ANALYSIS OF FRICTION ON THE RHVP

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For the

RED HILL VALLEY PARKWAY INQUIRY

November 2022

FM Consultants

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1 Introduction

The main objective of this report is to review all the available evidence regarding the frictional properties of the Red Hill Valley Parkway (RHVP) since its construction in 2007 to its resurfacing in 2019, for the Red Hill Valley Parkway Inquiry (the RHVPI or the Inquiry).

I delivered a report for the Inquiry dated April 2022 titled "Primer on Friction, Friction Management, and Stone Matrix Asphalt Mixtures" (the Primer) about which I testified on April 26, 2022. The Primer is attached as Appendix 1 and I will draw on it or reference it from time to time in this report.

As summarized in the Primer (at p.5): The frictional properties of pavements play a significant role in road safety, as the friction between tire and pavement is a critical factor in reducing potential crashes. When a tire free rolls in a straight line, the contact patch is instantaneously stationary with little to no friction developed at the tire/road interface, although there are some interactions that contribute to rolling resistance. However, when a driver begins to execute a maneuver that involves a change of speed or direction, forces develop at the interface in response to acceleration, braking, and/or steering that cause a friction reaction between the tire and the road. Friction enables the vehicle to speed up, slow down, or track around a curve. The reaction forces are limited by the dynamic friction available.

2 Frictional Properties

Both friction and macrotexture are important to provide a safe pavement surface on high-speed roads. This section presents a discussion of the various set of friction and macrotexture measurements conducted on the RHVP.

Because friction depends on the interaction between the tire and the pavement, different measurements are obtained for different testing conditions, such as wet and dry pavement, hot and cold weather, type and condition of the tire, and so on. This variety of measurements has led to the development of different testing devices that operate under different conditions. Friction testing equipment used in the highway industry measures wet friction after spreading a small amount of the water on the pavement. However, the various friction-measuring technologies available use different types of tires, water film thicknesses, and operating principles, so they do not produce a common, standardized measurement of friction.

Furthermore, the level of friction available also depends on the speed at which the tire is slipping with respect to the pavement surface. When a tire is free-rolling on dry pavement, there is virtually no slip. However, as the driver starts to brake or navigate a curve, the tire starts to slip with respect to the pavement, up to the point where the tire is locked—not rotating—and the rubber on the contact patch is slipping at a speed equal to the vehicle speed.

Many different devices have been developed over the years to measure pavement friction. They all rely on the broad principle of sliding rubber over a wet road surface and measuring the reaction forces developed. These forces are used to compute the coefficient of friction and, in some cases, this number is multiplied by 100 to compute what standards call Friction Number (FN), Skid Number (SN), or Grip Number (GN).

2.1 RHVP Friction Measurements

Several sets of friction measurements on the RHVP have been conducted over the years using different technologies and equipment. These have included measurements with Locked-wheel testers (ASTM E274-15) using a ribbed tire (ASTM E501-08) mostly at 90 km/hr, with a GripTester (ASTM E2340-11) at 50 km/hr and British Pendulum tests in accordance with ASTM E303. In addition extracted aggregate samples were tested to determine its Polished Stone Value and macrotexture measurements taken according to Sand Patch test (ASTM E965). These will be discussed in detail below.

2.1.1 Locked Wheel Friction Measurements

Friction measurements were taken with a Locked-wheel tester (ASTM E274-15) at 90 km/hr using a ribbed tire (ASTM E501-08) by the Ministry of Transportation of Ontario (MTO) from 2007-2012 and 2014, and by ARA in 2019 both before and after the resurfacing of the RHVP.

2.1.1.1 MTO Locked Wheel Trailer Measurements, 2007-2014

From 2007 to 2012, and again in 2014, the MTO monitored the friction on the section of the RHVP from Greenhill Ave in the south to the CNR OH Structure in the north (approximate 3.8 kilometers) using a ASTM E274 trailer with a ribbed tire at 90 km/hr.[1] The averages by lane of these tests are summarized in Figure 1 and some key observations are presented following.[2]

[1] MTO Pavement Friction Survey 2007, RHVP Southbound Lane 1 (GOL0002620); MTO Pavement Friction Survey 2007, RHVP Southbound Lane 2 (GOL0002621); MTO Pavement Friction Survey 2014, RHVP Northbound Lane 1 (MTO0022943); MTO Pavement Friction Survey 2014, RHVP Northbound Lane 2 (MTO0022944); MTO Pavement Friction Survey 2014, RHVP Southbound Lane 1 (MTO0022945); MTO Pavement Friction Survey 2014, RHVP Southbound Lane 2 (MTO0022946); MTO Friction Test Report, Red Hill Valley Parkway, 2008-2014 (HAM0054586_0001).

[2] I am aware that in 2010 the MTO testing was conducted at 100 km/hr rather than the 90 km/hr it was conducted at in the other years, and that the operator subsequently adjusted the FN in the 2010 results to account for this difference. I have used the adjusted 2010 results as they appear in the 2014 results.

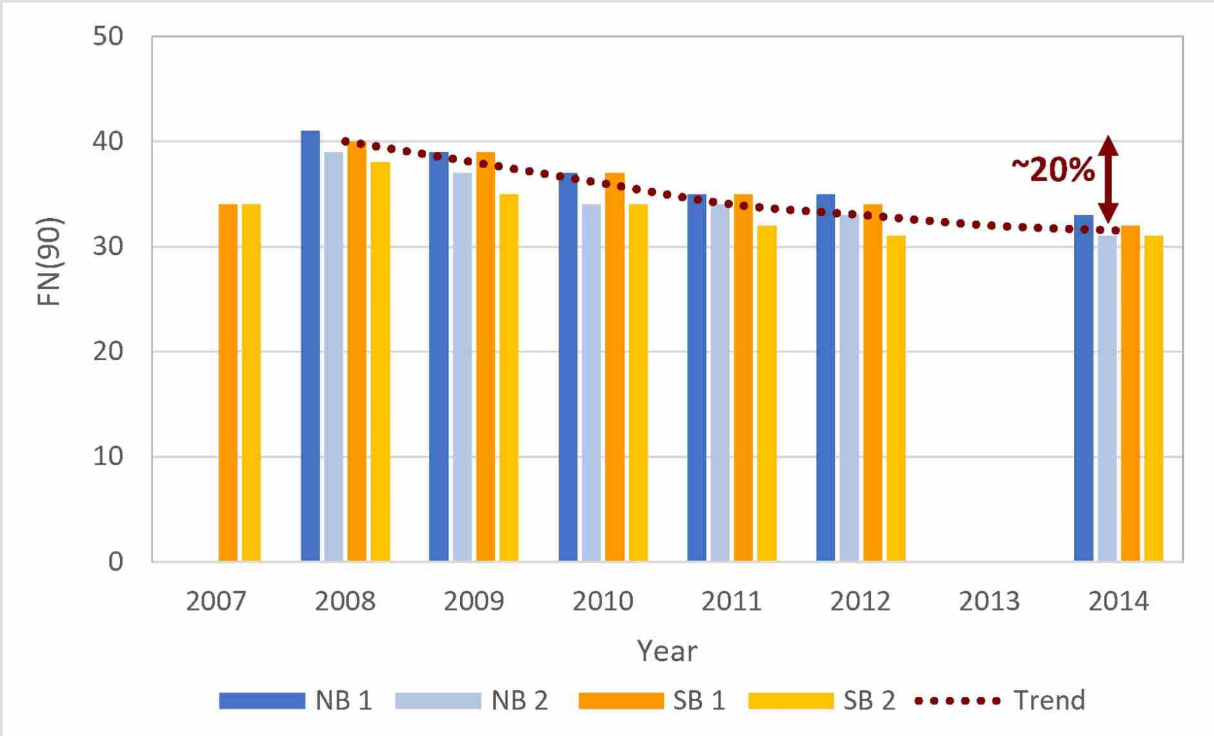

Figure 1. MTO Locked-Wheel tester average friction measurements in 2007-2012 and 2014.

For the initial MTO testing conducted in October 2007 shortly after construction and before the RHVP opened to public traffic, only the SB lanes were tested. The results of this testing showed low values, with an average FN(90) of 34 for both lanes with some individual results below FN(90)=30. However, it must be noted that these tests were conducted on a new SMA pavement before opening to traffic. Friction usually increases significantly during the first few weeks after opening. During the period shortly after construction, the traffic wears away the thin asphalt layer that initially covers the exposed aggregates thereby exposing their microtexture, which contributes to increasing the friction. As noted in the Primer, low friction for a short initial period has been identified as an issue for some SMA pavements.

In 2008, the values measured by the MTO were higher than in 2007, with FN(90) in the 39-41 range. This is consistent with the expected increase in friction after an initial period of lower friction on an SMA pavement immediately following construction.

In 2008 and in the subsequent years measured by the MTO, all four lanes were tested, both SB and NB, rather than just the SB lanes tested in 2007.

Beginning with the MTO testing conducted in 2011, some of the individual test results measured fiction numbers lower than FN(90)=30. I understand that FN(90)=30 is a value used by the MTO as a guideline for identifying road sections that may need a safety investigation, also considering other factors such as road geometry, collision history, etc. I further understand there was voluminous evidence on MTO friction management practices including its use of FN(90)=30 given by MTO witnesses at the Inquiry. I do not offer an opinion in respect of the

MTO's use of FN(90)=30 in respect of its friction management practices, but I note it as being a frictional value of some significance to the MTO.

The average friction values taken by MTO decreased by approximately 20% in the first 6 years (2008 to 2014), suggesting some level of aggregate polishing. There is a further discussion of polishing below.

As discussed below, when compared to testing conducted by entities other than the MTO, the RHVP friction values seem to have stabilized after 2014.

2.1.1.2 ARA Locked Wheel Trailer Measurements in 2019

Additional Locked-wheel tests were conducted by ARA in 2019 before (May) and after (September) the resurfacing of the RHVP that year. ARA conducted the testing at 90, 80, and 65 km/hr, using a ribbed tire.[3]

The ARA results taken at 90 km/hr are compared with the 2007-2014 MTO testing in Figure 2. While, as noted above, the MTO measurements cover only a subsection of the RHVP between Greenhill and the CNR OH Structure, the 2019 ARA measurements cover the entire length of the RHVP.

In fact, the ARA measurements go well beyond the limits of the SMA paving on the RHVP at both the north and south ends. For that reason, as set out in more detail below, I have calculated the averages for each of the four lanes by excluding the results at both ends that apparently were taken on different pavements than the RHVP SMA surface that was paved in 2007.

[3] Red Hill Valley Parkway – Surface Pavement Investigation Methodology Report, ARA, September 11, 2019 (HAM0009630_0001); RHVP Friction Testing Results, Northbound Lanes, ARA, May 2019 [Appendix A-I to Methodology Report] (HAM0009628_0001); RHVP Friction Testing Results, Southbound Lanes, ARA, May 2019 [Appendix A-II to Methodology Report] (HAM0009629_0001); RHVP Friction Testing Results, Northbound Lanes, ARA, September 2019 [Appendix A-I to Methodology Report] (HAM0009633_0001); RHVP Friction Testing Results, Southbound Lanes, ARA, September 2019 [Appendix A-II to Methodology Report] (HAM0009634_0001); see also Red Hill Valley Parkway – Surface Pavement Investigation Methodology Report, ARA, November 15, 2019 (HAM0009637_0001).

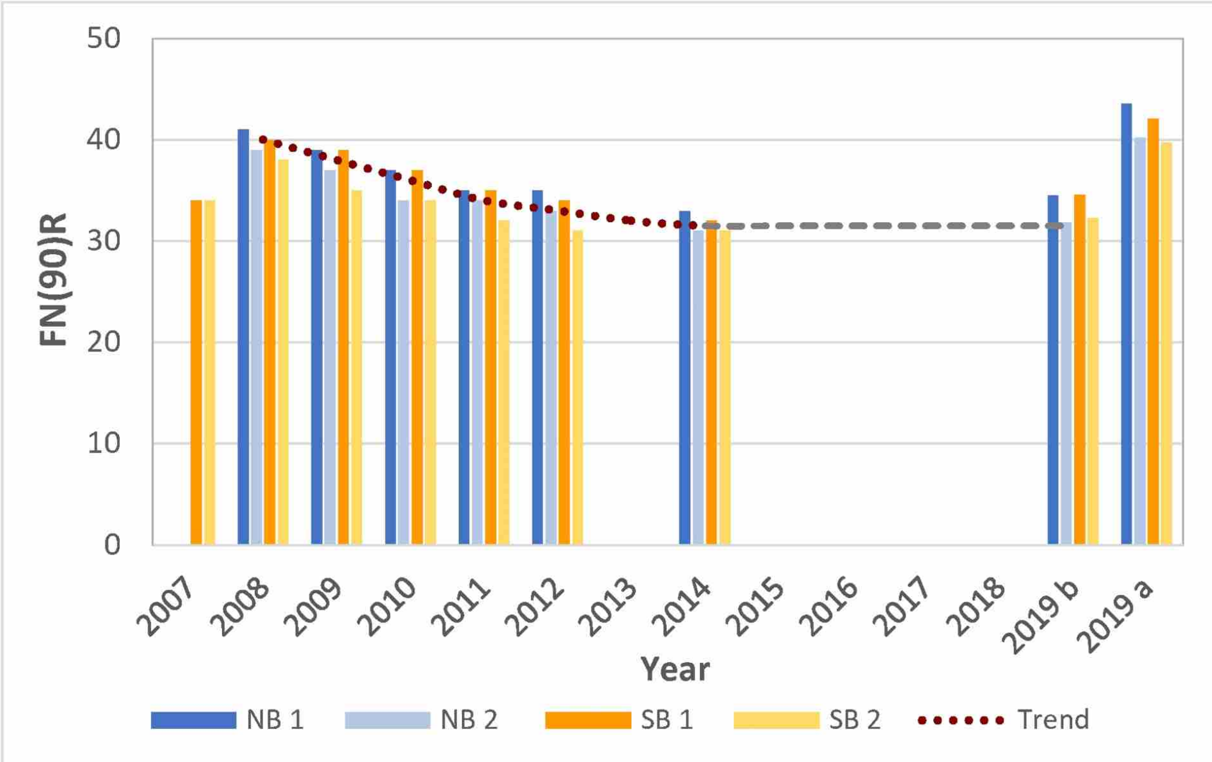

Figure 2. All Locked-Wheel tester average friction measurements at 90 km/hr: MTO 2007-2012 and 2014, and ARA 2019 before and after RHVP resurfacing.

The ARA measurements taken in 2019 before the RHVP resurfacing that year indicate that the average friction remained approximately at the same level as when the MTO performed its final testing in 2014. The average FN(90) for the four lanes ranged from 31 to 35.

To illustrate the trend along the section investigated, Figure 3 and Figure 4 present the FN(90)R values measured by ARA on the two main lanes in both directions, in 2019 prior to the resurfacing. It can be observed that several measurements (especially on lane 2 in both directions) are lower than FN(90)R = 30.

Furthermore, the plots from the ARA pre-resurfacing testing illustrated in Figures 3 and 4 also show that the friction is significantly higher in the adjacent sections at either end of the segment under consideration (RHVP). These are the LINC at the South end and the QEW interchange at the North end. I am advised by Commission Counsel that the LINC was resurfaced in 2011 by the City of Hamilton, and the QEW interchange paving was completed in or about late 2008 or early 2009 by the MTO. While I understand the exact limits of the RHVP SMA paving at the time of testing in 2019 are not available to me, the limits of that paving in 2007 are shown on the plans in Overview Document 3.1[4] and appear to roughly correlate to the locations where the ARA test results climb towards the north and south ends of the ARA testing.

[4] Overview Document 3.1 at pages 10 and 16

SOUTHBOUND LANES

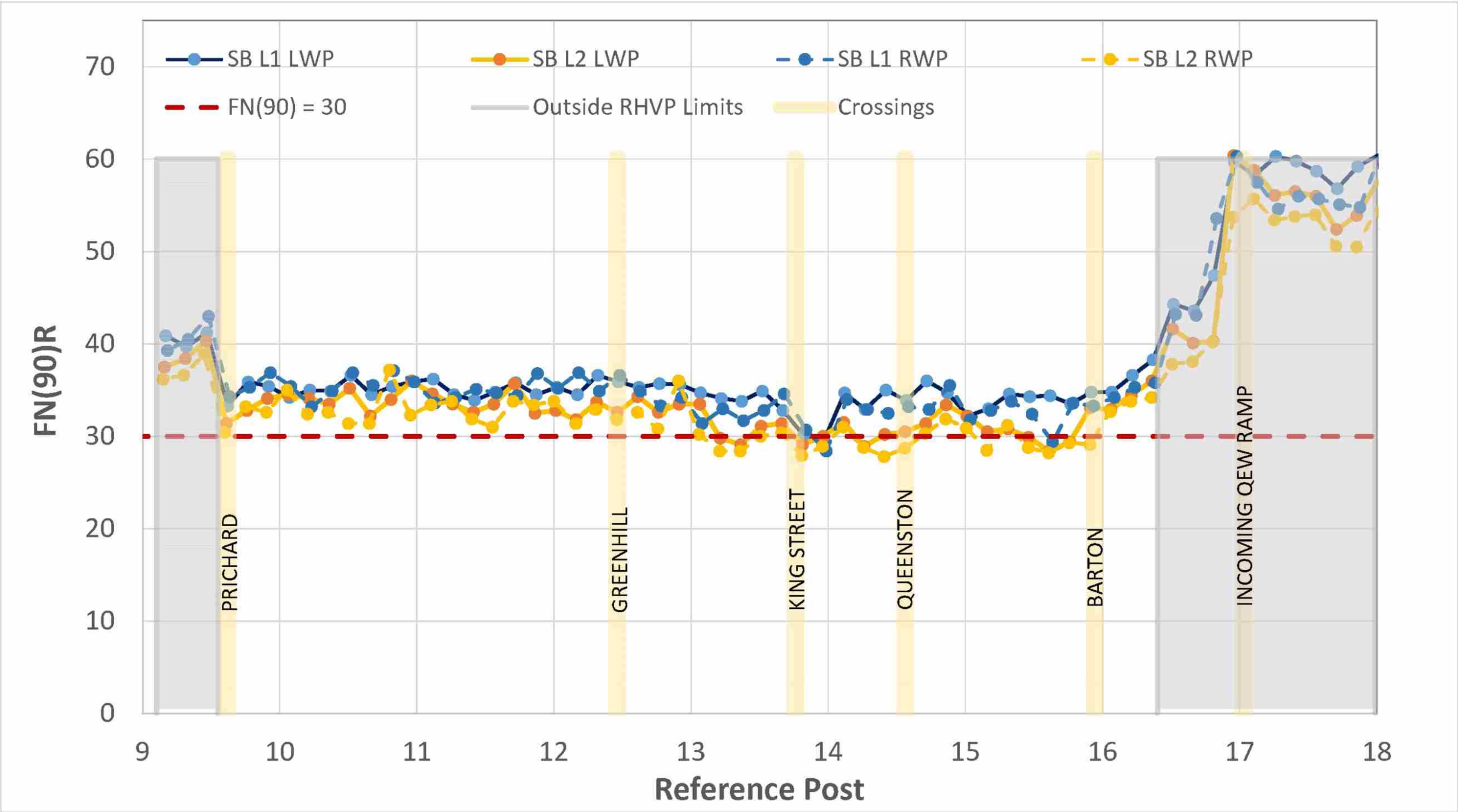

Figure 3. May 2019 Friction Measurements by ARA before resurfacing in the Southbound direction

Notes: The reference post is the chainage as provided in the files submitted by ARA. The shaded areas cover sections outside the results at both ends that were taken on different pavements than the RHVP SMA surface that was paved in 2007.

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NORTHBOUND LANES

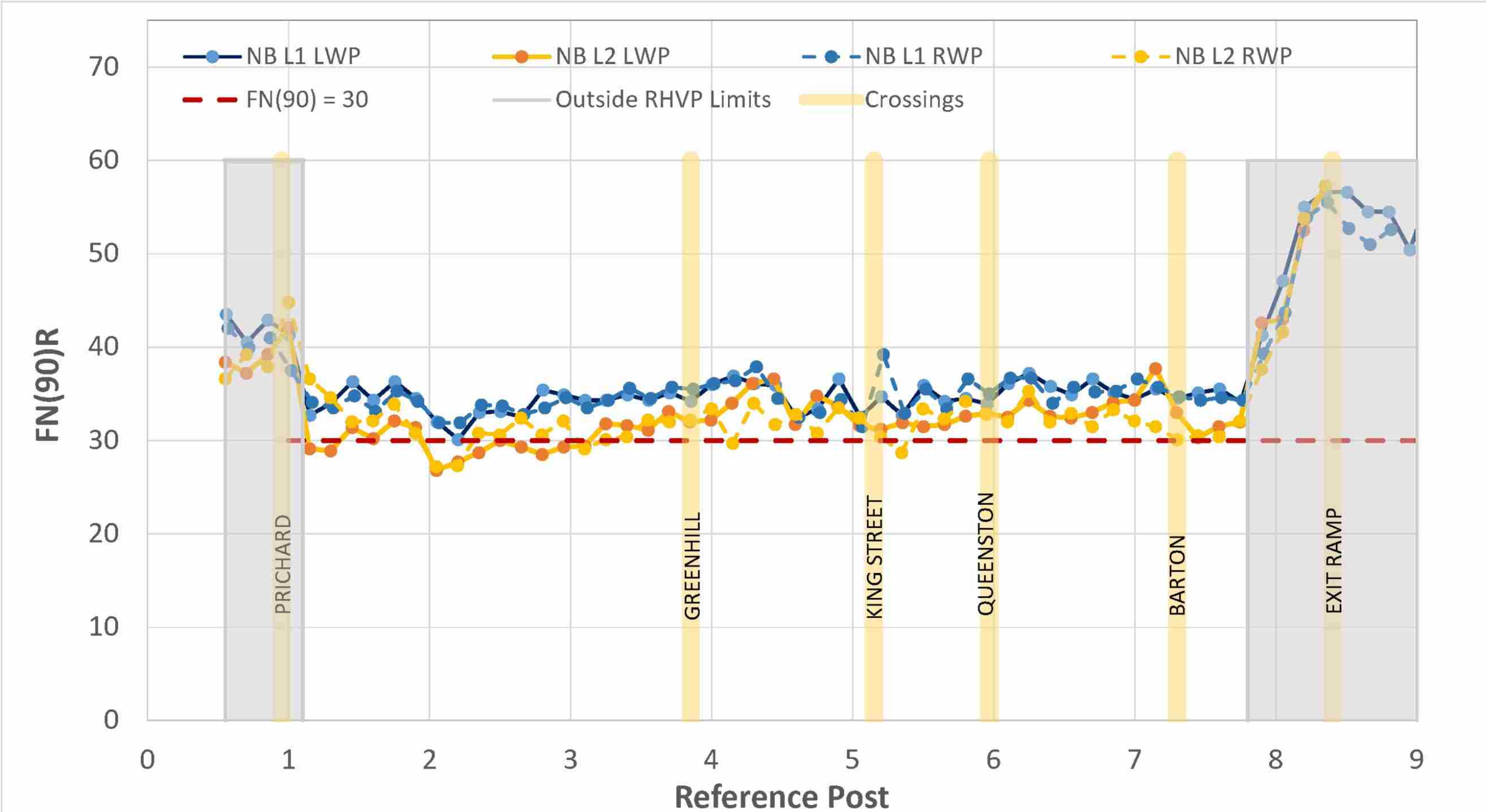

Figure 4. May 2019 Friction Measurements by ARA before resurfacing in the Northbound direction

Notes: The reference post is the chainage as provided in the files submitted by ARA. The shaded areas cover sections outside the results at both ends that were taken on different pavements than the RHVP SMA surface that was paved in 2007.

