Friction Courses for Moderate Traffic Highways

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The final evaluation of the performance of 17 bituminous test sections constructed in 1978 on Highway 7 near Lindsay, Ontario, is reported. The objective of the trial was to develop suitable surface friction course mixes for highways carrying moderate volume of traffic (about 5,000 AADT) at posted speed limit of 80 km/h. These mixes would provide and maintain adequate levels of surface friction to reduce wet pavement skidding accidents. Both open- and dense-graded type mixes were included in the evaluation. Two standard mixes were incorporated for control purposes. A specialty patented mix called DELUGRIP was also placed in the trial. Aggregates used consisted of crushed gravels, local sand, and screenings of various blends. Frictional properties of the test sections were measured three times within the 6-year monitoring period. Samples of the surface course mixes were periodically taken for laboratory testing and evaluation. Friction results indicate that the coarse aggregate content and quality is a major factor for determining the level of friction achievable in a mix. The mixes found suitable for moderate traffic are those containing at least 25 percent of hard igneous coarse aggregate with the coarse aggregate content in the mix greater than 60 percent. Open friction course mixes using granite/basalt coarse aggregate (without limestone) were found to perform best, but some of the dense friction course mixes also performed satisfactorily. Mixes containing a high proportion of limestone coarse aggregate from local supplies were found unsatisfactory both in terms of friction number and, in most cases, durability.

The Ministry of Transportation of Ontario initiated a research and development project in 1977 to develop hot laid surface course mixes with high-frictional qualities. The main objective of the project was to determine more economical friction course mixes for locations other than heavily trafficked high-speed freeways without resorting to the use of scarce, premium quality aggregates.

In particular, answers to the following questions were sought:

1. Can the friction properties of mixes be improved on by using marginal aggregates available locally?
2. What level of improvement can be expected of blending better quality aggregates and at what extra cost?
3. Would the open-graded surface course mixes using local limestone aggregates provide adequate frictional resistance and durability?

With these points in mind, 17 bituminous surface course mixes were designed and constructed in September 1978. The test site was monitored for 7 years. This paper summarizes the work done on field observation and laboratory evaluation of the performance of the test mixes over 7 years of service.

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MATERIALS

Aggregates

The aggregates employed for the trial are commonly available materials in the Province of Ontario. Coarse aggregates were of igneous gravels from the north and limestone from the south of the province. Fine aggregates were of local sand, local limestone screenings, and igneous screenings from northern Ontario.

The aggregates used in the test mixes were as follows:

1. Coarse aggregates: granite/basalt gravel, limestone gravel, and traprock stone.
2. Fine aggregates: screenings, washed or unwashed; natural sand; and limestone filler.

A brief description of these materials and some properties of the coarse aggregate are given in Table 1. The Maple Ridge igneous material is similar in characteristics to Havelock traprock.

The fine aggregate was from the same source as the coarse aggregates. Washed and unwashed fine aggregates and local natural sand were utilized.

Asphalt Cement

An 85/100 penetration grade asphalt cement was used for all the test mixes except mix No. 16 for which 60/70 penetration grade was obtained from Gulf Clarkson refinery. The penetration value of the original asphalt was 90 for the 85/100 grade and 54 for the 60/70 penetration grade.

Filler

Filler was used in mix No. 16 only. It was of the limestone type with a gradation conforming to the MTO specification (minimum 80 percent passing, 0.075 mm sieve).

MIX DESIGNS

There were several factors considered during the selection of the experimental mixes. Among them, previous MTO experiences with friction course mixes placed in the test sections on Highway 401 Toronto By-Pass (*1*) were taken into account. Technology on friction course mixes from other jurisdictions were also considered.

