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Selection of Pavement Friction Test Sites Using Collision Profiling

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Publication Title	**Selection of Pavement Friction Test Sites Using Collision Profiling**

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Abstract

The objective of this pilot was to determine if a relationship can be established between tested pavement friction, wet weather collision rate and or pavement condition indices.

Some 43 sites, mostly two lane highways, were identified in Ontario's Eastern Region, where the ratio of wet-to-dry collisions during period 2001 – 2003 equalled or exceeded 0.45. The 0.45 ratio value was selected as an arbitrary cut-off, being 50% above the provincial average of 0.3. Some 12 sites had to be dropped from the pilot test list, since they were resurfaced since 2001. Overall, some 8% of the regional road network was friction tested.

The 31 pilot sites were tested using the ministry standard equipment, in accordance with ASTM E274 and ASTM E501. Results were correlated against a number of parameters extracted from either the collision database or Ontario's Pavement Management System (PMS2).

The pilot determined that good correlation exists between wet-to-dry ratio and friction for undivided highways. Using wet-to-dry ratio of greater than 0.45 to short listed sections targeted for friction testing yielded a 40% "success" rate in the test producing low friction values, compared with 30% success rate for the current, visual inspection method, that mostly picks up flushing, but only rarely polishing. It is therefore recommended that the wet-to-dry collision ratio be used to augment the list of sites where friction testing is annually requested.

Unfortunately, no correlation was found between friction values determined by testing, pavement condition or collision information, so that the above shortlist (derived from high wet-to-dry collision ratio) could be reduced or prioritised. In order not to overtax available friction testing resources, the site request list will have to be manually prioritised, based on site tactile inspection and / or surface course aggregate test and friction performance data.

Key Words	Pavement friction, targeted network friction monitoring, test site selection, wet surface, wet-to-dry collision ratio, collision rate, polishing, flushing
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Executive Summary

For several decades, the MTO has been using a pro-active, front-end approach to pavement friction management through pre-approval of aggregate sources and policy over the selection of surface courses. Human error and poor construction practices have been relatively rare and this approach can be credited for generally good friction performance of our roads. For this reason, annual friction testing of the entire network, or a random sampling of significant part of it, would not be cost-effective in Ontario.

Pavement friction changes with time and exposure to traffic. The Ministry conducts targeted network friction monitoring and tests all road segments that are visually identified during the annual network pavement condition rating surveys as flushed (linked to lower friction levels). Polishing, which is not typically visible, is often missed, unless the OPP draws attention to a high skidding collision location.

The objective of this pilot was to determine a method to improve the regular annual friction testing to include sites where surface polishing is suspected, based on a high wet weather collision rate (collision profiling).

Thirty-one sites in Eastern Region, mostly two lane highways, were slated for friction testing based on the ratio of wet-to-dry collisions being 50% above the provincial average of 0.3. About 8% of the Eastern Region's road network was subsequently tested using standard Ministry equipment.

In the pilot, 40% of the short listed sites had low friction incidence, compared with a 30% incidence represented by the current visual identification method. It is therefore recommended that the collision profiling be used to augment the visual site inspection method as an integral part of the ministry's targeted network friction testing.

After the testing, an analysis was performed to determine if, based on traffic and/or pavement condition parameters, some of the 31 short listed sites could have been eliminated ahead of friction testing. A relationship between routinely collected parameters and friction was not found, thus; short listed sites will require either tactile site inspections or modelling of the surface course aggregate performance.

It was found that most low friction areas were on full width patches with hot mixes containing limestone, contrary to the current Ministry policy. Maintenance staff must ensure that limestone is not used on high traffic volume road surfaces.

If U.S. practice were to be emulated here, unscheduled friction restoration would be carried out on the pilot's 47.8 km (2.1% of the Regional network), in addition to the scheduled annual resurfacing, that is -typically performed on 7-8% of the network (12 – 15 year pavement cycle).

Disclaimer

This report does not constitute a Ministry standard or policy on pavement friction network testing, site investigation or pavement friction restoration.

If a reference is made to low pavement surface friction, or friction number ranges used by several U.S. jurisdictions to trigger a site investigation, these references must not be interpreted as trigger values for a friction restorative surface treatment or as a warrant for investigations in Ontario. These are yet to be established in near future. Each individual road segment requires a site-specific assessment so that an informed engineering decision can be made on whether a new surface treatment is appropriate, its form and timing. Such assessment is based on friction demand (in terms of likelihood drivers will use their brakes and horizontal and vertical alignment of the highway), wetness exposure (in terms of precipitation and surface drainage), collision consequences (in terms of level of service, traffic speed and roadside forgiveness) and pavement management (in terms of pavement condition rating and age).

The intention of the pilot was not to test a random, representative sample of Eastern Ontario provincial highways, but an intentionally biased sample of highways with the rate of wet weather collisions 50% and higher over the Provincial average. The relatively high incidence of low friction thus validates the methodology of test site selection, rather than representing general friction levels of Ontario highways.

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Introduction

For several decades the MTO has been using a pro-active, front-end approach to pavement friction management. This is accomplished through pre-approval of aggregate sources designated as suitable for use on Ministry contracts [Appendix A] and policy over the selection of surface courses on all roads and highways under ministry jurisdiction [Appendix B]. Human error and poor construction practices have been relatively rare and this approach can be credited for generally good friction performance of Ministry roads. By contrast, many U.S. jurisdictions use relatively weak front-end measures to ensure adequate friction and instead rely on post-construction friction testing and, or network-wide friction testing.

Pavement friction changes with time. The relatively gradual changes are related to choice of aggregate, design mix type, aggregate properties, site drainage, exposure to traffic and the level of winter sanding. As a method of detecting when friction levels drop below the level expected by the travelling public and demanded by traffic, the Ministry routinely tests all road segments that are identified as potentially friction deficient, during the annual visual network pavement condition rating surveys. In addition, newly constructed pavement surfaces are tested when low friction is suspected or when a request is received from the Ontario Provincial Police or the Provincial Coroners.

It is therefore feasible that some sites with low friction may be overlooked, since the annual scanning process is visual and involves one or more slow speed passes of each road segment. While asphalt flushing, which is associated with low friction, is typically identified, aggregate surface polishing is harder to spot from a moving vehicle and can be missed.

Since tire friction is lower on wet pavement, most experts agree that a higher collision rate is generally expected during wet surface conditions than during dry conditions. Focusing on wet weather collisions may be effective in identifying sites where low friction could be a contributing causative factor and where friction testing may be warranted.

The primary objective of this pilot is to determine if a correlation can be found between collision data parameters and low friction incidence. If good correlation is found, the method can be used to supplement the existing targeted pavement friction network monitoring. Additional test sites can be identified based on unusually high wet weather collision rate, yet low enough overall collision rate as not to appear on the radar screen of Traffic Sections during scrutiny of sites with above average collision rate.

The sites selected for testing based on collision information are of interest even when friction testing determines that the friction is not a concern. Such sites can be referred to the Regional Traffic Section for further investigation. This analysis may determine causes, other than low friction, to account for the unusually high incidence of wet weather collisions.

A similar program to analyse collision data for above average representation of wet pavement collisions was in operation at MTO in the late 70's [Kamil Nabil, 1980]. The main difference was that it required that short-listed sites undergo first a field inspection by the Traffic Section, followed by friction testing. Since the latter is more cost effective, if this pilot is successful, it is expected that friction shall be measured first, to eliminate it as a causative factor, and a field investigation may follow only when other causative factors are suspected.

Eastern Region Geotechnical Section volunteered to facilitate this pilot project. The success of the pilot may lead to better identification of low friction highway segments in areas of high friction demand Province-wide. By identifying such sites, and a treatment where required, the overall collision rate is likely to be reduced.

Approach

The intent of this study was to investigate the wet road surface collision rate during summer conditions, thereby eliminating the effect of winter conditions on pavement friction. Collisions where the road surface condition was listed as ice, loose snow, slush or packed snow were screened out so that the analysis focussed only on records where the pavement condition was recorded as bare-wet or bare-dry.

The Ministry's Eastern Region road network was used as the base for the pilot study. The Accident Information System (AIS) was used as the source of collision data. Head Office Traffic also supplied the Average Annual Daily Traffic (AADT) data.

As it was not practical to skid trailer test the entire Eastern Region road network, a parameter was needed to dictate whether a road segment would be included in the correlation analysis. Literature search [Seiler-Scherer, 2004 and Viner, 2005] reported that wet-to-dry or wet-to-total collision ratio was linked to incidence of low pavement friction.

Several U.S. jurisdictions use the ratio of wet-to-total collisions in network friction analyses. This approach was deemed inappropriate under Ontario conditions - where the winters are long and the inclusion of collisions attributed to winter road surface conditions, such as snow, ice or slush would distort the results and thereby introduce weather-based inequities between regions.

The Ontario Road Safety Annual Reports from 2001 to 2003 established that the Ontario provincial average of wet-to-dry collisions was roughly 0.3. Based on this research, it was arbitrarily decided that road segments with a wet-to-dry collision ratio exceeding 0.45 would be selected for inclusion in this pilot study, 0.45 being 50% above the provincial average or 0.3. The ratio 0.45 also corresponds to the wet / (wet +dry) ratio of 0.3 used in an earlier MTO program as a cut-off [Kamel, 1980].

Bare-wet and bare-dry collisions was therefore extracted for each road segments in the Linear Highway Referencing System (LHRS) to support the use of the wet-to-dry collision ratio to screen out sites most likely to yield low friction test values.

Pavement treatment historical data for the short listed sites were analysed and sites that had been resurfaced, either partially or fully, since 2001 were eliminated from the pilot. Friction changes are most dramatic in the first 5 years after the surface course is laid and typically stabilizes after the first couple of years; therefore, friction values measured during the pilot are assumed to be representative of the friction conditions during the pilot's three year collision data period, spanning 2001, 2002 and 2003. At sites where minor pavement preservation treatments were encountered, such as like rout and seal operations, the friction testing was performed in areas unaffected by the treatment.

The cumulative collision data for the Region was extracted for each site from the Accident Information System (AIS). Loss-of-control, single vehicle run-off-the-road and rear-end collisions are often linked to low pavement friction; however, these collision types are relatively rare and restricting the study to this narrow collision subset was deemed not sufficient to yield meaningful results. By similar rationale, the extracted collision numbers include all collision types and all collision severities (fatal, injury and property damage only - PDO). This approach was deemed reasonable, since it was not the pilot's purpose to use cost-benefit basis for short-listing test sites, where the cost of a fatality is the sole decisive factor.

Since meteorological information could not be ascertained with available resources, it was assumed that during the collision period, the pavement's wet time to dry time ratio was constant within the Region. Based on this assumption, the wet-to-dry collision rate would represent the relative risk of having a wet pavement collision relative to a dry pavement collision. This assumption is considered reasonable within the relatively small geographic area of a Region.

Not all loss-of-control and run-off-the-road accidents are reported. The number of unreported collisions is not expected to be a factor in the analysis and will not be considered. In the absence of research, it will be assumed that the percentage of unreported accidents would be the same for wet and dry condition collisions.

Research indicates that friction values vary slightly for the same pavement and surface condition within any given year. This is attributed to the pavement temperature at the time of testing, prolonged dry or wet periods and winter operations sanding / Spring cleanup. Typically, the Friction Number changes by approximately 1 for every 10^0 C [Yingjian, 2003]. Since all the sites were tested within a few days of each other and the temperature from one site to another differed by less than 5^0 C, the effect of friction value seasonal fluctuation was ignored in the pilot.

Sites that were rehabilitated after 2001 were naturally excluded from further analysis. It was also considered to eliminate sites with relatively few wet pavement surface collisions. This was rejected, which proved to be a correct decision, as discussed later.

The road segments that remained in the study varied from 0.6 km to 12.2 km, with the mean length being 6.2 km. Skid measurements were taken at manually established intervals ranging from 300m on shorter sites to 700 metres on longer sites.

Data

Head Office Traffic Section provided collision data collected between 2001 and 2003 (Table 1). All 34 sites shown had wet-to-dry collisions ratios greater or equal to 0.45. The majority of sites identified in the pilot were two lane undivided highways.

The data collection process involved excluding sites that had experienced rehabilitation after 2001. This limited the number of sites to be reviewed. Originally 43 sites were identified as having met the wet-dry collision ratio value, however, 9 of these sites were removed from the data set as they had undergone full or selective resurfacing in the last 4 years.

Three sites (Site ID's 21, 25, 27) were removed from the pilot because they were undergoing reconstruction work not identified prior to friction testing. The 31 sites left in the pilot represented roughly 8% of the Eastern Region pavement network.

The reliability of construction and maintenance records was poor; therefore, minor pavement surface treatments may have occurred over the course of 2001-2003. To be on the safe side, friction testing was conducted always outside such repairs, such as pothole patching and rout and seal, even though some of these may have been present during the collision data collection period.

