Constructed)														
Project	Mix Type	Producer	Site Manager Sample ID	Rut Average (in.)	Rut Index	Ave. AADT	Ndes	PG Grade	Hamburg Rut Depth (mm)	Ave Pass Max.	SIP	SIP Depth	Strip Slope	Creep Slope
Barre City NH 2961(2)	Type IVS w/ 20% RAP	VT Blacktop - Colchester, VT	jgehrig176F185100	0.14	86	9767	80	70-28	3.01	20000	NA	NA	NA	0.000075
			jgehrig176Q193941						3.44	20000	NA	NA	NA	0.000085
			jgehrig179H225230						8.93	20000	11052	2.48	0.001142	0.000118
			dsavage178F090948						1.99	20000	NA	NA	NA	
Charlotte FEGC F 019-4(20)	Type IIS w/ 20% RAP	Pike IND (736) - New Haven, VT	jglover176F094745	0.16	83.80	12000	80	58-28	6.62	20000	NA	NA	NA	0.000164
			jglover176D063539						10.28	18418	14059	6.58	0.000907	0.000313
			lrothlon1776173517						5.97	20000	NA	NA	NA	0.000191
			tcoletta1798151221						3.00	20000	NA	NA	NA	
			etaveres17A2071242						12.5	16362	8774	4.65	0.000801	0.000364
Danville - St. Johnsbury STP FPAV(9)	Type IVS w/ 20% RAP	Pike IND (702) - Waterford, VT	gporter179B070108	0.06	94.00	563	50	58-34	9.75	20000	13166	4.95	0.000499	0.000251
			tcoletta17A6093250						12.5	17266	8885	5.26	0.000818	0.000367
Essex Junction NH 2956(2)	Type IVS w/ 20%	VT Blacktop - Colchester, VT	lrothlon178D181538	0.141	85.9	11788	80	70-28	3.76	20000	NA	NA	NA	0.0001
Hardwick - Danville STP 2122(1)	Type IVS w/ 20% RAP	Pike IND (702) - Waterford, VT	bwaterma17AD065250	0.088	91.2	3038	65	58-34	12.5	16504	12704	7.62	0.000823	0.000447
			jbreton178H075133						11.33	20000	14154	6.07	0.000602	0.000257
			jbreton178J110802						9.41	20000	NA	NA	NA	0.000213
Hancock STP 2923(1)	Type IVS w/ 20%	VT Blacktop - Colchester, VT	dconnell176F0994849	0.05	94.80	1500	65	58-28	4.21	20000	NA	NA	NA	0.000116
			gporter1761051744						5.04	20000	NA	NA	NA	0.000122
Hartland STP FPAV(8)	Type IVS w/ 20%	Pike IND (720) - W. Lebanon, NH	rknapp178P102700	0.08	92.10	1689	50	58-34	12.5	14176	8930	6.86	0.00103	0.00052
			etavares179T150144						11.34	13000	7243	4.48	0.000995	0.00092
Randolph - Braintree STP FPAV(7)	Type IVS w/ 20% RAP	Pike IND (901) - Berlin, VT	dsavage176S63834	0.06	94.10	1379	50	58-28	12.5	11008	5625	3.31	0.00105	0.000377
			gporter177Q073537						12.5	14490	8903	5.13	0.001149	0.000362
			jgehrig177V132604						12.50	6068	NA	NA	NA	0.001201
Rochester ER STP 0162(21)	Type IIS w/ 20% RAP	Pike IND (720) - W. Lebanon, NH	gporter178N081704	0.093	90.7	858	50	58-28	4.33	20000	NA	NA	NA	0.000105
			tcoletta178Q150041						6.19	20000	NA	NA	NA	0.00021
			tcoletta178T161907						3.27	20000	NA	NA	NA	0.000086
			jglover176H112317						9.16	20000	NA	NA	NA	0.000268
			scrowley176Q105822						10.31	18550	11330	4.32	0.000444	0.000218
Rockingham - Springfield STP 2962(1)	Type IVS w/ 15% RAP	Cold River Materials - Walpole, NH	gporter176K153823	0.072	92.8	2453	65	58-28	12.5	16860	11113	4.43	0.001183	0.000237
			gporter176N101328						12.5	13820	8280	4.87	0.001327	0.000344
			jgehrig177Q114348						12.5	14380	11556	7.69	0.001241	0.000475
			jgehrig177V075536						12.5	14436	8081	4.43	0.000753	0.00031
			jglover175U062239						12.17	17896	12494	4.94	0.001276	0.000224
			jglover1778094343						7.54	20000	15335	4.19	0.000392	0.000147
			scrowley176E080529						12.5	14920	7090	4.46	0.0009	0.000441
			scrowley1777121000						8.02	18056	9623	3.44	0.000666	0.000155
			scrowley177C111000						12.5	17784	NA	NA	NA	0.000266
			scrowley1781105150						12.5	12770	7239	5.41	0.00096	0.000432
			scrowley1783070716						12.5	13688	6181	6.68	0.000829	0.000423
			scrowley1788084247						12.5	13224	8095	5.89	0.000915	0.00047
Roxbury - Northfield ER STP 0187(13)	Type IVS w/ 20% RAP	Pike IND (720) - W. Lebanon, NH	bwaterma17AV105641	0.061	93.8	1100	50	58-28	12.5	10522	6218	5.49	0.0014	0.000547
			tcoletta175G064353						5.5	20000	NA	NA	NA	0.000155
			tcoletta175G064353						10.52	20000	12963	4.82	0.000501	0.00021
			tcoletta179D061046						2.85	20000	NA	NA	NA	
			tcoletta179B063938						9.03	20000	11326	4.44	0.000482	0.000252
Roxbury - Northfield STP	Type IVS w/ 20%	Pike IND (720) - W. Lebanon, NH	gporter177602156	0.088	91.2	1121	50	58-28	12.5	12396	7764	7.27	0.001029	0.00059
			jgehrig1769051309						12.5	17916	10083	4.23	0.000841	0.000237
Rutland - Killington ER NH 020-2(36)	Type IIIS w/ 15% RAP	Wilk Paving - Center Rutland, VT	gporter1754062752	0.129	87.1	8291	80	70-28	4.76	30000	21270	3.65	0.000113	0.000092
			gporter176815923						5.34	20000	NA	NA	NA	0.000128
			gporter177H071513						3.76	20000	NA	NA	NA	0.00008
			jgehrig175G112708						3.96	20000	NA	NA	NA	0.00012
			jgehrig175O061423						3.44	20000	NA	NA	NA	0.000095
			jglover176L064548						4.39	20000	NA	NA	NA	0.000129
			lrothlon176D125700						4.49	20000	NA	NA	NA	0.000121
			lrothlon1761142144						5.96	20000	NA	NA	NA	0.000158
			scrowley177712111						4.41	20000	NA	NA	NA	0.000119
			tcoletta176Q161029						4.92	20000	NA	NA	NA	0.000176
South Burlington - Williston NH 2944(1)	Type IIS w/ 20% RAP	VT Blacktop - Colchester, VT	jglover17672114025	0.19	80.9	14816	80	70-28	3.44	20000	NA	NA	NA	0.000061
			lrothlon1789192238						3.26	20000	NA	NA	NA	0.00007
			dsavage179I041242						3.58	20000	NA	NA	NA	0.000125
			scrowley1780185257						4.3	20000	NA	NA	NA	0.00013
			jbreton17A6180835						4.26	20000	13661	2.75	0.000104	0.000087
St. Albans Town STP	Type IVS w/ 20%	VT Blacktop - Colchester, VT	gporter175G075800	0.096	90.4	2152	50	58-28	12.5	14762	NA	NA	NA	0.000493
			gporter175K061647						5.39	20000	NA	NA	NA	0.000161