TABLE 1 CHARACTERISTICS OF COARSE AGGREGATES

Item	PROPERTY	MAPLE RIDGE	BEAMISH	HAVELOCK
1	General Description:	(Gravel)	(Gravel)	(Traprock)
	a) Type of rock/stone	GRANITE/BASALT	LIMESTONE	BASALT
	b) Size used, mm	9.5 or 13.2	9.5 or 13.2	13.2
	c) Crushed material (% by wt) <LS607>	82	75	100
	d) Petrographic Number(PN) <LS609>	104	114	103
2	Los Angeles Abrasion Value (500rev), %loss <LS603>	16	26	12
3	Magnesium Sulphate Soundness (5 cycles), %loss <LS606>	.7	3.0	.7
4	Water Absorption, % by wt <LS604>	.5	.9	.6
5	Polished Stone Value (PSV) <BS812>	46	43	46
6	Aggregate Abrasion Value (AAV) <BS812>	2.2	Not tested	2.2

LS = MTO Laboratory Standard

BS = British Standard

The selected test mixes included

1. Six open friction course (OFC) mixes: 65 percent of coarse aggregate (CA) and washed screenings as fine aggregates (FA)
2. Eight dense friction course (DFC) mixes: 55 percent of CA and various blends of FA.
3. Two standard mixes: HL-3 and HL-1 containing 45 percent of CA and 55 percent of local sand.
4. DELUGRIP mix: designed by Dunlop Ltd.

Blends of coarse and fine aggregate components of the different designed mixes are shown in Figure 1 and aggregate gradation curves in Figures 2, 3, and 4. There is very little difference in aggregate gradation among the open mixes. In the case of DFC mixes, there was one exception: An additional 8 percent of passing 9.5 mm sieve was included in mix No. 7, and, in comparison with mix No. 11 and mix No. 7, it contained less fines passing 0.300 mm sieve. The gradation of standard mixes HL-3 and HL-1 is also plotted.

The DELUGRIP mix is quite different. It was designed to contain approximately 63 percent of coarse aggregate and about 7 percent of fines passing 0.075 mm sieve. It used a harder grade asphalt cement than all other test mixes. It is a unique design (*2*). A summary of the gradations and aggregate types used is given in Table 2.

LOCATION OF TEST SITE

The test site was part of a normal, scheduled resurfacing project on Highway 7 near Lindsay, Ontario, and located west of the junction of Highways 7B and 35. It covers about 2,200 m in length, and each of the 17 sections is approximately 127 m long.

The two-lane roadway is 7.3 m wide with partially paved shoulders to a total pavement width of 8.5 m.

A traffic survey carried out on the test section of Highway 7 showed that average annual daily traffic (AADT) for 1978 was 5,295 vehicles and for 1985 was 5,600 vehicles. For commercial vehicles, E/B, 10.0 percent and W/B, 15.0 percent.

The layout of the test sections is shown schematically in Figure 1. All of the OFC mixes were grouped and placed over