Table 1 – Traffic Data

Site ID	Highway Class	Length (km)	AADT	No. Collisions 2001-2003			Collision Ratio		Collision rate per 100 Million km travelled	
				Wet	Dry	Total	Wet/Dry	Wet/Total	Wet	Wet + Dry
1	Freeway	3.8	41400	18	40	97	0.45	0.19	10.45	33.67
2	Arterial	1.5	9200	2	4	7	0.50	0.29	13.24	39.71
3	Arterial	6.4	4900	7	11	29	0.64	0.24	20.38	52.42
4	Arterial	0.6	16500	7	14	21	0.50	0.33	64.57	193.72
5	Arterial	7.7	5600	9	17	39	0.53	0.23	19.06	55.07
6	Arterial	5	22000	2	4	7	0.50	0.29	1.66	4.98
7	Arterial	1.4	11300	6	11	18	0.55	0.33	34.64	98.14
8	Arterial	3.1	4450	3	6	14	0.50	0.21	19.86	59.58
9	Arterial	10.6	4600	19	40	77	0.48	0.25	35.59	110.50
10	Arterial	11.3	2950	5	11	16	0.45	0.31	13.70	43.83
11	Arterial	10.1	2250	9	11	26	0.82	0.35	36.17	80.37
12	Arterial	10	2550	10	17	30	0.59	0.33	35.81	96.70
13	Arterial	9.8	3500	3	4	11	0.75	0.27	7.99	18.64
14	Local	7.4	390	1	1	2	1.00	0.50	31.64	63.29
15	Freeway	1.4	27800	5	10	20	0.50	0.25	11.73	35.20
16	Arterial	9.9	1500	6	7	19	0.86	0.32	36.90	79.95
17	Arterial	12.2	1400	7	11	26	0.64	0.27	37.43	96.24
18	Arterial	3.5	1400	2	3	7	0.67	0.29	37.28	93.19
19	Arterial	3.8	1400	2	3	7	0.67	0.29	34.33	85.83
20	Arterial	8.6	3700	6	9	30	0.67	0.20	17.22	43.05
21	Arterial	0.2	3200	1	1	2	1.00	0.50	142.69	285.39
22	Collector	4.2	3000	4	7	14	0.57	0.29	28.99	79.73
23	Arterial	8.4	5700	23	16	48	1.44	0.48	43.87	74.39
24	Arterial	3.5	4500	5	3	8	1.67	0.63	28.99	46.39
25	Local	9.4	11000	1	2	6	0.50	0.17	0.88	2.65
26	Collector	11.3	5400	4	7	15	0.57	0.27	5.99	16.46
27	Arterial	3	20200	2	1	7	2.00	0.29	3.01	4.52
28	Arterial	1.3	2500	3	5	8	0.60	0.38	84.30	224.80
29	Arterial	5.2	600	2	3	8	0.67	0.25	58.54	146.35
30	Arterial	8	5200	8	17	30	0.47	0.27	17.56	54.88
31	Arterial	8	5500	6	13	24	0.46	0.25	12.45	39.44
32	Collector	1.6	5500	2	4	8	0.50	0.25	20.76	62.27
33	Collector	9.5	1350	4	8	16	0.50	0.25	28.48	85.45
34	Collector	4.2	2700	4	3	8	1.33	0.50	32.21	56.37

In this study, the Friction Number (FN) is defined as the ratio of the horizontal force acting on the fully locked wheel to the vertical force acting on the wheel. In some jurisdictions, the friction number is also known as the Skid Number (SN). The equipment used to test friction in the field were ASTM E274 (Skid Resistance of Paved Surfaces Using a Full-Scale Tire) and ASTM E501 (Standard Rib Tire for Pavement Skid Resistance Tests). The testing was conducted at the posted speed, which in most cases was 80km/hr.

The test locations were chosen manually between 300 and 700 metres apart, depending on the variability of the pavement surface (the greater the variability, the more frequently the testing was performed). The test equipment operator avoided performing the test over any highly localised features, such as sealed cracks or an expansion joints.

Typically only one direction was tested – that with the more pronounced downhill gradient, where it was assumed that a higher portion of the collisions had taken place. It was further assumed that the directional traffic and the percentage of commercial vehicles were balanced in both travel directions.

Adjustments were not made to Friction Number (FN) values to account for seasonal friction variation because all the testing was carried out under similar conditions, in less than two weeks.

The results of the friction testing are presented in Table 2. For each site several friction test readings were taken and the FN80 value is the average reading. For ease of interpretation, the results were segregated into three ranges, corresponding to typical U.S. practice, and colour coded, for easier distinction and manipulation (Figure 1). The friction management practice in each U.S. state will be soon documented in the NCHRP report, based on TRB Research Project 1-43 "Pavement Friction Guide", that was due for completion by the end of February 2006.

Friction Range	**Typical U.S. practice**	**Identification Code**
$FN_{PS} \leq 25.9$	Detailed Site Investigation is Warranted	Red
$26 \leq FN_{PS} \leq 30.9$	Review Collision History and Monitor Site	Yellow
$FN_{PS} \geq 31$	No Immediate Action Warranted	Green

Figure 1 – Friction Number Colour Coding

Next, a concept of "critical site-segment" was introduced, using the following convention:

- Where any test reading within the site belonged to the first range (coded red), then the critical site-segment became also red;
- Whenever any test reading within the site belonged to the middle range (coded yellow), then the critical site-segment became also yellow; and lastly;
- When all readings of the site were in the third range (coded green), then the critical site-segment became also green.

Subsequently, all test sites were sorted to place those with critical site-segment coded red at the top of the list, followed by yellow coded critical site-segments, and lastly green coded segments.

The next step was to prioritise the sites within their respective colour groups. To do so, a sort was performed based on a construct parameter called "Exposure Factor", derived as product of the critical site segment FN value, site-segment length, and the Average

Annual Daily Traffic (AADT). The sites each received a ranking from 1 to 31, in reflection of the above sort.

When low friction is localised to a short road segment, it is a worse than when low friction is uniform throughout the segment length or the road network. Drivers may receive a false sense of security while routinely braking over an area with higher friction and increase their speed. Should they subsequently encounter a portion of roadway with localised low friction, their speed may be too high to safely negotiate a sharp curve, or perform emergency braking. This rational explains why the above approach of using critical site-segments and prioritisation by exposure factor was adopted. Alternatively, if the test data for each site were averaged, for all sites with at least one test reading coded red or yellow (Table 1), the average site FN comes to FN = 30. This would lead to a misleading impression that the friction is within tolerable bounds.

To investigate the relationship between high incidence of wet weather collisions, friction and other pavement surface characteristics, the following key geotechnical parameters were extracted from the MTO Pavement Management System (PMS2) - Table 3:

- International Roughness Index (IRI);
- Distress Manifestation Index (DMI);
- Pavement Condition Index (PCI); and,
- Ride Condition Index.

Unfortunately, pavement texture (both the macro, or the micro) data was unavailable and couldn’t be investigated as a parameter in this study.

Table 2 - Friction Value Variability

Site ID	Total Tested Length	Site-segment 1		Site-segment 2		Site-segment 3		Site-segment 4		Site-segment 5		Site-segment 6		Site-segment 7		Site-segment 8	
		Length	FN80	Length	FN80	Length	FN80	Length	FN80	Length	FN80	Length	FN80	Length	FN80	Length	FN80
26	11.3	3.7	23.0	2.4	37.3	2.2	27.2	3.0	36.7								
30	8	0.9	20.4	0.7	30.4	0.8	22.9	1.5	35.1	1.4	22.7	0.7	31.4	2.0	28.3		
9	10.6	2.8	35.7	0.7	22.3	0.4	30.3	1.7	22.6	3.0	40.2	1.4	26.0	0.6	42.0		
7	1.4	1.4	24.6														
5	7.7	2.5	21.8	5.2	42.0												
12	10	3.2	22.8	4.8	46.6	2.0	23.6										
34	4.2	2.1	24.3	0.6	26.3	1.2	24.3	0.3	26.0								
22	10.2	3.8	24.8	2.0	34.3	4.4	24.3										
19	3.8	3.8	21.1														
18	3.5	0.7	28.4	1.9	24.4	0.3	28.2	0.6	25.0								
20	8.6	0.6	22.4	8.0	48.3												
33	9.5	4.3	38.5	5.2	25.0												
23	8.4	8.4	29.7														
3	6.4	2.1	27.6	0.4	25.3	3.9	27.6										
11	10.1	0.7	39.1	4.7	28.3	1.1	25.1	1.1	32.5	2.5	25.7						
17	12.2	5.9	43.3	3.4	29.5	1.9	24.7	1.0	31.8								
2	1.5	0.1	33.7	0.5	30.1	0.9	35.2										
31	15	4.5	32.3	1.1	28.8	1.9	36.7	1.7	26.5	3.3	31.8	1.1	30.8	1.4	32.1		
29	5.2	0.6	37.1	0.1	28.4	0.3	32.8	0.2	28.2	1.9	33.5	0.6	29.4	1.3	34.1	0.2	28.6
1	3.8	3.8	42.0														
6	5.0	5.0	31.0														
15	1.4	1.4	52.4														
13	9.8	9.8	41.0														
10	11.3	11.3	40.3														
24	3.5	3.5	37.2														
16	9.9	9.9	44.6														
8	3.1	3.1	38.5														
4	0.6	0.6	45.7														
32	1.6	1.6	53.8														
28	1.3	1.3	37.6														
14	7.4	7.4	47.6														

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Table 3 - Friction and Potentially Related Parameters

Site ID	Surface Type	Length (km)	AADT	Collisions 2001-2003			Ratios		Collision Rate per 100 million km driven		Critical Site-segment		Site Mean	Exposure Factor	Exposure Rank	Pavement Condition			
				Wet	Dry	Total	Wet/Dry	Wet/Total	Wet	Wet + Dry	FN80	% Site	FN80			IRI	RCI	DMI	PCI
26	HL-4	11.3	5400	4	7	15	0.57	0.27	5.99	16.46	24.0	48%	30.6	29289.6	1	1.7	6.9	8.4	76.7
30	HL-3	8	5200	8	17	30	0.47	0.27	17.56	54.88	21.6	44%	26.6	18304.0	2	2.1	6.1	7.2	61.9
9	HL-4	10.6	4600	19	40	77	0.48	0.25	35.59	110.50	23.8	37%	31.3	18041.2	3	2.1	6.1	5.6	47
7	HL-1	1.4	11300	6	11	18	0.55	0.33	34.64	98.14	24.6	100%	24.6	15820.0	4	1.9	6.5	6.2	54.6
5	HL-1	7.7	5600	9	17	39	0.53	0.23	19.06	55.07	21.8	32%	29.1	13798.4	5	2.2	6.2	6.1	51.4
12	HL-4	10	2550	10	17	30	0.59	0.33	35.81	96.70	23.0	47%	30.1	11985.0	6	1.1	8.1	8.9	85.8
34	RHM	4.2	2700	4	3	8	1.33	0.50	32.21	56.37	24.8	100%	24.8	11340.0	7	1.3	7.7	9.3	87.9
22	HL-4	4.2	3000	4	7	14	0.57	0.29	28.99	79.73	24.6	81%	26.5	10206.0	8	1.6	6.9	7.3	66.6
19	HL-4	3.8	1400	2	3	7	0.67	0.29	34.33	85.83	21.1	100%	21.1	5320.0	9	1.4	7.4	8.3	77.3
18	HL-4	3.5	1400	2	3	7	0.67	0.29	37.28	93.19	25.8	100%	25.8	4900.0	10	2.1	6.1	8.0	69.2
20	HL-4	8.6	3700	6	9	30	0.67	0.20	17.22	43.05	22.4	7%	45.1	2227.4	11	1.4	7.4	7.6	71.0
33	HL-4	9.5	1350	4	8	16	0.50	0.25	28.48	85.45	25.0	8%	36.8	1026.0	12	2.5	5.6	6.3	50.8
23	RHM	8.4	5700	23	16	48	1.44	0.48	43.87	74.39	29.7	100%	29.7	47880.0	13	1.7	6.8	7.2	65.4
3	RHM	6.4	4900	7	11	29	0.64	0.24	20.38	52.42	27.5	100%	27.5	31360.0	14	1.4	7.5	7.7	72.5
11	HL-4	10.1	2250	9	11	26	0.82	0.35	36.17	80.37	27.1	85%	28.7	19316.3	15	1.6	7.1	7.9	72.6
17	HL-4	12.2	1400	7	11	26	0.64	0.27	37.43	96.24	28.5	50%	33.9	8540.0	16	1.6	7.1	8.5	77.2
2	HL-4	1.5	9200	2	4	7	0.50	0.29	13.24	39.71	30.7	40%	33.3	5520.0	17	1.5	7.2	9.1	83.7
31	HL-3	8	5500	6	13	24	0.46	0.25	12.45	39.44	26.5	12%	31.6	5280.0	18	0.9	8.9	9.1	89.4
29	SC/HL-4	5.2	600	2	3	8	0.67	0.25	58.54	146.35	29	20%	32.6	624.0	19	2.3	7.7	5.9	55.4
1	HL-1	3.8	41400	18	40	97	0.45	0.19	10.45	33.67	42.0	100%	42	157320.0	20	1.1	8.2	8.7	83.7
6	HL-1	5	22000	2	4	7	0.50	0.29	1.66	4.98	31.0	100%	20%	110000.0	21	1.3	7.7	9.0	84.9
15	DFC	1.4	27800	5	10	20	0.50	0.25	11.73	35.20	52.4	100%	52.4	38920.0	22	0.4	9.3	9.2	90.5
13	HL-4	9.8	3500	3	4	11	0.75	0.27	7.99	18.64	41.0	100%	41	34300.0	23	1.9	6.4	7.6	67.4
10	HL-4	11.3	2950	5	11	16	0.45	0.31	13.70	43.83	40.3	100%	40.3	33335.0	24	1.1	8.3	8.9	85.9
24	HL-4	3.5	4500	5	3	8	1.67	0.63	28.99	46.39	37.2	100%	37.2	15750.0	25	1.3	7.6	8.6	81.4
16	HL-4	9.9	1500	6	7	19	0.86	0.32	36.90	79.95	44.6	100%	44.6	14850.0	26	1.0	8.4	9.3	89.4
8	HL-1	3.1	4450	3	6	14	0.50	0.21	19.86	59.58	38.5	100%	38.5	13795.0	27	1.0	8.7	9.9	95.9
4	HL-1	0.6	16500	7	14	21	0.50	0.33	64.57	193.72	45.7	100%	45.7	9900.0	28	1.4	7.5	6.4	61.1
32	HL-1	1.6	5500	2	4	8	0.50	0.25	20.76	62.27	53.8	100%	53.8	8800.0	29	1.6	7.0	7.6	70.0
28	HL-4	1.3	2500	3	5	8	0.60	0.38	84.30	224.80	37.6	100%	37.6	3250.0	30	0.9	8.8	9.2	89.4
14	ST	7.4	390	1	1	2	1.00	0.50	31.64	63.29	47.6	100%	47.6	2886.0	31	2.5	6.7	6.7	60.8

The photos below illustrate several “red-coded” sites (Figures 2, 3 and 4).