Table 11 – 2018 VAOT Pavement Sections Containing Laboratory Wheel Tracking Test Results

Hamburg Rutting vs Measured Field Performance (2018)														
Project	Mix Type	Producer	Site Manager Sample ID	Rut Average	Rut Index	Ave. AADT	Ndes	PG Grade	Hamburg Rut Depth (mm)	Ave Pass Max.	SIP	SIP Depth	Strip Slope	Creep Slope
Bennington NH 2966(1)	Type IVS w/ 20% RAP	WE Dailey/Peckham (45) - S Shaftsbury, VT	tarel188D061826	0.114	88.5	6803	65	70-28	10.32	20000	12660	5.11	0.000501	0.000257
			scrowley1896145508						6.72	20000	13509	3.78	0.000349	0.000198
			dconnoll189R084245						5.88	20000	15507	4.42	0.000307	0.00017
			tarel18A2093504						3.88	20000	NA	NA	NA	0.000126
Bennington - Wilmington NH SURF(51)	Type IVS w/ 20% RAP	WE Dailey/Peckham (45) - S Shaftsbury, VT	tcoletta1865055842	0.11	89.30	4325	65	58-28	12.5	12744	6992	4.4	0.000824	0.000391
Montpelier STP 2950(1)	Type IVS w/ 20% RAP	Pike IND (901) - Berlin, VT	jjacobso186Q075145	0.155	84.51	4888	65	70-28	10.49	19566	13334	2.99	0.001326	0.000150
St. Albans City STP 2957(1)	Type IVS w/ 20% RAP	VT Blacktop - Colchester, VT	scrowley185D180532	0.12	88.35	9200	80	70-28	3.16	20000	NA	NA	NA	0.000086
			tarel186J165054						3.41	20000	NA	NA	NA	0.000099
			jbreton187G173419						2.55	20000	NA	NA	NA	0.000064
Reading - Windsor STP FPAV(11)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	gporter189D105848	0.01	99.03	1427	50	70-28	3.49	20000	NA	NA	NA	0.000096
			tarel189J145920						2.6	20000	NA	NA	NA	0.000072
			tarel189O094332						3.92	20000	NA	NA	NA	0.000106
Weathersfield - Reading STP FPAV(12)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	jjacobso187R061056	0.02	98.40	1256	50	70-28	4.17	20000	NA	NA	NA	0.000111
			ldonavan187R080905						3.13	20000	NA	NA	NA	0.000086
			gporter1888073953						3.58	20000	NA	NA	NA	0.000093
			tarel189Q133152						2.57	20000	NA	NA	NA	0.000072
			jbreton18A4054111						4.41	20000	16845	3.94	0.000141	0.000119
			etavares18AC055915						3.5	20000	NA	NA	NA	0.000088
			etavares18AI145911						3.61	20000	NA	NA	NA	0.000095
Weathersfield - Windsor STP FPAV(13)	Type IVS w/ 20% RAP	Pike IND (720) - W Lebanon, NH	gporter188D075139	0.015	98.51	954	50	70-28	3.28	20000	NA	NA	NA	0.000092
Cabot - Danville FEGC F 028-3(26)C/2	Type IIS w/ 20% RAP	Pike IND (733) - Waterford, VT	gporter1896095514	0.09	91	4100	65	58-28	4.56	20000	NA	NA	NA	0.000153
			gporter1896095514						5.44	20000	NA	NA	NA	0.000177
			tarel186D084429						7.29	20000	11103	4	0	0.000178
Enosburgh - Richford STP 2969(1)	Type IVS w/ 20% RAP	Pike IND (905) - Coventry, VT	jjacobso187U070012	0.053	94.9	4656	65	70-28	3.80	20000	NA	NA	NA	0.000107
			gporter189I102550						3.14	20000	14213	2	0	0.000088
	Type IIIS w/ 20% RAP	Pike IND (801) - Swanton, VT	tcoletta18A1080242						4.69	20000	18176	4	0	0.000152
			tcoletta18AC111912						5.28	20000	15941	4	0	0.000147
Essex NH 2931(2)	Type IIS w/ 20% RAP	Pike IND (736) - New Haven, VT	tcoletta189K074609	0.023	97.7	7605	65	70-28	3.63	20000	NA	NA	NA	0.000099
			tcoletta189O094819						3.89	20000	NA	NA	NA	0.000114
			tcoletta189O094819						3.71	20000	NA	NA	NA	0.000097
		Pike IND (800) - Williston, VT	dconnoll18AA072024						4.38	20000	NA	NA	NA	0.000112
	e IVS w/ 20%		tarel18B9081105						2.73	20000	NA	NA	NA	0.000072
Waterbury - Stowe STP 2945(1)	Type IIS w/ 20% RAP	VT Blacktop - Colchester, VT	scrowley1859182253	0.019	98.1	12226	80	70-28	3.20	20000	NA	NA	NA	0.000089
			scrowley185H193057						3.22	20000	NA	NA	NA	0.000087
			scrowley185O184722						3.41	20000	NA	NA	NA	0.000089
			scrowley186F033031						3.44	20000	NA	NA	NA	0.000084
			jbreton1866175418						3.63	20000	NA	NA	NA	0.000076
			jbreton1871174622						3.57	20000	NA	NA	NA	0.000094
			jbreton1879173230						3.13	20000	NA	NA	NA	0.000078
			jbreton187C175522						2.80	20000	NA	NA	NA	0.000067
			jbreton187Q173832						2.93	20000	NA	NA	NA	0.000060
			jbreton1885171752						3.21	20000	NA	NA	NA	0.000022
			jbreton188R170621						3.12	20000	NA	NA	NA	0.000080
			jbreton18888163354						3.06	20000	NA	NA	NA	0.000065
			jbreton188M174557						2.51	20000	NA	NA	NA	0.000056
			jbreton1899174832						3.09	20000	NA	NA	NA	0.000070
			jbreton189I173143						3.93	20000	NA	NA	NA	0.000094
			jbreton189C175153						2.89	20000	NA	NA	NA	0.000066
			jbreton189K174401						2.76	20000	NA	NA	NA	0.000069
			jbreton189T001654						2.82	20000	NA	NA	NA	0.000085
			jbreton18A9174700						2.48	20000	NA	NA	NA	0.000064
			jbreton18AM171512						2.86	20000	NA	NA	NA	0.000057
			jbreton18AM171512						3.11	20000	NA	NA	NA	0.000063