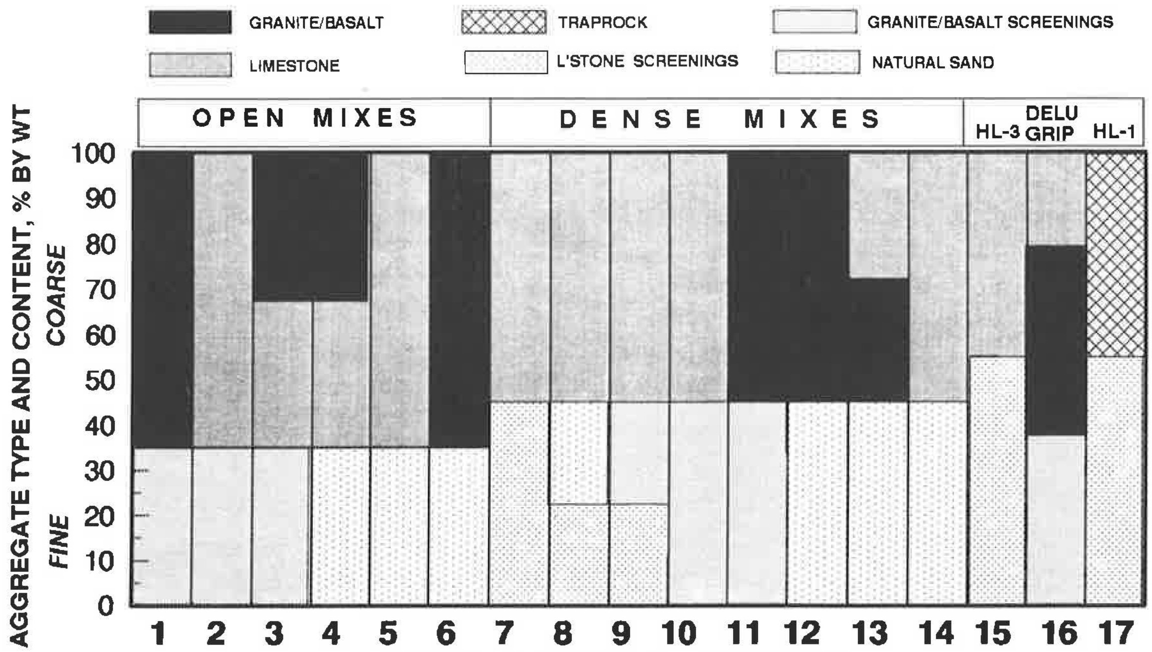

FIGURE 1 Layout of test sections and types of aggregates used in trial mixes (Highway 7, Ontario).

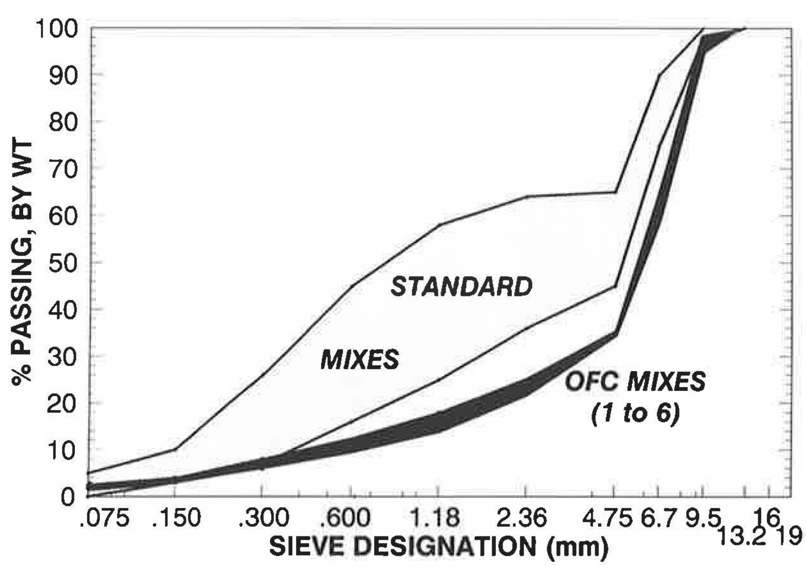

FIGURE 2 Aggregate gradation chart: mixes 1–6.

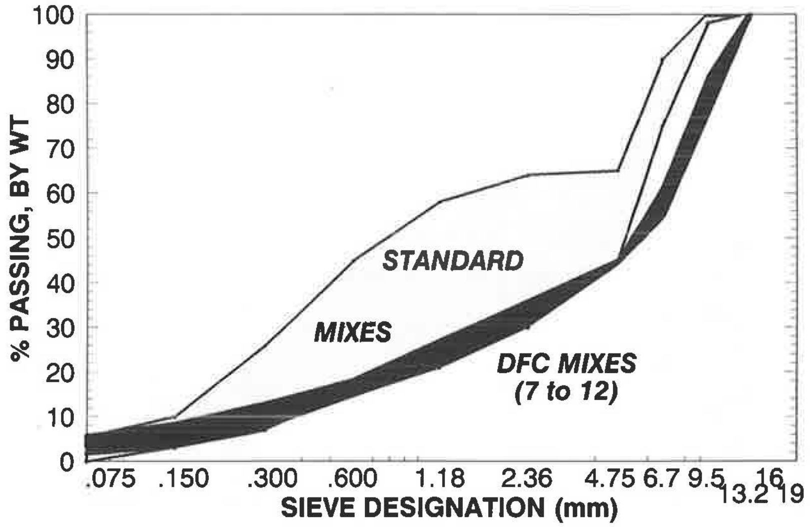

FIGURE 3 Aggregate gradation chart: mixes 7–12.