Figure 2 - Site 23 Slight Coarse Aggregate Loss

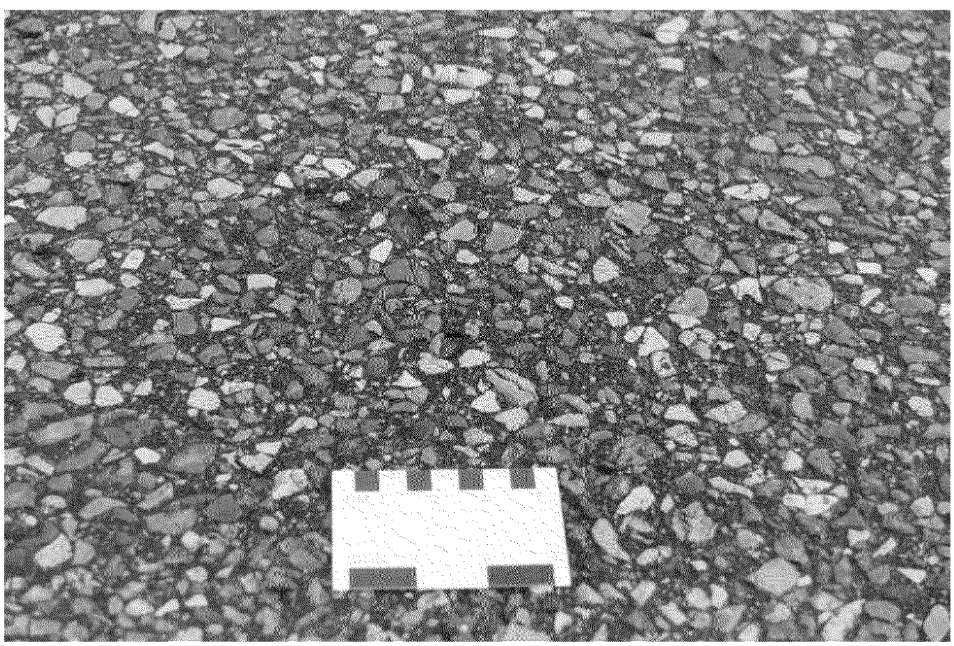
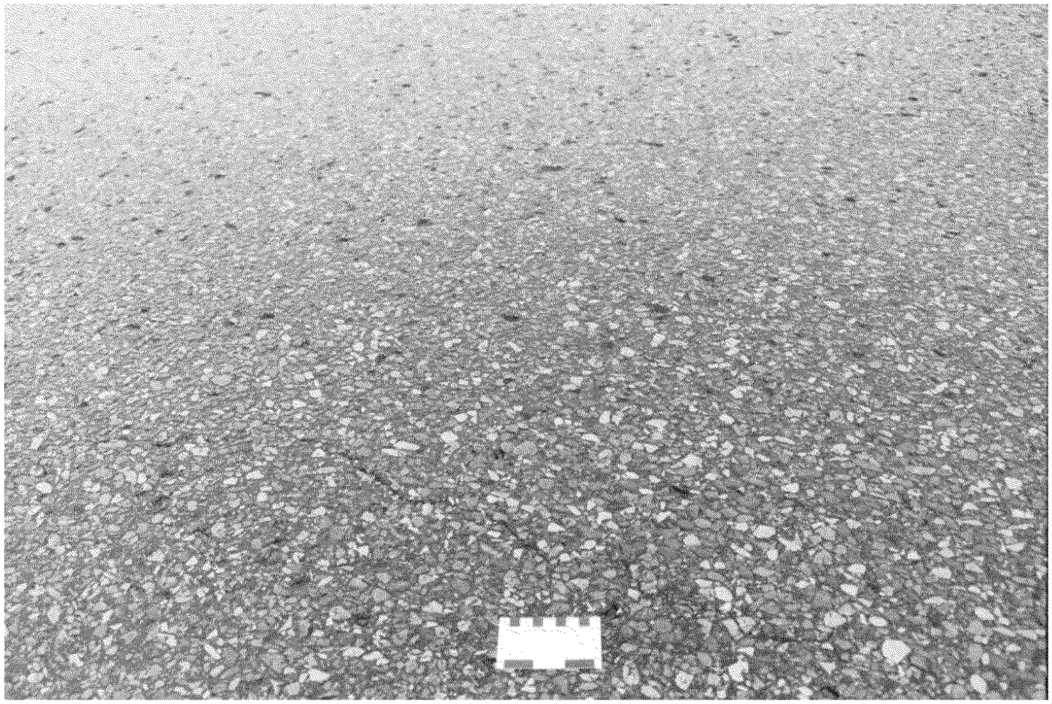


Figure 3 - Site 23 Close-up of Polished Limestone Coarse Aggregate



Analysis

The three principal goals in this pilot were to establish:

- If a wet-to-dry collision rate is a reasonable predictive factor for low friction incidence;
- If some sites with high wet-to-dry collision ratio can be excluded from friction testing; and
- If there are common causative factors for low friction.

Current Predictive Success Rate

Presently, MTO staff typically identify sites for friction testing based on:

- Visual identification during the annual Pavement Condition Rating (PCR) network survey;
- Concerns raised by MTO Operations or Traffic staff (or external sources); and,
- Poor experience with similar material, design, or technology.

The existing, mostly visual method to compile a list of sites to be tested for friction yields an average success rate of 30% (Table 4) in predicting sites where low friction typically warrants (Figure 1) further action.

	Ontario Friction testing, excluding testing new aggregate sources, new pavement mixes or new paving equipment			Sites (of total tested) where friction in red or yellow ranges was encountered (%)
	The average FN value at posted speed			
	FN ≤ 25.9 Code: Red	**26 ≤ FN ≤ 30.9 Code: Yellow**	**FN ≥ 31 Code: Green**	
2002	**17**	**14**	**26**	**54%**
2003	**10**	**12**	**71**	**24%**
2004	**10**	**2**	**55**	**18%**
			3 Year Average	**30%**

Table 4 – The average success rate of the visual inspection method 2002 - 2004

In the above table the FN values represent test site averages. The three year average represents approximately a 30% "success rate" in the friction tests validating the visually based prediction of possibly low friction.

In the pilot, using test site average FN values of 31 tested segments, four belonged to the red coded range and eight in the yellow coded range, thus jointly representing approximately a 40% of sites where the actual friction tests validated the (wet-to-dry collision ratio based) prediction of possibly low pavement friction.

By using the cumulative length of the site segments, rather than measuring the percentage of sites, testing of the 206.3 km of roadway resulted in 47.8 km of red coded readings and 36.6 km of yellow coded readings. Jointly these represent approximately 40% of the total pilot length tested, confirming the method's predictive "success rate". Nineteen of the thirty-one sites tested had yellow or red coded site segments. No similar statistics are available for the visual inspection method. Figure 5 illustrates how the two methods compare.

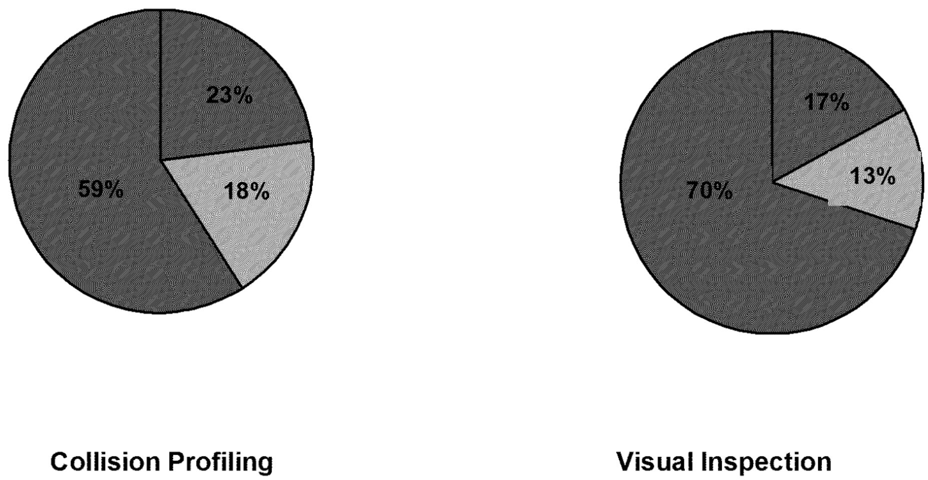


Figure 5 – Comparison between the Visual Inspection and the Collision Profiling Methods

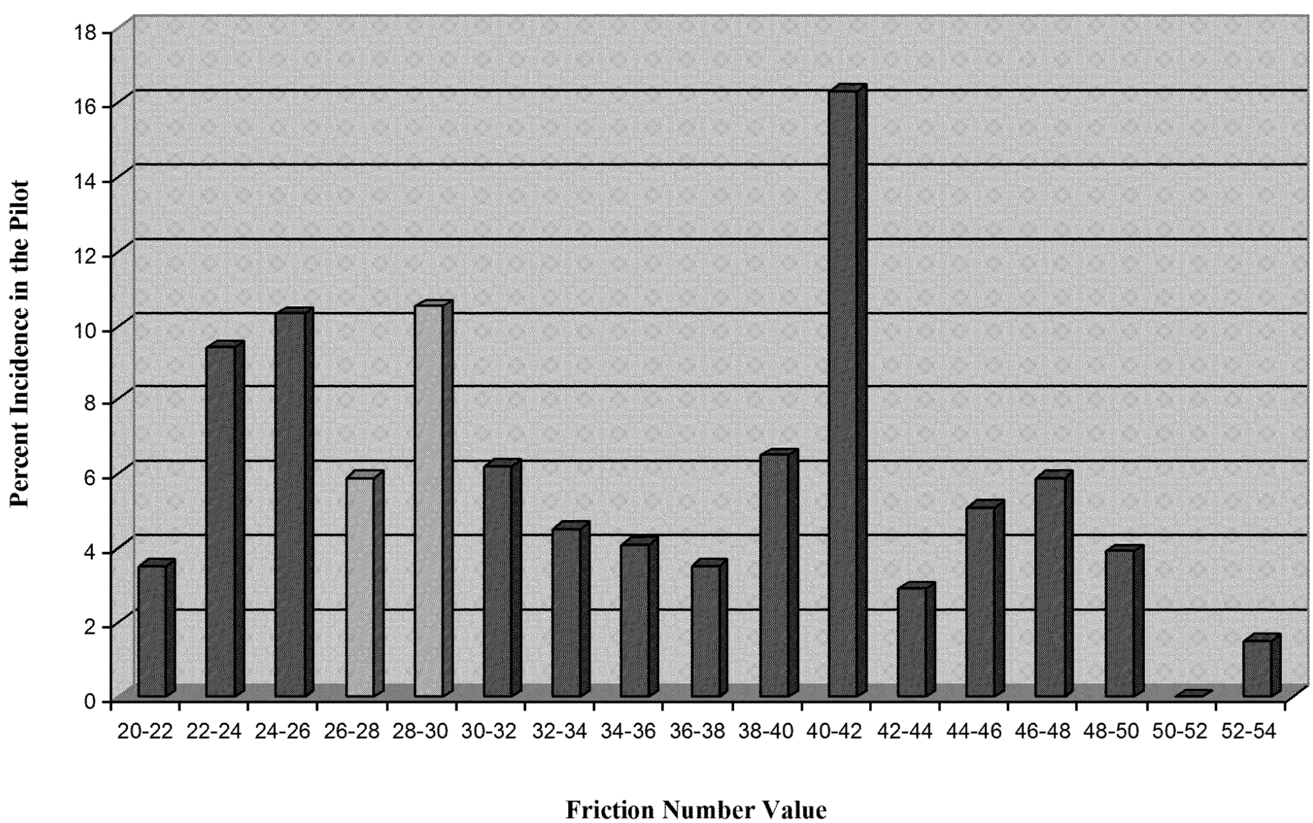


Figure 6 - Distribution of FN Values in the Pilot

Geotechnical Analysis

Nine of the twelve red coded sites were visited by staff of MTO Soils and Aggregates Section in order to identify common causes, if any, of the lower than desired friction readings. Specifically, Site IDs 26, 30, 7, 12, 34, 22, 19, 18 and 23 were visited.

All the red coded site-segments had surprisingly one thing in common - all were locally milled and resurfaced since they were constructed with limestone aggregate. The Ministry policy defining what type of course and aggregate can be used on a particular class of highway [Appendix B and Ontario Provincial Standard (Material) Specification 1003] were not followed, likely because the repairs pre-dated these. The limestone was typically polished and slippery to touch. At the majority of red coded site-segments there was slight aggregate cracking or loss (Figure 2 and 4), likely having no or negligible effect on the pavement friction measurement.

Collision Analysis

To be able to compare sites of unequal length and pilot sites with the rest of the Regional network, total collision rates were calculated per 100 million vehicle kilometers per year to form a basis for comparison. The Ontario Road Safety Annual Report (ORSAR) states the combined provincial and municipal road network has been steady over the past years, standing at 200 collisions per 100 million vehicle kilometers traveled. Table 5 provides a breakdown.

The Average Collision Rates in Ontario in 2001 - 2003	
Road Function / Classification	**Collisions per 100 million-km**
Freeways	60
Arterial and Collector Highways	70-80
Secondary Highways	110-120
Municipal Roads	300
All municipal & provincial roads (average)	200

Figure 5 – Collision Rates by Highway Classification

The Eastern Region average provincial road network total collision rate was calculated as the product of Total Collisions x 10^8 / (3 x 365 x AADT x Site Length) is 71.

For the 4 sites where the mean FN coded red the average total collision rate was 105. The average total rates for the 8 yellow coded sites and the 19 green coded sites were 82 and 96 respectively. The number of sites in this pilot is too small for these rates to be relied on with any degree of certainty.

Collision Data and Low Friction Correlation

In the initial crude effort to establish if the pilot sites can be prioritised or eliminated from the need for friction testing, the sites were sorted by all available parameters (Table 3). This was done with hopes that the ranking would be preserved after the sort. This exercise was unsuccessful for all parameters by which a sort was attempted.
It was found, however, that by sorting by the total collision rate in descending order, the vast majority of red and yellow coded sit es occupied the upper two thirds of Table 6.

The number of sites tested was not large enough to conclusively find a relationship between the friction and the total collision rate.