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In the QEW interchange area abutting the North end of the RHVP, the FN(90) quickly climbs into the 50's. On the LINC abutting the South end of the RHVP, the FN(90) also climbs sharply (but not as much) into the mid/high 30's and low 40's. The latter is consistent with the Tradewind Scientific GripTester measurements which, as discussed in the next section, included the LINC. The difference between the friction on the adjacent highway sections at either end of the RHVP compared with that on the RHVP itself makes the relatively low friction on the RHVP even more problematic. Those drivers reaching the RHVP from adjacent highway sections with higher friction may have an expectation of friction levels that are not available on the RHVP.

The ARA FN(90) measurements taken in September 2019 after the RHVP resurfacing averaged, by lane, between 40 and 44. These results were significantly higher than those collected by ARA in May 2019 prior to the resurfacing, and slightly higher than the friction values measured by the MTO in 2008 (average FN(90) 38-41).

2.1.2 GripTester Measurements

Additional measurements were taken using a fixed-slip continuous friction measurement equipment (GripTester), by Tradewind Scientific on November 20, 2013, and by Englobe in May 2019 prior to the RHVP resurfacing.

As outlined in the Primer, there are difficulties in comparing friction test results obtained by using different testing devices at different speeds. As I testified on April 26, 2022, all other things being equal, a GripTester will return higher GN (Grip Number) than a Locked-wheel tester will return FN (friction number). So, the GN reported by Tradewind and Englobe are not immediately comparable to the MTO and ARA results. Directionally, one would expect the GripTester GN to be higher than the Locked-wheel tester FN.

Nevertheless, I consider the GripTester results by both Tradewind and Englobe to be generally confirmatory of, and consistent with, the Locked-wheel tester results obtained by the MTO and ARA, for reasons that I will explain after discussing the GripTester results themselves.

2.1.2.1 Tradewind Scientific GripTester Measurements

On November 20, 2013, Tradewind Scientific conducted continuous friction testing with a GripTester (ASTM E2340-11) at 50 km/hr, using a 0.25mm water film thickness and an ASTM 1844 test tire inflated at 140 KPa (20 psi) on the four lanes of the RHVP and the LINC.[5]

Measurements taken by Tradewind for the RHVP are presented in Figure 5 (Southbound lanes) and Figure 6 (Northbound lanes).

[5] RHVP & LINC Friction Testing Results, Tradewind Scientific Ltd., November 20, 2013 (TRW0000092); Friction Testing Survey Summary Report, Lincoln Alexander & Red Hill Valley Parkways (Hamilton), Tradewind Scientific Ltd., November 20, 2013 (GOL0001113).

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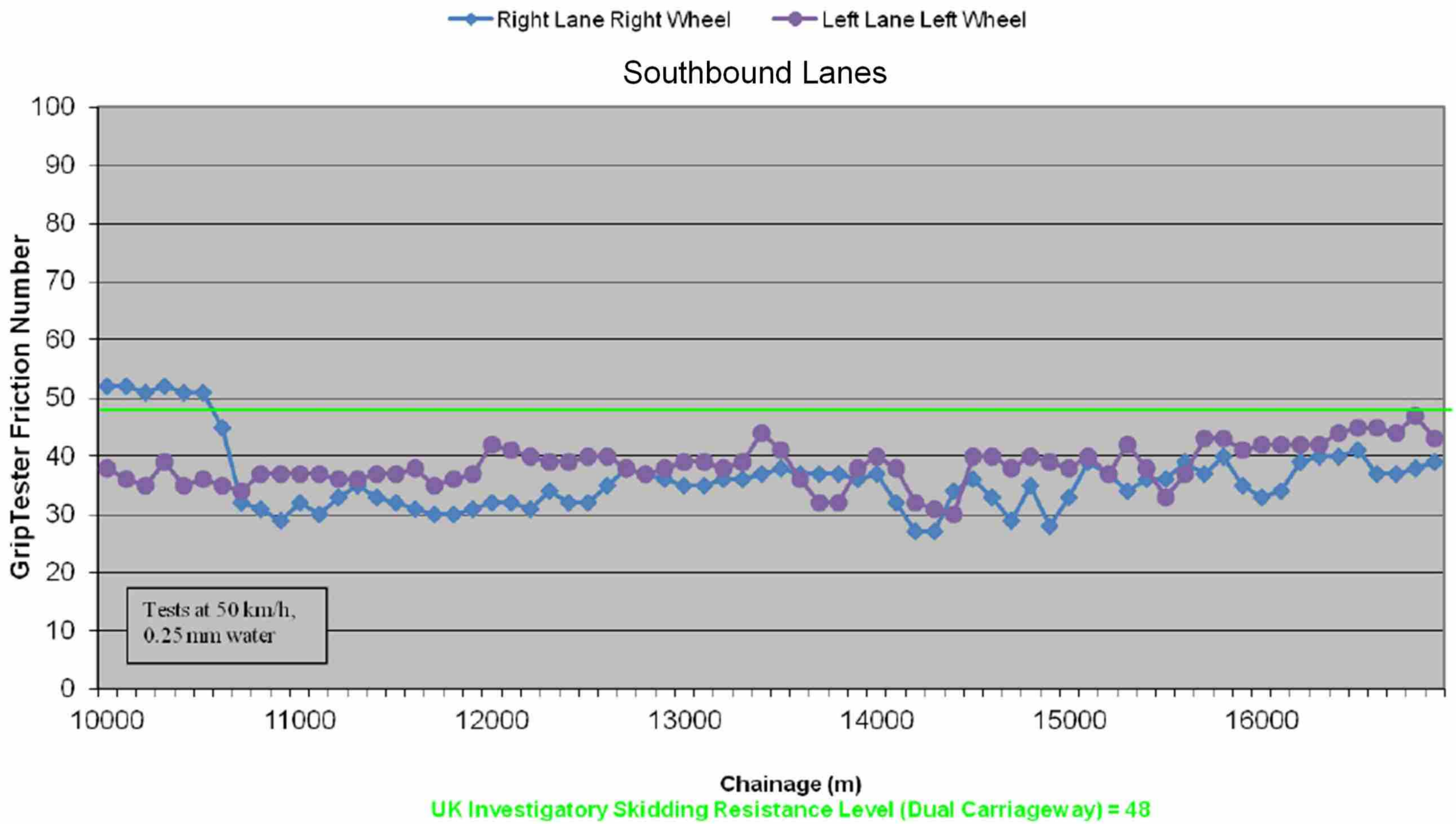

Figure 5. GripTester measurement by Tradewind on the Southbound direction, November 20, 2013 (Tradewind, 2013).

Note: This Figure is reproduced from the Tradewind Report except that in the Tradewind Report the "Southbound Lanes" are referred to as the "Westbound Lanes" as a continuation from the LINC westbound lanes.

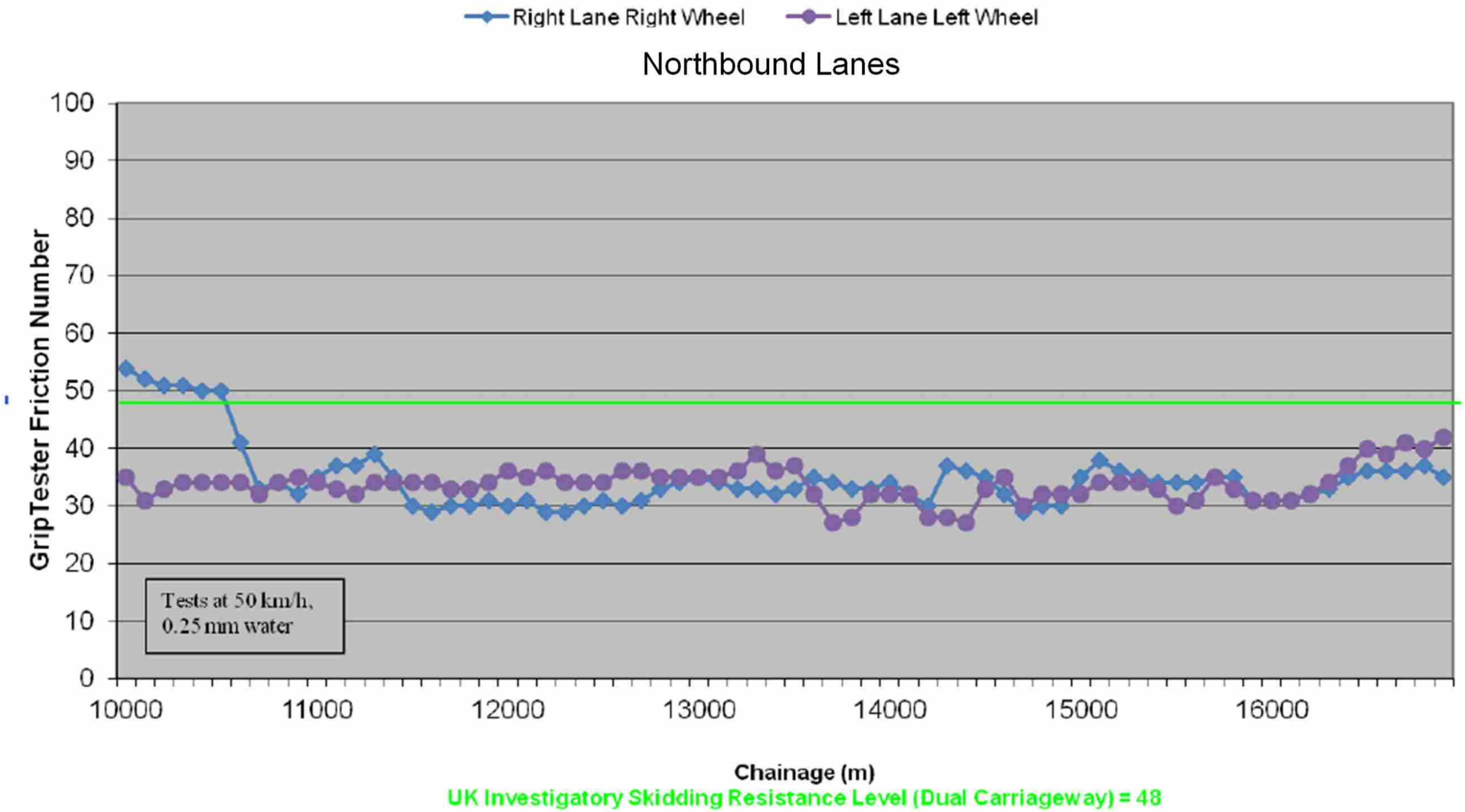

Figure 6. GripTester measurement by Tradewind on the Northbound direction, November 20, 2013 (Tradewind, 2013).

Note: This Figure is reproduced from the Tradewind Report except that in the Tradewind Report the "Northbound Lanes" are referred to as the "Eastbound Lanes" as a continuation from the LINC eastbound lanes.

My key observations about the Tradewind Report are presented following:

- Although the GripTester is used mostly on airports, it has been used to test highway pavements in different countries. As set out in the Primer, it has the advantage over the ASTM E274 Locked-wheel trailer of providing continuous measurements and, thus, minimizing the chances of missing localized areas of low friction. In addition, it is more reflective of the anti-lock braking systems used in modern vehicles.
- Using approximate conversions and adjusting for speed, the average values are similar to those collected by the MTO in 2014 with the ASTM E274 trailer. This conversion and comparison is discussed later in this report.
- The GripTester Friction Numbers on the RHVP were considerably lower than in the LINC. As set out in the Tradewind Report, with the exception of about 600m on the outside lanes where the RHVP joins the LINC (discussed more below), the RHVP wheel path results were mostly in the range of GN30-40 on the outside lanes and GN30-45 on the inside lanes (with some below GN30) while the LINC results were more consistent and generally in the range of GN50-60.
- The Tradewind Report indicated that overall friction averages on the RHVP were "below or well below the same UK Investigatory Level 2 (GN of 48)" and recommended that "a more detailed investigation be conducted and possible remedial action be considered to enhance the surface texture and friction characteristics of the Red Hill Valley Parkway, based on the friction measurements recorded in the current survey."[6]
- I concur with this recommendation. Although Tradewind used an earlier table with an earlier conversion to convert the Investigatory Levels for the SCRIM to GN and only one investigatory level for each demand category, the same conclusion would have been reached using the levels reported by UKPMS (2005) and reproduced in Table 1, which was the UK standard at the time of the Tradewind Report.
- Tradewind's average measured values on all four lanes are below the GN=41 values recommended for motorways or highways in Table 1, and thus, a detailed safety investigation was warranted based on that British standard.
- Furthermore, higher investigatory values could have been applied because of the curves and ramps in the section under investigation. Therefore, according to the British standards, the measured GN should have given rise to consideration of whether friction was a contributing factor to collisions along with other relevant factors.
- In addition, as pointed out in the Tradewind Report, "there are some localized sections with quite low friction values, reaching 27-30 in several areas." That is, GN measurements significantly lower than the averages. These locations should have been investigated to determine if the low friction was contributing to collisions, especially in wet conditions.

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[6] Friction Testing Survey Summary Report, Lincoln Alexander & Red Hill Valley Parkways (Hamilton), Tradewind Scientific Ltd., November 20, 2013 (GOL0001113 at image 13).

Table 1. Adaptation of the UK Investigatory Levels for a Mark 2 GripTester using a conversion factor of 0.85 (after UKPMS 2005)[7].

Site category and definition			Investigatory level (IL) at 50 km/h							
		SFC	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
		GN	35	41	47	53	59	65	71	76
A	Motorway									
B	Non-event carriageway with one-way traffic									
C	Non-event carriageway with two-way traffic									
Q	Approaches to and across minor and major junctions, Approaches to roundabouts and traffic signals									
K	Approaches to pedestrian crossings and other high risk signal									
R	Roundabouts									
G1	Gradient 5-10% longer than 50 m (see note 6)									
G2	Gradient >10% longer than 50 m (see note 6)									
S1	Bend radius < 500 m – carriageway with one-way traffic									
S2	Bend radius < 500 m – carriageway with two-way traffic									

- Notes: Reference should be made to Chapter 4 of HD 28/04 and in particular, the notes to Table 4.1 (of HD 28/04) for guidance on interpretation.
- Dark Grey indicates the range of ILs that should generally be used for roads carrying significant levels of traffic.
- Light Grey in cells indicates a lower IL that may be appropriate in lower risk situations, such as low traffic levels or where the risks present are mitigated by other means, providing this has been confirmed by the crash history.

- Measurements were also taken by Tradewind in the center of the outside lane (lane 2) in the Northbound direction (referred to as "Eastbound" in the Tradewind Report) and those results were higher than the measurements taken on the wheel paths in each lane (approximately 23% higher that the average of the wheelpaths of the two lanes in the same direction and 18% higher than the average on the wheelpaths in all four lanes in both directions), supporting the assumption that the aggregate had polished on the wheel paths and that the drop in friction was at least partially due to this polishing. To maintain appropriate levels of friction over time, it is important that the aggregates exposed on the surface of the pavement maintain its microtexture. The microtexture is the fine-scale texture, with amplitude lower than about 0.5 mm, on the surface of the coarse aggregate that interacts directly with the tire rubber on a molecular scale and provides adhesion. Although there is always some wear or polishing due to the abrasive effect of the tire on the pavement, if the coarse aggregate sources are susceptible to polishing, the reduction in friction over time can be significant as discussed later in this report.
- The southernmost approximately 600m of the right (outside) lanes in both directions that the Tradewind Report includes as part of the RHVP (adjacent to what Tradewind defines as the LINC) have similar GN (slightly above GN=50) to Tradewind's results for the LINC. Indeed those two 600m segments continue at virtually the same GN as the immediately adjacent

[7] UKPMS (2005). *UK Pavement Management System User Manual Volume 3: Machine Data Collection for UKPMS.* http://www.ukroadsliaisongroup.org/en/utilities/document-summary.cfm?docid=6FC2D12A-93EE-4DE6-B2C3879F57EF918F (accessed April 2020)

LINC segments. The same is true for the center reference measurements taken by Tradewind in the middle of the Northbound right (outside) lane. I am advised by Commission Counsel that there is evidence that those 600m segments were paved as part of the 2011 resurfacing of the LINC and so is a different pavement surface than the RHVP SMA surface paved in 2007. That would certainly make sense given the discrepancy between those 600m segments and the rest of what Tradewind defined as being the RHVP. It would also be consistent with ARA's 2019 RHVP pre-resurfacing Locked-wheel test results for the southernmost area tested as described earlier in this report.

- Just as the MTO 2014 and the ARA 2019 pre-resurfacing Locked-wheel tests showed a levelling off of the previously declining FN(90) results, as will be discussed below, the Englobe 2019 pre-resurfacing GripTester results are similar to the Tradewind test results taken in late 2013. This alignment supports the conclusion that the Tradewind results are reliable.

In my opinion , the methodology used by Tradewind and the test results as reflected in the Tradewind Report are sound (apart from applying the incorrect UK standard which did not change the result). I concur with the conclusions and recommendations in the Tradewind Report.

In January 2014 Golder prepared a report about the RHVP titled "Performance Review after Six Years in Service" which attached the Tradewind Report and referred to the "Friction Number (FN)" values in the Tradewind Report as being "relatively low".[8] While the nomenclature of using "FN" for GripTester results is incorrect, I agree with Golder that the Tradewind results were "relatively low". They were below the UK investigatory level referred to above.

As set out in the Primer, I recognize that, unlike in some other jurisdictions (notably as discussed in the Primer, the UK, Australia, and New Zealand, and some U.S. states) there are no published provincial or national standards in Ontario or Canada respecting highway friction investigatory or intervention levels. However, in my view that does not mean standards imported from other jurisdictions for the purposes of evaluating the frictional qualities of pavements have no meaning or ought to be disregarded. To the contrary, the British standards reproduced in Table 1 can provide a good reference.

2.1.2.2 Englobe GripTester Measurements

In May 2019, prior to the RHVP resurfacing, Englobe continuously measured the friction of the entire RHVP section with a GripTester (ASTM E2340-11) at 50 km/hr using a 0.25mm water film thickness and an ASTM 1844 test tire inflated at 140 KPa (20 psi).[9]

The GN measured by Englobe in the Northbound and Southbound directions of the RHVP are presented in Figure 7 and Figure 8, respectively. It must be noted that the plots on the Y axis of Figures 7 and 8 display the friction coefficient and not the GN (Grip Number). To obtain the GN requires multiplying the friction coefficient by 100; for example, a friction coefficient of 0.4 is GN40.

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[8] Red Hill Valley Parkway Performance Review after Six Years in Service, Golder, January 2014 (GOL0002981 at image 10).

[9] Red Hill Valley Parkway Friction Testing, Englobe, September 10, 2019 (HAM0009626_0001).