A first attempt at comparing the Hamburg Wheel Tracking rutting and the corresponding pavement section field rutting is shown in Figure 34. The results show a poor correlation when attempting to simply compare rutting values without applying any filtering. Incorporating the asphalt mixtures' design gyration level, Figure 35 was produced. Figure 35 shows that typically lower Hamburg rutting occurred at higher design gyration levels (80) while resulting in some of the higher magnitudes of field rutting. The opposite occurred for the lower design gyration level (50) where more Hamburg rutting occurred with lower magnitudes of field rutting. Since the Hamburg Wheel Tracking test's loading and test parameters are not loaded for the expected traffic level, asphalt mixture designed for lower traffic levels (i.e. – lower design gyration levels, neat asphalt binders, lower angularity values, etc) will show greater amounts of rutting in the laboratory. However, since those same materials are exposed to very little traffic in the field, the field rutting is low.

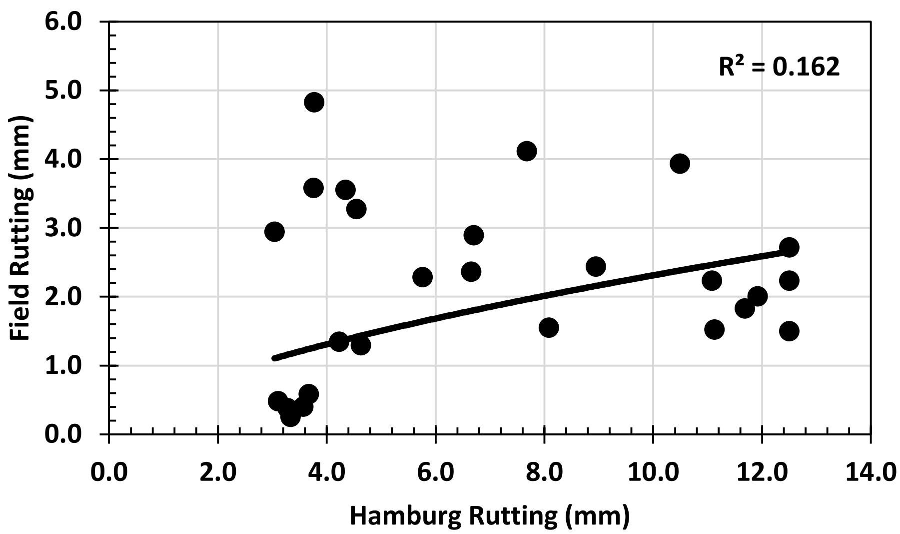


Figure 34 – Comparison of VAOT's Hamburg Wheel Tracking Rutting and Field Rutting

The same analysis from Figure 35 was also conducted but with asphalt binder grade (Figure 36). In Figure 36, it appears that the high temperature grade clearly influenced the laboratory test results as the PG70 asphalt binders showed much lower Hamburg Wheel Tracking rutting when compared to the PG58 asphalt binders.