Sect ID	Length	AADT	Collisions 2001-2003			Ratios	Total	Partial Segment		Segment
			Wet	Dry	Total	Wet/Dry	Rate	FN80	%	FN80
29	5.2	600	2	3	8	0.67	234.16	28.7	20%	32.6
28	1.3	2500	3	5	8	0.60	224.80	37.6	100%	37.6
4	0.6	16500	7	14	21	0.50	193.72	45.7	100%	45.7
9	10.6	4600	19	40	77	0.48	144.22	23.8	37%	31.3
17	12.2	1400	7	11	26	0.64	139.02	28.5	50%	33.9
18	3.5	1400	2	3	7	0.67	130.46	25.8	100%	25.8
19	3.8	1400	2	3	7	0.67	120.16	21.1	100%	21.1
16	9.9	1500	6	7	19	0.86	116.85	44.6	100%	44.6
33	9.5	1350	4	8	16	0.50	113.93	25.0	8%	36.8
12	10	2550	10	17	30	0.59	107.44	23.0	47%	30.1
11	10.1	2250	9	11	26	0.82	104.49	27.1	85%	28.7
7	1.4	11300	6	11	18	0.55	103.91	24.6	100%	24.6
22	4.2	3000	4	7	14	0.57	101.47	24.6	81%	26.5
8	3.1	4450	3	6	14	0.50	92.68	38.5	100%	38.5
23	8.4	5700	23	16	48	1.44	91.55	29.7	100%	29.7
20	8.6	3700	6	9	30	0.67	86.10	22.4	7%	45.1
3	6.4	4900	7	11	29	0.64	84.45	27.5	100%	27.5
32	1.6	5500	2	4	8	0.50	83.02	53.8	100%	53.8
5	7.7	5600	9	17	39	0.53	82.60	21.8	32%	29.1
30	8	5200	8	17	30	0.47	65.86	21.6	44%	26.6
34	4.2	2700	4	3	8	1.33	64.43	24.8	100%	24.8
14	7.4	390	1	1	2	1.00	63.29	47.6	100%	47.6
1	3.8	41400	18	40	97	0.45	56.31	42.0	100%	42
31	8	5500	6	13	24	0.46	49.81	26.5	12%	31.6
15	1.4	27800	5	10	20	0.50	46.93	52.4	100%	52.4
24	3.5	4500	5	3	8	1.67	46.39	37.2	100%	37.2
2	1.5	9200	2	4	7	0.50	46.32	30.7	40%	33.3
10	11.3	2950	5	11	16	0.45	43.83	40.3	100%	40.3
13	9.8	3500	3	4	11	0.75	29.29	41.0	100%	41
26	11.3	5400	4	7	15	0.57	22.45	24.0	48%	30.6
6	5	22000	2	4	7	0.50	5.81	31.0	100%	31

Table 6 – Site Prioritization Based on the Total Collision Rate

Also tried was the ratio of wet plus dry collisions, divided by total collisions, wet collisions per kilometre and parameter products. In each case, the sort scrambled the ranking randomly. This exercise was repeated with products of parameters, but again, without a success.

A regression analysis was performed to further investigate the relationship between both the wet-to-dry collision ratio and the test-determined friction number (FN80). Figures 7 and 8 illustrate the findings.

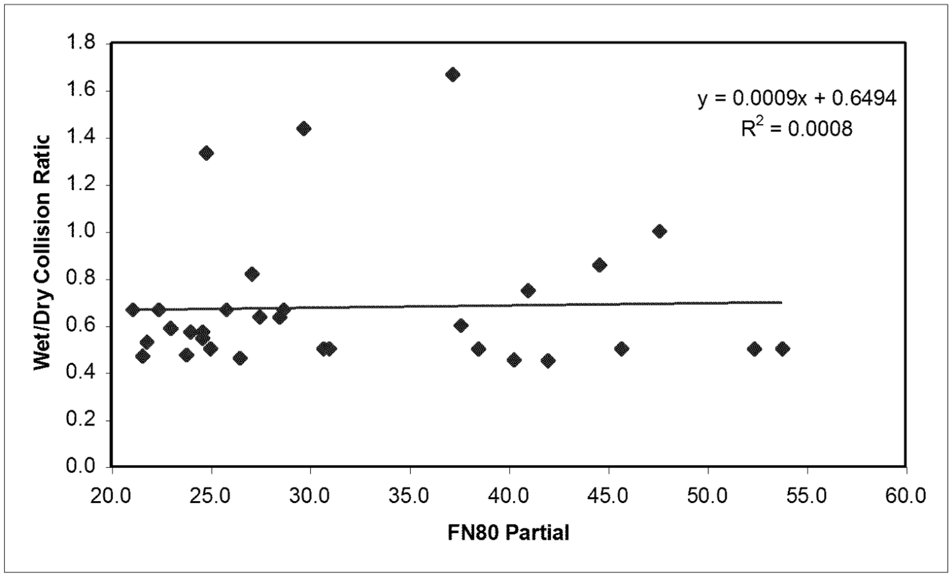


Figure 7 – FN_{80} versus Wet-Dry Collision Ratio

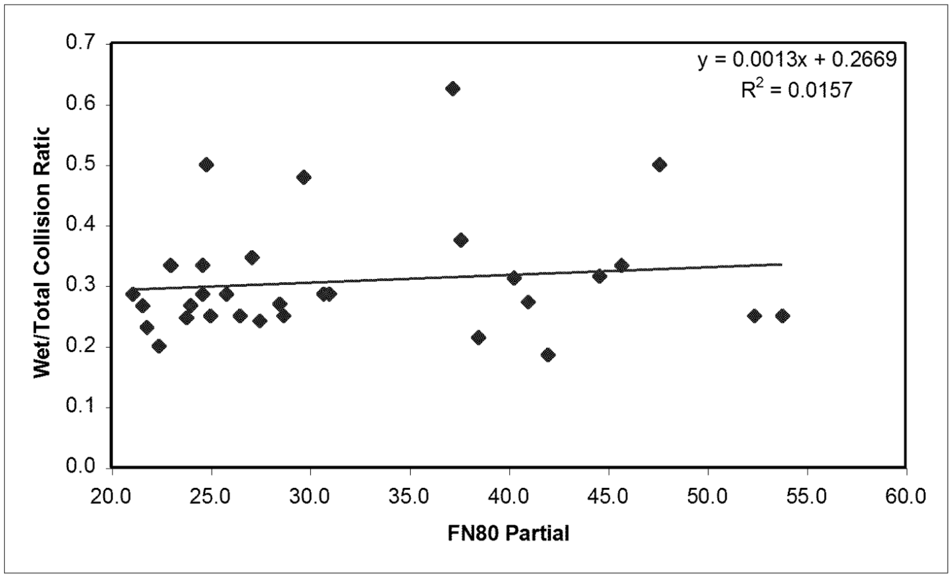


Figure 8 – FN_{80} versus Wet-Total Collision Ratio

The resulting regression equations have very low correlation coefficients for both Figures 7 and 8. This indicates that the relationship between collision ratio and friction ratio is non-existent or highly improbable. One factor that may have led to the low correlation coefficient values is the vast span of AADT values of the data set.

A consequent analysis was completed after sorting the data into five traffic volume ranges and selecting the top 3 data points with the highest wet-to-dry ratio (Figure 9).

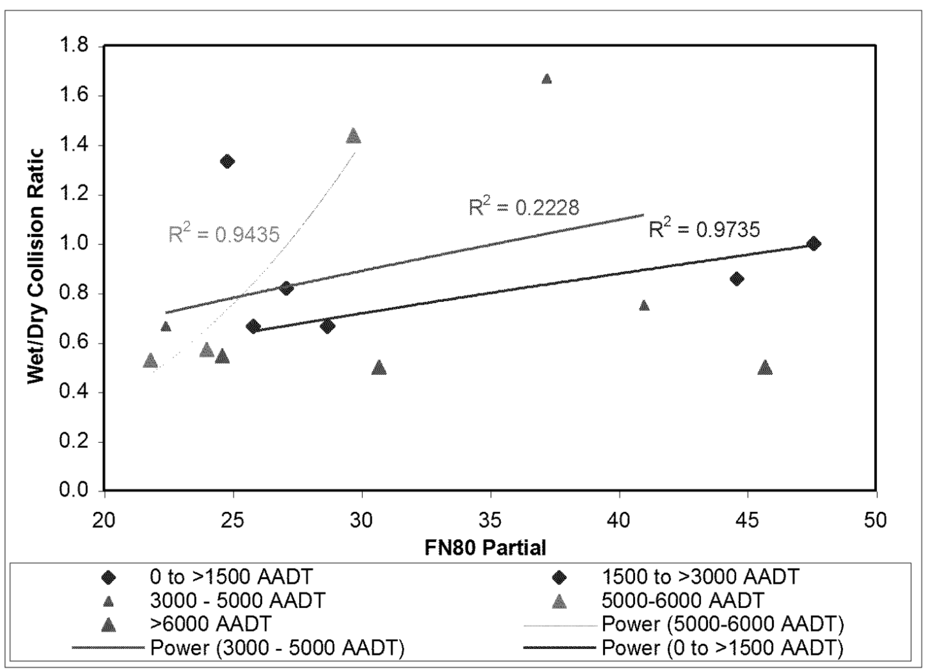


Figure 9 – FN_{80} versus Wet-Dry Collision Ratio (sorted by AADT)

Sorting the data set into AADT ranges resulted in trend curves / lines sloping in the opposite way from what was intuitively expected. This could be attributed to the small number of sites used and / or the hypothesis that friction demand, rather than low friction incidence, is the key influencing factor for the wet-to-dry collision rate. If this pilot is expanded Province-wide, another correlation should be attempted.

The data was analysed to determine the influence that the predominant pavement type possibly has on the wet-to-dry collision ratio to the friction number (Figure 10).

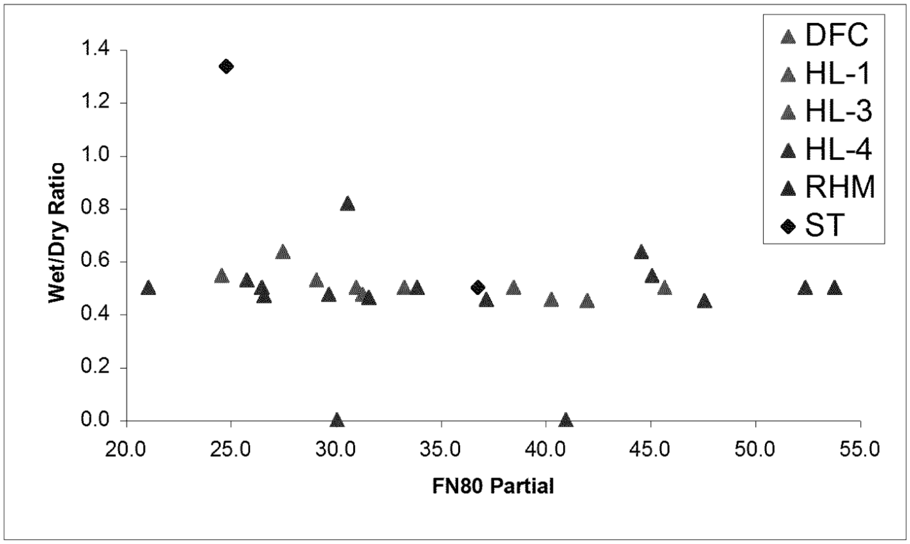


Figure 10 – FN_{80} versus Wet-Dry Collision Ratio (sorted by pavement type)

The results of the plot demonstrate that hot mix asphalt pavement type was unrelated to the level of friction encountered. This can be perhaps partially attributed to frequent incidence of patching that does not necessarily correspond to the dominant pavement type.

Pavement Condition and Correlation with Friction and Collisions

The relationship between the pavement condition parameters and the friction number was also investigated (Figures 11 to 17).

Previous research in Sweden [Ihs, 2002] had shown a correlation between the total collision rate and the IRI value. The study indicated that with higher IRI values more collisions are experienced. It was assumed that if collisions and IRI were correlated, then perhaps friction and IRI were also to some degree correlated. To test this theory the relationship between friction and IRI was examined (Figure 18).