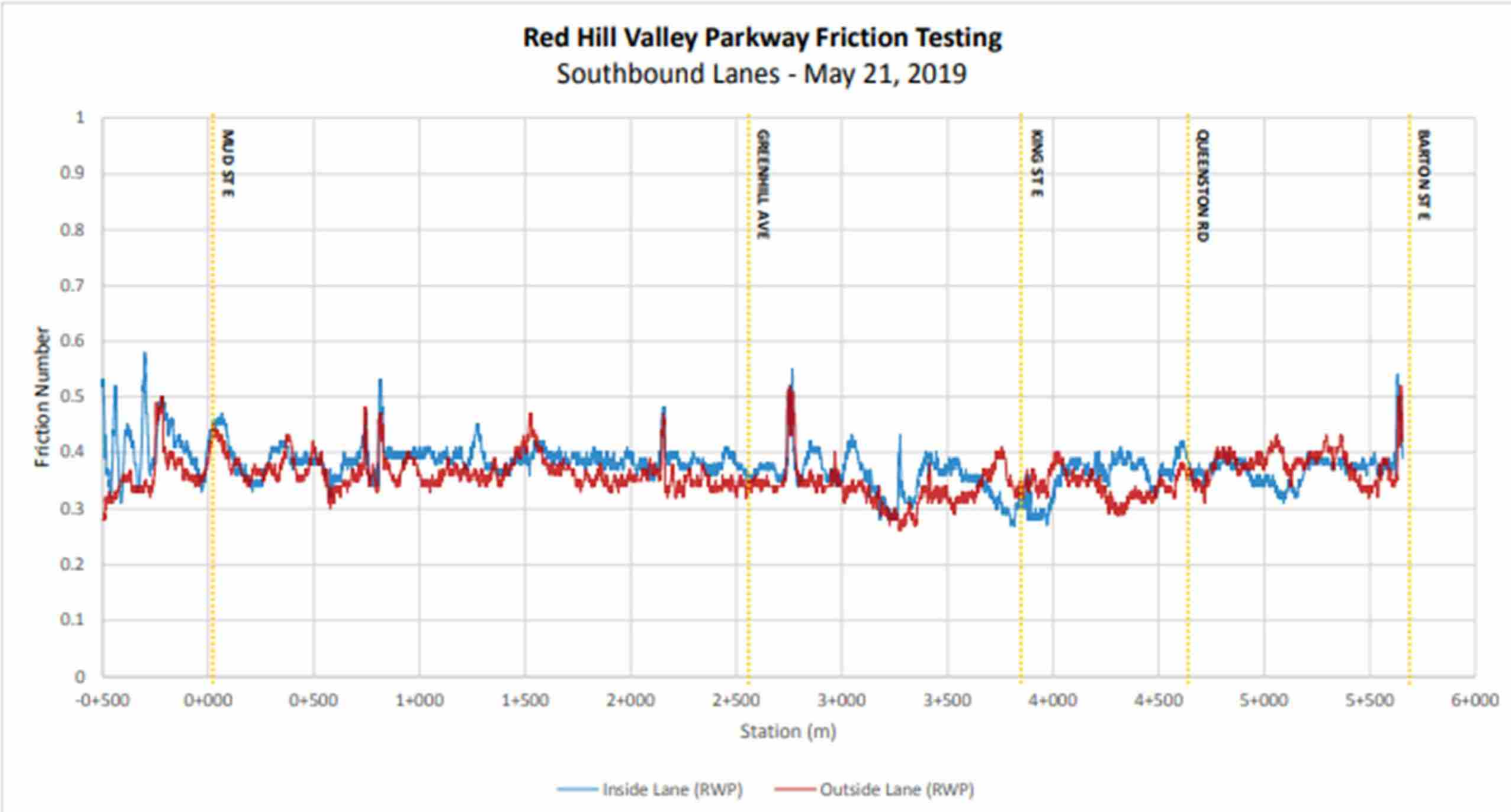

Figure 7. GripTester measurements taken by Englobe pre-resurfacing on the Southbound Lanes, May 2019.

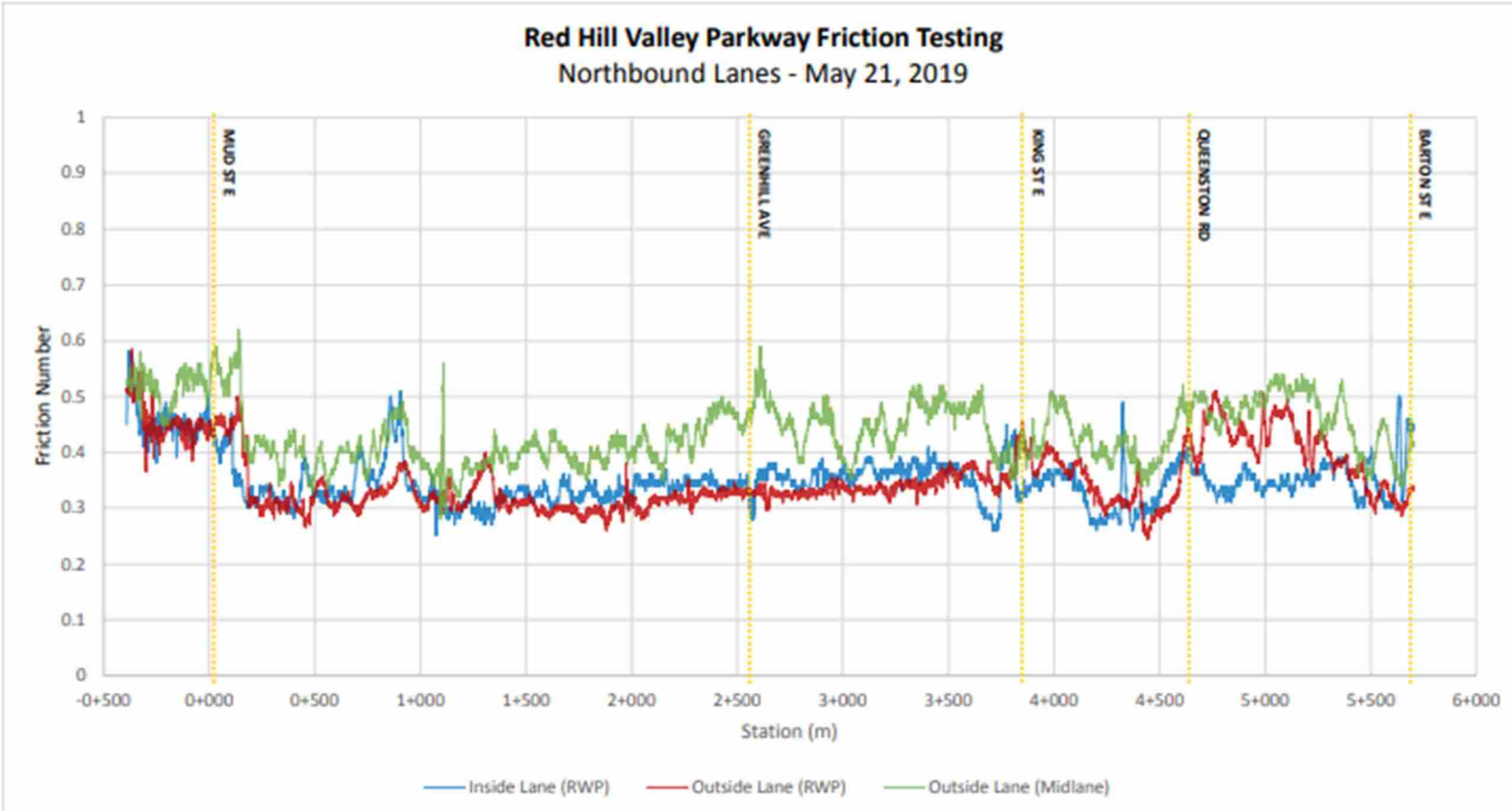

Figure 8. GripTester measurements taken by Englobe pre-resurfacing on the Southbound Lanes, May 2019.

The GN values obtained by Englobe in May 2019 with a GripTester were very similar to those measured by Tradewind in 2013 with the same type of equipment, confirming the MTO and ARA Locked-wheel results indicating that the friction had stabilized after 2014. The average GN for each of the four lanes as measured by Tradewind in November 2013 and Englobe in May 2019 is compared in Figure 9.

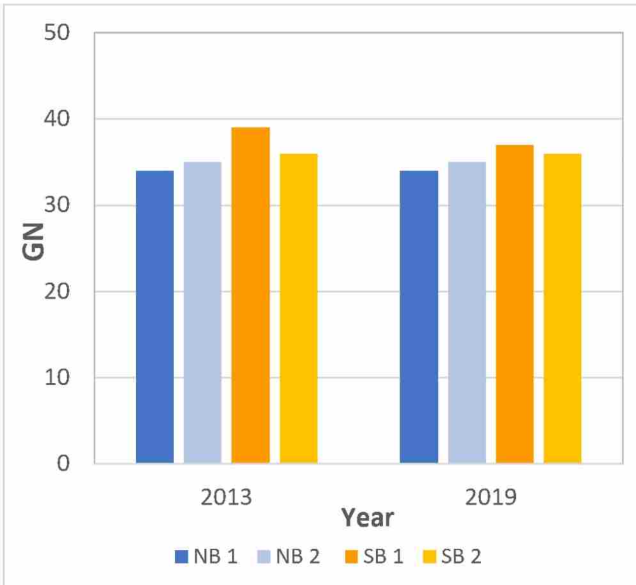

Figure 9. Evolution of GripTester average measurements by lane between those taken by Tradewind in November 2013 and Englobe in May 2019.

The Englobe results as shown in Figures 7 and 8 also confirm the presence of localized areas with lower friction, as observed in the Tradewind report.

Measurements were also taken by Englobe in the middle of lane 2 (outside lane) in the Northbound direction and those results were higher than the measurements taken on the right wheel path in each lane (approximately 23% higher), again supporting the assumption that the aggregate had polished on the wheel paths and that the drop in friction was due to this polishing, as discussed for the Tradewind measurements.

2.1.2.3 Comparison between GripTester and Locked-Wheel Trailer Measurements

Figure 10 compares the GripTester results by Tradewind and Englobe with those obtained using the Locked-wheel trailer by MTO and ARA discussed in the previous section.

This comparison required converting the GripTester Numbers (GN) to FN(90)R using approximate equations and adjusting for the testing speed. The specific steps to convert the values include the following steps:

1. Convert the GripTester number (GN) to SCRIM reading (SR) as it is used in the U.K. based on the equation provided in

SR = GN*0.89 (Dunford, 2010)[10]

2. Correct the SR to the side force friction number SFN at 50 km/hr

   $SFN(50) = SR(UK)/0.78$

3. Convert the SFN to a FN measured with a locked-wheel trailer at 40 mph or 65 km/hr

   $FN(65) = 0.87 * SFN(50) - 1.5$ (de Leon et al. 2019)[11]

4. Correct the measurements to FN(90)R using the average change in friction from the measurements at 90 km/hr and 65 km/hr in the May 2019 ARA Measurements.

   $FN(90) = FN(65) * 0.96$

The results set out in Figure 10 suggest that the results of the 2014 MTO testing and the November 2013 Tradewind testing are consistent and show relatively low average friction levels 6 to 7 years after construction. Similarly, it suggests that the results of the pre-resurfacing 2019 ARA and Englobe testing are consistent and show that the friction levels had levelled off after 2013/2014.

The Primer contains a section on interconversion of friction measurements (at p.16) in which I indicated that "the interconversions are not very accurate and may not apply to pavements not included in their development". That remains true. However, in the case of the RHVP I am confident that the conversion of GN to FN(90), while not exact, is reasonably accurate. This is because the equations listed in this section are current and the converted values agree with the ASTM Locked-wheel measurements. Thus, I believe that the conversions are at least reasonably appropriate.

2.1.3 Comments on Friction Measurement and Temperature

Since the various tests have been conducted at different temperatures, it is important to also discuss the impact of temperature on friction measurements.

As stated in the Primer: Because both hot mix asphalt surfaces and tires are viscoelastic materials, temperature also affect their properties. Furthermore, the water viscosity also changes with temperature. Research has indicated that tire-pavement friction decreases if the tire temperature increases (AASHTO 2008).[12]

Some standards provide a range of allowed temperature for measuring friction; for example, AASHTO TP 143-21 recommends a pavement temperature range between 5°C to 50°C for measuring friction using the SCRIM.

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[10] Dunford, A., (2010) GripTester Trial - October 2009, TRL Published Project Report PPR497, for ADEPT, Devon County Council, Wokingham, Berkshire, U.K.

[11] de León Izeppi, E., Flintsch, G., Katicha, S., McGhee, K., and McCarthy, R. (2019). *PFM Program Utilizing Continuous Friction Measurement Equipment and State-of-the-Practice Safety Analysis Demonstration Project Final Report.* Federal Highway Administration Report, Washington, DC

[12] AASHTO (2008). *Guide for Pavement Friction, American*, American Association of State Highway and Transportation Officers, Washington, DC.

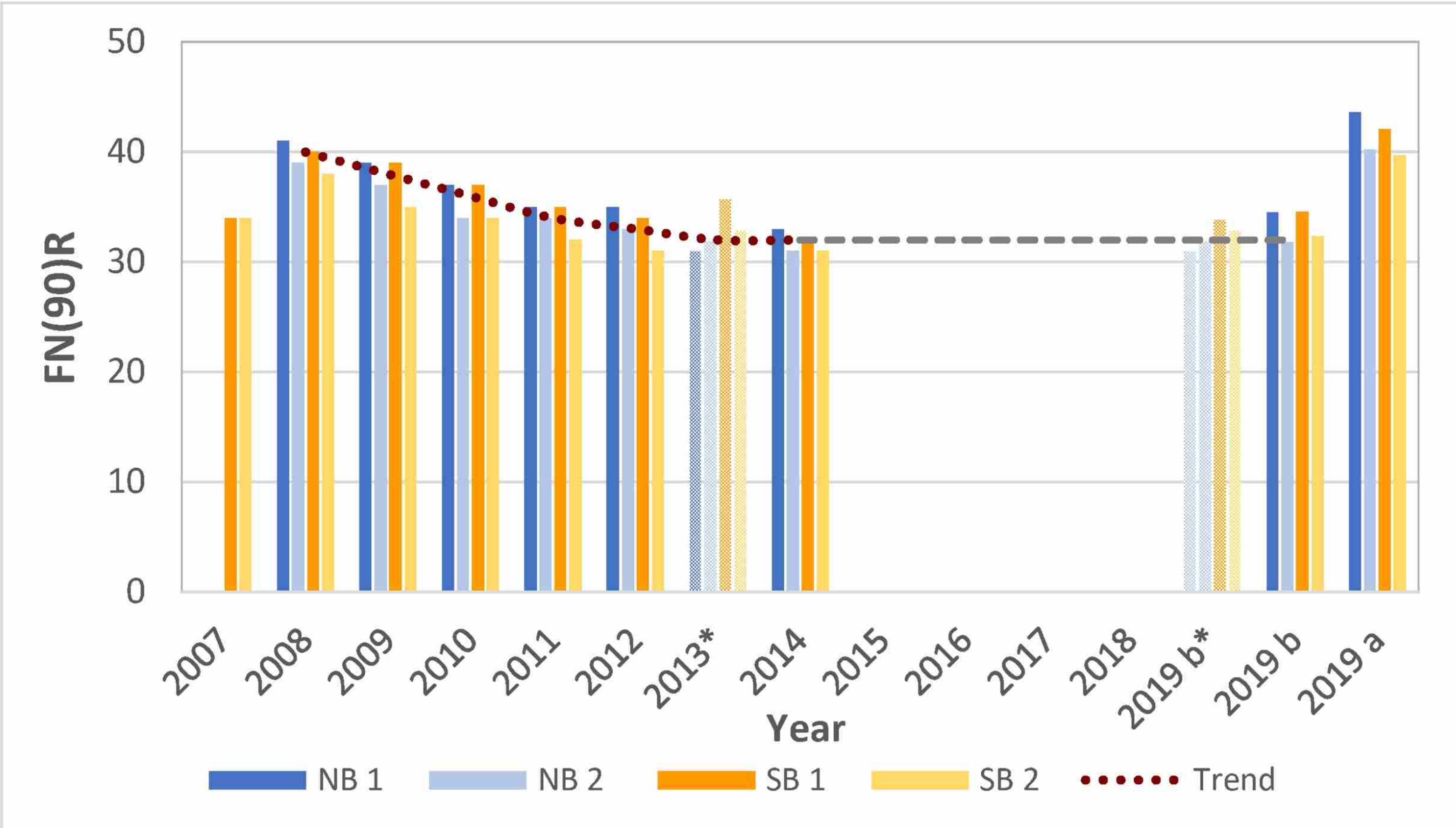

Figure 10. GripTester friction measurements overlapped with the Locked-wheel tester data.

Note: The GripTester results converted to FN(90) are the more lightly shaded bars shown in 2013 (Tradewind) and 2019 before resurfacing (Englobe)

Other standards, such as the one for the British Pendulum testing (ASTM E303-93) and locked-wheel friction testers (ASTM E274-15), do not recommend a temperature range, but indicate that the measurement temperature be reported with the results. In addition, ASTM E274-15 provides a range of ambient temperature for verifying the requirements for the equipment instrumentation, 4°C to 40°C.

I would personally recommend that friction testing generally be conducted with pavement temperatures range between 5°C to 50°C, as recommended by AASHTO TP 143-21. It is important to note that the pavement temperature is generally higher than the air temperature, particularly during the daytime hours.

Furthermore, no measurements should be taken for temperatures below 0°C, as the water may freeze and make the results invalid.

Although, tire-pavement friction generally decreases as tire temperature increases, I did not make any temperature adjustments to the friction measurements above described in this report. This is because there are no established standards to do this accurately. As the results described above are generally confirmatory of one another, I am comfortable not creating additional uncertainty by making temperature adjustments of questionable accuracy.

2.1.4 British Pendulum Testing

Golder Associates conducted British Pendulum testing in accordance with ASTM E303 in several locations along the RHVP in December 2017 and the results were reported in a report from Golder dated February 28, 2019.[13] The results are summarized in Figure 11.

The Figure shows that the British Pendulum results are very variable and show several very low values.

The Golder February 28, 2019 report notes that testing was carried out at night when the temperature was below 0°C and there was light snow fall; therefore, the results were not considered reliable by Golder.

As noted in the Primer: the British Pendulum test standard (ASTM E303-93) does not recommend a temperature range, but indicates that the measurement temperature be reported with the results.

Although the ASTM standard does not establish a range of temperatures for testing, I agree with Golder's assessment that the results are unreliable as the measurements should not have been taken at below freezing temperatures, as the water may freeze.

[13] Evaluation of Pavement Surface and Aggregates – Red Hill Valley Parkway, Golder Associates Ltd., February 28, 2019 (HAM0029042_0001 at images 1-3, 6).

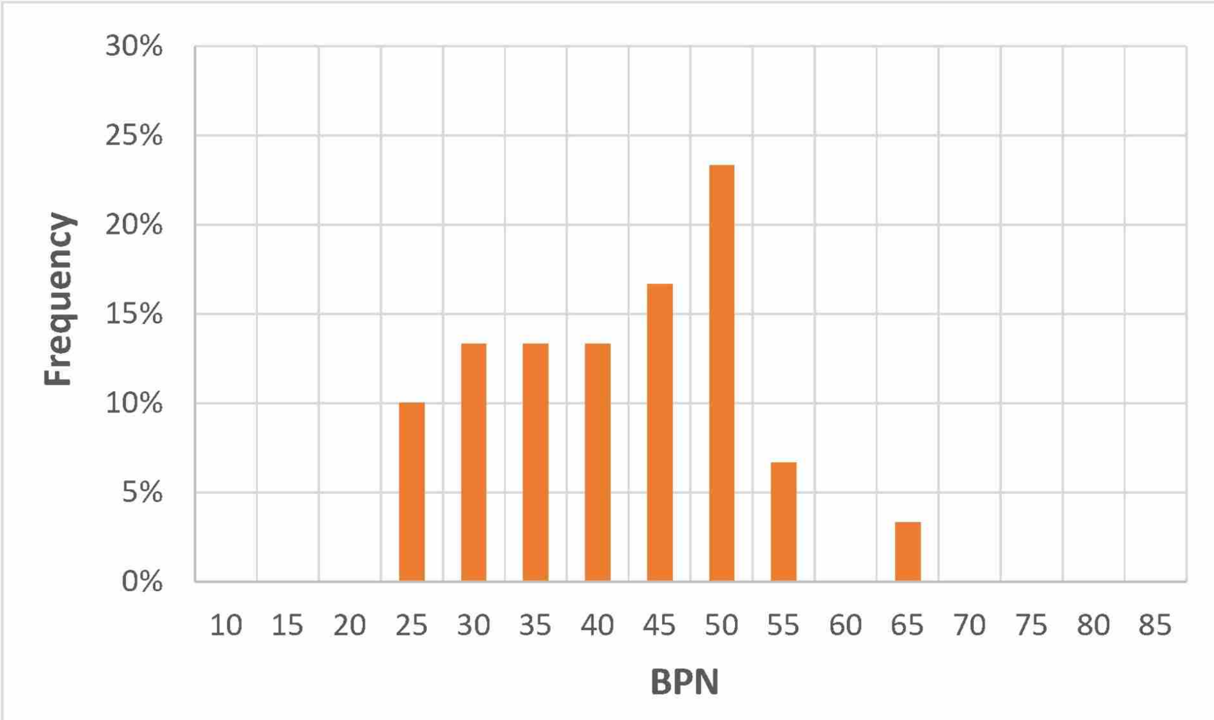

Figure 11. Summary of BPN tests conducted by Golder in December 2017.

2.1.5 Polished Stone Value (PSV)

As set out in the Primer: The coarse aggregates in the surface of the pavement are in contact with the tire and thus are subject to the adhesion forces that contribute to the friction and grip needed to safely operate vehicles. These adhesion forces generated between the rubber and aggregates abrade or polish the aggregate particles by eliminating some of the asperities. This lowers the microtexture and produces a reduction in friction over time. Some aggregates have better resistance to polishing than others. Therefore, aggregate polishing characteristics are important to maintain long-term friction. The polished stone value (PSV) of coarse aggregate is often used to measure the ability of coarse aggregate to resist the polishing action of tires. The PSV is used to characterize the ability of coarse aggregate to maintain a certain coefficient of friction even after tire abrasion.

To determine the Polished Stone Value (PSV), aggregate coupons (aggregates embedded in epoxy resin) are fabricated, subjected to accelerated polishing (using the British polish wheel) for a specified time (usually 9 hrs.), and then tested for frictional resistance using the British Pendulum Tester. The British pendulum number (BPN) value associated with accelerated polishing is defined as the polished stone value (PSV). This number is a quantitative representation of the aggregate's terminal frictional characteristics. Higher values of PSV indicate greater resistance to polish (AASHTO 2008)[14].

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[14] ASHTO (2008). *Guide for Pavement Friction, American*, American Association of State Highway and Transportation Officers, Washington, DC.

Aggregates recovered from asphalt cores taken from the field by Golder in December 2017 were sent to James Fisher Testing Services (Ireland) Ltd to determine the aggregate PSV in accordance with BS EN 1097-8.[15] The aggregates tested were of course placed in 2007 when the RHVP was initially paved.

The PSV of 45 obtained is considered relatively low compared to the British standards[16] as well as the MTO requirement of a minimum average PSV no less than 50, with no value less than 48) required for inclusion on the MTO's Designated Source for Materials List.[17] This indicates that the aggregate is susceptible to polishing. This result from the in-service RHVP pavement in December 2017 (PSV=45) is consistent with the results the MTO obtained from 1992 (reported by the MTO in December 2007), but lower than the value (PSV=52) the MTO reported for the same aggregate source obtained from the quarry in 2008.[18] The variation of PSV over time for a quarry is not uncommon as different rock seams are exploited over time. The relatively low PSV of 45 obtained from the samples taken in December 2017 is consistent with the significant drop in friction (approximately 20%) observed between 2008 and 2014 described above. An aggregate susceptible to polishing losses its macrotexture because of the abrasive effect of traffic, and it contributes to a decrease in friction as observed in the RHVP and discussed in the previous sections.