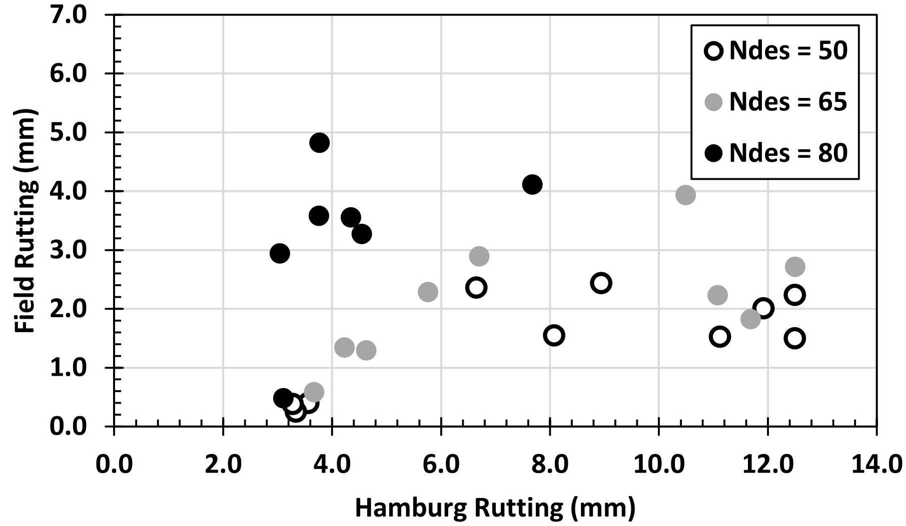


Figure 35 – Comparison of VAOT's Hamburg Wheel Tracking Rutting and Field Rutting While Identifying Design Gyration Level

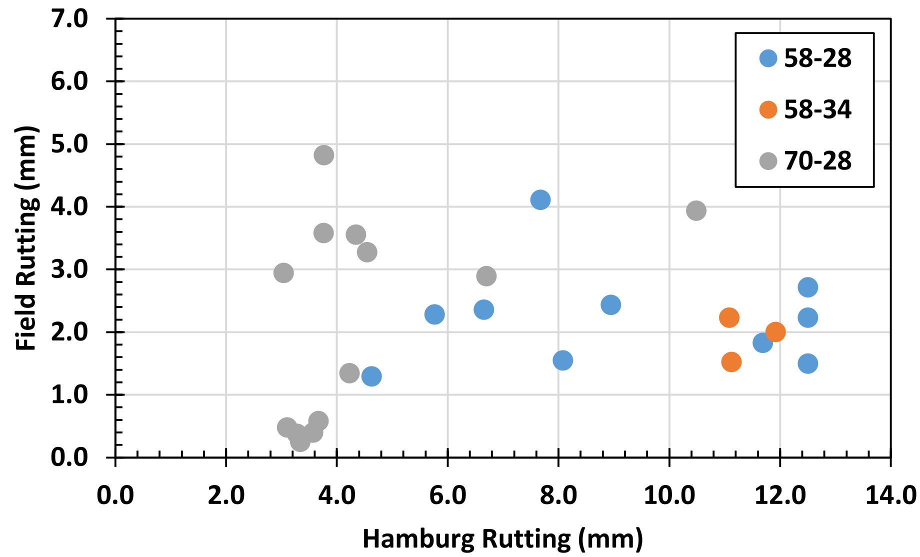


Figure 36 - Comparison of VAOT's Hamburg Wheel Tracking Rutting and Field Rutting While Identifying Asphalt Binder Grade

The field rutting was compared to the respective traffic level of the pavement section. As shown in Figure 37, the field rutting was found to be directly related to the AADT of the pavement section. Therefore, any Hamburg rutting criteria should take the AADT into consideration since the testing parameters of the test method will remain constant.

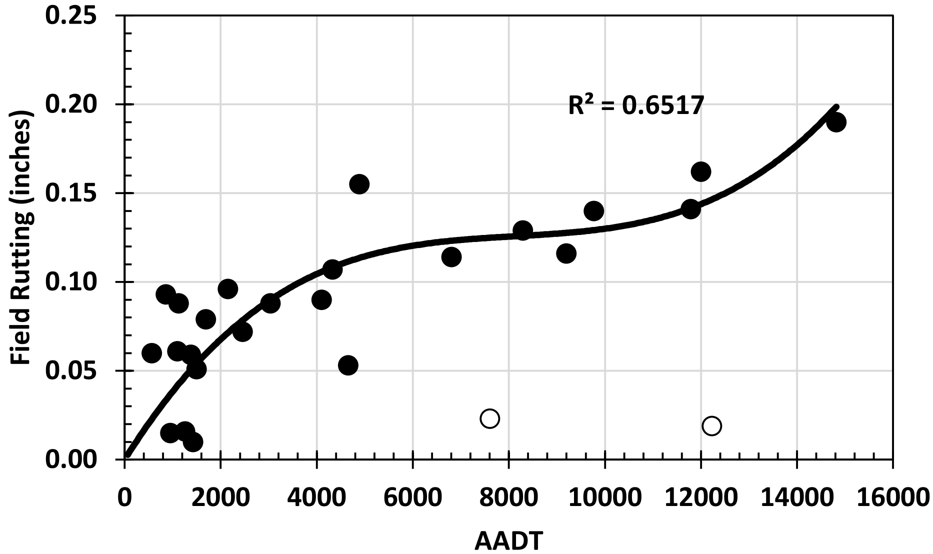


Figure 37 – VAOT AADT and Corresponding Field Rutting (Statistical Outlier = White Circles)

With traffic shown to be a critical factor, the field rutting rate was calculated by dividing the magnitude of field rutting by the AADT, which results in rutting rate parameter of mm/AADT. The resultant relationship between rutting rate and Hamburg rutting is shown in Figure 38. Figures 39 and 40 show the same information, but filtered by design gyration level and PG grade, respectively. By normalizing the field rutting with the applied traffic, a much better comparison between laboratory mixture performance and field performance was found. In addition, Figure 39 shows that the higher the gyration level, the lower the rutting rate. Both the 65 and 80 design gyration mixes showed much lower rutting rates than the 50 design gyration asphalt mixtures. Figure 40 showed that PG grade had a much greater influence on the laboratory Hamburg rutting performance than the actual field rutting rate.