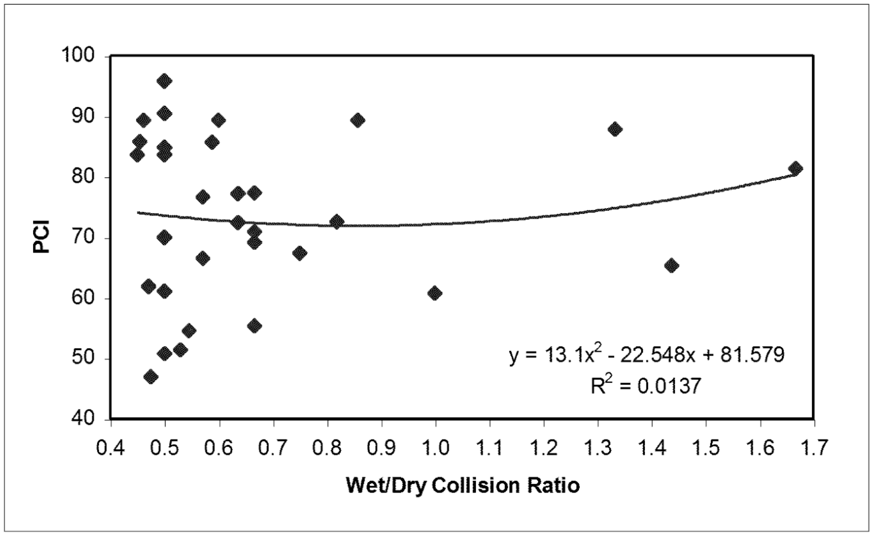


Figure 11 – Wet-Dry Collision Ratio versus Pavement Condition Index - PCI

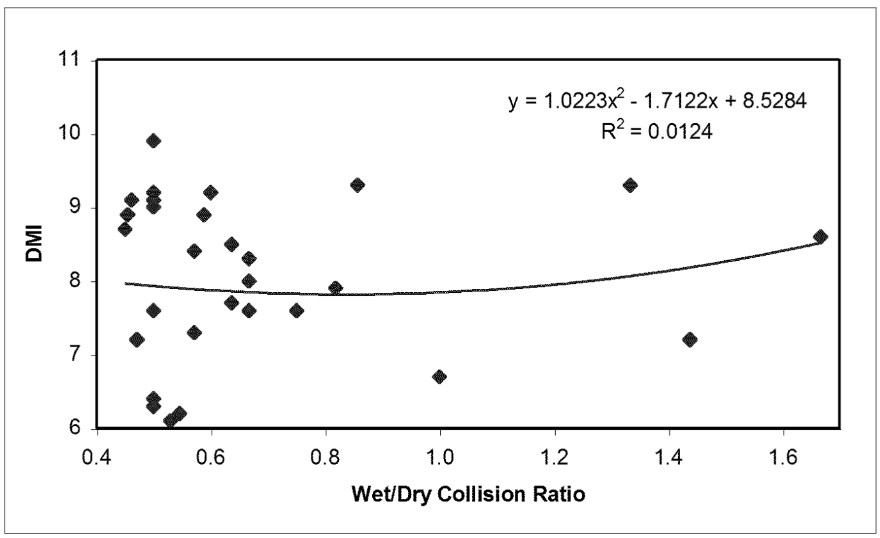


Figure 12 – Wet-Dry Collision Ratio versus Distress Manifestation Index - DMI

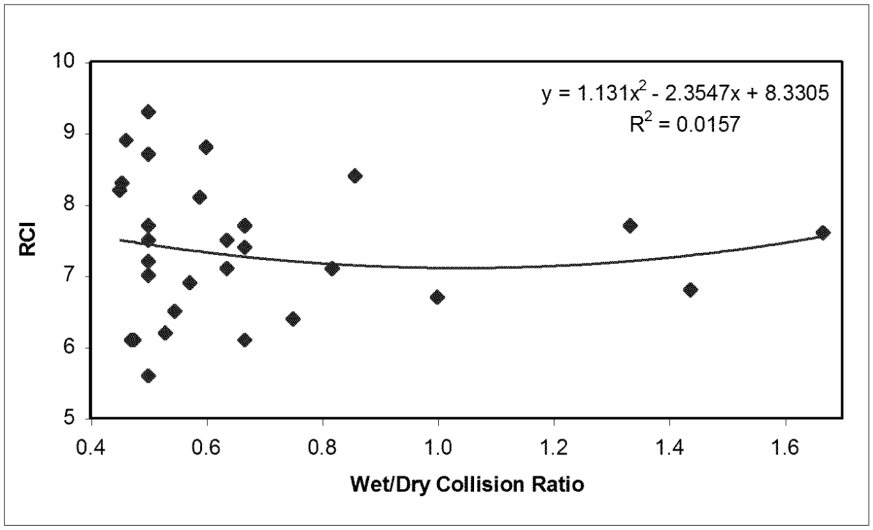


Figure 13 – Wet-Dry Collision Ratio versus Ride Condition Index - RCI

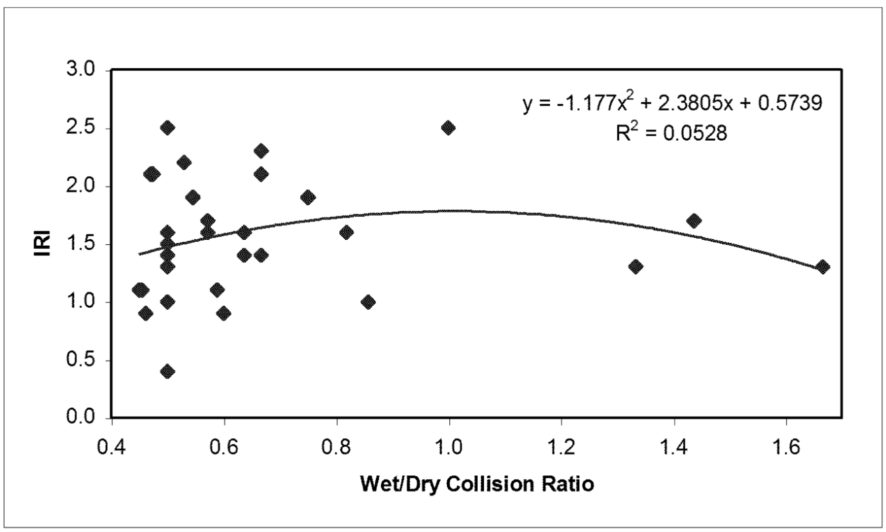


Figure 14 – Wet-Dry Collision Ratio versus International Roughness Index - IRI

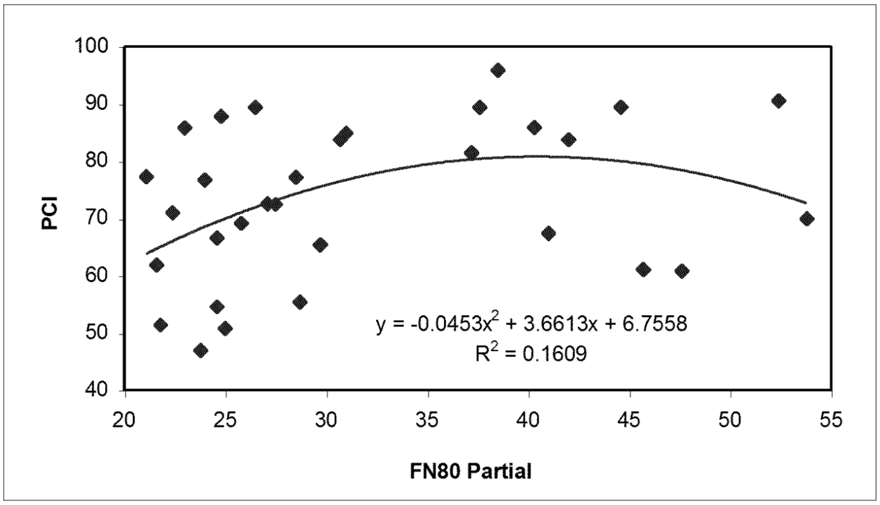


Figure 15 – FN_{80} versus Pavement Condition Index - PCI

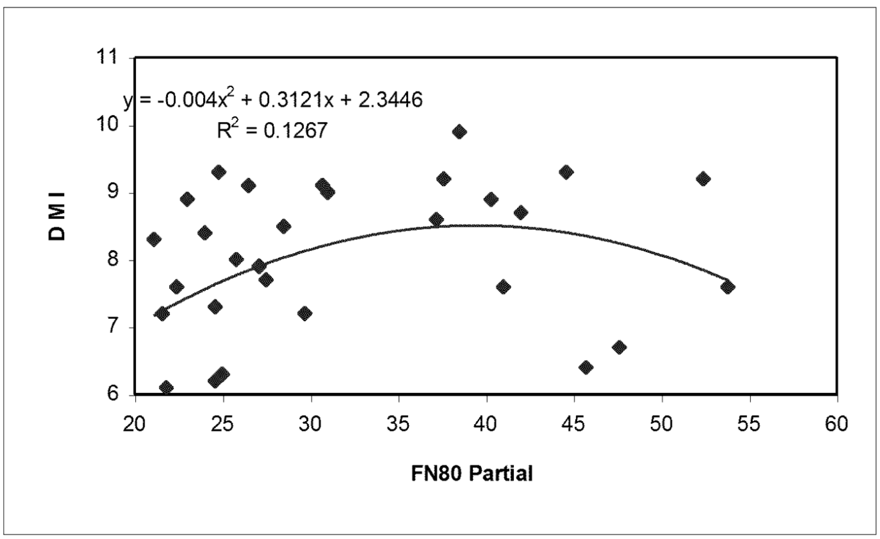


Figure 16 – FN_{80} versus Distress Manifestation Index - DMI

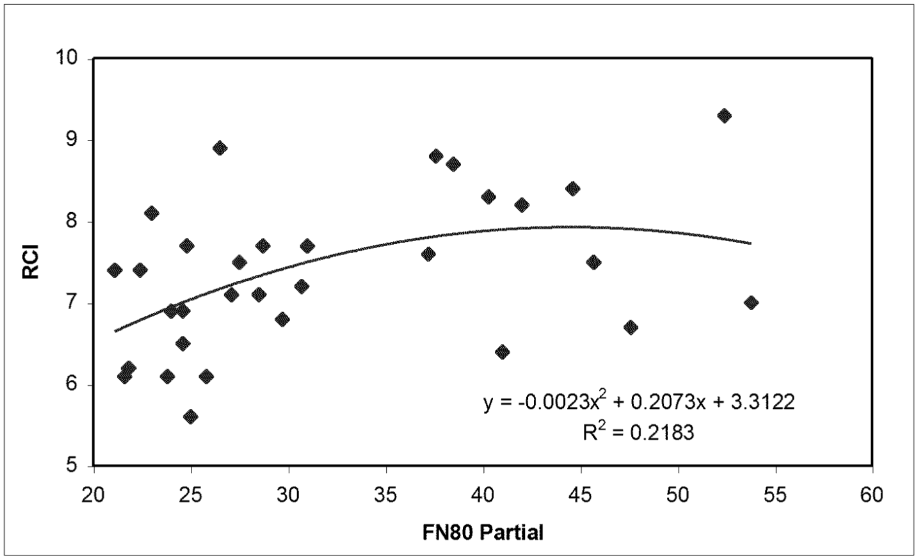


Figure 17 – FN_{80} versus Ride Condition Index - RCI

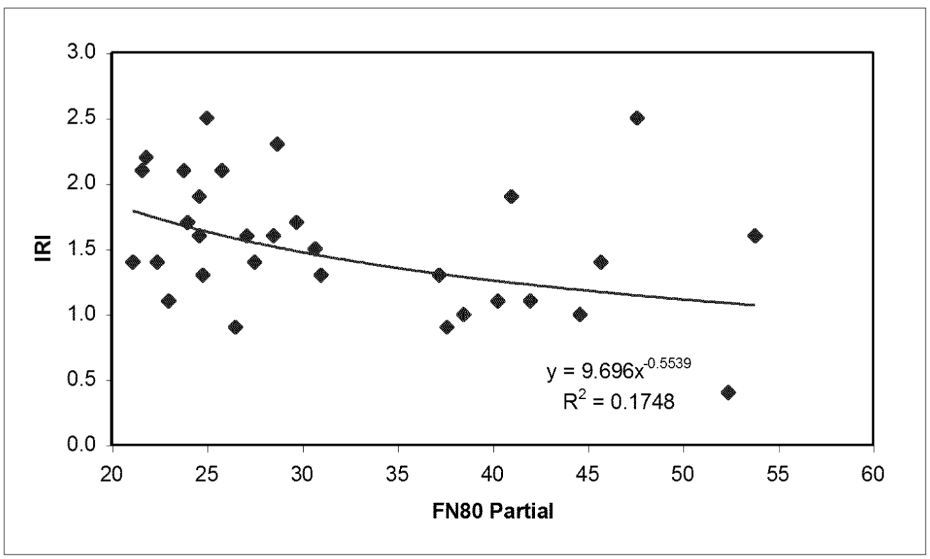


Figure 18 – FN_{80} versus International Roughness Index - IRI

As was fully expected, the regression equations presented above have very low correlation coefficients for Figures 11 through 18, thereby indicating the traditional pavement condition parameters do not correlate with low pavement friction or a high wet-to-dry collision ratio.

Discussion

Good correlation between the wet-to-dry collision rate and the incidence of low friction was reported in TRB Record 623. The article documents research that was performed in West Germany and France in 1976. Research in England [Viner, 2005] and Switzerland [Seiler-Scherer, 2004] resulted in good correlation for undivided highways only. This pilot also found good correlation between the wet-to-dry collision rate and the incidence of low friction in a set of sites short-listed for friction testing because of their above-average wet-to-dry collision ratio. All attempts to identify one or more parameters linked to low friction incidence within the short-listed set of sites were unsuccessful.

It is important to use a minimum of three years of collision data for an analysis. This is to minimise the effect of the regression to the mean phenomena and to minimize any undue effects of collisions that while recorded on wet pavement, on closer examination of records are attributable to factors unrelated to wetness, such as driver impairment or distraction.

It is likely that a percentage of collisions reported as having taken place on a dry pavement surface took place under slightly damp surface conditions, where the available friction is greatly reduced. While this may result in some sites not "making" the short list, this may only affect borderline sites where the wet-to-dry collision ratio is near the cut-off limit of 0.45. In any case, this would have no impact on the findings of this study.

There were 40% of the tested sites, by length, that had less than desired friction values. The remaining 60% had the desired or higher friction values, but made the short-list because other causative factors were involved in the over-representation of wet pavement surface collisions at these sites. These causative factors may include poor pavement surface drainage (water ponding), low delineation visibility during night-time wet conditions and in general, any conditions that contribute to above average incidence of a driver error. To eliminate the unwarranted friction testing at the projected 60% of the sites short-listed by collision profiling, one could utilize either a tactile inspection, or identify the surface course aggregate type and source from records. Then, based on the year of the last surface treatment, the aggregate Polished Stone Value and test section friction monitoring data associated with the pre-approval of the aggregate source, one may model the likely level of friction currently present. Only as the last resort should the testing be prioritised by deferring testing at sites with the lower end of the total collision rate, as presented on Table 5.

Sample sites with low friction were surveyed from a mineralogical perspective. One common denominator was discovered - the areas were locally resurfaced with hot mixes containing soft limestone, contrary to the current Ministry policy. Sites with limestone repairs had high variability of friction values within the segment length.

Limestone is rarely, if ever used for full or partial pavement repairs of the pavement surface in the two northern regions. It follows that the collision profiling may not be as successful there as was the case in the pilot Eastern Region. On the other hand, what is unique to the two northern regions is that the residents can use legally studded tires. Scandinavian experience indicates that hard aggregates, i.e. those containing quartz, are highly susceptible to polishing by studded tires. We may find that should the studded tire usage become widespread, over the years, the collision profiling approach may yield good results in Northern Ontario. Consequently, the collision profiling may prove to be successful province-wide in the long term.

In the pilot it was observed that even the yellow and green coded sites, where the friction was found to be marginal and satisfactory, respectively, had an above average total collision rate (Table 5). The explanation lies in the sampling method, which was designed to short-list sites with a high incidence of wet weather collisions. While pavement surface course friction may play a minor role in dry and winter condition collisions, the hypothesis is that it does not account for all of the pilot sites having a collision rate significantly higher than the total collision rate for provincial highways – 70 collisions per 100 million vehicle kilometers driven. The explanation likely lies in the biased selection process that singles out sites with high friction demand (with or without low friction segments being present).

Indeed, site inspections revealed that most sites have multiple low radius horizontal and vertical curve combinations, multiple or partially obscured commercial entrances, and isolated curves, thus confirming the high friction demand site hypothesis. The inspections found that high fiction demand was unrelated to incidence of low friction. This is not surprising, since the Ministry's policy (Appendix B) has been to link the surface course type and aggregate to the road traffic volume / function, and not to geometrics or other causative factors responsible for high friction demand at a site. However, high friction demand sites with highly localized incidence of braking or acceleration, such as intersection approaches, are likely to exhibit polishing.

Contrary to expectations, sites with relatively few wet weather collisions were just as likely to be tested with low friction as sites with high collision numbers or rates, even though the probabilistic approach would put need for testing at such sites in doubt. The wet-to-dry collision ratio was thus confirmed as the only reliable predictor of low friction.

It is also apparent that even though low friction likely manifests itself most prominently in wet surface collisions, it also, to some degree, is responsible for higher total (including dry and winter condition) collision rate.

It is likely that the baseline collision rate is closer to 92 (combined for the green and yellow coded sites) than the national regional average of 70; lets say conservatively 90. The rational for this is that the pilot sites all have above average friction demand, which jointly with low friction incidence, contributes to a high collision rate.