2.1.6 Macrotexture

As stated in the Primer: Macrotexture represents surface irregularities, with amplitudes ranging between approximately 0.1 and 20 mm. As water film thickness increases, the pavement's macrotexture provides water drainage paths beneath the tire, reducing hydroplaning potential and allowing for greater tire/pavement adhesion (a function of the pavement's microtexture). Macrotexture also provides friction through hysteresis (energy loss due to asymmetrical deformation of the tire). The hysteresis effect exponentially increases with increasing vehicle speed, so it is critical to providing good friction at high speeds. While the microtexture is primarily affected by the type of aggregate used, mostly its surface asperities and polishing characteristics, macrotexture is the result of the type and properties of the asphalt mixture used in the surface of asphalt pavements and the type of texturizing used in concrete pavements.

Macrotexture can be measured using both highway speed profilers and static methods. The oldest method is the volumetric patch test. In this test, a known volume of sand, glass beads, or grease is spread evenly into a circular patch on the road surface (where sand is used, it is commonly called a "sand patch test"). The area is measured, and the average depth below the peaks in the surface is calculated to give a value known as mean texture depth (MTD).

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[15] Evaluation of Pavement Surface and Aggregates – Red Hill Valley Parkway, Golder Associates Ltd., February 28, 2019 (HAM0029042_0001 at images 1-2, 6).

[16] CD 236 Surface course materials for construction

[17] December 13, 2007 MTO Letter to Demix Agrégats (MTO0000042) attaching MTO Laboratory Test Data, Demix Agrégats (Varennes, Quebec), 1992 (MTO0000043)

[18] December 13, 2007 MTO Letter to Demix Agrégats (MTO0000042) attaching MTO Laboratory Test Data, Demix Agrégats (Varennes, Quebec), 1992 (MTO0000043); December 4, 2008 MTO Letter to Demix Agrégats (MTO0000044) attaching MTO Laboratory Test Data, Demix Agrégats (Varennes, Quebec), July 2008 (MTO0000045).

On wet pavements, as the vehicle speed increases, skid resistance decreases to an extent that depends on the macrotexture (Figure 4). Generally, surfaces with greater macrotexture have greater friction at high speeds for the same low-speed friction (Roe and Sinhal 1998), but this is not always the case.

Macrotexture measurements were conducted using the sand patch method in accordance with ASTM E965 in December 2017 by Golder and May 2019 by ARA pre-resurfacing. These are summarized in Figure 12.

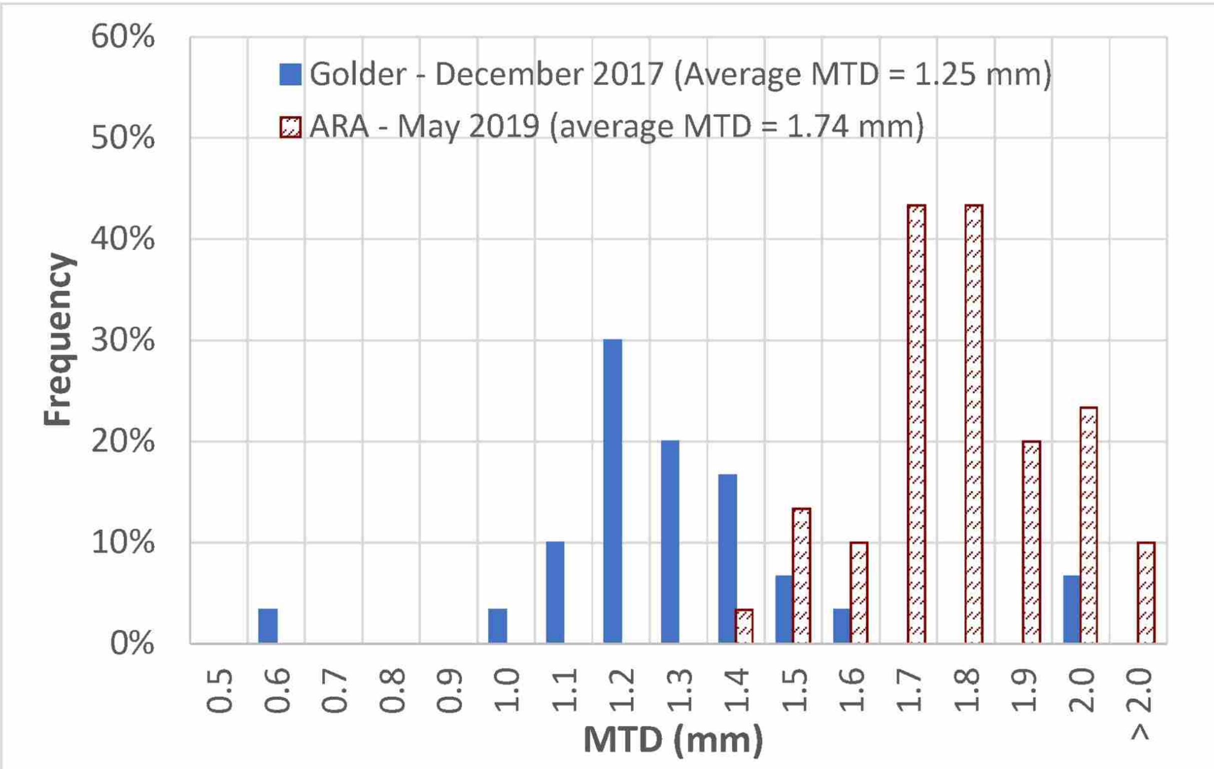

Figure 12. Macrotexture measurements in 2017 by Golder and in May 2019 by ARA.

These macrotexture measurements are acceptable and can be described as follows.

The Golder December 2017 measurements were taken after 10 years of service and showed an average mean texture depth (MTD) value of 1.25 mm and a standard deviation of 0.26 mm. Only two locations in one lane (NB lane 2) had relatively low values (MTD of 0.57 mm and 0.91 mm). I agree with Golder's statement that a pavement with good macro-texture should have a texture depth of about 1.0 mm.[19] These values are also considered appropriate according to the standards from other countries, such as the one from New Zealand presented in the Primer (page 22) and reproduced below in Table 2.

[19] Evaluation of Pavement Surface and Aggregates – Red Hill Valley Parkway, Golder Associates Ltd., February 28, 2019 (HAM0029042_0001 at images 1-2, 5).

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Table 2. New Zealand Minimum Macrotexture Requirements (NZTA 2013)[20]

Legal Speed Limit	Minimum Macrotexture MPD (mm)					
	Chipseals		Asphaltic Concrete ESC ≥ 0.4		Asphaltic Concrete ESC < 0.4	
	ILM	TLM	ILM	TLM	ILM	TLM
50 km/hr and less	1.0	0.7	0.4	0.3	0.5	0.5
Less than or equal to 70 km/hr but >50 km/hr	1.0	0.7	0.4	0.3	0.7	0.5
Greater than 70 km/hr	1.0	0.7	0.9	0.7	0.9	0.7

Note: ILM = investigatory level macrotexture; TLM = threshold level macrotexture; ESC = Seasonally adjusted sideforce friction coefficient (measured with the SCRIM)

The ARA May 2019 Measurements were taken after about 11.5 years of service and showed significantly higher MTD values than those taken by Golder in 2017, with a an average MTD value of 1.74 mm and a standard deviation of 0.16 mm.[21]

The increase in macrotexture values between the Golder and ARA tests may be attributed to pavement surface deterioration. However, in either case, the macrotexture values are considered appropriate and similar to those observed in other SMA surfaces. Thus, I am not of the view that macrotexture was a contributing factor to the high number of collision and the high percentage of collisions on wet surfaces.

3 Laboratory and Production Results

The friction and macrotexture of hot mix asphalt may be influenced by the mix design and production, including mix placement and compaction.

I have reviewed the SMA 12.5 mix design prepared by Trow Associates Inc for Dufferin.[22] I have also reviewed the various SMA test results submitted for approval, the quality control and quality assurance test results, as well as the production records for the SMA 12.5 mix placed on the RHVP and Overview Document 3.[23] I have also reviewed the affidavit of Ludomir Uzarowski affirmed September 30, 2022 respecting approval of the aggregate used for the SMA.[24]

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[20] NZTA (2013). "Specification for State Highway Skid Resistance Management." *T10 Specification*, New Zealand Transport Agency (NZTA).
[21] Red Hill Valley Parkway – Surface Pavement Investigation Methodology Report, ARA, September 11, 2019 (HAM0009630_0001); Sand Patch Data, Red Hill Valley Parkway, ARA, May/June/July 2019 [Appendix C to Methodology Report] (HAM0009627_0001); see also Red Hill Valley Parkway – Surface Pavement Investigation Methodology Report, ARA, November 15, 2019 (HAM0009637_0001).
[22] GOL0001631
[23] List of documents reviewed to follow
[24] RHV0001024

From that review I have the following general observations:

- The mix design was consistent with current mix design practices for SMA, based on my experience.
- Although the records indicate some departures from the mix design values, none of them would be expected to have a significant negative impact on the frictional properties of the pavement surface.
- While the low compaction observed in asphalt nuclear density test results for the mix placed in early August 2007 in some of the sections could have a negative impact on durability, in my view the low compaction would not have contributed to low friction. Nor, in my view, would cracking or breaking of the aggregates due to over-compaction contribute to low friction.

4 Additional Discussion

4.1.1 CIMA

The 2013 Red Hill Valley Parkway Safety Review by CIMA reviewed the safety performance of a portion of the RHVP between Dartnall Road and Greenhill Roads and recommended viable potential measures that could be implemented to increase the safety performance and/or drivers' sense of security.[25] The report identified friction testing, installing 'slippery when wet' signs and enforcement of travel speed as possible overall countermeasures, as well as installing high-friction surfaces (HFS) on some of the ramps.

The 2015 Red Hill Valley Parkway Detailed Safety Analysis by CIMA analyzed the safety of the entire Parkway and found that 50% of the collisions occurred on wet surfaces, suggesting friction problems.[26] In particular, the NB mainline in the segment including the King Street interchange showed a very high percentage of wet collisions. The report also recommended friction testing as one of the countermeasures that should be considered. Additional possible improvements recommended included conducting speed enforcement (e.g., speed feedback signs) and installing 'slippery when wet' signs.

The January 2019 CIMA Roadside Safety Assessment reported an even higher percentage of collisions occurring on wet surface (64% of the mainline collisions that included road surface condition information), and that its findings "suggest that inadequate skid resistance (surface polishing, bleeding, contamination) and excessive speeds may be contributing factors to collisions". It further noted that the mainline collisions involving wet surface conditions "present extremely high proportions between Greenhill Avenue and King Street, and between King Street and Queenston Road (up to 88%)" and that "In combination with potential skid resistance and excessive speed issues, curve radii compatible with a design speed of 100km/h around the King

[25] Red Hill Valley Parkway Safety Review, CIMA, October 2013 (HAM0041871_0001).
[26] Red Hill Valley Parkway Detailed Safety Analysis, CIMA, November 2015 (HAM0056684_0001).

Street interchange may explain this concentration of collisions (operational speed may exceed the design speed)".[27] This report recommended, among other countermeasures, to "ensure the pavement design for the upcoming resurfacing considers the history of wet surface collisions and investigates the need for higher friction surface".[28]

In a Legal Opinion dated February 4. 2019, David G. Boghosian, LL.M evaluated a series of documents and reported on a telephone conversation with Brian Malone of CIMA on December 11, 2018. Mr. Boghosian wrote:[29]

> *When asked to rank, in order of greatest contribution, to the inordinate number of wet road crashes, Mr. Malone advised as follows:*
>
> - *Slipperiness of the road surface (i.e., the road is slipperier when wet than other roads which leads to greater accidents then on roads with similar large numbers of horizontal curves in wet road conditions);*
> - *Speeds exceeding the capability of the highway given the curvature of the road;*
> - *Curves in the road (there are a number of sharp curves having design speeds of 100 km/hr, whereas a high proportion of vehicles are substantially exceeding that speed);*
> - *The close proximity of on/off-ramps to each other leading to losses of control and/or drivers' errors as traffic attempts to merge onto the highway or cut across lanes to get off the highway.*

I have reviewed the portion of the rough transcript of Mr. Malone's testimony at the Inquiry on October 31, 2022 in which he testified regarding the ranking in Mr. Boghosian's memo, and that he felt the bullet points were all contributing factors, but interrelated, and that he did not think he would rank them.

I agree that the proportion of RHVP collisions that occurred on a wet surface was high. I also agree that all of the listed factors, including slipperiness of the road surface (low friction) probably contributed to this unusually high percentage of wet road collisions. However, I do not have enough scientific evidence to comment on the order of greater contribution attributed to Mr. Malone in the memo.

Mr. Boghosian also stated in his February 4, 2019 letter that Mr. Malone also indicated that:

> *Because of the large aggregates, however, SMA holds much more water on the road that does not drain away than conventional asphalt because the water sits in pockets between the large aggregates, creating "micro-ponds". He speculates this is the reason for the high number of accidents on the RHVP in combination with the high number of curves and excessive speeds at which the highway is driven.*

I believe that Mr. Malone was referring to the pavement macrotexture. I do not agree that the larger aggregate on the surface contributed to the high number of collisions in wet weather. In

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[27] Roadside Safety Assessment, CIMA, January 2019 (HAM0054495_0001 at image 23)
[28] Roadside Safety Assessment, CIMA, January 2019 (HAM0054495_0001 at image 40)
[29] Hamilton re: Red Hill Valley Parkway Legal Opinion, David G. Boghosian, February 4, 2019 (HAM0064331_0001 at images 8-9).

general, for high-speed freeways higher levels of macrotexture have been associated with lower number of collisions (AASHTO 2008; Flintsch et al., 2012[30]). The voids provided by the valleys between the coarse aggregates provide water drainage paths beneath the tire, reducing hydroplaning potential and allowing for greater tire/pavement adhesion (a function of the pavement's microtexture).

The Report by CIMA, review of Red Hill Valley Parkway Friction Test results, from May 2020 reviewed the results of the MTO testing from 2008-2014, the 2013 Tradewind testing, and the ARA 2019 friction measurements taken after the resurfacing that year. However, this May 2020 CIMA report does not reference the Englobe GripTester results from 2019 (taken before resurfacing), nor does it reference the ARA pre-resurfacing Locked-wheel testing that year.[31] This report highlights the 20% reduction in friction from 2008 to 2014 mentioned in section 2.1.1. The report also recommends to continue to monitor the 'new" pavement. I agree with this recommendation.

4.1.2 Golder

Golder's January 2014 report titled "Performance Review after Six Years in Service" which attached the Tradewind Report contained recommendations for dealing with longitudinal top down cracking in certain areas of the RHVP by resurfacing, and on the remaining portion to rout and seal cracks followed by applying a single layer of microsurfacing. The report stated that "by carrying out the mill and overlay where required and applying microsurfacing, the issue of relatively low FN on the RHVP would also be addressed."[32]

While I cannot opine on the cracking issue, I agree with Golder that the combination of resurfacing in some areas and microsurfacing on the rest of the RHVP (done properly – see my comments on microsurfacing in the Primer at p.32), would have addressed the low friction issue at that time.

A letter report from Golder on February 28, 2019 reviewed the data it had collected in 2017 and suggested that "an immediate, effective treatment to address a concern with frictional characteristics of the SMA surface course on the RHVP would be to carry out shot blasting/skidabrading of areas of concern on the existing pavement surface."[33] I agree with that shot blasting can be a good short term solution to address low friction. However, in my view resurfacing (which was done several months later) was probably a better and longer-term solution.

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[30] Flintsch, G., McGhee, K., de León Izeppi, E., and Najafi, S. (2012). *The Little Book of Tire Pavement Friction, Version 1.0.*
[31] Review of Red Hill Valley Parkway Friction Test Results, CIMA, May 2020 (CIM0022320).
[32] Red Hill Valley Parkway Performance Review after Six Years in Service, Golder, January 2014 (GOL0002981 at image 11).
[33] Evaluation of Pavement Surface and Aggregates – Red Hill Valley Parkway, Golder Associates Ltd., February 28, 2019 (HAM0029042_0001 at image 3).

4.1.3 *City of Hamilton Annual Collision Reports 2017-2021*

I have reviewed the City of Hamilton's Annual Collision Reports for 2017[34], 2018[35], 2019 [36], 2020[37], and 2021[38].

The Annual Collision Reports show a very high percentage of collisions occurred when the pavement surface was wet, and of single motor vehicle collisions. This is considered an indication that pavement surface friction is not adequate to attend the demand of the vehicles traveling on that road segment. However, those types of collision have declined in recent years, following the resurfacing of the RHVP in mid-2019 and other countermeasures. I have reviewed the report of Russell Brownlee prepared for the Inquiry and I agree with and adopt his observations and conclusions about the Annual Collision Reports.

5 General Observations and Conclusions

This report reviewed all the available evidence regarding the frictional properties of the RHVP since its construction in 2007 to its resurfacing in 2019, for the RHVPI.

Several sets of friction measurements on the RHVP have been conducted over the years using different technologies and equipment. These have included measurements with Locked-wheel testers (ASTM E274-15), with a GripTester (ASTM E2340-11) and British Pendulum tests (ASTM E303). In addition extracted aggregate samples were tested to determine its Polished Stone Value and macrotexture measurements taken according to Sand Patch test (ASTM E965).

Summarizing all the information reviewed in the previous sections, the following observations can be drawn:

The Tradewind report and the MTO measurements showed a significant decrease of friction in the first six to seven years of service. This suggests that some level of wear, including aggregate polishing (reflecting a decline in microtexture), which, in my view, is consistent with the relatively low PSV value resulting from testing aggregate recovered from asphalt cores extracted in 2017. This is further supported by the satisfactory macrotexture measurements obtained by Golder in 2017 and ARA in 2019.

The subsequent measurements and reports by ARA and Englobe shortly before resurfacing in 2019, showed similar levels of friction along the RHVP and indicated that friction levels stabilized after the initial reduction.

Although there are no published investigatory level friction standards in Canada, according to international standards, both the Tradewind report and the MTO measurements suggest that after about six to seven years of service the RHVP had levels of friction that should have warranted a

[34] City of Hamilton 2017 Annual Collision Report (HAM0013587_0001).
[35] City of Hamilton 2018 Annual Collision Report (RHV0000597).
[36] City of Hamilton 2019 Annual Collision Report (RHV0000609).
[37] City of Hamilton 2020 Annual Collision Report (RHV0000908).
[38] City of Hamilton 2021 Annual Collision Report (RHV0001001).

more detailed safety investigation to determine whether friction was a contributing factor to collisions along with other relevant factors for investigation, at least for the sections with the lowest friction values. The Tradewind report showed GripTester Numbers below the investigatory levels used in the U.K. The 2014 MTO measurements showed, in addition to the approximately 20% reduction in FN(90)R overall since 2008, several locations with a FN(90)R lower than 30, which I understand is a value used by the MTO as a guideline for identifying road sections that may need a safety investigation, at least where warranted by the collision history.

The various safety analysis and collision reports show a consistently high percentage of collisions on wet surface conditions (prior to resurfacing), which is an indication that the friction demand exceeded the friction supplied by the pavement when the surface was wet.

In conclusion, it is my view that that the very high percentage of collisions during wet conditions combined with the friction test results in the Tradewind report, as well as the MTO measurements, was an indication that the relatively low friction contributed to those collisions, together with excessive speeds and the geometry of the freeway which give rise to elevated friction demand and thus collectively supported the previously stated need for a detailed safety analysis that could have resulted in a decision to apply a treatment to improve the frictional properties of the pavement surface, such as resurfacing or microsurfacing.

APPENDIX A

PRIMER ON FRICTION, FRICTION MANAGEMENT, AND STONE MATRIX ASPHALT MIXTURES

PRIMER ON FRICTION, FRICTION MANAGEMENT, AND STONE MATRIX ASPHALT MIXTURES

Prepared by

Gerardo W. Flintsch, P.E., Ph.D.

For the

RED HILL VALLEY PARKWAY INQUIRY

April 2022

FM Consultants

Table of Contents

List of Acronyms

AASHTO: American Association of State Highway Transportation Officials

ABS: anti-lock braking systems

ASTM: ASTM International, formerly the American Society for Testing and Materials

BPN: British pendulum number

CFME: continuous friction measuring equipment

CSC: characteristic SCRIM coefficient

DMRB: Design Manual for Road and Bridges [U.K. Standards for Highways]

ESC: equilibrium SCRIM coefficient

FHWA: Federal Highway Administration

FN: friction number [measured with a locked-wheel tester]

GN: grip number

HFST: High Friction Surface Treatment

IL: investigatory level [for friction]

ILM: investigatory level for macrotexture

MPD: mean profile depth

MTD: mean texture depth

NCAT: National Center for Asphalt Technology

NZTA: New Zealand Transport Agency

PIARC: Permanent International Association of Road Congresses

PFM: pavement friction management

PSV: polished stone value

SBS: styrene-butadiene-styrene

SCRIM: Sideway-force Coefficient Routine Investigation Machine

SFC: sideway force coefficient

SFN: sideway force number

SR: SCRIM Reading

SMA: stone-matrix asphalt

SN: skid number [measured with a locked-wheel tester]

Superpave: Superior Performing Asphalt Pavements

TLM: threshold level for macrotexture

1 Introduction to Pavement Friction

The frictional properties of pavements play a significant role in road safety, as the friction between tire and pavement is a critical factor in reducing potential crashes. When a tire free rolls in a straight line, the contact patch is instantaneously stationary with little to no friction developed at the tire/road interface, although there are some interactions that contribute to rolling resistance. However, when a driver begins to execute a maneuver that involves a change of speed or direction, forces develop at the interface in response to acceleration, braking, and/or steering that cause a friction reaction between the tire and the road. Friction enables the vehicle to speed up, slow down, or track around a curve (Flintsch et al. 2012). The reaction forces are limited by the dynamic friction available.