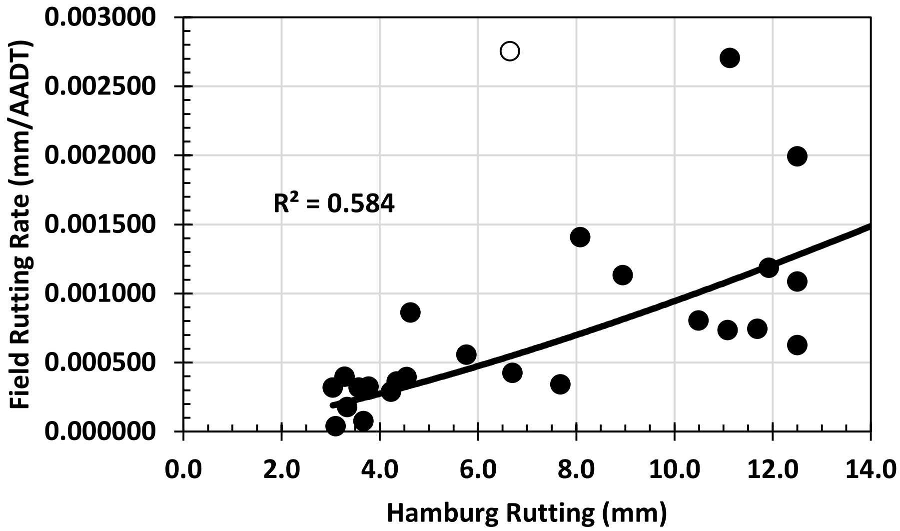


Figure 38 – Field Rutting Rate vs Hamburg Wheel Tracking Rutting

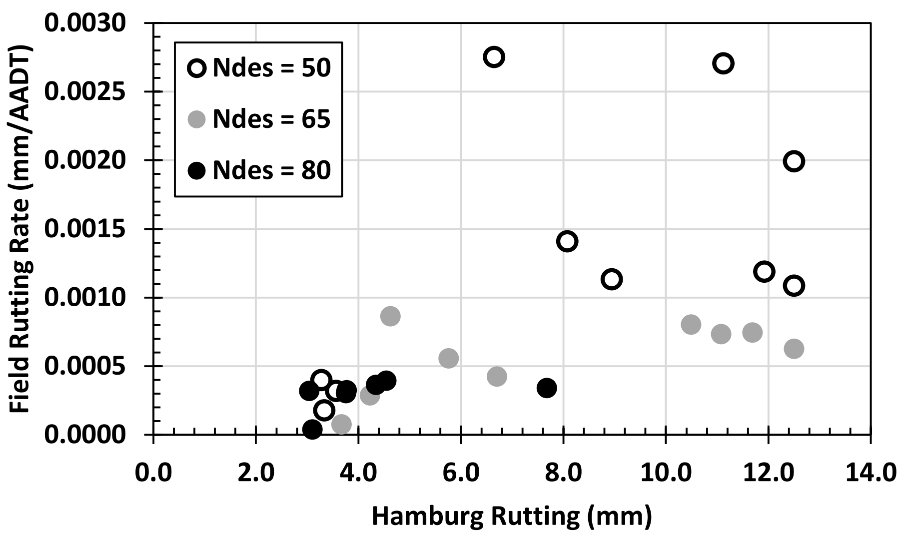


Figure 39 – Field Rutting Rate vs Hamburg Wheel Tracking Rutting with Design Gyration Level

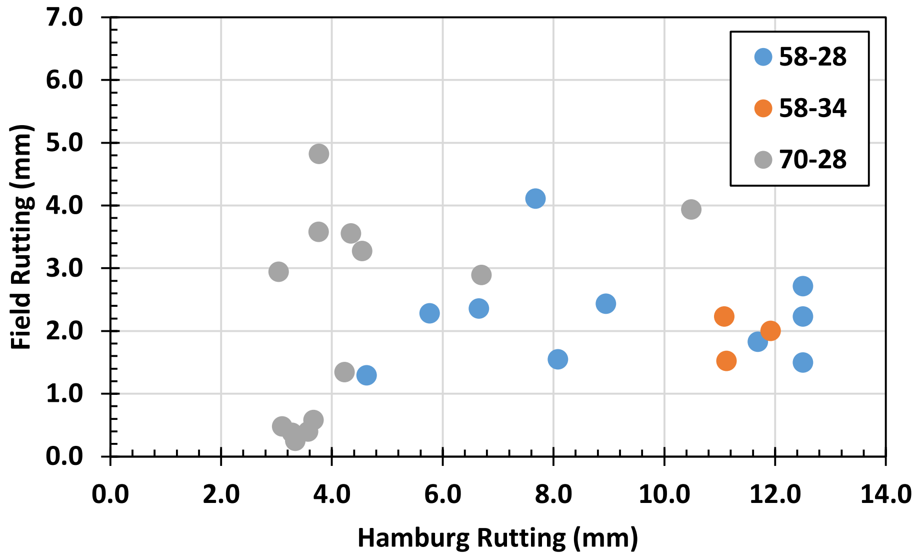


Figure 40 – Field Rutting Rate vs Hamburg Wheel Tracking Rutting with Asphalt Binder Grade

Rutting – Final Rcommendations

The final preliminary Hamburg Wheel Tracking rutting criteria is shown as Figure 41. The criteria is based on the design gyration level of the proposed asphalt mixture as this was found to be sensitive to the rutting rate of the field sections. The criteria indicates that for both the 50 and 65 design gyration asphalt mixtures, a maximum Hamburg rutting at 20,000 passes should be 12.5 mm. Both of these asphalt mixtures showed good field performance, even when some of the respective asphalt mixtures achieved close to an average of 12.5 mm rutting in the Hamburg. Meanwhile, the 80 design gyration level asphalt mixtures must meet a maximum of 6.0 mm of Hamburg rutting after 20,000 cycle. This value was found to be on the higher end of the average Hamburg rutting while still showing good field performance. Figure 41 shows a transition at an AADT of 7,000 in the proposed criteria as this mirrored where the 80 design gyration level asphalt mixtures appeared to begin being placed at (Figure 42).