As a rough estimate, the benefit of restoring the friction on the red coded sites can be estimated as $(90 - 105) / 105 \approx 15\%$. It is not possible, on such a small sample of sites as tested in this pilot, to derive a reliable linkage between collision rate and friction. Further, by the nature of the site selection procedure, the sample is biased towards sites with high friction demand, which was subsequently confirmed in the field. It is likely that for randomly selected sites, or for the entire network, where high and low friction demand sites are all represented, there is a high probability that no relationship between friction and collision rate can be established. Until an analysis of the province-wide data becomes available, the author advises against using cost benefit analyses to justify or prioritize friction restoration projects.

The safety benefit (15%), established for high friction demand sites, may be smaller than expected, because some collisions may migrate, after the surface of a red coded site was treated, to the adjacent segment, if its friction is marginal. This is because drivers are likely, based on vehicle response to routine braking, to expect as high friction ahead as that experienced on the treated site. They may thus gain a false sense of security and enter the adjacent segment at too high a speed for conditions.

The wet-to-dry collision rate is a complementary tool to the current visual assessment method to assist in selecting sites for friction testing. It helps to reveal polished surface locations on pavements, which would not be apparent to a casual observer during a typical drive-by inspection. Conversely, the visual inspection method focuses on flushed surface sites that would not otherwise be picked by the collision profiling method, if they are in low friction demand areas, and thus have few or no collisions. The two methods are thus complementary and should preferably be used together.

One drawback of the collision profiling method is that it is biased towards selecting high friction demand sites, where friction is only a minor collision causative factor. The testing effort is not wasted if high friction values are found, provided this leads to investigations of other possible causative factors, that may be followed by corrective measures.

Because of the relatively large sample size from which the pilot sites were generated (the entire Eastern Region – 2,280 km), it is a fair assumption that this methodology will have similar predictive success rate in other regions.

The collision profiling method may be unsuitable for evaluation of low speed road segments, since none of the sites included in the pilot had posted speed below 80 km/h or included signalised intersections. Further, the particular skid trailer used produces good correlation for pavement friction and surface macro-texture and therefore representative of skid resistance at higher speeds, while microtexture comes to play at lower speeds. For this reason the collision profiling may not be as successful a predictor of low friction on lower speed roads.

The collision profiling will not necessarily identify all sites with the potential to exhibit low friction. Since collisions are fortunately uncommon, there could be polished road segments with collisions yet to happen, irrespective of the level of friction demand. Used in combination, the visual inspection and the collision profiling methods are the optimum tool to schedule friction testing with available resources, short of dramatically expanding the scope of testing and periodically testing the entire network.

The method used in this pilot may not have success at detecting low friction for divided highway segments, since the pilot did not include an adequate sample of divided highways. Because of strict aggregate selection policies on the divided highway network, polishing is highly unlikely to be encountered. It is much more likely that on freeways, low night-time visibility of lane dividing lines at high friction demand sites, e.g. weaving or merging sections, is mostly responsible for the high wet-to-dry collision ratio.

As the testing progresses over the years, it is expected that the number of sites with high wet-to-dry collision ratios will be reduced. It is not cost effective to lower the threshold (wet-to-dry collisions = 0.45), but instead, suspend using the method for a number of years, until aggregate aging and polishing leads to emergence of a new set of sites that meet the adopted criterion.

Recommendations

1. Continue with the existing targeted approach to network friction monitoring.

2. Adopt collision profiling for selecting sites for pavement friction testing, in addition to the existing visual inspection method, province-wide.

3. Monitor the collision profiling method's predictive success rate, particularly for freeways, and consider suspending it for a number of years, when encountering diminishing returns.

4. Monitor the collision rates versus friction and establish a basis for using cost-benefit methodology to make decisions on friction restorative treatments.

5. Provide for the field staff reference cores or aggregate coupons, so that they may, with a degree of confidence, identify sites where the degree of polish warrants friction testing.

6. Investigate segments with similar method of pavement repair, age, aggregate source and similar traffic volume to the low friction sites, in order to identify polished sites with low friction demand (too few or no collisions).

7. Refer sites identified by collision profiling which test with normal friction for an optional follow-up Traffic analysis to determine other causative factors.

8. Ensure that contractual agreements, inspection / QA manuals and training is available to ensure that limestone and other aggregates susceptible to polishing are not used on higher volume roads, not even for partial or temporary repairs.

9. Ensure that for existing highways, Area Term Contracts specify not only average FN values, but also minimum individual friction test values..

10. In the longer term, enhance the targeted network friction monitoring to include polished pavement sites with low friction demand that collision profiling overlooks. This can be accomplished by aggregate friction performance modelling based on DSM aggregate source acceptance friction monitoring historical data and PMS2-stored aggregate source and year last resurfaced information.

Acknowledgements

The authors would like to thank the following Ministry of Transportation employees:

Todd Comfort, Head, Geotechnical Section, Eastern Region, for volunteering to facilitate the pilot in Eastern Region and for assigning the Engineering Development Trainee, Laura Donaldson, to the project;

Peter Gaboury, Head, Traffic Information Management, Traffic Office, for extracting the required collision data from the Accident Information System; and,

Li Ningyuan, Senior Pavement Management Engineer, Pavements and Foundations Section, Materials Engineering and Research Office, for extracting the required pavement condition data from the Pavement Management System (PMS2).

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Appendix A

Acceptance Guidelines for Designated Sources for Materials (DSM) List 3.05.25

Aggregates, Coarse for Superpave 12.5 FC 1, Superpave 12.5 FC 2, SMA, HL1, DFC, and OFC; and Aggregates, Fine for Superpave 12.5 FC 2, SMA, DFC, and OFC

Requirements for inclusion of sources of aggregate for premium quality asphalt pavements on DSM list #3.05.25 are as follows:

- Submission of a letter requesting consideration for DSM list 3.05.25 to the Manager of the Soils and Aggregates Section, Ministry of Transportation of Ontario, Materials Engineering and Research Office (MERO), Building C, Room 220, 1201 Wilson Avenue, Downsview, Ontario, M3M 1J8 (telephone [416] 235-3734).

- Satisfactory nature and consistency of the source, as determined from a geological examination by staff of the Soils and Aggregates Section.

- Suitability of the production facilities, as determined from an inspection by staff of the Soils and Aggregates Section.

- Sampling by Soils and Aggregates Section staff of 1,000-tonne (approximately) stockpiles of coarse and fine aggregate meeting relevant grading requirements.

- Satisfactory quality, as determined from testing conducted by the Soils and Aggregates Section on the stockpile samples and from subsequent testing. The requirements of OPSS 1001 and 1003, and such special provisions that alter these specifications, must be met. In addition, an average PSV (Polished Stone Value) of no less than 50 (with no value less than 48) and an average AAV (Aggregate Abrasion Value) of no more than 6.0 must be maintained.

- Submission to the Manager of the Soils and Aggregates Section of a quality control plan describing the procedures and processes followed to ensure product quality. Contamination of the aggregate by weathered material and undesirable lithologies such as marble and mica-bearing pegmatite must be avoided.

- Arranging for construction of a test section on a ministry contract utilizing Superpave 12.5 FC 1, Superpave 12.5 FC 2, SMA, HL1, DFC, or OFC (as relevant). The section must be at least 500 metres in length, across the full width (driving and passing lanes) of the pavement in one traffic direction; and in a typical Superpave 12.5 FC 1, Superpave 12.5 FC 2, SMA, HL1, DFC, or OFC (as relevant) location which has a 100 kilometre/hour speed limit, and is an at- or near-horizontal, straight section of highway, with typical truck percentage of average annual daily traffic.

Identification signs, no larger than 30 centimetres by 30 centimetres, may be erected at both ends of the test section near the fence line and parallel to the highway at the option of the proponent. The Manager of the Soils and Aggregates Section must approve the location of the test section and be provided with the asphalt mix design for approval by staff of the Bituminous Section of MERO. The requirement for a Superpave 12.5 FC 1 or HL1 test section is waived if the coarse aggregate is mineralogically and texturally similar, as determined by the Soils and Aggregates Section, to that from another source on DSM list #3.05.25.

- Satisfactory performance of the aggregate in the test section during a two-year period, as determined by the Soils and Aggregates Section from visual examination of the pavement and skid-resistance surveys conducted by the Pavements and Foundations Section of MERO.

- Registration with The Road Authority (www.roadauthority.com; [905] 459-9200), for which there is an annual fee.

- Payment by cheque (payable to the Minister of Finance) of $5,350.00 (which includes 7% GST) to the Manager of the Soils and Aggregates Section.

The Ministry of Transportation of Ontario evaluates products in the context of its own needs only. Ministry approval does not constitute a general or specific endorsement, and must not be used by the recipient of such approval to promote sale of a product, service, or process. Any violation of this prohibition may result in the withdrawal of any approval granted.
The evaluation of aggregate for premium quality asphalt pavements is described in the following publication, which is available from the Soils and Aggregates Section:

Rogers, C., Gorman, B., and Lane, B., 2003: Skid-Resistant Aggregates in Ontario; Ministry of Transportation of Ontario, Materials Engineering and Research Office, Report MERO-005, 30p.

July 15, 2004

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Appendix B

Ministry of Transportation

Ministère des Transports

Ontario

MINISTRY DIRECTIVE

Program: Policy, Planning & Standards

Directive: PLNG-C-003

Issuing Authority: Executive Director, Highway Engineering Division

Date of Issue: 78 11 23 **Effective Date:** Immediate **Revised:** 03 08 06

TO: Assistant Deputy Ministers, Executive Directors, Regional Directors, Directors, District Engineers, Regional Managers, Office Managers

SUBJECT: The Use of Surface Course Types on Provincial Highways.

ALTERNATIVE INDEX LISTING (S): Highway Surfaces
Bituminous Mix Types
Alternative Bid

REFERENCE: This directive supersedes Directive C-16 dated 78-11-23; revised 84-5-23 and 88-05-03; OPSS 1150; and special provisions 311, 310 and 321.

PURPOSE: To establish policy to ensure consistent application of standards for selecting surface course types for all highway improvement projects in Ontario. These standards consider the present state of knowledge and experience in North America.

BACKGROUND: The previous Directive used Annual Average Daily Traffic divided by the number of lanes (AADT/lane) as the sole criterion for selecting the pavement surface course type. A document was published designating the highways in each region that would have HL3/HL4, HL1, and DFC as the surface course bituminous mix type [1].

According to the previous directive, HL3/HL4 was recommended for low volume roads with less than 2500 AADT/lane, HL1 for highways with 2500 to 5000 AADT/lane, and DFC for highways with 5000 or more AADT/lane. MTO no longer uses HL3 mix for Freeways and King's Highways. Superpave mixes and Stone Mastic Asphalt (SMA) have only recently been introduced into Ontario and were not included in the previous directive.

It was recognized that in addition to AADT, there are other factors which must be considered in selecting surface course types as part of highway construction, rehabilitation and maintenance projects. They include:

- Alternative Bid criteria for freeway paving contracts;
- Continuity within a specific origin/destination corridor;
- Rut resistant mixes (e.g. Dense Friction Course, DFC or Stone Mastic Asphalt, SMA) to withstand heavy truck loads based on Equivalent Single-Axle Load (ESAL);
- Satisfactory pavement surface friction characteristics; and
- Need for special mix types (e.g. HL1 modified, HL3 modified, HL4 modified, Superpave mixes and SMA).

POLICY: Considering the above factors and following consultation with the regions and other jurisdictions, the existing standards for selecting pavement surface course types for Freeways and King's Highways have been updated. The new standards for the selection of surface course types, which are provided in Table 1, are based on either ESAL or AADT criteria. Maps indicating the surface course types required for provincial highways for all regions are attached (Appendices 1-6) for use in conjunction with this directive. For additional information or clarification, the appropriate Geotechnical Section should be contacted.

TABLE 1: Selection of Surface Course Types

ESALS /design lane /year (or AADT/lane)				
AADT<500	AADT 500 - 2500	AADT 2500-5000	1<ESAL<3 Million or AADT>5000	ESAL> 3 Million
HL4 or Surface Treatment [a]	HL4 or Superpave 12.5	HL1 [b] or Superpave 12.5FC 1	DFC [c, d] or Superpave 12.5FC 2	SMA [d]

[a] Surface treatment type should be selected according to the guidelines given in the Pavement Design and Rehabilitation Manual [3].
[b] HL4 modified or HL3 modified (meeting the polishing and wear requirements of HL1) may be substituted for HL1 upon recommendation by the Heads of Geotechnical Sections.
[c] During the EA process, the use of Open Friction Course (OFC) may be considered for the purpose of noise reduction in urban residential areas where significant noise issues have been identified.
[d] Alternative Bid freeway paving contracts requiring the preparation of one concrete and one asphalt pavement design are to be used for all new construction and full depth reconstruction projects in the order of five 2-lane kilometres in length or longer, where close to one million or more ESALs are anticipated in the design lanes within 5 years of construction.

In general, the current ESAL or AADT value is considered for the selection. However, if the current value is close to the maximum threshold value, then an anticipated increase in traffic volume should be considered. In some cases, the criteria given in Table 1 may identify two possible surface course types, in which case the mix type satisfying the higher threshold value should be selected, as illustrated in the example given below.

<u>Example</u> A highway with less than one million ESALs/design lane/year and greater than 5000 AADT/lane would require either HL1 or DFC, based on ESAL or AADT criteria respectively. In this case, DFC mix should be selected as it satisfies the higher AADT threshold value. The ESAL (or AADT) threshold values for different mixes are based on research and experience of different agencies including MTO [2].

MAJOR CONTRIBUTORY FACTORS TO SURFACE COURSE SELECTION

Equivalent Single-Axle Load

The concept of ESALs was originally developed by AASHTO for converting mixed mode traffic to an equivalent number of 80 kN single-axle loads for use in pavement design. The process of collecting mixed traffic data and converting it to ESALs for Ontario road conditions is described in an MTO report [4].

In this directive, different surface course types are identified for each traffic category. It is important to note that the ESAL or AADT criteria in Table 1 should be used in conjunction with other factors, as described below, to match the unique features and requirements of different highways.

Alternative Bid Criteria

On December 3, 2001, MTO initiated an Alternative Bid (AB) process for freeway paving contracts. Under the AB process, bidders determine their "Construction Bid" for a concrete or asphalt pavement design option. The bidder then adds a "Bid Adjustment Factor" to their Construction Bid. Bid Adjustment Factors for the concrete and asphalt pavement options are calculated by MTO in advance based on life cycle costing information and are included in the tender documents. The lowest "Total Adjusted Bid" wins.