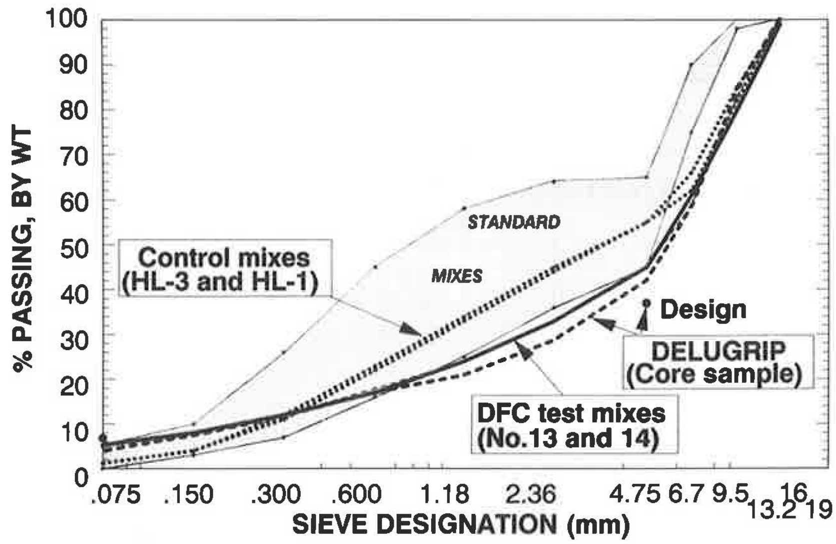

FIGURE 4 Aggregate gradation chart: mixes 13–17.

a 38 mm binder course. The other test sections have been placed on a 19 mm sand asphalt leveling course.

CONSTRUCTION

Details of the production and the construction work are given in Kamel and Corkill (*3*) and summarized as follows:

- Placement of the test mixes was carried out in good weather conditions (mid-September) and was completed in 5 days.
- Open mixes (1–6) and mix No. 16 were compacted by using a 10-ton steel-wheeled roller only. Both steel and rubber-tired rollers were used on all other test sections. The paver was equipped with a vibratory screed.
- No special problems were encountered in placing the mixes.

POSTCONSTRUCTION MEASUREMENTS

Water permeability and ASTM brake force trailer measurements were carried out to determine initial water drainage capability and frictional properties of the experimental mixes, respectively.

A permeability test was carried out within the first week after construction, using a procedure developed by the Johns-Manville Co. (*4*).

Results showed that all of the mixes were too permeable to measure because of the rapid water drainage (>25 ml/min is considered permeable) with the exception of Nos. 15 and 17 (control mixes), which were impermeable, and Nos. 7 and 8, which gave a result higher than 275 ml/min.

Surface friction measurements were carried out for the first time 1 month after construction, in October, using a skid

TABLE 2 MIX DESIGN GRADATIONS

Test Section No.	Aggregate Type		Passing Sieve Size (mm), % By Wt									
	COARSE	FINE	.075	.150	.300	.600	1.18	2.36	4.75	6.7	9.5	13.2
1	M	M	1.4	3	6	10	14	22	35	63	98	100
2	L	M	1.7	4	8	12	17	25	35	59	95	100
3	ML	M	1.9	4	8	12	18	25	35	61	97	100
4	ML	L	1.9	3	6	10	16	24	35	63	96	100
5	L	L	1.8	3	6	10	16	24	35	62	95	100
6	M	L	2.1	4	6	10	14	22	35	64	97	100
7	L	S	1.8	3	8	17	26	35	45	57	86	100
8	L	SL	3.4	6	11	18	26	35	45	56	81	100
9	L	SM	3.4	6	11	18	26	35	45	56	81	100
10	L	M	4.6	7	12	17	23	33	45	55	80	99
11	M	M	4.0	6	10	15	21	30	45	58	78	99
12	M	L	5.3	8	12	17	24	33	45	61	80	99
13	ML	L	5.3	8	12	17	24	33	45	61	83	100
14	L	L	5.5	8	12	18	24	33	45	59	83	100
15	L	S	1.2	4	11	22	33	44	55	66	85	100
16	ML	M	6.9			DELUGRIP			37		MIX	
17	T	S	1.2	4	12	23	34	45	55	62	82	100