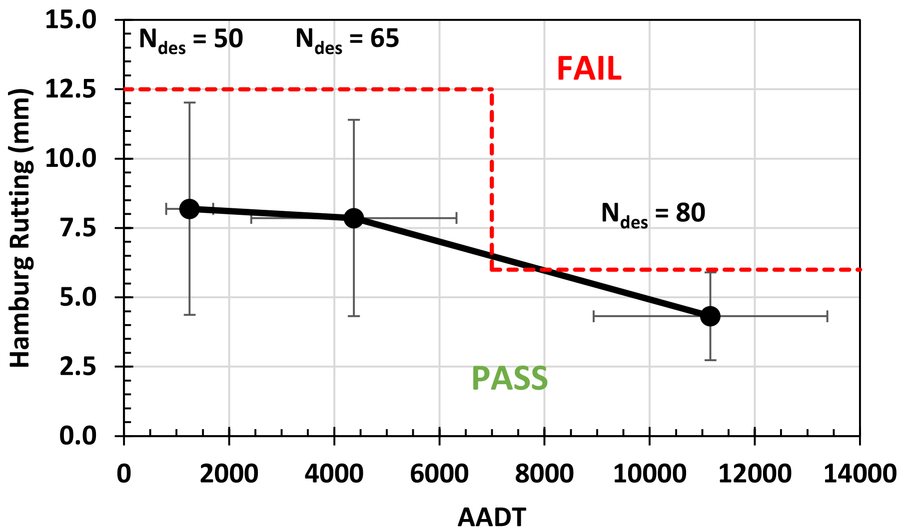


Figure 41 – Proposed Tentative Hamburg Wheel Tracking Rutting Criteria for VAOT's Asphalt Mixtures

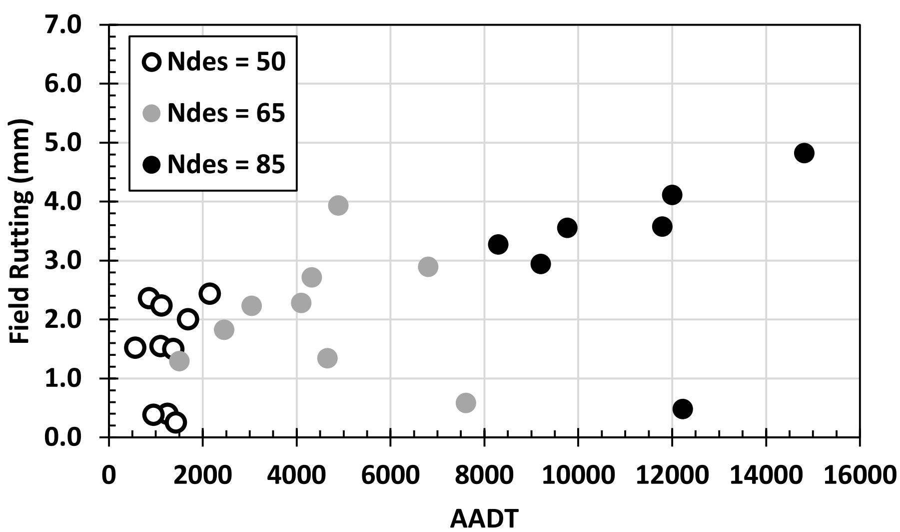


Figure 42 – Field Rutting vs AADT for Different VAOT Design Gyration Level Asphalt Mixtures

In addition to the preliminary criteria, Table 12 contains pavement sections in Vermont that are currently showing POOR rutting performance. To help further validate the proposed criteria, it is recommended that VAOT take field cores from the poor performing rutting sections and test in the Hamburg Wheel Tracking test.