AB freeway paving contracts are to be used for all new and full depth reconstruction projects in the order of five 2-lane kilometres or longer in length, where close to one million or more ESALs are anticipated in the design lanes within 5 years of construction.

Projects that do not meet the AB criteria should not be automatically discounted, but with the approval of the Regional Director, an assessment should be made on the merits of awarding them as AB contracts. The advice of the Estimating Office and Materials Engineering and Research Office is available to assist the region in these situations.

Continuity Within A Specific Origin/Destination Corridor

Adjacent sections within a specific corridor should be considered when selecting a surface course type on a highway to ensure continuity.

Examples

1. If a highway between two communities meets HL1 criteria except for a short section, HL1 should be used for the entire length to ensure frictional continuity.

2. Continuity is also important in terms of winter maintenance requirements. The choice of surface course type should be consistent with the winter maintenance activities required for the adjacent or existing surface course types.

Rutting Resistance

Table 1 requires HL4 mix for highways between 500-2500 AADT/lane. However, in some cases, HL4 mix may require higher percent crushed particles for both coarse and fine aggregates to resist rutting due to large volumes of trucks, even if traffic volume is less than 2500 AADT/lane. In addition, highways with slow moving trucks particularly on steeper upgrades and at major intersections may require high stability mixes to resist rutting. The percent crushed particle requirement for high stability HL4 mix shall be determined in consultation with the Regional Geotechnical Section.

Surface Friction Characteristics

Bituminous mix consists of about 95 % aggregates, which have a great influence on the skid resistance or the frictional characteristics of the pavement. The skid resistance of wet pavements depends not only on the mix type but also on the physical properties of the aggregates used in the mix and the traffic volume and speed [5]. Thus, highways with AADT greater than 5000 vehicles/lane require high stone content in stable mixes with high wear and polish resistant aggregates.

On highways with 500-2500 AADT/lane, the designated HL4 mix may require good quality aggregates to resist polishing and wear due to large volumes of trucks, even if traffic volume is less than 2500 AADT/lane. In this case, HL4 may be modified to include aggregates meeting the physical property requirements of HL1.

Use of Special Mix Types

There are special mix types such as HL1 modified, HL3 modified and HL4 modified mixes that are being used in some regions to address specific local problems. The potential applications of these mixes in certain circumstances are discussed below. Superpave mixes and SMA have only recently been introduced into Ontario and a brief summary of these mix types follows.

HL1 modified

In some locations in Eastern Region, HL1 modified mix (maximum of 10% natural blending sand in a DFC mix) is being used in lieu of DFC for the surface course on a trial basis to improve durability and workability of the mix.

HL3 modified and HL4 modified

HL3 modified and HL4 modified mixtures are being used in some areas of Northern Ontario as the surface course mix in lieu of HL1 mix. The aggregates used are not included in the Designated Sources of Materials (DSM) list but shown on regional aggregate sources lists. The use of these mixes is permitted to keep costs to a reasonable level until sufficient local suppliers, if any, meet the DSM requirements for HL1. HL3 modified aggregates meet the physical property and gradation requirements of HL1 aggregates. HL4 modified aggregates meet the requirements of HL4 aggregates in addition to meeting the frictional property requirements of HL1 aggregates.

If a Superpave 12.5FC 1 mix is selected for a contract where an HL3 modified and/or HL4 modified mix has been historically used, the Superpave aggregates should be selected from the regional aggregate sources list specifically identified for this purpose to ensure that the required physical properties are met.

Superpave Mix

Superpave mix is a hot mix asphalt designed according to Superpave criteria. Superpave, which stands for Superior Performing Asphalt Pavements, was introduced in 1992 by the Strategic Highway Research Program (SHRP) under the sponsorship of the Federal Highway Administration of the U.S. The Superpave methodology incorporates a performance-based asphalt materials characterization system to improve the long-term pavement performance under diverse environmental conditions. It presently consists of the following elements:

- Asphalt cement specifications (now fully adopted in Ontario as Performance Grade Asphalt Cements (PGAC)).
- Revised aggregate specifications which include gradation control points, "consensus" properties such as fractured faces and clay content, as well as "agency" properties which are properties specified at the discretion of the agency; and
- A new mix design method using the Superpave Gyratory Compactor.

Superpave designates hot mix types by the Nominal Maximum Aggregate Size, which represents the sieve size, in mm, through which at least 90 % of the aggregate passes. Under this system, the most common surface course type on Ontario highways is expected to be Superpave 12.5. The Ministry has added two mix types to the Superpave suite of mixes: Superpave 12.5FC 1 and Superpave 12.5FC 2. The "FC" stands for friction course. The "1" requires that the coarse aggregate fraction for this mix type must be obtained from a Designated Sources for Materials (DSM) list. The "2" requires that the coarse and fine aggregates for this mix type must be obtained from a source listed on the DSM.

Stone Mastic Asphalt

Stone Mastic Asphalt (SMA) is a heavy duty gap graded hot mix asphalt with a relatively large proportion of stones and an additional amount of mastic-stabilized asphalt cement. The SMA mixture has an aggregate skeleton with coarse aggregate stone-on-stone contact to withstand damage due to heavy truck loads.

The additional amount of asphalt cement binder is required primarily to provide increased durability and resistance to aging and cracking to a mix, which by choice of aggregates and gradation, is already quite resistant to rutting. The stabilization of the extra asphalt cement and in particular, prevention of binder run-off during construction are achieved by: 1) an increase in fines and filler, 2) addition of organic or mineral fibre, 3) polymer-modification, or 4) a combination of all three.

DEVIATIONS FROM THE RECOMMENDED PLAN FOR SURFACE COURSE TYPES

It is recommended that the need for the use of special mixes not identified in Table 1 or on the attached maps be documented in the Pavement Design Report and submitted to the Geotechnical Committee for review and endorsement. In the event that deviations are required in the form of upgrading or downgrading the mix types identified in this directive to address local rutting problems and/or to ensure continuity, the Regional Heads of Geotechnical Sections would determine the need, if any, to request a review by the Geotechnical Committee. While implementing this directive, caution must be exercised so as not to create short, isolated sections, which may result in different maintenance requirements and varying pavement performance characteristics.

IMPLEMENTATION:

Implementation of this directive is effective immediately. In northern Ontario, implementation shall coincide with the establishment of regional aggregate sources lists for HL3 modified and HL4 modified aggregates not included in the Designated Sources of Materials (DSM) list. If implementation of this directive would result in a change in the surface course type in a contract requiring revision of contract documents, the Regional Director, at their discretion may opt to not implement the directive for that particular project.

REFERENCES

1. "HL1 and DFC Surface-Course Hot Mix Types for Designated Freeways and Other King's Highways (Excluding Secondary Highways)", Highway Design Office, Highway Engineering Division, April, 1984, revised May 1988.

2. The Benefits of New Technologies and Their Impact on Life-Cycle Models, ERES Consultants, and Report prepared for MTO, October 26, 2000.

3. Pavement Design and Rehabilitation Manual, Ministry of Transportation, SDO-90-01, January 1990.

4. Hajek, J., "Procedures for Estimating Traffic Loads for Pavement Design", Pavements and Foundations Section, MTO, January 1995.

5. Rogers C., Gorman, B., Lane, B. "Skid Resistant Aggregates in Ontario', Proceedings, 46th Canadian, Technical Asphalt Association Conference, Toronto, November 19-21, 2001.

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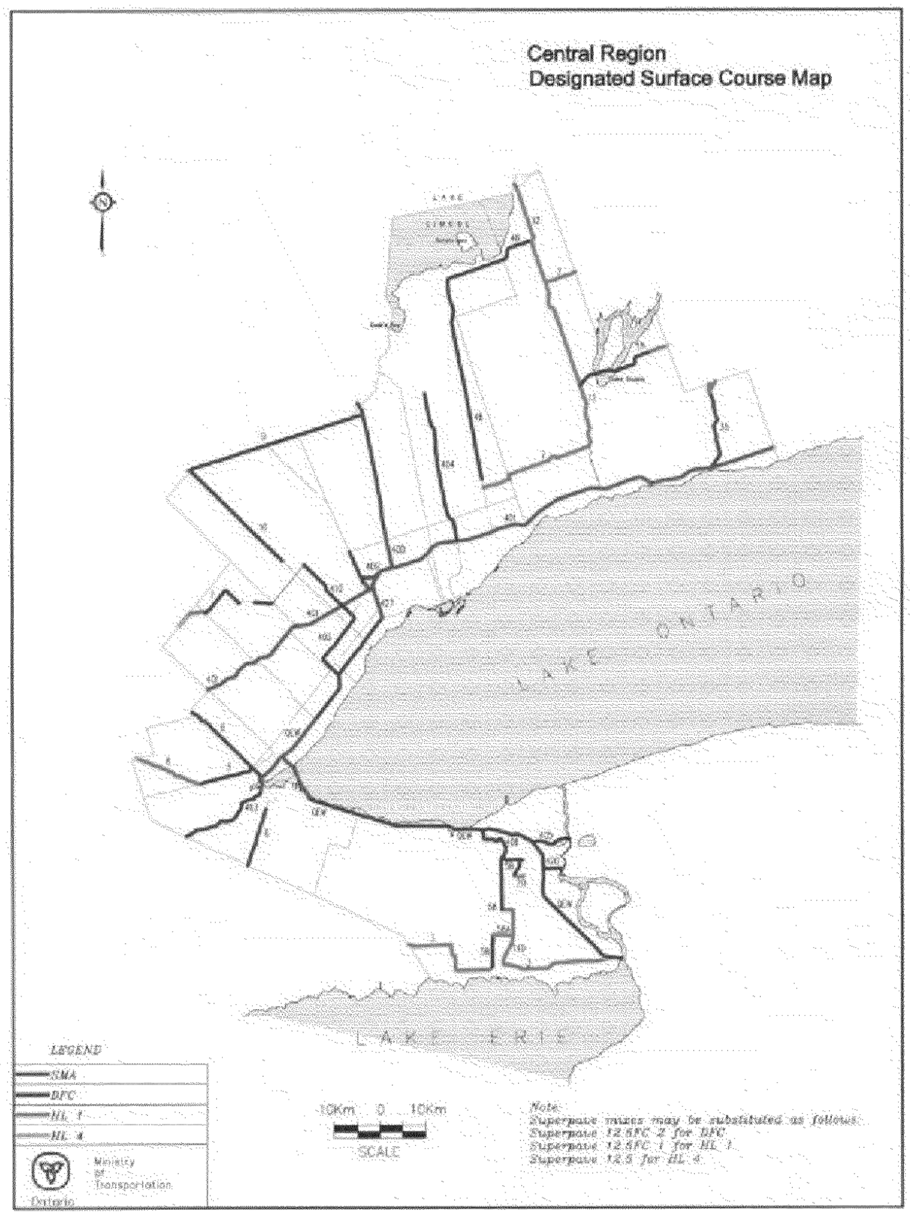
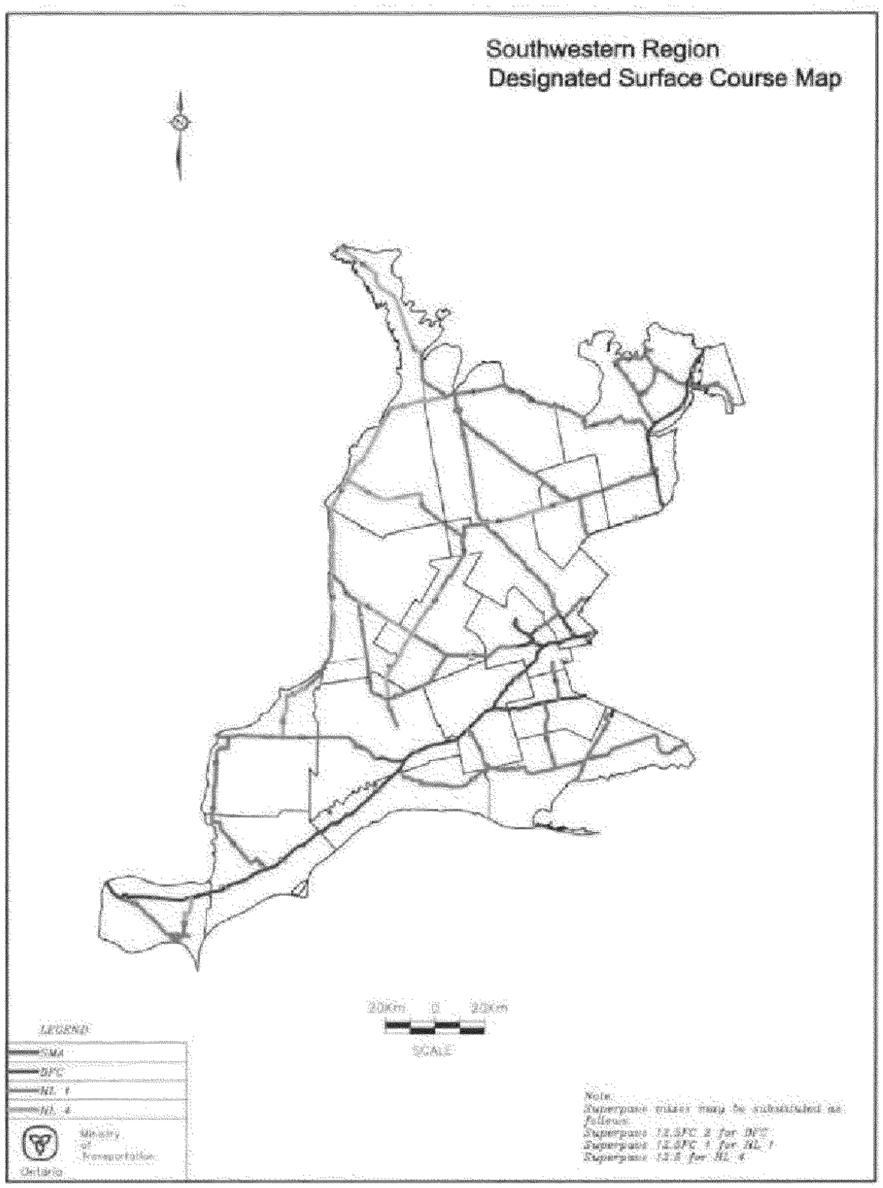
Appendix 1