SYMBOLS:

Coarse Aggregate	Fine Aggregate
M - Granite/Basalt Gravel	M - Granite/Basalt Screenings
L - Limestone	L - Limestone Screenings
T - Traprock	S - Natural Sand

NOTES:

1) 9.5 mm max. size coarse aggregate was used for test sections 1 to 6 (open mixes) and 16 (Delugrip).
2) Washed screenings were used for test sections 1 to 6.
3) ML,SL,SM are 1:1 blends of respective aggregates, except for section 16 where M to L blend was 1.7:1.

trailer conforming to ASTM E 274, at 50 and 80 km/h. Friction number (FN) was determined for each test section at the two speeds. The results are given in Table 3.

FIELD SAMPLING AND OBSERVATIONS

Sampling

Mix samples were taken both at the plant (at discharge) and from the job site after placement of the mix but before compaction. The samples were tested in the laboratory for mix compositions and Marshall values on recompacted mixes.

Pavement cores were taken from each of 17 test sections within a week after construction of the test sections as well as at the third and seventh year of service. These samples were tested for mix gradation and asphalt cement content, penetration and viscosity of recovered AC, Marshall properties of recompacted mix, and pavement compaction.

Observations

Detailed survey of the test sections was carried out after 6 years of service to establish if there were any relationships between laboratory test results and field observations. Table

TABLE 3 POSTCONSTRUCTION FRICTION NUMBER VALUES

Test Section No.	FN at 50 km/h LANE (S) W/B	E/B	BOTH	FN at 80 km/h LANE (S) W/B	E/B	BOTH
1	47	44	45.5	37	36	36.5
2	42	39	40.5	35	34	34.5
3	44	43	43.5	36	35	35.5
4	40	41	40.5	33	30	31.5
5	38	38	38.0	31	30	30.5
6	44	44	44.0	33	35	34.0
7	43	38	40.5	30	32	31.0
8	40	38	39.0	29	30	29.5
9	38	39	38.5	30	31	30.5
10	44	43	43.5	33	35	34.0
11	48	48	48.0	37	37	37.0
12	46	45	45.5	35	35	35.0
13	44	44	44.0	33	33	33.0
14	41	40	40.5	28	30	29.0
15	40	38	39.0	29	27	28.0
16	48	48	48.0	38	37	37.5
17	46	45	45.5	34	33	33.5

4 gives a summary of the crack map data gathered during the field survey. It includes longitudinal, transverse, and other types of cracks. The pavement surface "Crack Index," based on crack severity and crack type weight factors, was introduced for comparing the performance of different sections. The index was derived from the Distress Manifestation Index (*5*) and relates to crack damages only. Total Crack Index was calculated and ranking numbers were assigned to each of the sections. Also, a quotient of overall crack length to the length of each test section is shown in the table.

It can be seen that the OFC sections showed much less transverse cracking than most of the DFC sections. The difference could be due to variations in the mixes and better base supports on which these mixes were laid. It can be expected that apart from the mix properties the poor pavement base stabilities can account for the increased incidence of crackings and roughnesses.

The test section with a good ranking is No. 5 (i.e., ranked 1 in Table 4), which is followed by No. 2. The poorest ranked mixes in respect to cracking are placed in sections No. 16 (DELUGRIP) and 10 (DFC with 100 percent limestone CA). The heavy sand raveling and cracking found during the survey in those two sections had resulted in the harsh surface texture leading to an increase in the Friction Number (FN) values.