Table 12 – Poor Rutting Performance Pavement Sections in Vermont

Poor Rutting Measurements										
Route Name	ETE_From	ETE_To	ETE_Road	Begin Town	AADT	Rut Average	Rut Index	Last Work Project Name	Last Work Project Number	Last Work Year
VT 9	3.1	3.2	V009	BENNINGTON	4520	0.34	66.5	Bennington	NH 2966(1)	2018
	3.3	3.4	V009	BENNINGTON	5113	0.29	71.4	Bennington	NH 2966(1)	2018
	3.5	3.6	V009	BENNINGTON	4900	0.30	70.5	Bennington	NH 2966(1)	2018
	3.7	3.8	V009	BENNINGTON	4900	0.54	45.8	Bennington	NH 2966(1)	2018
	3.8	3.9	V009	BENNINGTON	4900	0.35	64.5	Bennington	NH 2966(1)	2018
	3.9	4	V009	BENNINGTON	4900	0.30	70.5	Bennington	NH 2966(1)	2018
	4	4.1	V009	BENNINGTON	4990	0.38	61.6	Bennington	NH 2966(1)	2018
	4.1	4.2	V009	BENNINGTON	5200	0.29	71.4	Bennington	NH 2966(1)	2018
	4.2	4.3	V009	BENNINGTON	5200	0.32	68.5	Bennington	NH 2966(1)	2018
	4.3	4.4	V009	BENNINGTON	5302	0.29	71.4	Bennington	NH 2966(1)	2018
	4.4	4.5	V009	BENNINGTON	8600	0.30	70.5	Bennington	NH 2966(1)	2018
	4.5	4.6	V009	BENNINGTON	8600	0.45	54.7	Bennington	NH 2966(1)	2018
	4.6	4.7	V009	BENNINGTON	8600	0.30	70.5	Bennington	NH 2966(1)	2018
	4.7	4.8	V009	BENNINGTON	8600	0.36	63.6	Bennington	NH 2966(1)	2018
	4.8	4.9	V009	BENNINGTON	8600	0.38	61.6	Bennington	NH 2966(1)	2018
	4.9	5	V009	BENNINGTON	7973	0.39	60.6	Bennington	NH 2966(1)	2018
	5	5.1	V009	BENNINGTON	6700	0.35	64.5	Bennington	NH 2966(1)	2018
	5.2	5.3	V009	BENNINGTON	6126	0.31	69.5	Bennington	NH 2966(1)	2018
	5.3	5.4	V009	BENNINGTON	6000	0.38	61.6	Bennington	NH 2966(1)	2018
	5.4	5.5	V009	BENNINGTON	6000	0.30	70.5	Bennington	NH 2966(1)	2018
VT 12	58.4	58.5	V012	MONTPELIER CIT	3301	0.77	23.2	Montpelier	STP 2950(1)	2018
	58.5	58.6	V012	MONTPELIER CIT	3300	0.67	33.0	Montpelier	STP 2950(1)	2018
	58.6	58.7	V012	MONTPELIER CIT	3300	0.40	59.6	Montpelier	STP 2950(1)	2018
	58.8	58.9	V012	MONTPELIER CIT	3300	0.75	25.1	Montpelier	STP 2950(1)	2018
	58.7	58.8	V012	MONTPELIER CIT	3300	0.63	37.0	Montpelier	STP 2950(1)	2018
VT 14	93.3	93.4	V014	ALBANY	2100	0.49	50.8	District Paving	NE19PAV901	2018
	93.4	93.5	V014	ALBANY	2100	0.31	69.5	District Paving	NE19PAV901	2018
VT 67A	2.2	2.3	V067A	BENNINGTON	5800	0.32	68.5	Bennington	STP 2973(1)	2018
	2.4	2.5	V067A	BENNINGTON	5800	0.27	73.4	Bennington	STP 2973(1)	2018
	3	3.1	V067A	BENNINGTON	5800	0.27	73.4	Bennington	STP 2973(1)	2018
	3.3	3.348	V067A	BENNINGTON	6400	0.28	72.4	Bennington	STP 2973(1)	2018
VT 100	27.9	28	V100	WILMINGTON	4800	0.39	60.8	District Paving	NE19PAV101	2018
VT 131	0.2	0.3	V131	CAVENDISH	2800	0.58	41.9	District Paving	NE19PAV201	2018
	0.3	0.4	V131	CAVENDISH	2800	0.56	43.9	District Paving	NE19PAV201	2018
	1.1	1.2	V131	CAVENDISH	2900	0.39	60.6	District Paving	NE19PAV201	2018
	1.2	1.3	V131	CAVENDISH	2900	0.36	63.6	District Paving	NE19PAV201	2018
	1.3	1.4	V131	CAVENDISH	2900	0.48	51.7	District Paving	NE19PAV201	2018
	1.7	1.8	V131	CAVENDISH	2900	0.30	70.5	District Paving	NE19PAV201	2018
	1.8	1.9	V131	CAVENDISH	2900	0.51	48.8	District Paving	NE19PAV201	2018
	1.9	2	V131	CAVENDISH	2900	0.48	51.7	District Paving	NE19PAV201	2018
	2	2.1	V131	CAVENDISH	2823	0.45	54.7	District Paving	NE19PAV201	2018
	2.1	2.2	V131	CAVENDISH	2200	0.41	58.6	District Paving	NE19PAV201	2018
US 2	101.2	101.3	U002	CABOT	4100	0.51	49.0	District Paving	NE19PAV702	2018
	101.3	101.4	U002	DANVILLE	4100	0.49	50.8	District Paving	NE19PAV702	2018
	101.4	101.5	U002	DANVILLE	4100	0.57	42.9	District Paving	NE19PAV702	2018
	101.5	101.6	U002	DANVILLE	4100	0.33	67.5	District Paving	NE19PAV702	2018
	101.6	101.7	U002	DANVILLE	4100	0.42	57.6	District Paving	NE19PAV702	2018
	101.7	101.8	U002	DANVILLE	4100	0.34	65.5	District Paving	NE19PAV702	2018
	101.8	101.9	U002	DANVILLE	4100	0.45	54.7	District Paving	NE19PAV702	2018
	101.9	102	U002	DANVILLE	4100	0.28	72.4	District Paving	NE19PAV702	2018
	102.8	102.9	U002	DANVILLE	4100	0.37	62.6	District Paving	NE19PAV702	2018
US 5	186.4	186.5	U005	DERBY	10700	0.27	73.4	District Paving	NE19PAV902	2018
	186.8	186.9	U005	DERBY	10700	0.44	55.7	District Paving	NE19PAV902	2018
	187.2	187.3	U005	DERBY	11320	0.28	72.5	District Paving	NE19PAV902	2018
	187.5	187.6	U005	DERBY	8600	0.43	56.9	District Paving	NE19PAV902	2018
US 7	10.2	10.3	U007	BENNINGTON	6700	0.38	61.6	Bennington	NH 2966(1)	2018
	10.3	10.4	U007	BENNINGTON	6700	0.43	56.7	Bennington	NH 2966(1)	2018
	10.4	10.5	U007	BENNINGTON	6915	0.38	61.6	Bennington	NH 2966(1)	2018
	10.5	10.6	U007	BENNINGTON	7200	0.38	61.6	Bennington	NH 2966(1)	2018
	10.6	10.7	U007	BENNINGTON	7200	0.28	72.4	Bennington	NH 2966(1)	2018
	10.8	10.9	U007	BENNINGTON	8084	0.40	59.6	Bennington	NH 2966(1)	2018
	10.9	11	U007	BENNINGTON	8764	0.59	40.9	Bennington	NH 2966(1)	2018
	11	11.1	U007	BENNINGTON	9100	0.45	54.7	Bennington	NH 2966(1)	2018
	11.1	11.2	U007	BENNINGTON	9100	0.42	57.6	Bennington	NH 2966(1)	2018
	11.2	11.3	U007	BENNINGTON	9100	0.71	29.1	Bennington	NH 2966(1)	2018
	11.3	11.4	U007	BENNINGTON	8607	0.72	28.1	Bennington	NH 2966(1)	2018
	11.4	11.5	U007	BENNINGTON	7400	0.28	72.4	Bennington	NH 2966(1)	2018
	11.5	11.6	U007	BENNINGTON	8800	0.36	63.6	Bennington	NH 2966(1)	2018
	11.6	11.7	U007	BENNINGTON	9400	0.35	64.5	Bennington	NH 2966(1)	2018
	11.7	11.8	U007	BENNINGTON	9400	0.41	58.6	Bennington	NH 2966(1)	2018
	145.9	146	U007	MILTON	13900	0.58	41.9	District Paving	NE19PAV501	2018
I 89 SB	52.5	52.6	I089-S	MONTPELIER CIT	11300	0.26	73.7	ntpelier-Waterb	IM SURF(59)	2018
I 91 NB	11.9	12	I091	BRATTLEBORO	8200	0.30	70.2	ilford-Brattlebo	NH SURF(60)	2018