Appended to Directive PLNG-C-003
Formerly PHY-C-016
Revised August 6, 2003

Appendix 2



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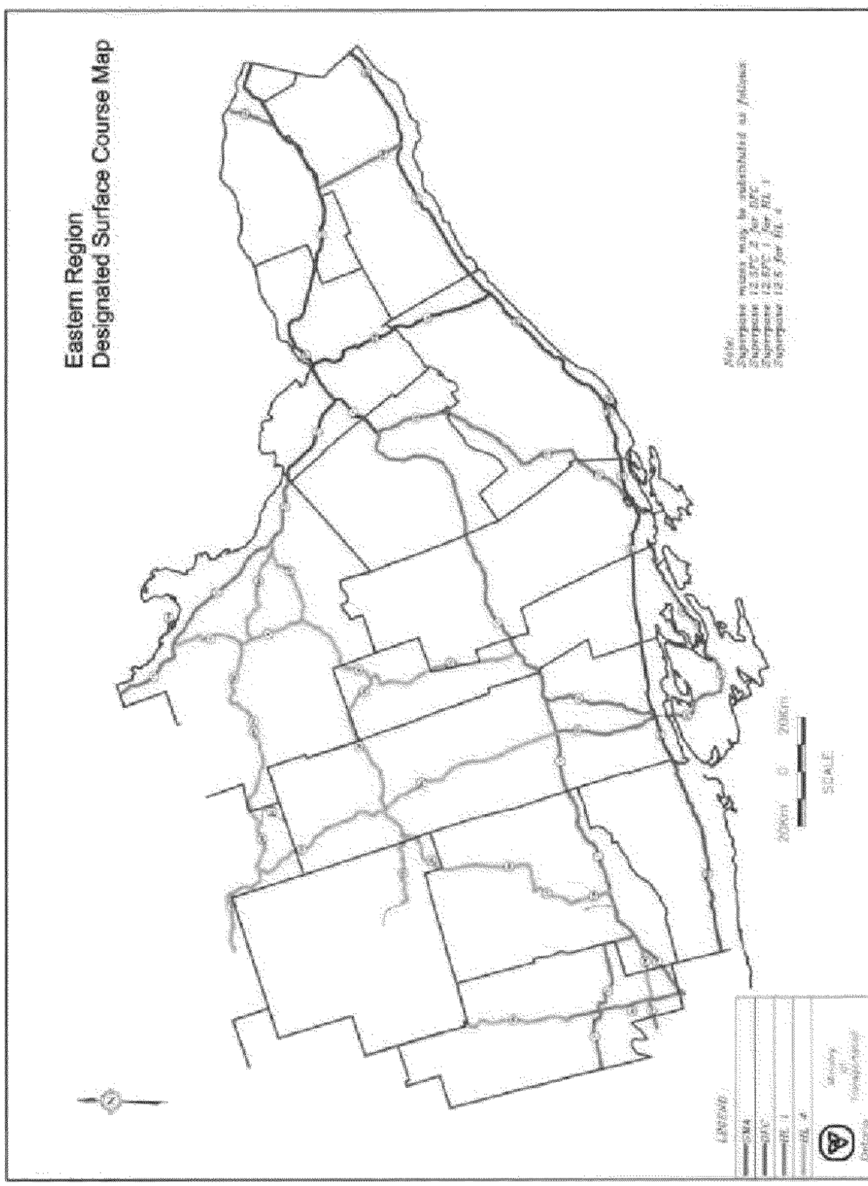
Appendix 3



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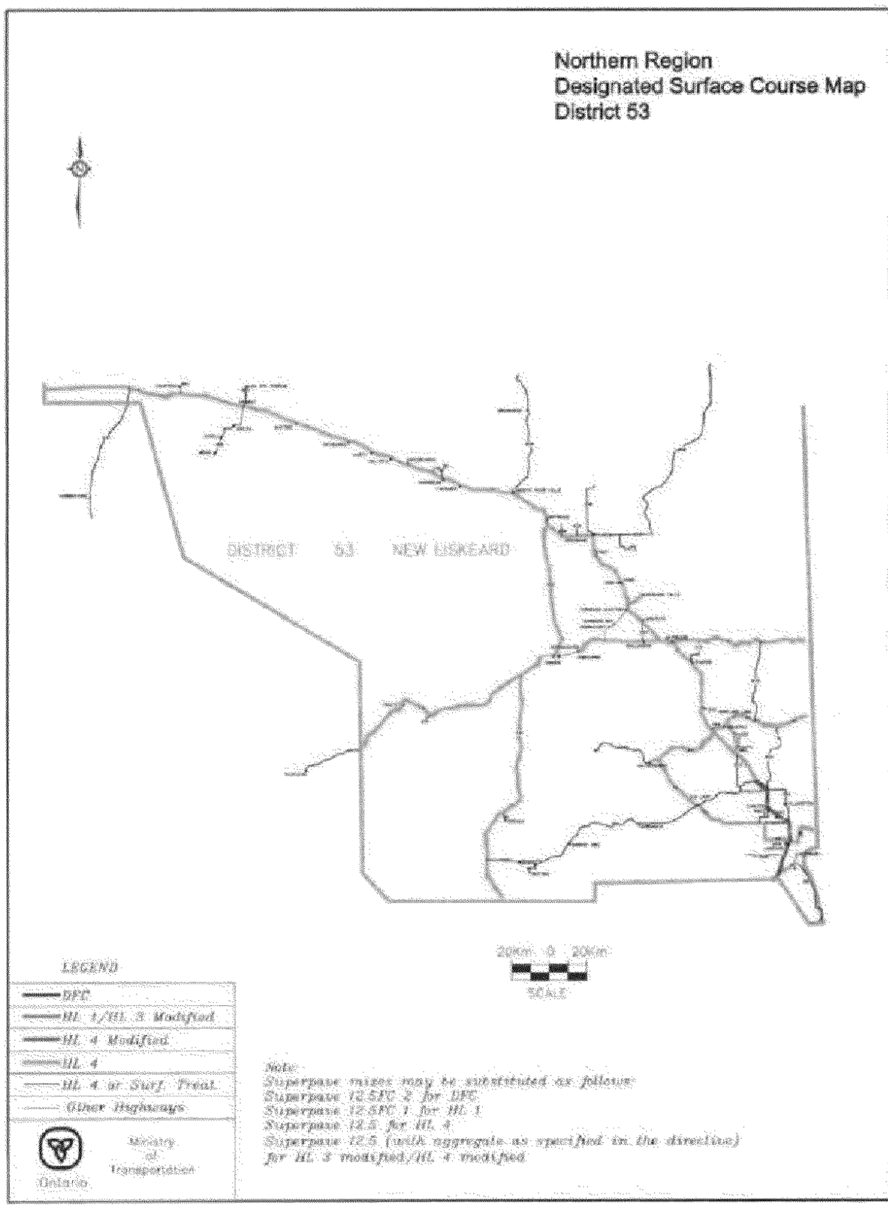
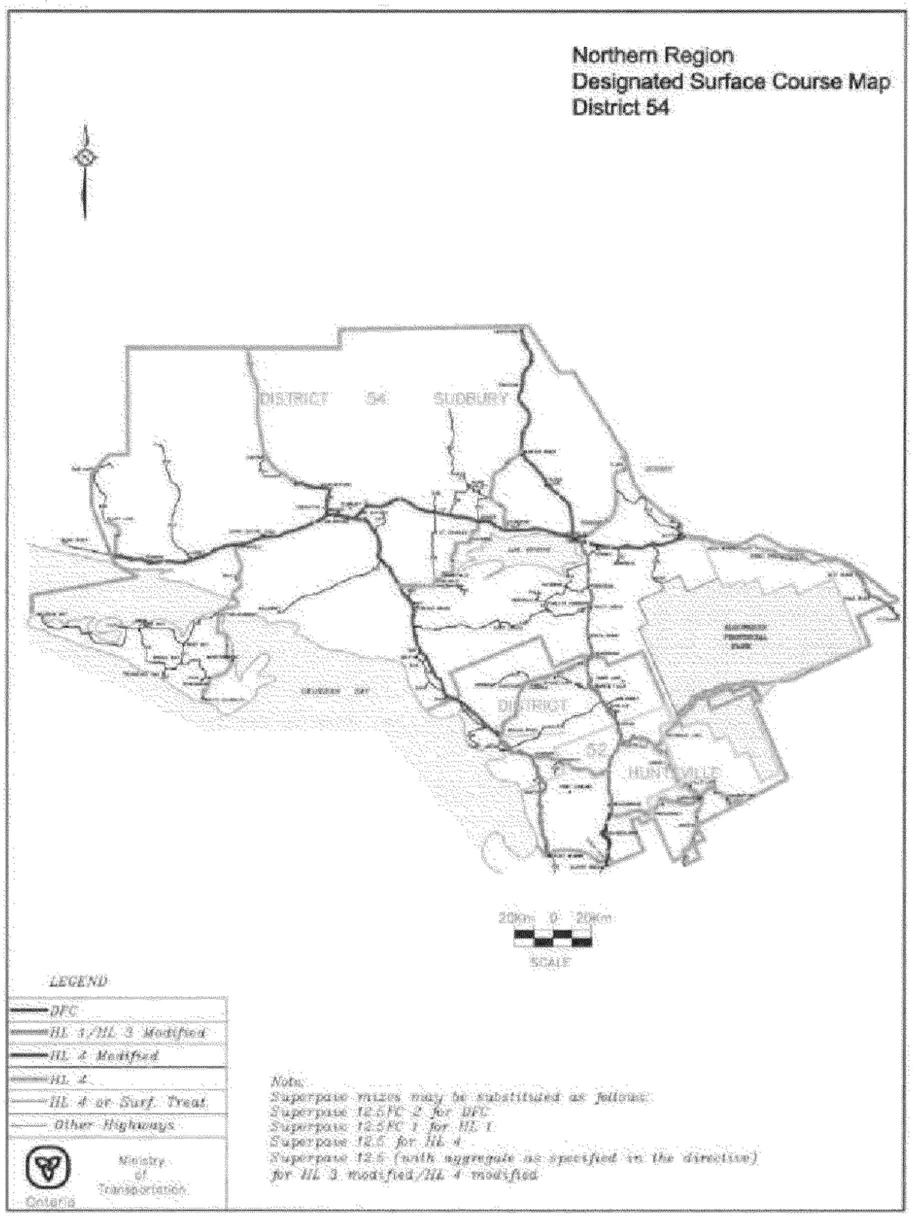
Appendix 4


Northern Region
Designated Surface Course Map
District 53
DISTRICT 53 NEW LISKEARD
20Km 0 20Km
SCALE
LEGEND
DFC
HL 1/HL 3 Modified
HL 4 Modified
HL 4
HL 4 or Surf. Treat.
Other Highways
Ministry of Transportation
Ontario
Note:
Superpave mixes may be substituted as follows:
Superpave 12.5FC 2 for DFC
Superpave 12.5FC 1 for HL 1
Superpave 12.5 for HL 4
Superpave 12.5 (with aggregate as specified in the directive)
for HL 3 modified/HL 4 modified

Appendix 5



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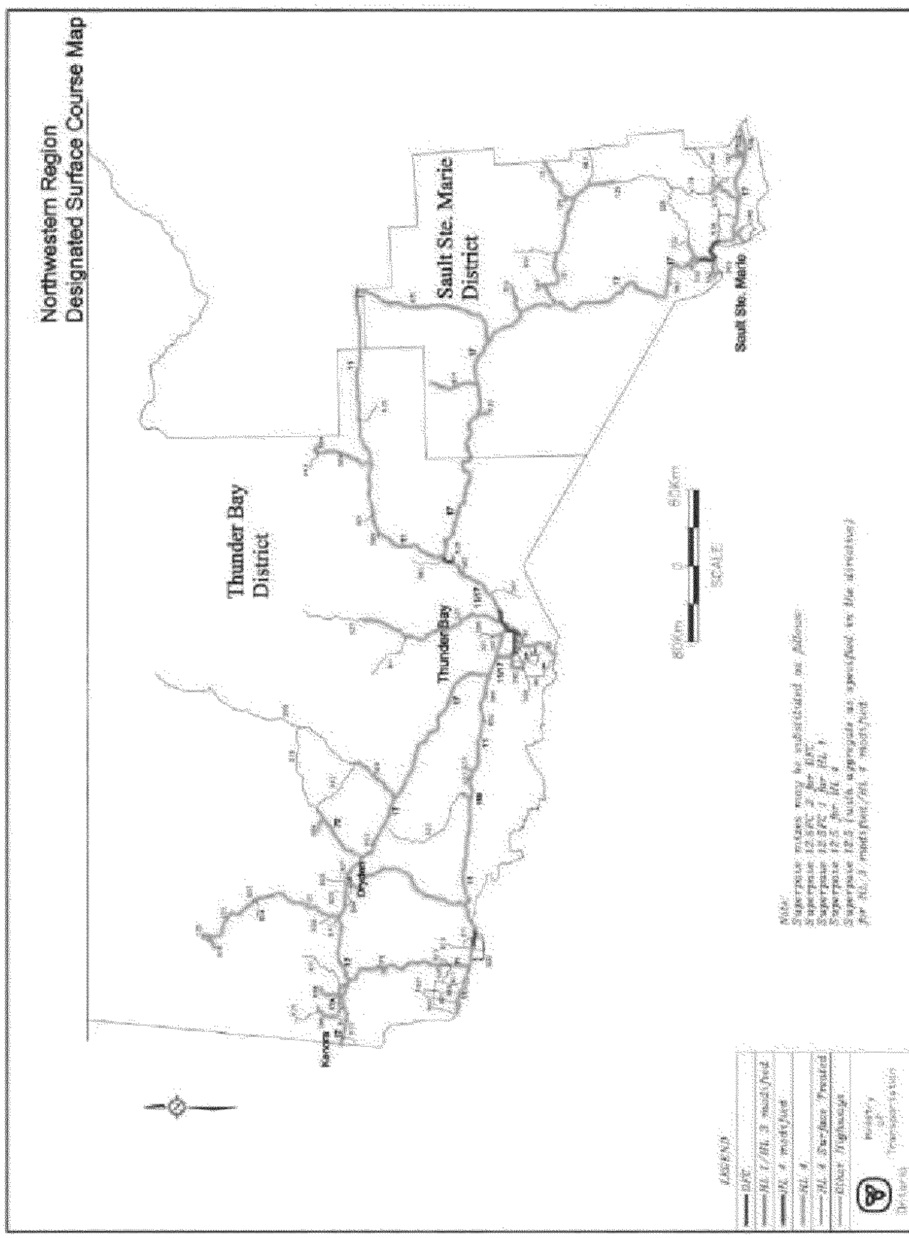
Appendix 6



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