1.1 Definition of Pavement Friction

According to the American Association of State Highway Transportation Officials (AASHTO) *Guide for Pavement Friction*, "Pavement friction is the force that resists the relative motion between a vehicle tire and a pavement surface" (AASHTO 2008). This *Guide* was developed under the National Cooperative Highway Research Program (NCHRP) project 1-43 and the final report of this project (Hall et al, 2009) contains the *Guide,* as well as additional technical details and background not included in the document published by AASHTO. The friction force between tire and pavement is generally characterized by a dimensionless coefficient, known as the coefficient of friction (μ), which is the ratio of the tangential force at the contact interface to the longitudinal force on the wheel.

1.1.1 The Physics Behind Friction

The friction that can develop between a vehicle's tires and the pavement is the result of the interaction between the tire, the pavement, and the conditions on the road surface, so it is not a property of the tire or the road surface individually. Tire-pavement friction depends also on the amount of water and other contaminants present between the tire and the pavement, the vehicle's maneuver, and the environmental conditions.

In terms of physics, tire pavement friction is the result of two main forces: adhesion and hysteresis (Figure 1). Adhesion is the molecular bonding between the tire and the pavement surface, while hysteresis is the energy loss due to tire deformation. In addition to contributing to friction, the bonding is responsible for tire wear as increased forces from vehicle braking or maneuvering tears the rubber.

Hysteresis forms as the tire touches the pavement and the pavement surface texture causes deformation in the tire rubber. This deformation stores potential energy in the tire. As the tire relaxes, part of this energy is recovered and another part is dissipated in the form of heat. This generated heat (energy loss) is known as hysteresis. Both hysteresis and adhesion are related to surface characteristics and tire properties (AASHTO 2008), as explained in the following section.

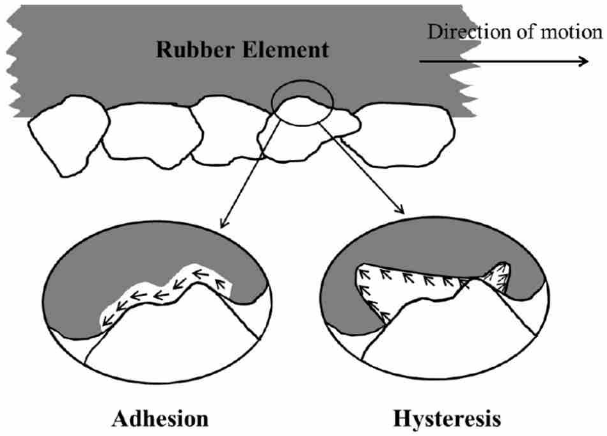

Figure 1. Main components of tire-pavement friction (after AASHTO 2008)

1.1.2 The Contribution of the Pavement to Tire-Pavement Friction

The properties or characteristics of the pavement surface that affect friction are defined by the texture in the surface. Pavement texture is defined as "the deviations of the pavement surface from a true planar surface" (AASHTO 2008). These deviations vary from microscopic asperities on the aggregate surface, to valleys and crests in between the aggregates that form the surface of the pavement, to bumps in the road that affect the vehicle dynamics and driver comfort (referred to as *roughness* or *smoothness* in the highway industry).

There are two main components of the texture spectrum that affect tire-pavement friction: microtexture and macrotexture (Wambold 1995). These are illustrated in Figure 2 and described as follows.

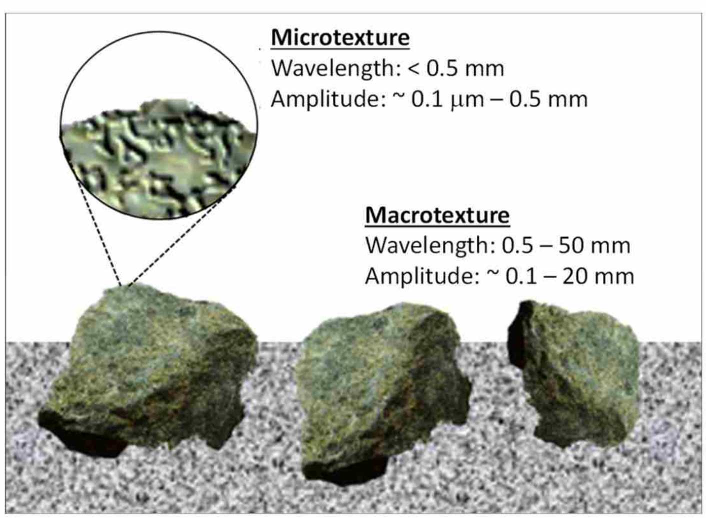

Figure 2. Texture Properties that Influence Skid Resistance

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1. *Microtexture* is the fine-scale texture, with amplitude lower than about 0.5 mm, on the surface of the coarse aggregate in asphalt or the sand in concrete pavements that interacts directly with the tire rubber on a molecular scale and provides adhesion. This property is important to provide adequate friction on both wet and dry roads. It needs to be present at any speed but is especially important at lower speeds.
2. *Macrotexture* represents slightly bigger surface irregularities, with amplitudes ranging between approximately 0.1 and 20 mm. As water film thickness increases, the pavement's macrotexture provides water drainage paths beneath the tire, reducing hydroplaning potential and allowing for greater tire/pavement adhesion (a function of the pavement's microtexture). Macrotexture also provides friction through hysteresis (energy loss due to asymmetrical deformation of the tire). The hysteresis effect exponentially increases with increasing vehicle speed, so it is critical to providing good friction at high speeds.

While the microtexture is primarily affected by the type of aggregate used, mostly its surface asperities and polishing characteristics, macrotexture is the result of the type and properties of the asphalt mixture used in the surface of asphalt pavements and the type of texturizing used in concrete pavements.

The coarse aggregates in the surface of the pavement (which provide the microtexture as shown in Figure 2) are in contact with the tire and thus, are subject to the adhesion forces that contribute to the friction and grip needed to safely operate vehicles. These adhesion forces generated between the rubber and aggregates abrades or polishes the aggregate particles by eliminating some of the asperities. This lowers the microtexture and produces a reduction in friction over time. Some aggregates have better resistance to polishing than others. Therefore, aggregate polishing characteristics are important to maintain long-term friction. The polished stone value (PSV) of coarse aggregate (discussed further in section 1.2.5) is often used to measure the ability of coarse aggregate to resist the polishing action of tires. The PSV is used to characterize the ability of coarse aggregate to maintain a certain coefficient of friction even after tire abrasion.

1.1.3 Friction During Braking

The dynamic coefficient of friction varies with the relative degree of *slipping* of the tire with respect to the pavement surface. During braking along a straight section of road, as the braking force increases, the reacting force increases until it approaches a point at which the peak coefficient of friction available between the tire and the road is exceeded (this normally occurs between 18 percent and 30 percent slip). At this point (commonly known as *peak friction*), the tire continues to slow down relative to the vehicle speed and to slip over the road surface, even though the wheel is still rotating. If the braking force continues, the tire slips even more. Eventually complete locking of the wheel occurs, at which time the wheel stops rotating and the tire contact patch skids over the road surface. Figure 3 illustrates this phenomenon.

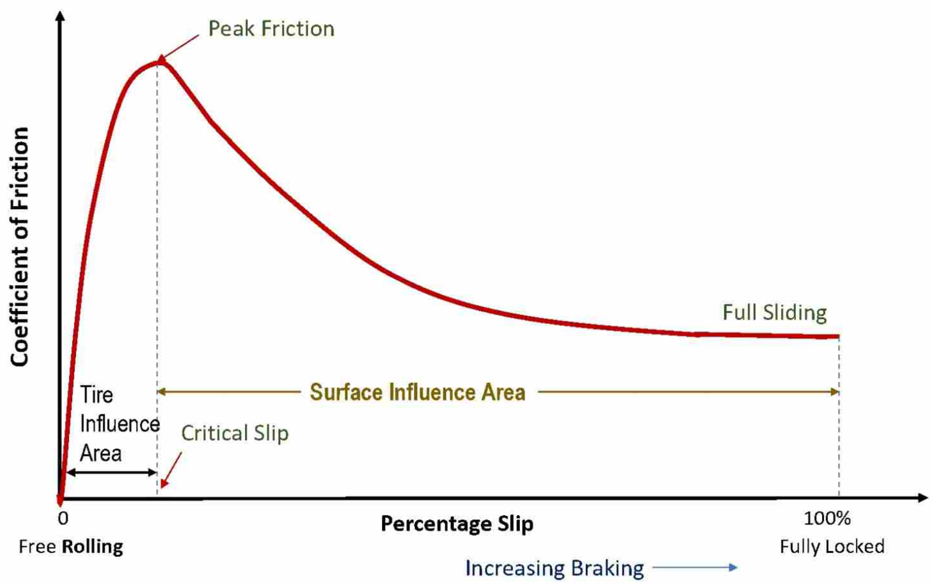

Figure 3. Friction Versus Slip (not to scale; after Henry, 2000)

On a dry road surface, the difference between peak and sliding friction is small and speed has relatively little effect. This is illustrated by the blue dotted line in Figure 4. On a wet road, however, peak friction is often lower than in dry conditions, the sliding friction is typically lower than peak friction, and both usually decrease with increasing speed.

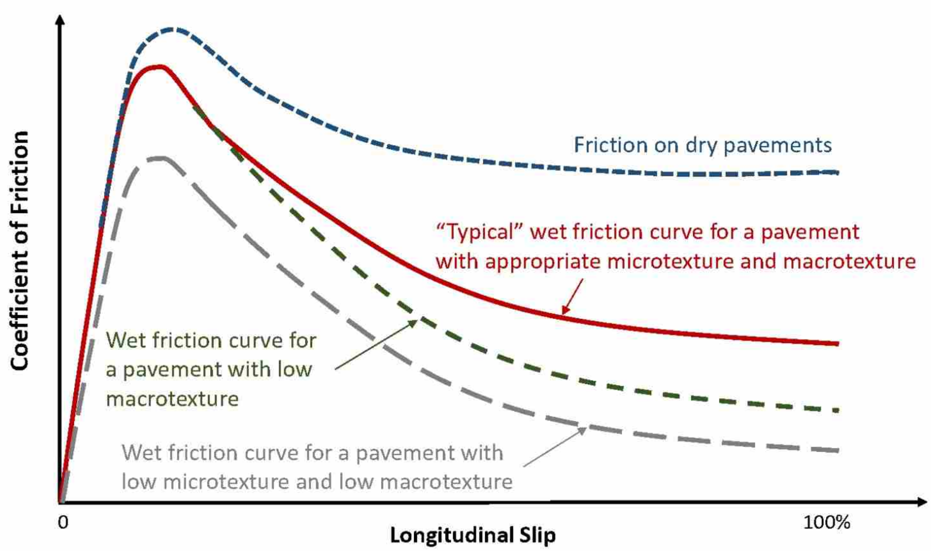

Figure 4. Illustration of the Effect of Microtexture and Macrotexture on the Coefficient of Friction Available at Different Percentage of Slip (not to scale)

The differences between wet and dry conditions, as well as peak and sliding friction, depend not only on vehicle speed and tire properties, but also to a large extent on the characteristics of the road surface and the amount of water and other contaminants on the pavement. The importance of these factors is discussed further in section 1.2.4.

Except for the top blue dotted line, all illustrative curves in Figure 4 represent friction on a wet pavement. Under these conditions, macrotexture is the main property that affects how fast the friction decreases with speed. In pavement with low macrotexture, the right side of the coefficient of friction curve is steeper and the wet coefficient of friction decreases greatly with increasing speed, as shown by the green dotted line. For this reason, roadways with high posted speeds need pavements with high macrotexure to reduce the rate at which friction decreases as speed increases on wet pavement.

1.1.4 Friction While Cornering

Similarly, when the vehicle needs to maneuver a curve, cornering generates transversal forces at the tire pavement interface that allow the vehicle to follow the curved path. If the combination of forward speed and the effective radius of curvature of the maneuver, influenced by the geometry of the road and steering angle, result in a demand, or need, for friction that exceeds what the road can provide, the wheel may slip sideways, causing the vehicle to yaw (friction demand is discussed in part 2). In this situation, a marked difference between peak and sliding friction could lead to a rapid loss of control.

1.1.5 Simultaneous Cornering and Braking

The situation is exacerbated when braking and cornering occur simultaneously, because the available friction has to be shared between the two mechanisms. The available friction has to provide enough forces for the vehicle to deaccelerate and to maintain the path along the curve. If the combination of cornering and braking exceeds the critical slip (corresponding to the peak friction; see Figure 3 and Figure 4), the available total friction will decrease and the operator may lose control of steering.

This is why anti-lock braking systems (ABS) are important. These systems detect the onset of wheel slip and momentarily release and then re-apply the brakes to make sure the critical slip is not exceeded. This reduces the likelihood of side-slip occurring and helps the driver to maintain control. Some modern vehicle control systems use similar approaches to reduce the risk of side-slip under simultaneous acceleration and cornering.

1.2 Measuring Friction

Because friction depends on the interaction between the tire and the pavement, different measurements are obtained for different testing conditions, such as wet and dry pavement, hot and cold weather, type and condition of the tire, and so on. This variety of measurements has led to the development of different testing devices that operate under different conditions. Friction

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testing equipment used in the highway industry measures wet friction after spreading a small amount of the water on the pavement. However, the various friction-measuring technologies available use different types of tires, water film thicknesses, and operating principles, so they do not produce a common, standardized measurement of friction.

Furthermore, as previously discussed, the level of friction available also depends on the speed at which the tire is slipping with respect to the pavement surface. When a tire is free-rolling on dry pavement, there is virtually no slip. However, as the driver starts to brake or navigate a curve, the tire starts to slip with respect to the pavement, up to the point where the tire is locked—not rotating—and the rubber on the contact patch is slipping at a speed equal to the vehicle speed.

1.2.1 Types of Friction Measuring Equipment

Many different devices have been developed over the years to measure pavement friction. They all rely on the broad principle of sliding rubber over a wet road surface and measuring the reaction forces developed. These forces are used to compute the coefficient of friction discussed previously and, in some cases, this number is multiplied by 100 to compute what standards call Friction Number (FN), Skid Number (SN), or Grip Number (GN). Figure 5 shows some of the most commonly used friction measuring equipment for roadways.

(a) Locked-Wheel Friction Tester

(b) GripTester

(c) SCRIM

Figure 5. Examples of Friction Testers

There are several general measuring principles:

i. *Sliders*, attached either to the foot of a pendulum arm or to a rotating head, which slow down on contact with the road surface. The rate of deceleration is used to derive a value representing the skid resistance of the road. A variant of this approach, still used by police forces in some parts of the world, is to measure the reaction force when a sled (with sliders representing car tires) is dragged over the road surface. The most commonly used device in this category is the British Pendulum Test (ASTM E303-93)

ii. *Longitudinal friction coefficient* measurement equipment uses an instrumented measuring wheel mounted in line with the direction of travel. A fixed gear, or braking system, forces the test wheel to rotate more slowly than the forward speed of the vehicle. Consequently, the tire contact patch slips over the road surface and a frictional force is developed that can be measured. Typically, the ratio of drag to vertical forces is calculated (averaged over a fixed measuring length) to provide the recorded value representing the friction. Individual devices in this category use a wide range of slip ratios. Examples of these types of devices include:

    a. Fixed-slip friction testers (e.g., GripTester); and

    b. Locked-wheel friction testers, which completely lock the brake of the measuring wheel and produce 100% slip (e.g. ASTM E274-15 standard skid tester). Locked-wheel testers can either use ribbed or smooth tires. Measurements using ribbed tires are known to be less sensitive to pavement macrotexture and water film depth than those taken using smooth tires.

iii. *Sideway force coefficient* measurement equipment uses an instrumented measuring wheel set at an angle to the direction of travel of the vehicle. Because the normally freely rotating tire is set at an angle, the tire slips over the road surface, and the resulting force along the wheel axle (the sideways force) is measured. The ratio of vertical and side forces averaged over a defined measuring length provides the recorded value that represents skid resistance. The wheel angle determines the slip ratio, and this ratio combined with the vehicle speed determines the slip speed. The most common type of this equipment is the *Sideway-force Coefficient Routine Investigation Machine* (*SCRIM*). These systems report a sideway-force friction coefficient (SFC).

iv. *Decelerometers* are typically custom-made units mounted in a test vehicle and are used to measure the deceleration of a vehicle under emergency braking. Widely used by police forces to assess road surface friction for collision investigations, and more recently in experimental naturalistic driving studies, these devices are not suitable for road network assessment or quality control purposes.

v. *Friction estimates based on vehicle kinematics and sensors* are also becoming more popular, but they are not used regularly in practice yet.

While some systems measure friction in short, localized sections of the road (for example, the ASTM E274-15 standard skid tester), others measure with the tire partially slipping continuously with respect to the pavement surface and are known as continuous friction measuring equipment (CFME – examples being the GripTester and the SCRIM). Different types of CFME use different operational principles and measuring modes.

1.2.2 Friction Measuring Standards

Friction testing and interpretation are done according to standard procedures, which are normalized by national and/or international bodies. The most commonly used standards in North America are those produced by AASHTO and ASTM International, formerly the American Society for Testing and Materials.

Most highway agencies in North America have traditionally used locked-wheel friction testers or "skid trailers" to measure friction. These tests are normalized by ASTM E274-15, *Standard Test Method for Skid Resistance of Pavement Surfaces Using a Full-Scale Tire*. The trailer fully locks one of the wheels of a trailer (generating 100% slip) to simulate emergency braking without anti-lock brakes, which were uncommon at the time the technology was developed. The measurements can be done using a ribbed tire (ASTM E501-08) or a smooth tire (ASTM E524-08).

The friction values measured, reported as friction numbers (FN) or skid numbers (SN), using the two tires are not consistent as they are affected differently by the two main pavement texture properties, microtexture and macrotexture. ASTM E274-15 reports friction as a skid resistance number that includes the speed of testing and the type of tire: R or S, for ribbed or smooth, respectively. For example, SN40R indicates that the test was run at a test speed of 40 mph (64 km/h) with a standard ribbed tire. When the standard international metric system is used, the test speed is placed in parentheses, for example, SN(65)R. AASHTO uses a similar notation but refers to the number as friction number or FN.

While measurements using the smooth tire are sensitive to both microtexture and macrotexture, measurements using the ribbed tire, are impacted mostly by the microtexture of the pavement. Ribbed tire measurements are not very sensitive to the surface macrotexture and some agencies have added macrotexture measurements to capture the full friction curve. In addition, friction measurements with the ribbed tire are also less susceptible to the testing speed and are typically higher than those produced by smooth tires at high speeds.

A key limitation of locked-wheel testers is that they can only sample the pavement surface by repeatedly collecting data on short segments of road and do not effectively differentiate the changes in friction along the route corridor. Furthermore, these devices are difficult to utilize in critical high friction demand locations, such as horizontal curves or intersections, which tend to experience greater tire scrubbing and polishing that lead to loss of pavement friction (FHWA 2021). As discussed in the previous section, the locked-wheel tester is a two wheel trailer that fully locks one of the wheels while testing. If the testing occurs on a sharp curve, the trailer may start to sway and the operator may lose control of the vehicle. The risk is reduced on curves with high radius of curvature and appropriate superelevation.

In contrast, most airports use CFME and report the coefficient of friction, not multiplied by 100. The most common equipment used in these facilities is fixed-slip CFME that measures friction at a low slipping speed. Examples include the GripTester (manufactured by Findlay Irvine). Because the systems use different configurations and operational conditions, the various CFME technologies produce different friction measurements and also different from those obtained with the locked-wheel trailers. The CFME operates at a low slip (Figure 6), so it is impacted mostly by the microtexture of the pavement and is not very sensitive to the surface macrotexture. Their measurements are often complemented by macrotexture measurements. As the reduction of friction with increasing slip depend on the macrotexture of the pavement, the relationship between the measurements of different friction measuring equipment is also a function of the macrotexture. The difference is higher for lower values of macrotexture.

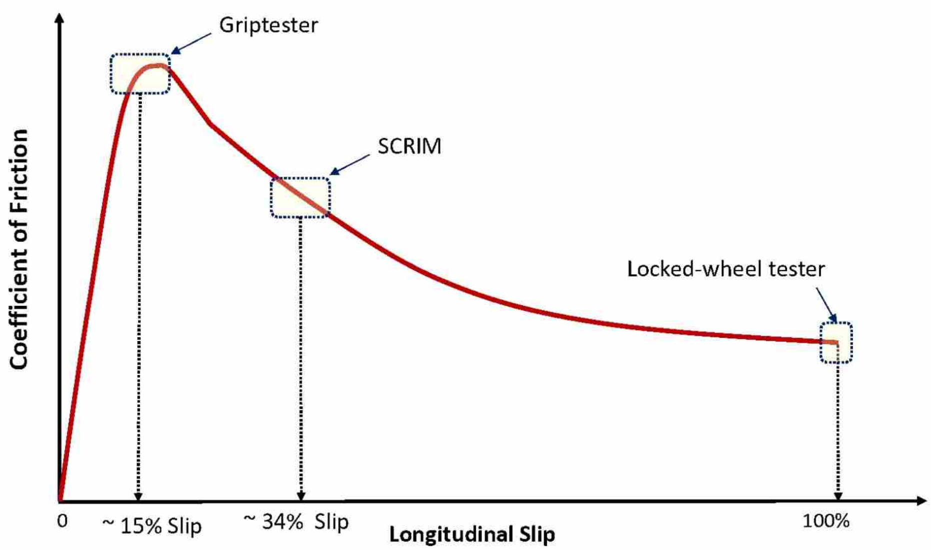

Figure 6. Illustration of the Slip Operational Ranges for Different Friction Measuring Equipment (not to scale).

Since around 2008, highway agencies in the United States have also started experimenting with the use of continuous friction testing on road networks (Flintsch et al. 2019). The initial experiments and demonstrations were done using GripTesters, but most recent efforts have typically measured the sideway force friction using SCRIMs. The first demonstration using a SCRIM in the U.S. started in 2015. First developed by the Transport Research Laboratory (TRL) in the United Kingdom, this type of friction measuring equipment has been used for roads in Europe and other parts of the world since the 1970's.