Final Conclusions

A study was conducted to help determine preliminary performance criteria for Connecticut, Maine and Vermont Agency of Transportations for potential use in Performance Related Specifications and Balanced Mixture Design. Different levels of data availability illustrated different procedures for how to develop initial criteria.

For the Connecticut DOT, laboratory performance test data was not available. Therefore, GOOD and POOR performing pavement sections were identified from the Pavement Management System (PMS). It was recommended that the field sections be cored and the recovered cores tested in the laboratory for their respective laboratory performance. Field to laboratory relationships, taking into consideration factors such as field traffic, design gyration level, asphalt binder grade, can then be developed and evaluated.

For the Maine DOT, laboratory performance data was available along with the corresponding field distress information. An extensive PMS database was not provided. The results of the analysis recommended:

- Fatigue Cracking: IDEAL-CT Index @ 25ºC > 150
    - Additional research also recommended to evaluate intermediate temperatures more representative to Maine's climatic conditions
- Rutting: Hamburg Wheel Tracking Test @ 45ºC and 20,000 Passes
    - AADT < 5,000: Rutting < 12.5 mm
    - AADT > 5,000: Rutting < 7.0 mm

For the Vermont AOT, laboratory performance data and a full PMS database was available for the analysis. After reviewing all of the available data and performance history, the following were recommended:

- Fatigue Cracking: SCB Flexibility @ 25ºC > 8.0
    - Additional research also recommended to evaluate intermediate temperatures more representative to Vermont climatic conditions
- Rutting: Hamburg Wheel Tracking Test @ 45ºC and 20,000 Passes
    - AADT < 7,000: Rutting < 12.5 mm
    - AADT > 7,000: Rutting < 6.0 mm

Lastly, there may be a means to develop "Regional" performance tests and criteria if state agencies are willing to compare and agree on tests methods and criteria. The benefit of this would be that asphalt producers that work in adjacent states would only need to worry about one set of performance tests and criteria, as opposed to many. For example, it is clear that the Hamburg test results from both Maine and Vermont are similar and general agreement could be made that when AADT < 6,000, rutting is < 12.5 mm, while AADT > 6,000, rutting is < 6.5 mm. A small compromise between state agencies resulting is a common specification.

Regarding fatigue cracking, this too could accomplished. However, at this time, Maine and Vermont AOT's do not utilize the same test procedure. By developing a comparative database of performance results, one could relate one test method to the other. Figure 43 shows an example of test data developed by Rutgers University using asphalt mixtures (plant and lab produced) for both New Jersey and New York. The comparison between the IDEAL-CT values and SCB Flexibility Index values are quite good ($R^2 = 0.79$, n = 138). Using the linear regression equation in the figure;

- SCB Flexibility Index of 8.0 = IDEAL-CT Index of 132
- IDEAL-CT Index of 150 = SCB Flexibility Index of 9.3

Interesting enough, the results of each other's fatigue cracking requirements are somewhat similar, when using the relationship for the NJ and NY materials. Although the data does not represent asphalt mixtures native to Maine and Vermont, conceptually the identical method could be conducted to help regionalize performance test criteria.

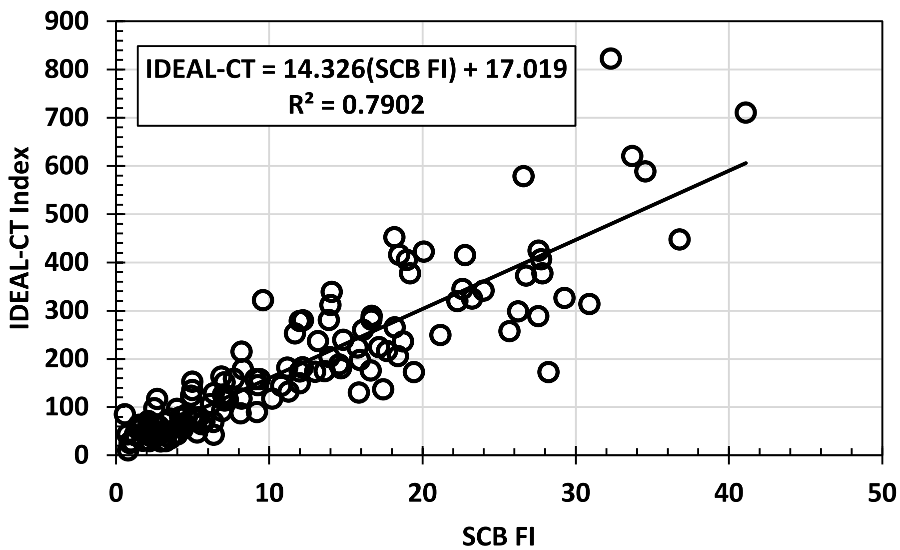


Figure 43 – Rutgers University Database Comparison of IDEAL-CT and SCB Flexibility Index Tested at 25°C