To facilitate the adoption of this technique in the U.S., the *Pavement Surface Properties Consortium – Managing the Pavement Properties for Improved Safety* (TPF-5(345)), developed a standard test, *Continuous Measurement of Sideway-Force Friction Number for Highway Pavements*. The standard has been recently approved by AASHTO and published as AASHTO standard TP 143-21 in 2021. This standard uses the SCRIM reading at 40 mph (SR40) that is different from the CSC used in the U.K. [measured at 50 km/h and multiplied by an index of SFC of 0.78; as discussed in DMRB (2021)]. The SR40 is multiplied by 100 to provide the sideway-force friction number (SFN).

1.2.3 Macrotexture Measuring Technologies

A measure of macrotexture is often needed to complement the friction measurements to obtain the full spectrum of frictional properties at various slipping speeds. As shown in Figure 4, relatively high macrotexture is critical to maintain an appropriate level of friction at high speed, e.g., higher that 80 km/h. This is especially critical in areas of high friction demands, such as curves in high-speed freeways. For example, as discussed following in section 1.3, macrotexture can be used to compute the speed constant (S_p) that allows estimating the friction at different slipping speeds in ASTM E1960-07, *Standard Practice for Calculating International Friction Index of a Pavement Surface.*

Macrotexture can be measured using both highway speed profilers and static methods. The oldest method is the volumetric patch test. In this test, a known volume of sand, glass beads, or grease is spread evenly into a circular patch on the road surface (where sand is used, it is commonly called a "sand patch test"). The area is measured, and the average depth below the peaks in the surface is calculated to give a value known as mean texture depth (MTD).

In more recent years, laser displacement sensors, which measure along a narrow line traversed by the laser (rather than across the area of a patch of sand or glass beads), have been used to determine a surface profile from which a number of different parameters may be calculated to represent the texture depth. The most widely used parameter is the mean profile depth (MPD). The MPD is normalized in the ASTM E1845-15 standard, which attempts to estimate the average depth below the peaks in a 100-mm segment of the surface profile.

On wet pavements, as the vehicle speed increases, skid resistance decreases to an extent that depends on the macrotexture (Figure 4). Generally, surfaces with greater macrotexture have greater friction at high speeds for the same low-speed friction (Roe and Sinhal 1998), but this is not always the case.

1.2.4 Operational Factors That Affect Friction Measurements

Several operational factors affect the friction measurement. A good understanding of these factors is important to understand the various friction measuring technologies and standards (Flintsch et al. 2012).

i. *Water film thickness*: As mentioned in the previous section, water film thickness is one of the factors that have been proven to affect the friction measurements. The water on the pavement surface decreases the tire-pavement contact area and so reduces the available friction force. Thicker films of water produce lower friction measurements.

ii. *Type and condition of the tire*: Worn tires are known to be more sensitive to water film thickness and provide less friction than tires in good condition, especially on wet surfaces. Pavement macrotexture and tire treads can provide channels for water to escape through the tire pavement contact area, which results in increasing the traction forces between tire and pavement surface.

iii. *Vehicle and sliding speeds*: Speed is also a factor. Both the vehicle speed and the speed at which the tire is slipping with respect to the pavement surface will affect dry and wet friction. Friction decreases as the vehicle and slipping speeds increase.

iv. *Temperature*: Because both hot mix asphalt surfaces and tires are viscoelastic materials, temperature also affect their properties. Research has indicated that tire-pavement friction decreases if the tire temperature increases (AASHTO 2008). Some standards provide a range of allowed temperature for measuring friction; for example, AASHTO TP 143-21 recommends a pavement temperatures range between 5°C to 50°C for measuring friction using the SCRIM.

    Other standards, such as the one for the British Pendulum testing (ASTM E303-93) and locked-wheel friction testers (ASTM E274-15), do not recommend a temperature range, but indicate that the measurement temperature be reported with the results. In addition, ASTM E274-15 provides a range of ambient temperature for verifying the requirements for the equipment instrumentation, 4°C to 40°C.

v. *Contaminant*: Contaminants such as oily liquids, dust, rubber accumulation, and other substances also affect the available friction and can cause localized areas of low friction.

1.2.5 Measuring Aggregate Polishing Properties

Aggregate properties are the predominant factor that determines frictional performance of asphalt surfaces and they are the primary contact medium with the vehicle tires (AASHTO 2008). Aggregate generally is viewed as two distinct sizes—coarse aggregate and fine aggregate. Coarse aggregate pieces are greater than the No. 4 sieve (4.75 mm).

To minimize the use of coarse aggregates that are susceptible to polishing, which results in loss of friction over time, some agencies require the use of test that measure the resistance of the aggregate particles to abrasion, wear and/or polishing. Common tests used for this purpose include Micro-Deval test for coarse aggregates (AASHTO T 327, *Standard Method of Test for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus*) and the Polished Stone Value (PSV) test (AASHTO T 279, *Standard Method of Test for Accelerated Polishing of Aggregates Using the British Wheel*).

To determine the Polished Stone Value (PSV), aggregate coupons (aggregates embedded in epoxy resin) are fabricated, subjected to accelerated polishing (using the British polish wheel) for a specified time (usually 9 hrs.), and then tested for frictional resistance using the British Pendulum Tester. The British pendulum number (BPN) value associated with accelerated polishing is defined as the polished stone value (PSV). This number is a quantitative representation of the aggregate's terminal frictional characteristics. Higher values of PSV indicate greater resistance to polish (AASHTO 2008).

1.3 Interconversion of Friction Measurements

To be able to compare measurements taken by different types of equipment, measurements should be adjusted to a common scale. This process is called harmonization.

ASTM has defined harmonization of measurements as "the adjustments of the outputs of different devices used for the measurement of a specific phenomenon so that all devices report the same value" (ASTM E 2100-04). Several studies dealing with harmonization of friction measurement equipment have been conducted around the world: (1) the World Road Association (PIARC) International Experiments from the early 1990s (Wambold et al., 1995); (2) the NASA Friction Workshops at Wallops Flight Facility (Yager 2005); (3) the European HERMES project (Descornet et al. 2006); (4) the Virginia Tech Transportation Institute (VTTI) Pavement Surface Properties Consortium Rodeos (TPF-5(141)); and (5) the "Tyre and Road Surface Optimisation for Skid resistance and Further Effects" (TYROSAFE) (Scharnigg et al. 2011), among others.

The PIARC experiment (Wambold et al., 1995) developed the International Friction Index (IFI) to compare and harmonize between various methods used around the world to measure friction and texture (Wambold et al. 1995). The IFI is composed of two parameters: a speed constant (S_p) and a friction number at 60 km/hr (*F60*). A macrotexture measurement is used to compute the speed constant (S_p), and it allows estimating the friction at different slipping speeds. The higher the S_p, the faster the friction decrease with speed. The IFI has been normalized in ASTM E1960-07, *Standard Practice for Calculating International Friction Index of a Pavement Surface.*

The IFI harmonization procedure has been available for many years. However, it is not widely used because the results are very dependent on the equipment used and the surfaces tested to determine the interconversion coefficients, which are used to determine the harmonization coefficients. Furthermore, several studies have questioned the use of the reference devices chosen for the standard (e.g., Flintsch et al. 2009, Barrantes et al. 2018).

Though there are many problems converting friction measurements obtained with the different types of equipment discussed in the previous sections, some recent studies have provided guidance to conduct approximate interconversion among the three main types of equipment used to measure highway friction: the locked-wheel tester, SCRIM, and GripTester. These procedures use the principles in the International Friction Index but eliminate the use of static reference measurements.

De León et al. (2019) provide procedures for interconverting SCRIM measurements and locked-wheel testers' friction numbers, using smooth and ribbed tires, based on a national study sponsored by the Federal Highway Administration (FHWA). Similarly, de León et al. (2017) provide equations to interconvert GripTester and locked-wheel tester measurements. However, the interconversions are not very accurate and may not apply to pavements not included in their development.

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2 Pavement Friction Management

This section discusses the various approaches that highway agencies use to specify and manage the fictional properties of pavements. There are different types of factors that contribute to highway crashes, including those related to drivers, vehicles, and highway conditions (Treat et al., 1979). Of these three categories, only highway conditions can be partially controlled by highway agencies, through design, construction, maintenance, and management practices and policies. Among the various highway-related conditions that influence safety (e.g., curvature, intersections, and roadsides), friction and texture play a key role: if deficient, they can contribute to crashes.

Though deficient friction is seldom the main cause of a crash, there are situations where low friction can cause crashes in the presence of other contributing circumstances. For example, if human error makes an emergency maneuver necessary, a crash may occur if the friction demanded by the maneuver is greater than the friction that the road surface can provide in that location. If the available friction is exceeded, skidding or wheel slipping may lead to a loss of control or to a collision (Flintsch et al. 2012). On the other hand, if the friction is high, the collision may be avoided or its severity reduced.

Road sections with poor friction, or skid resistance, because of the materials they are made of and/or how those materials have been polished by traffic, may contribute to crashes. To minimize the contribution of friction problems to road crashes, highway agencies typically employ friction management approaches to detect such situations and take appropriate action. Pavement friction management includes engineering practices to provide a pavement surface with adequate and durable friction during construction, and it includes periodic data collection and analysis to ensure the effectiveness of these practices.

Countries such as UK, Australia, New Zealand, and Germany have established pavement friction management programs or policies to provide a framework by which road engineers can monitor the condition of their networks and, based on objective evidence, make appropriate judgments regarding treating or resurfacing the road where required. These judgments balance the risk of a crash occurring with the costs and practicalities of providing adequate friction. Because high levels of friction and macrotexture enable vehicles to reduce speeds more rapidly and allow longer retention of control, they may prevent a crash or reduce its consequences in terms of death or severity of injury. Though crashes will probably never be completely eliminated, an effective policy can reduce collision risk and reduce the severity of those crashes that do happen.

An effective approach to provide adequate pavement friction requires strategies at both the management and design levels of a highway pavement program. The management component requires policies and practices to monitoring friction and crashes, and proper and timely responses to potentially unsafe roadway surfaces (AASHTO 2008). Thus, a pavement friction management program involves building pavement surfaces with appropriate friction and macrotexture, monitoring of skid resistance on the network with the appropriate measuring equipment, establishing values of friction that would trigger an investigation for each road category, and defining appropriate interventions for places where deficiencies are identified.

2.1 Relationship between Crashes and Friction

Pavement friction is very important to roadway safety (AASHTO 2008; Henry 2000). Several studies over the years have repeatedly shown that sites with low friction have more crashes than sites with high friction. Because a large percentage of the skidding problems occur when the road surface is wet, the focus over many years has been on the link between wet crashes and friction. For example, a study in Kentucky in the 1970s revealed that the rate of wet crashes increases as the surface friction drops below a certain value, as illustrated in Figure 7 (Rizenbergs et al. 1973). This led many U.S. state highway agencies to focus on friction in their Skid Accident Reduction Programs or Wet Accident Reduction Programs, which concentrated on areas with high numbers of wet crashes (Anderson et al. 1998).

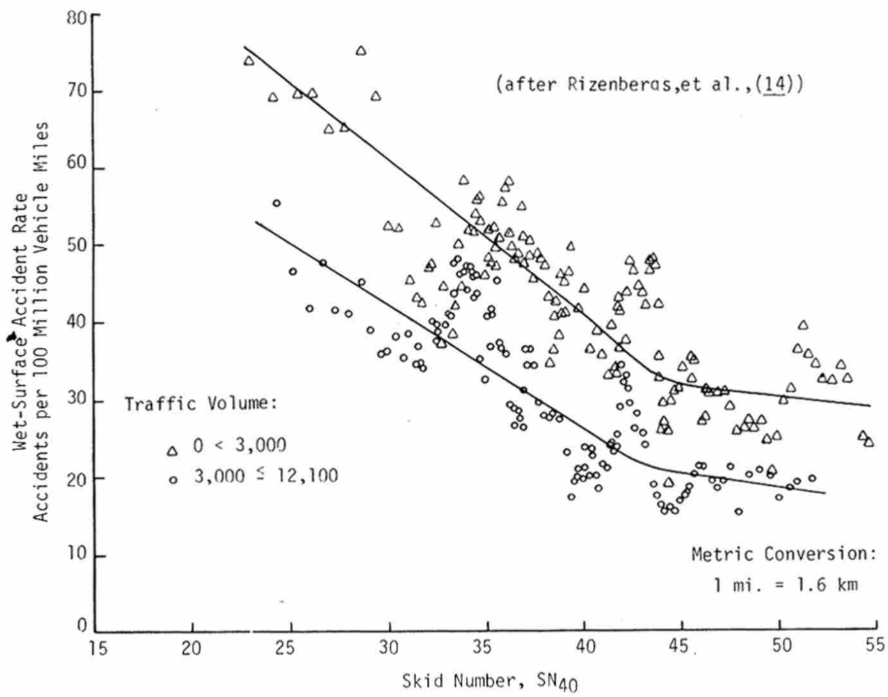

Figure 7. Example Illustration of the Relationship between Wet-Weather Crash Rates and Pavement Friction for Kentucky Highways (Rizenbergs et al. 1973)

The higher impact of friction on crashes when the pavement surface is wet, versus dry, has often led to the assumption that skid resistance is sufficient on dry surfaces. However, recent studies have found that both dry and wet crash rates increase with decreasing friction. For example, Mayora and Piña (2009) and Najafi et al. (2015) have shown that skid resistance affects both dry and wet crashes. For example, Figure 8 shows how both the wet and dry crash rates decrease as friction increases on Virginia roadways. However, it is important to note that the impact is higher on wet crashes than on dry crashes (McCarthy et al., 2021). This is illustrated in Figure 9, which presents an example of the estimated percent change in dry and wet crashes as a function of friction based on the models developed by McCarthy et al. (2021). For the types of

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pavements investigated, a roadway with a SFN of 40 can be expected to have 54% more wet crashes and 25% more dry crashes than one with a SFN of 60. It is important to note that these values are illustrative only as they are specific for the network investigated.

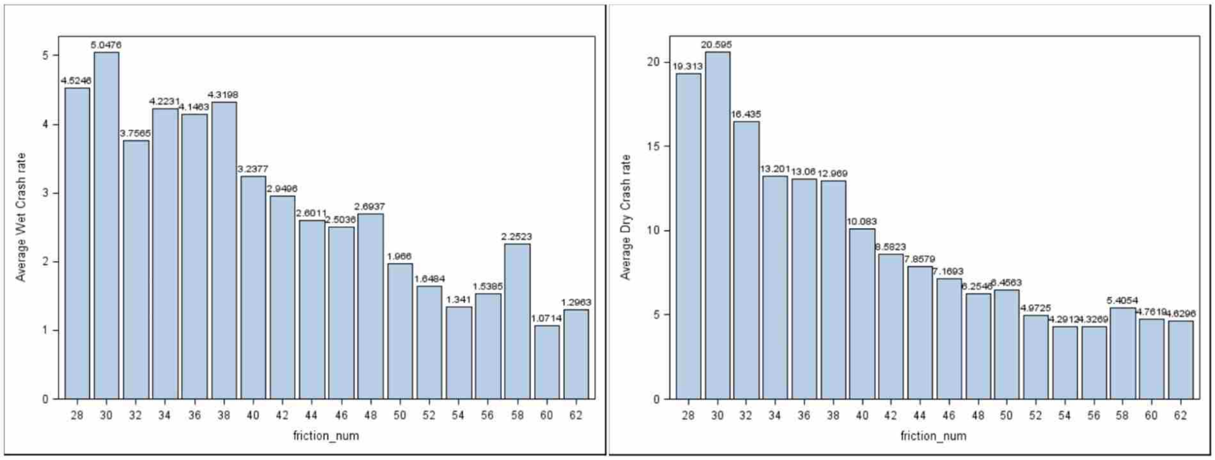


Figure 8. Average Wet- and Dry-crash Rates by FN40S Level for Virginia (Smith et al. 2011)

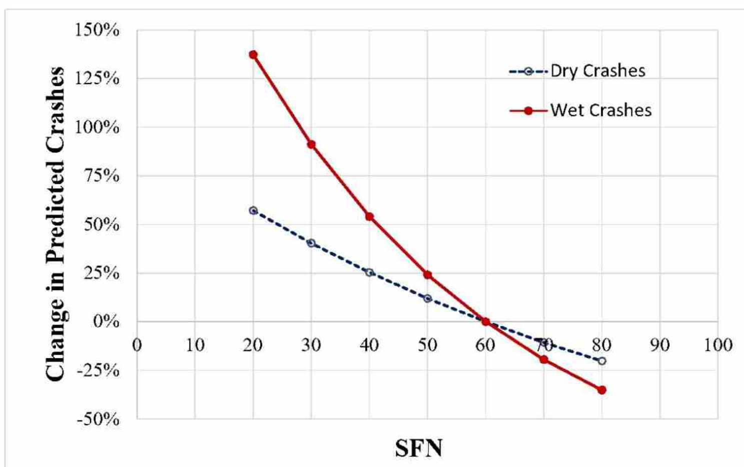


Figure 9. Illustrative example of estimated changes of Average Wet- and Dry-crash Rates vs. Friction (SFN) (after McCarthy et al., 2021)

2.2 Designing for Friction

Pavement friction design involves utilizing proper materials and construction techniques to achieve high levels of microtexture and macrotexture in pavement surface. The type of aggregates used in the surface mix directly affects the microtexture, while gradation and aggregate size governs the macrotexture properties of pavement surface. In asphalt mixtures, large aggregates govern the frictional properties of the surface, while for concrete mixes, fine aggregates control the frictional properties (AASHTO 2008).

The wear characteristics of aggregates are also important in maintaining proper friction level. Aggregate mineralogy and hardness directly affect the durability and resistance to polishing of the aggregates (as discussed in section 1.2.5). It is generally better to have aggregates with different size and wear characteristics in the mix so they can constantly renew the surface (AASHTO 2008).

2.3 Friction Demand

Not all vehicles need the same friction under all circumstances. Furthermore, factors such as traffic volume, geometrics (curves, grades, cross-slope, sight distance, etc.), potential for conflicting vehicle movements, and intersections will impact how much friction is needed. For this reason, many highway agencies have defined friction demand categories to help identify areas where more friction is needed.

Friction demand is the level of friction needed to safely accelerate, brake, and steer a vehicle. Highway agencies seek to assure that pavement surface friction supply (the maximum friction that the surface can provide) meets or exceeds friction demand at all times. Because the demand varies along different types of roads and also along any given road because of the presence of sharp curves, grades, or intersections, agencies often establish friction demand categories systematically based on highway alignment, highway features/environment, and highway traffic characteristics.

Ideally, friction demand categories should be established for individual highway classes, facility types, or access types. There will be significant sections of the network, especially lightly trafficked routes or major highways, where that will not require much friction because situations likely to involve skidding are generally rare. On the other hand, in places where it is known that drivers frequently need to brake or turn at speed, for instance, needed friction levels are likely to be higher than would be adequate elsewhere.

The countries that have focused on improving friction to reduce crashes, led by the United Kingdom (UK), have defined friction demand categories that reflect the risk associated with driving along each demand category. The UK has defined 10 highway demand categories (DMRB 2021), which divide the roads based on their design standard (high-level highways, divided highways, and two-lane roads) and whether or not the sections include an “event.” A non-event roadway section is a tangent section of roadway with a gradient less than 5 percent and with no intersection, ramp, or crossings. Events include sharp curves, intersections, ramps, crossings, and sections with gradient greater than 5 percent.

Similarly, the *Guide for Pavement Friction* (AASHTO 2008) recommends that highway agencies establish investigatory level and intervention level values for pavement friction and texture in accordance for each friction demand category. However, recent proposed revisions to the *Guide* recommend eliminating the use of intervention levels because agencies are unlikely to trigger treatments without a detailed project investigation (de León et al. 2019).

2.4 Friction Investigatory Levels

Friction demand categories are typically assigned threshold values of skid resistance, called investigatory levels, that trigger investigation of pavement sections with measured skid resistance at or below the threshold value to determine the cause of the deficiency and whether a safety countermeasure is necessary. The primary function of a skid resistance policy is to produce the adequate friction properties across a pavement network by assigning thresholds of friction that maintain an acceptable level of crash risk (DMRB 2021). The following section provide examples of friction management policies.

2.4.1 United Kingdom

In the United Kingdom, the standard for skidding resistance, HD 28, was implemented in 1988. The recommendations in this standard were based on decades of research into the relationship between skidding accidents and pavement characteristics. The concept of separating the British pavement network into friction demand categories based on skidding crash risk was first proposed by Cyril Giles in 1956 (Roe & Caudwell 2008).

The standard has been periodically updated in response to changes in factors, such as traffic volume, that affect the crash risk across the Strategic Road Network. Table 1 reproduces the latest standard, CS 288, and shows the recommended ranges of investigatory levels (IL) of SCRIM CSC (characteristic SCRIM coefficient) for 10 friction demand categories. In the context of the standard, the SCRIM CSC is an SFC (sideway force coefficient) that has been corrected to a survey speed of 50 km/h, multiplied by an index of SFC (0.78), and corrected for seasonal variation (DMRB 2021).

Standard CS 288 has replaced previous Standards HD28/04 and HD 28/15 (Highways England 2015). The main differences with the superseded versions are additional notes provided to expand the criteria for selecting the most appropriate investigatory levels when several values are listed.

In most cases, the investigatory levels are presented as ranges and the standard provides specific guidance on how to select the specific value most relevant for a particular section based on a detailed site investigation. For example, for highways (category A, motorways), an IL of 0.35 (denoted by ST) will be appropriate in almost all circumstances, but it can be changed to 0.30 in exceptional cases if, following a detailed site investigation, it is clear that the crash risk associated with a skid resistance below 0.35 is low (DMRB 2021). Similarly, for other divided highways (category B, non-event carriageway with one-way traffic), the IL should be increased to 0.40 for special cases, such as areas where pedestrians or other vulnerable road users are common but category K is not appropriate, junctions where the geometry does not justify using category Q, etc.

Table 1. UK Friction Demand Categories and SCRIM Investigatory Levels (DMRB 2021)

Site category and definition		Investigatory level (IL) of CSC (skid data speed corrected to 50 km/h and seasonally corrected)							
		0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
A	Motorway	LR	ST						
B	Non-event carriageway with one-way traffic	LR	ST	ST					
C	Non-event carriageway with two-way traffic		LR	ST	ST				
Q	Approaches to and across minor and major junctions, Approaches to roundabouts and traffic signals				ST	ST	ST		
K	Approaches to pedestrian crossings and other high risk signal					ST	ST		
R	Roundabouts				ST	ST			
G1	Gradient 5-10% longer than 50 m (see note 6)				ST	ST			
G2	Gradient >10% longer than 50 m (see note 6)				LR	ST	ST		
S1	Bend radius < 500 m – carriageway with one-way traffic				ST	ST			
S2	Bend radius < 500 m – carriageway with two-way traffic				LR	ST	ST		

'ST' indicates the range of ILs that should generally be used for roads carrying significant levels of traffic.

'LR' in cells indicates a lower IL that may be appropriate in lower risk situations, such as low traffic levels or where the risks present are mitigated by other means, providing this has been confirmed by the crash history.

NOTE 1 Sites with the same site category can have different levels of risk of skidding crashes. There is therefore the flexibility to set different ILs for different sites within the same category.

NOTE 2 This allows sites where the risk of skidding crashes is potentially higher to have a higher IL and possibly be treated to maintain a higher level of skid resistance.

NOTE 3 The objective of setting an IL is to assign a level of skid resistance appropriate for the risk on the site, at or below which further investigation is required to evaluate the site specific risks in more detail.

NOTE 4 Advice for selecting an appropriate IL is provided in Appendix A of the standard. The range of ILs for each site category has been developed as a result of UK research and reflects the variation in crash risk within a site category.

The UK Pavement Management System User Manual Volume 3: Machine Data Collection for UKPMS also provided the values for the GripTester as an alternative device (UKPMS 2005). These are reproduced in Table 2. The GripTester values used in the table were calculated using a conversion factor of 0.85 based on a correlation study conducted by the Transport Research Laboratory (TRL) (Frankland 2004).

More recently, this document has been replaced and the current website (UKRLG 2021) recommends using a conversion factor of 0.89 based in a more recent TRL correlation study (Dunford, 2010). Furthermore the site indicated that the correlation applies only "to the specific surface types assessed as part of PPR 497. If a GripTester is used to monitor a network then appropriate Investigatory Levels (IL) should be calculated for the GripTester results rather than converting the GripTester data into SC data and using the ILs defined for sideways force devices."

Therefore, it important to note that the conversions are approximate and dependent on the pavement surfaces used for their development, as discussed in section 1.3. Moreover, in general, macrotexture values in the U.K., are significantly higher than those in North America, because the U.K. has established minimum macrotexture requirements and this will impact the interconversion relationships between the measurements.

Table 2. Adaptations of the UK Investigatory Levels for a Mark 2 GripTester using a conversion factor of 0.85 (after UKPMS 2005).

Site category and definition			Investigatory level (IL) at 50 km/h							
		SFC	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
		GN	0.35	0.41	0.47	0.53	0.59	0.65	0.71	0.76
A	Motorway									
B	Non-event carriageway with one-way traffic									
C	Non-event carriageway with two-way traffic									
Q	Approaches to and across minor and major junctions, Approaches to roundabouts and traffic signals									
K	Approaches to pedestrian crossings and other high risk signal									
R	Roundabouts									
G1	Gradient 5-10% longer than 50 m (see note 6)									
G2	Gradient >10% longer than 50 m (see note 6)									
S1	Bend radius < 500 m – carriageway with one-way traffic									
S2	Bend radius < 500 m – carriageway with two-way traffic									

Notes: Reference should be made to Chapter 4 of HD 28/04 and in particular, the notes to Table 4.1 (of HD 28/04) for guidance on interpretation.

Dark Grey indicates the range of ILs that should generally be used for roads carrying significant levels of traffic.

Light Grey in cells indicates a lower IL that may be appropriate in lower risk situations, such as low traffic levels or where the risks present are mitigated by other means, providing this has been confirmed by the crash history.

2.4.2 Australia

In Australia, Austroads is responsible for developing Australian "national guidance documents" on topics such as road safety and asset management (Hillier 2012). Consequently, Austroads is responsible for developing and managing skid resistance policies in Australia. The most recent policy is the *Guidance for the Development of Policy to Manage Skid Resistance (AP-R374/11)*, which Table 3 summarizes (Pratt & Neaylon 2011).

The Austroads guidelines for managing skid resistance features seven friction demand categories and a range of SCRIM side-force friction coefficients (SFC) investigatory levels assigned to each. Currently, the state and local road authorities are encouraged, but not required, to develop a strategy for managing skid resistance across the roadway networks. It is noted that the measurements are conducted a lower testing speed for the last two friction demand categories.

Table 3. Friction Demand Categories and Investigatory Levels used in Australia (from Pratt & Neaylon 2011)

Site category	Site description	Investigatory level of SFC_{50} at 50 km/h or equivalent						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
		Corresponding risk rating						
		1	2	3	4	5	6	7
1 (see notes)	Traffic light controlled intersections; pedestrian/school crossing; railway level crossings; roundabout approaches			INVESTIGATION ADVISED				
2	Curves with tight radius ≤ 250 m; gradients ≥ 5% and ≥ 5 m long; freeways/highways on/off ramps							
3 (see notes)	Intersections							
4	Maneuver-free areas of undivided roads							
5	Maneuver-free areas of divided roads							
Site category	Site description	Investigatory level of SFC_{20} at 20 km/h or equivalent						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
		Corresponding risk rating						
		1	2	3	4	5	6	7
6	Curves with tight radius ≤ 100 m			INVESTIGATION ADVISED				
7	Roundabouts							
Key to thresholds at or below which investigation is advised								
(dark grey)	All primary roads, and for secondary roads with more than 2500 vehicles per lane per day							
(light grey)	Roads with less than 2500 vehicles per lane per day							

Notes:

- Investigatory levels are based on the minimum of the four-point rolling average skid resistance for each 100 m section length.
- Investigatory levels for site categories 1 and 3 are based on the minimum of the four-point rolling average skid resistance for the section from 50 m before to 20 m past the feature, or the 50 m approaching a roundabout.

Source: Austroads (2003).

2.4.3 New Zealand

The New Zealand (NZ) policy for managing skid resistance on the state highway network, known as the T10 specification, was introduced in 1997 (Cook et al. 2014; Owen et al. 2008). The T10 specification is based on the UK approach but adapted for the NZ environment (Owen 2014). The specification has been updated on several occasions.

The current T10 specification, shown in Table 4, features five friction demand categories and a range of ESC investigatory levels assigned to each. The equilibrium SCRIM coefficient (ESC) is SFC corrected for the "SFC factor," survey speed, temperature, and seasonal variation. In addition to the investigatory level, the New Zealand Transport Agency (NZTA) also assigned another value, called a threshold level (TL), that is an intervention level used to trigger immediate remedial action when ESC is at or below the TL. The TL assumes the maximum of two possible values: 0.10 ESC units below the investigatory level or 0.30 ESC (NZTA 2013).

Table 4. Friction Demand Categories and SCRIM ESC Investigatory Levels Used in New Zealand (from NZTA 2013)

Site category	Skid site description	Investigatory level (IL), units ESC						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
1	Approaches to: a) Railway level crossings b) Traffic signals c) Pedestrian crossings d) Stop and give way controlled intersections (where state highway traffic is required to stop or give way) e) Roundabouts One lane bridges a) Approaches and bridge deck							
2	a) Urban curves < 250 m radius							
	b) Rural curves < 250 m radius				L	M	H	
	c) Rural curves 250-400 m radius			L	L	M	H	
	a) Down gradients 10% b) On ramps with ramp metering							
3	a) State highway approach to a local road junction b) Down gradients 5-20% c) Motorway junction are including on/off ramps d) Roundabouts, circular section only							
4	Undivided carriageways (event-free)							
5	Divided carriageways (event-free)							

Notes to Table 1:

- When using seasonally corrected data, ILs are for mean skidding resistance within the appropriate averaging length. This is referred to as the Skid Assessment Length (SAL). The SAL for each site category is detailed in table 2.
- The curve risk rating on rural curves with radii 0-400 m is shown as H, M or L (high, medium or low-risk curves) in the appropriate greyed IL band under site categories 2b and 2c. Two options are available for rural low-risk sites with radii between 250 m and 400 m. Urban curves with a radius less than 250 m are site category 2a.
- The units for IL in table 1 are ESC, being the average of the left and right wheelpaths. Where seasonally corrected data is not available, SCRIM coefficient (SC) may be used as an approximation to ESC with further checks undertaken when seasonal corrections are available.
- Where the length of the feature is less than the SAL, the actual length shall be averaged and considered.

New Zealand also monitors macrotexture (MPD) at the network level and compares the values to established requirements, set forth in terms of investigatory level macrotexture (ILM) and threshold level macrotexture (TLM). Table 5 presents these requirements (NZTA 2010). The guidelines allow ILM and TLM reductions of up to 0.008 in (0.2 mm) in accordance with crash risk deviations between a region and the national average.

Table 5. New Zealand Minimum Macrotexture Requirements (NZTA 2013)

Legal Speed Limit	Minimum Macrotexture MPD (mm)					
	Chipseals		Asphaltic Concrete ESC ≥ 0.4		Asphaltic Concrete ESC < 0.4	
	ILM	TLM	ILM	TLM	ILM	TLM
50 km/hr and less	1.0	0.7	0.4	0.3	0.5	0.5
Less than or equal to 70 km/hr but >50 km/hr	1.0	0.7	0.4	0.3	0.7	0.5
Greater than 70 km/hr	1.0	0.7	0.9	0.7	0.9	0.7

Notes to Table 3

- On curves where the advisory speed limit is 45 km/h or less, consideration may be given to the use of ILM and TLM (as per table 3) for asphaltic concrete where the permanent speed limit is 50km/h and less
- The TLM for chipseals is set at 0.7 mm MPD. In urban areas, where the surveyed macrotexture is equal to or higher than required for asphaltic concrete (i.e., 0.5 mm MPD), maintenance to improve the macrotexture may be delayed provided that:
  - The ESC is above TL.
  - ESC levels are stable, i.e., they have not reduced by more than 0.05 ESC since the previous annual survey.
  - Inspections are programmed and resources are available to ensure prompt treatment is undertaken, should macrotexture levels continue to drop.

2.4.4 Canada

I am unaware of any published provincial or national standards in Canada respecting highway friction investigatory or intervention levels, and the provinces have developed different approaches to manage friction. I have conducted a review, including consultation with colleagues, to confirm my understanding.

I have been advised by Commission Counsel to the Red Hill Valley Parkway Inquiry that a number of individuals from the Ontario Ministry of Transportation (MTO) will be called as witnesses at the public hearings, who will testify as to MTO practice and policy respecting highway friction management in Ontario. This will include, but not be limited, to MTO use of approved aggregate sources, and its use of the ASTM E274 locked wheel tester and application of the results from such testing.

2.5 Pavement Friction Management in the United States

In the United States, the traditional approach to solving friction problems has been to designate a group from the pavement field-testing unit to test the friction of specific roadway locations identified as having “high crash counts” or “hot spots.” The values selected to define high crash counts (typically wet-pavement crashes) have been chosen by various methods and are not uniform. Agencies then use a friction threshold value to decide if a section should be investigated for a friction-improving treatment. McGovern et al. (2011) reviewed the practice for reducing wet-weather skidding crashes in the U.S. and provided examples of these practices in California, Florida, Michigan, New York and Virginia.

The majority of agencies use only one threshold, which does not discriminate the roadway type or site type (e.g., whether it is located on a tangent, curve, vertical curve, etc.). For example, New York DOT uses locked-wheel friction testing at each 0.16-km (0.1-mi) segment of the qualifying location in each direction. If a section has one or more FN40R readings less than 32, it is recommended for treatment. Friction restoration treatments typically include either a 38-mm (1.5-in) asphalt concrete overlay using non-carbonate aggregates or a thin microsurfacing (Lyon and Persaud 2008).

Conversely, the safety management approach proposed in the *AASHTO Guide for Pavement Friction* recommends that adequate levels of friction be maintained on all roadway sections based on the friction demand needed for the different types of roadway segments (as it is done in the U.K., Australia, etc.). Different friction threshold values are set based on roadway types (interstate, primaries, etc.), geometry of the roadway section (intersection, curve, grade, etc.), and so on.

When friction thresholds are not met, detailed pavement and safety evaluations can be done to verify if an increase in the friction level is warranted to reduce the crash risk (e.g., of roadway departure fatalities and serious injuries). For example, a study conducted by the Maryland Department of Transportation recommended design FN40R for five demand categories, ranging from 35 for low demand sections to 55 in the highest demand locations (Chelliah et al. 2002).

The *AASHTO Guide for Pavement Friction* (AASHTO 2008) contains guidelines and recommendations for managing and designing for friction on highway pavements. In addition to emphasizing the importance of providing adequate levels of friction for the safety of highway users, the *Guide* (1) discusses the factors that influence friction and the concepts of how friction is determined; (2) presents methods for monitoring the friction of in-service pavements, identifying where friction deficiencies exist, and determining appropriate actions for addressing friction deficiencies (friction management); and (3) presents aggregate tests and criteria for ensuring adequate microtexture and discusses how paving mixtures and surface texturing techniques can be selected to impart sufficient macrotexture to achieve the design friction level (friction design).

The *Guide* provides three methods for establishing investigatory and intervention threshold friction levels. The first method uses historical trends of friction loss determined by plotting friction versus pavement surface age for a specific friction demand category. The investigatory level is set at the friction value where the friction deterioration rate begins to accelerate significantly, and the intervention level is set at a lower friction. The second method compares the historical pavement friction and crash rate data; the investigatory level is set to correspond to a significant increase in the rate of friction deterioration, while the intervention level is set when there is a significant increase in crashes.

Method 3 uses the distribution of friction data and the crash rates that correspond with each level of friction. The investigatory level is set at the point where the wet-to-dry crashes begin to increase significantly, and the intervention level is set at a lower level of friction determined subjectively by looking at the trends. However, as mentioned previously, de León et al. (2019) proposed that it is not appropriate to define intervention levels because highway agencies will not automatically trigger any kind of maintenance treatment to correct any deficiency without a proper investigation. Interventions are only triggered if the investigation concludes that it is necessary.

The Federal Highway Administration (FHWA) *Technical Advisory T 5040.1738—Skid-Accident Reduction Program* (FHWA, 2010) provides technical information and guidelines for implementing a pavement friction management program. This program aims to minimize friction-related vehicle crashes by ensuring that new pavement surfaces are designed, constructed, and maintained to provide adequate and durable friction properties; identifying and correcting sections of roadways that have elevated friction-related crash rates; and prioritizing use of resources to reduce friction-related vehicle crashes in a cost-effective manner.

Furthermore, a recent study from the U.S. Federal Highway Administration (FHWA 2021) has documented the potential economic and social benefits of implementing a pro-active PFM approach using continuous friction measurement equipment (CFME) data (de Leon et al. 2021). The project: (1) collected and analyzed pavement friction, crash, traffic, and other geometric data in four states; (2) demonstrated methods for establishing investigatory levels of friction for different friction demand categories; and (3) recommended a proactive systemic approach for developing pavement friction management plans using proven safety analysis methods, as described in the AASHTO Highway Safety manual (AASHTO 2010).

The methodology proposed, which has been included in a proposed revision of the *AASHTO Guide for Pavement Friction*, include the following steps:

1. Collect network-level friction, macrotexture and geometric data, as well crash data.
2. Subdivide the highway network into pavement friction demand categories, separating different types of roads, as well as localized areas that require more friction, such as curves and intersections.
3. Develop statistical models to relate crashes to friction, macrotexture, and other roadway characteristics for each friction demand category and perform network-level analysis.
4. Identify sections with friction deficiencies that may benefit from friction enhancement treatments.
5. Evaluate and select roadway segments for surface friction enhancement treatments, and optimal treatments for each of these segment, using economic analysis based on estimating the number of crashes that could be reduced by different treatments.

The methodology proposed in the FHWA report has been recently implemented and enhanced by the Virginia DOT to develop a pilot PFM program for the Corridors of Statewide Significance (CoSS) in Virginia. The project collected friction, macrotexture, and geometric data; processed and filtered the data; and conducted a systemic analysis of the network. The analysis investigated the relationship between crashes and friction and other roadway properties, and developed statistical models, called Safety Performance Functions (SPFs), to quantify this relationship. The SPFs were then used in empirical Bayes analyses to estimate crash counts before and after friction enhancement treatment and identify sections with friction deficiencies that may benefit from them (de Leon Izeppi et al, 2021). The methodology identifies roadway sections on which a friction enhancement treatment would yield positive economic benefits and thus, should be subjected to a detailed safety investigation. The application of the selected friction enhancement treatment to the candidate sections could result in a reduction of up to approximately 20% of crashes in the network analyzed. The effort also highlighted the importance of collaboration between the safety, maintenance and design groups within the agency.

2.6 Methods for improving low pavement friction

The traditional approach to treat areas with deficient frictional properties (friction or macrotexture), is to resurface or mill and replace the questionable surface assuming that the problems are due to polishing and wear of the pavement surface. However, there are also many different safety and preservation treatments that can be used to improve microtexture, macrotexture, or both. For example, high-friction surfaces (HFS) provide an effective (though costly) solution in areas of very high demand for friction, such as approaches to intersection or sharp curves on roadways with relatively high speeds.

Examples of treatment that can be used to restore or enhance frictional properties include the following technologies (see Figure 10 for illustrations of the various examples):

- ***High-friction surface treatment*** (HFST) is a safety treatment, rather than a pavement preservation treatment, that dramatically increases pavement friction and macrotexture to reduce crashes associated with friction demand issues. It can also be used to restore pavement surface friction where traffic has polished existing pavement surface aggregates. The treatment is installed by spreading a thin layer of polymeric resin binder over the pavement surface, then spreading or dropping a 1- to 3-mm abrasion and polish-resistant aggregate onto the resin layer. According to the FHWA, HFST is a highly effective and mature safety countermeasure for reducing both wet and dry pavement friction-related crashes. Despite its high cost, when applied at appropriately selected locations and installed properly, exceptional benefit/cost ratios have been realized by many agencies in the U.S. (Merritt et al., 2021).
- ***Chip seals*** or surface treatments are pavement preservation treatments that if properly designed and constructed provide long-term friction and macrotexture. To apply a chip seal, an asphalt binder (commonly asphalt emulsion) is applied to the existing asphalt pavement surface followed by the immediate application of aggregate chips that are rolled using a compactor to achieve the anticipated aggregate embedment and increase the retention of aggregate chips. The primary use of chip seal is to seal the pavement surface and provide a new surface with enhanced surface friction performance. Although Peshkin et al. (2011) recommended the use of chip seals for high volume roadways, many agencies only use of this type of treatment for low to medium traffic roadways. In addition, Li et al. (2012) reported drastic decreases in friction after 12 months on some of a series of chip seal applications investigated.
- ***Ultrathin Overlays,*** *ultrathin bonded wearing courses* or *ultra-thin friction courses* consist of thin layers of a fine HMA (generally using gap-graded aggregate an polymer-modified aggregate) typically applied as a preservation treatment (Merritt et al. 2015).

Typical thicknesses are between 10 and 20 mm. These treatments usually have good friction and macrotexture. However, some applications have resulted in surface with low macrotexture.

- ***Microsurfacing*** is a common preservation for high-volume, high-speed roadways. Microsurfacing is a mixture of crushed, well-graded aggregate, mineral filler (Portland cement), and latex-modified emulsified asphalt spread over the full width of pavement with either a squeegee or spreader box (Peshkin et al. 2011). While microsurfacing can be designed and constructed to have good friction and macrotexture, some applications can have relatively low macrotexture, which can be a problem on high-speed roadways.
- ***Micro-milling***. Micro-milling is a surface treatment in which a milling head is used to remove a thin layer of the pavement surface. Micro-milling differs from conventional milling in that the cutting head uses teeth that are spaced closely together, leaving a much less aggressive surface texture than conventional milling (which leaves a texture that is too rough). Whereas milling is typically used to remove pavement in preparation for an overlay, micro-milling leaves a much less aggressive surface texture that can be opened to traffic as a final surface. Although micro-milling is used regularly as part of pavement rehabilitation in preparation for a new overlay, there is very limited usage to date for improving frictional properties (Merritt et al. 2015).
- ***Grooving*** is a treatment usually used in airfield runways in which narrow grooves are sawcut into the pavement surface, typically in the direction of traffic, and typically 20 mm apart. The grooves increase pavement macrotexture, providing a path for bulk water drainage. Grooving is typically used on concrete pavements, but can also be done on asphalt pavements.
- ***Shotblasting*** or abrading is a surface treatment in which steel pellets or "shot" are fired at the pavement surface at high velocity to pit or abrade away a superficial layer of the pavement surface. Shotblasting removes contaminants from the surface and also pits the surface of the aggregates to improve microtexture. It is frequently used to remove rubber or oil deposits on the pavement surface on runways.

  Skidabrading is a special shotblasting technology that uses a high-speed wheel to propel steel shot in a controlled pattern towards a substrate. The high-speed impact of the steel shot abrades and removes contaminants while etching the surface. (Skidabrader 2022).

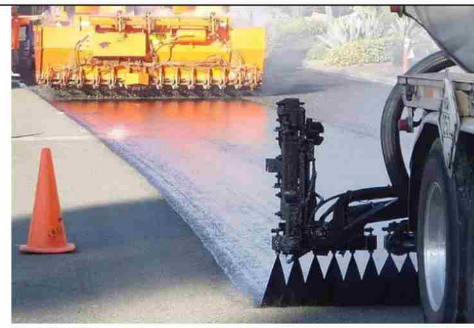
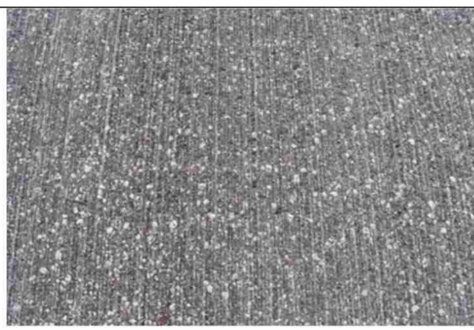
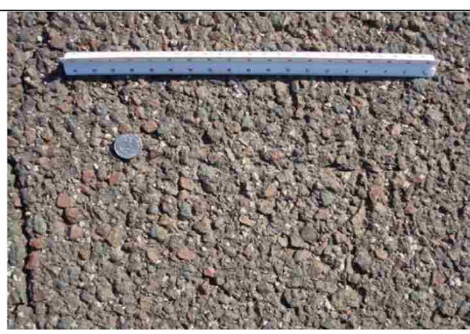
 HFST (1)	 Chip Seals (1)
 Microsurfacing (1)	 Ultrathin Overlays (1)
 Micro-milling (1)	 Grooving (1)
 Shotblasting (1)	 Skidabrading (2)

Figure 10. Illustrations of various friction improving treatments
[sources: (1) Merritt et al. (2015), (2) Skidabrader (2022)]

2.7 References

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3 Stone-Matrix Asphalt

Stone-Matrix or Stone-Mastic Asphalt (SMA) is an asphalt concrete mixture developed in Germany in the 1960s to provide heavily trafficked roads with a durable, rut-resistant wearing course using a gap-graded aggregate structure and a modified asphalt binder at elevated asphalt contents. The SMA technology was introduced in North America in the early 1990s, and it is used mostly as a surface layer (upper 1.5 to 3 inches of the pavement) on high-traffic freeways (NAPA 2002).

The most commonly hot-mix asphalt (HMA) used in North America are dense-graded mixes. These mixes used a well-graded aggregate (even distribution of aggregate particles from coarse to fine) and asphalt binder. They are typically classified based on the nominal maximum aggregate size (NMS) of the aggregate in the mix. This is defined in the Superpave mix design system as, "one sieve size larger than the first sieve to retain more than 10 percent". Dense-graded mixes are considered the workhorse of HMA since they may be used effectively in all pavement layers, for all traffic conditions. Surface mixes typically have 4.75, 9.5 or 12.5 NMS (NAPA 2001).

SMA is a gap-graded HMA with a stable stone-on-stone skeleton held together by a rich mixture of AC, filler, and stabilizing agents such as fibers and/or asphalt modifiers. SMA is often considered a premium mix because of higher initial costs due to increased asphalt contents and the use of more durable aggregates. Cubical, low abrasion, crushed stone and manufactured sands are recommended because the mixture gains most of its strength from the stone-on-stone aggregate skeleton. The skeleton is held together by a mixture of manufactured sands, mineral fillers, and additives (fibers and polymers) that make a stiff matrix. Mineral fillers and additives also reduce the amount of asphalt drain down in the mix during construction, increasing the amount of asphalt used in the mix, improving its durability (NAPA 2001). Figure 11 illustrates the aggregate structure of an SMA mix compared with a conventional dense-graded mixture.

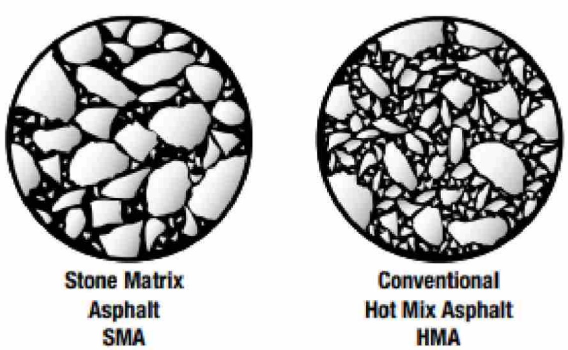


Figure 11. Comparison of the aggregate structure on conventional and SMA mixtures (NAPA 2002).

3.1 SMA Cost and Durability

The primary advantage of SMA is the extended life with improved pavement performance compared to conventional dense-graded hot-mix asphalt (HMA). Other reported advantages are noise reduction, improved frictional resistance, and improved visibility (NAPA 2002). SMA is designed to improve rut resistance and durability by using a stable stone-on-stone skeleton held together by a rich mixture of asphalt cement, along with stabilizing agents such as fibers and/or asphalt modifiers, as discussed in the previous section. SMA mixtures can also be used successfully in thin overlay and mill-and-fill resurfacing applications. For example, several districts in Virginia use SMA on most of their interstate highways.

The SMA mixes are typically more expensive (20%-25%) than the traditional HMA (NAPA 2002). The extra cost comes from the use of higher quality aggregates, more and typically more expensive polymer-modified binder, and more mineral filler than conventional mixtures. SMA mixtures also require adding fibers to stabilize the high quantities of binder and require higher mixing temperatures (because of the polymer-modified binders), which increases energy use during production. However, for high-traffic highways, the extra service life obtained because of the enhanced durability typically compensates for the extra cost.

McGhee and Clark (2007) reported that SMA outperformed dense-graded hot-mix asphalt in Virginia when placed in similar conditions. In most cases, the premium price for SMA is justified by the anticipated increase in performance. The researchers concluded that SMA was the most cost-effective hot-mix material for use in maintaining pavements on Virginia's interstate system. More recently, Yin and West (2018) reported increases in service life between 32% and 47% compared with traditional HMA mixes, designed using the Superpave methodology in some states; however, the SMA mixes did not produced higher service life in all the states investigated.

3.2 SMA Functional Properties

Several authors have also reported that SMA also has enhanced functional properties compared with traditional dense-graded asphalt. Data collected at the National Center for Asphalt Technology (NCAT) Test Track in Alabama showed that an SMA section provided a maximum 2 dB(A) reduction in noise and an approximately 15% increase in surface friction compared to the traditional dense-graded asphalt section with the same granite aggregates and styrene-butadiene-styrene (SBS) modified asphalt binder (Yin and West, 2018). The SMA section had better macrotexture and friction measured with a locked-wheel tester using a ribbed tire than a dense-graded mix with the same granite aggregate, as illustrated in Figure 12.

Early tests conducted at the Virginia Smart Road have also showed that an SMA section had higher macrotexture than the most common HMA mixes, designed using the Superpave methodology, used in the facility and slightly lower but similar friction (TPF 2016).

Similarly, a study in Japan also found that a high-performance SMA had improved frictional properties compared with a traditional dense-graded asphalt mixture (Tanaka and Maruyama 2018).

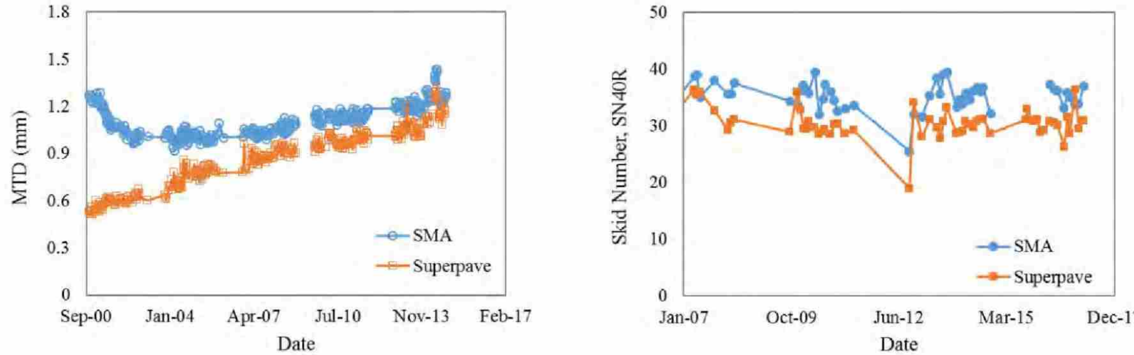


Figure 12 . Friction and Macrotexture Measurements over Time for SMA and Conventional HMA at the NCAT Facility (Yin and West, 2018)

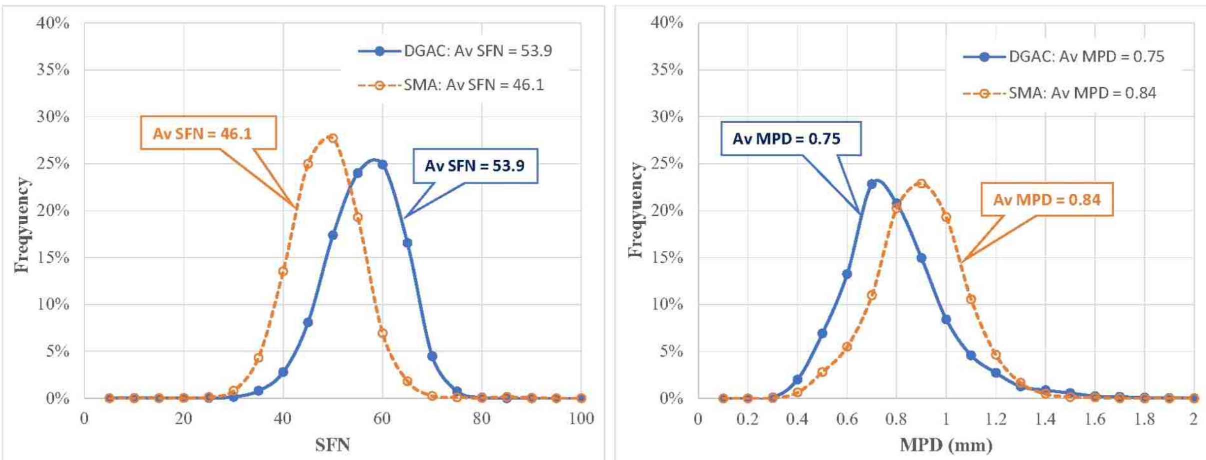
One potential concern with SMA surfaces is the potential low friction when the surface is new. McGhee et al. (2005) found that some of the SMA surface mixes placed in Virginia had relatively low early friction just after construction, but subsequent tests have shown a significant increase in available friction for all mix types.

Similarly, the European EAPA (2018) reported concerns in some countries that initial skid resistance during the first few weeks of trafficking may be lower than expected due to the thicker binder film on the surface compared to most other conventional asphalt types. However, the same publication also reported that European studies showed that SMA offered a sufficient skid resistance at this initial stage. Schreck (2004) reported that sand (often precoated with asphalt binder) is sometimes added to the surface of SMA in Germany and rolled in while it is hot. The construction practice is illustrated in Figure 13. This construction practice has also been used in the U.K. (Richardson 1999) and New Zealand (Baran and Lowe 2011).

(a) Application of Grit to SMA on Autoban 3, Near Passau, Germany	(b) Gritted (Foreground) and Non-Gritted SMA

Figure 13. Example of gritting of SMA in Germany (after Prowell et al, 2004)

A recent study to develop a pavement friction management program for the Corridors of State Significance in Virginia (de León Izeppi et al. 2021) collected network level data using a SCRIM system on road with different surfaces. Figure 14 compares the friction (SFN) and macrotexture (MPD) distributions for a sample of road in Virginia collected as part of this study . This plots show that SMA mixes have on average lower SCRIM friction (SFN, which reflects the pavement microtexture) but higher macrotexture than traditional dense graded mixes. It is notated that macrotexture is more critical to maintaining appropriate friction on wet pavements at high speed.



(a) Sideways Friction (b) Macrotexture

Figure 14. Comparison of SMA and dense-graded HMA friction and macrotexture properties for selected roads in Virginia

3.3 References

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APPENDIX B

LIST OF DOCUMENTS PROVIDED & REVIEWED

DOCUMENTS PROVIDED & REVIEWED		
DocID or Doc #	Document Date	Title
Inquiry Exhibit #3		RHVPI Overview Document #3: Construction of the RHVP
Inquiry Exhibit #3.1		RHVPI Overview Document #3.1: RHVP Design & Geometry
Inquiry Exhibit #4		RHVPI Overview Document #4: The Ministry of Transportation of Ontario and Friction Testing
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GOL0001735	7/27/2007 0:00	H-07-379 test strip ramp.pdf
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GOL0001726	8/7/2007 0:00	06-1181-230 SP12 FC2-H-07-414.pdf
GOL0001727	8/7/2007 0:00	06-1181-230 SP12 FC2-H-07-408.pdf
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GOL0000112	8/9/2007 0:00	06-1181-230 TBL26 2007 '08'02 SP 12,5 FC2 SA#H-07-416 07-652.pdf
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GOL0000160	8/17/2007 0:00	Image
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GOL0003091	8/17/2007 0:00	sta 26+450 low point connecting cross slopes.JPG
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GOL0003790	8/20/2007 0:00	FW: Mix Designs 12.5 FC2, HL3 HS (PW-06-243)
GOL0003791	8/17/2007 0:00	SP 12.5 FC2.pdf
GOL0003792	8/17/2007 0:00	INC - SP 12.5 Alternate Mix Design Review.pdf
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GOL0000229	8/20/2007 0:00	Image
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GOL0001668	8/20/2007 0:00	06-1181-230 TBL53 2007 '08'15 SP 12,5 FC2 SA#H-07-452 07-0721.pdf

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GOL0001671	8/21/2007 0:00	06-1181-230 TBL56 2007 '08'15 SP 12,5 FC2 SA#H-07-453 07-0724.pdf
GOL0001672	8/21/2007 0:00	06-1181-230 TBL57 2007 '08'14 SP 12,5 FC2 SA#H-07-451 07-0725.pdf
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GOL0002028	8/21/2007 0:00	SP 12.5 FC2 RHV - PW-06-243aug20-1.pdf
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GOL0000126	8/22/2007 0:00	06-1181-230 TBL65 2007 '08'20 SP 12,5 FC2 SA#H-07-466 07-0745.pdf
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GOL0000128	8/29/2007 0:00	06-1181-230 TBL77 2007 '08'27 SP 12,5 FC2 SA#H-07-480 07-0802.pdf
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GOL0005378	10/8/2007 0:00	Extraction and Gradation Work Sheet MTO LS-282 Sample # A 07 689
GOL0005233	10/13/2007 0:00	Gyratory Density and Air Voids Lab # A 07 1060
GOL0005222	10/15/2007 0:00	Superpave Extraction and Gradation Work Sheet MTO LS-282 Sample # A-07-1090
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GOL0005224	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1090-1

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GOL0005229	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1089-1
GOL0005230	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1089-2
GOL0005231	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1089-2
GOL0005234	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1060-1
GOL0005235	10/19/2007 0:00	Troxler 4141 Gyratory Compactor Sample ID 1060-2
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GOL0005232	10/22/2007 0:00	Superpave Extraction and Gradation Work Sheet MTO LS-282 Sample # A-07-1060
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GOL0000129	10/22/2007 0:00	06-1181-230 TBL89 2007 '10'11 SP 12,5FC2 07-1089.pdf
GOL0000130	10/22/2007 0:00	06-1181-230 TBL90 2007 '10'11 SP 12,5FC2 07-1090.pdf
MTO0000042	12/13/2007 12:00	Re: Approval of Your Varennes Quarry for SP 12.5 FC1 Coarse and SP 12.5 FC2 Coarse and Fine Aggregates
MTO0000043	12/13/2007 0:00	Table 1 Laboratory Test Data Demis Agregats Varennes, Quebec
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MTO0024003	6/18/2008 0:00	
MTO0024004	6/18/2008 0:00	
MTO0024005	6/18/2008 0:00	
RHV0000593	8/7/2008 8:38	Sustainable Pavements--Making the Case for Longer Design Lives for Flexible Pavements
MTO0000044	12/4/2008 12:00	Re: Approval of your Varennes Quarry for SP 12.5 FC1 Coarse and SP 12.5 FC2 Coarse and Fine Aggregates
MTO0000045	12/4/2008 0:00	Table 1 Laboratory Test Results Demix Agregats Varennes, Quebec
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MTO0005229	5/8/2009 15:18	
MTO0005230	5/8/2009 15:18	
MTO0005231	5/8/2009 15:18	
MTO0005232	5/8/2009 15:18	
MTO0005228	5/8/2009 15:18	
RHV0000588	9/8/2009 11:54	Construction of Durable Longitudinal Joints--The Courage to Use Innovations Pays Off
MTO0034018	4/1/2010 0:00	
MTO0034019	4/1/2010 0:00	
MTO0034020	4/1/2010 0:00	
MTO0034021	4/1/2010 0:00	
MTO0034022	4/1/2010 0:00	
MTO0007198	5/26/2011 11:19	
MTO0007199	5/26/2011 11:19	
MTO0007200	5/26/2011 11:19	
MTO0007201	5/26/2011 11:19	
MTO0007202	5/26/2011 11:19	
RHV0000594	11/11/2011 14:47	Using Instrumentation Data on an Active Highway For Pavement Management
MTO0007828	4/12/2012 14:49	
MTO0007829	4/12/2012 14:49	
MTO0007830	4/12/2012 14:49	
MTO0007831	4/12/2012 14:49	
MTO0007832	4/12/2012 14:49	
RHV0000591	5/18/2012 10:55	Instrumentation in RHVP Providing Data for Long Term Pavement Management
GOL0004440	1/1/1900 0:00	None
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TRW0000092	1/21/2014 17:50	HamiltonRoads
GOL0006588	1/24/2014 0:00	Friction numbers on RHVP
GOL0002620	10/17/2007 0:00	RedHill S1.xls
GOL0002621	10/17/2007 0:00	RedHill S2.xls
GOL0006591	1/24/2014 0:00	2009-06 Ponniah-Tam-Dziedziejko-Dhillon-Brown Early Age Low SMA Friction.pdf
GOL0001113	1/26/2014 13:40	Hamilton_LA-RHV_Rev2
GOL0002981	1/31/2014 0:00	13-1184-0026 Draft 31'January'14.pdf
MTO0009307	7/25/2014 15:12	
MTO0009308	7/25/2014 12:36	
MTO0009309	7/25/2014 12:51	
MTO0009310	7/25/2014 13:35	
MTO0009311	7/25/2014 14:10	
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GOL0003888	12/17/2015 0:00	FHWA-Assessment of Friction-Based Pavement Methods and Regulations_1177874969.pdf
GOL0001457	12/11/2017 0:00	Page 1 of 8.pdf
GOL0001458	12/11/2017 0:00	Page 2 of 8.pdf
GOL0001459	12/11/2017 0:00	Page 3 of 8.pdf
GOL0001460	12/11/2017 0:00	Page 4 of 8.pdf
GOL0001461	12/11/2017 0:00	Page 5 of 8.pdf
GOL0001462	12/11/2017 0:00	Page 6 of 8.pdf
GOL0001463	12/11/2017 0:00	Page 7 of 8.pdf
GOL0001464	12/11/2017 0:00	Page 8 of 8.pdf
HAM0054182_0001	12/18/2018 0:00	
GOL0005769	2/15/2020 0:00	18100695 Draft Report RHVP HIR Study - Dec 21, 2018.pdf
HAM0013587_0001	2/20/2019 12:00	2017 Hamilton Collision Report
HAM0012715_0001	2/5/2019 0:00	
HAM0054585_0001	2/13/2019 0:00	
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HAM0054587_0001	6/29/2007 0:00	
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CIM0017101.0001	2/25/2019 14:47	B0001014B_Model Results_e00v02
CIM0017099.0001	2/25/2019 16:03	B0001014B_Model Results_e00v03
HAM0028974_0001	2/26/2019 0:00	
HAM0036336_0001	2/26/2019 0:00	
HAM0029042_0001	3/1/2019 0:00	Microsoft Word - 1791724 RPT Evaluation of Pvmt Surface and Aggregates on RHVP February'2018.docx
GOL0006583	11/21/2019 0:00	18100695 RPT HIR Feasibility Strudy for RHVP March 11 2019.pdf
RHV0000585	6/28/2019 15:15	'Perpetual' pavement helps Hamilton meet goals for Red Hill Valley Project _ Rock To Road
HAM0009626_0001	9/10/2019 0:00	Report - Template Report (recto)
HAM0009627_0001	9/11/2019 0:00	
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HAM0009629_0001	9/11/2019 0:00	
HAM0009630_0001	9/11/2019 0:00	
HAM0009631_0001	9/23/2019 0:00	
HAM0009632_0001	10/15/2019 0:00	
HAM0009633_0001	11/14/2019 0:00	
HAM0009634_0001	11/14/2019 0:00	
HAM0009635_0001	11/15/2019 0:00	
HAM0009636_0001	11/15/2019 0:00	
HAM0009637_0001	11/15/2019 0:00	
RHV0000597	12/3/2019 10:40	2018 Hamilton Collision Report
HAM0009638_0001	2/18/2020 0:00	

DOCUMENTS PROVIDED & REVIEWED		
DocID or Doc #	Document Date	Title
HAM0009639_0001	2/18/2020 0:00	
HAM0009640_0001	2/18/2020 0:00	
HAM0009641_0001	2/18/2020 0:00	
CIM0022143	5/28/2020 8:43	B001173_Hamilton_RHVP Analysis_Final_E04
RHV0000609	5/29/2020 15:47	4.3 City of Hamilton 2019 Annual Collision Report
CIM0022320	6/1/2020 10:36	B001211_Hamilton_RHVP Friction Study_e04
RHV0000590	7/24/2020 18:28	Innovative, Comprehensive Design and Construction of Perpetual Pavement on the Red Hill Valley parkway in Hamilton
RHV0000595	7/24/2020 18:33	Verification of Pavement Design Metholodogies Using Measured In-Situ Response on an Urban Highway
RHV0000908	2/12/2019	
RHV0001001		
HAM0064331_0001	2/4/2019 0:00	
RHV0001024		Affidavit of Ludomir Uzarowski, affirmed September 30, 2022
MTO0022943	7/25/2014	RedHillValleyPkwy N1_SMA_DSM_MUN.xls
MTO0022944	7/25/2014	RedHillValleyPkwy N2_SMA_DSM_MUN.xls
MTO0022945	7/25/2014	RedHillValleyPkwy S1_SMA_DSM_MUN.xls
MTO0022946	7/25/2014	RedHillValleyPkwy S2_SMA_DSM_MUN.xls
RHV0000889	6/17/2021	RHVPI, Letter from Tradewind Responding to May 31 letter re CIMA Reports....PDF