LABORATORY EVALUATION

Aggregates

As the type of aggregates used in a mix determines the frictional properties and durability of pavement surface, factors such as aggegate abrasion, susceptibility to polishing, absorption, gradation, nominal size, percentage crushed, particle shape, and cleanliness need to be carefully considered during mix designs. Some of these factors and their relationships to friction and durability are examined in the following evaluation.

The performance of aggregates was evaluated by the change in their gradations over the years of service. It was found that mixes containing relatively soft coarse aggregate (e.g., limestone) had the most change in gradation (mixes No. 2, 5, 8, 10), whereas mixes containing very hard coarse aggregates changed very little (mixes No. 3, 6, 7, 9, 15, 17).

The changes in gradation could occur both internally in the matrix owing to degradation and cracking and externally by the action of tire wear and weathering of the aggregates. The effects of these factors were observable in the field where severe raveling took place in sections No. 2, 5, 8, and 10. These mixes had the highest changes in gradation curves (Figures 2–4).

TABLE 4 PAVEMENT CONDITION RANKING RESULTS

Test Section No.	Pavement Surface Crack Index* PER CRACK TYPE				Crack per Section Length (m/m)	Surface Condition Ranking**
	Longitu- -dinal	Trans- -verse	Other Cracks	TOTAL		
1	265.0	8.5	0.0	273.5	1.89	5
2	132.5	74.0	0.0	206.5	1.79	2
3	299.0	70.0	6.0	375.0	2.54	7
4	199.0	7.0	3.0	209.0	0.88	3
5	143.0	0.0	14.5	157.5	0.98	1
6	175.5	0.0	91.0	266.5	1.65	4
7	160.5	50.0	272.0	482.5	2.74	11
8	217.0	27.5	93.5	338.0	1.84	6
9	205.5	190.5	59.5	455.5	1.90	9
10	443.0	191.5	114.0	748.5	3.21	16
11	325.0	84.0	1.5	410.5	1.83	8
12	456.0	110.0	75.5	641.5	3.05	14
13	376.5	158.0	125.5	660.0	2.84	15
14	332.0	144.0	81.0	557.0	2.78	13
15	88.5	282.5	121.0	492.0	2.25	12
16	810.5	341.5	238.0	1390.0	5.30	17
17	51.0	136.5	287.5	475.0	1.42	10

*) The index is defined as:
SUM[SUM(crack length * severity weight factor)] * type weight factor

**) The test sections are ranked by number from 1 to 17 based on the crack index; 1 represents the highest rank, 17 - the lowest

Asphalt Cement

Asphalt cement content of the test mixes from samples taken during the monitoring period was within the ±0.3 percent deviation limits. Because of aggregate absorption and weathering effects, the amount of AC extracted from the mixes was slightly lower than the initial results; that is, the average change was OFC mixes, 0.6 percent; DFC mixes, 0.3 percent; DELUGRIP, 1.0 percent; and control mixes, negligible.

The hardening effects of the AC after 7 years are reflected in the retained penetration and the increased viscosity. These changes are shown:

Mix Type	*Penetration* (% ret.pen.)	*Viscosity* (% increase)	*Highest change*
OFC	30.0	168	Mix No. 5
DFC	46.5	91	Mix No. 10
DELUGRIP	35.7	362	—
Control	75.2	18	Mix No. 15

Because asphalt cement aging is closely related to the air void content of asphalt mixtures, the biggest change took place in mixes with high void content and high proportion of limestone aggregates (Figures 5 and 6). DELUGRIP mix, containing a harder original AC, had a percentage of retained penetration between the OFC and DFC. For mixes with the same air void content, variations in penetration and viscosity values obtained could be due to the hardening of AC by temperature fluctuations during the production of the mixes.

The aging in the asphalt cement in mixes No. 5, 6, and 10 was the worst, whereas the least occurred in control mixes (HL-1 and HL-3). The AC in mix No. 7 aged much less than in all other dense mixes because of the lower air void content.

Mix Properties

The mix properties changed with time at different degrees during the 7 years of service. Marshall test results on recom-

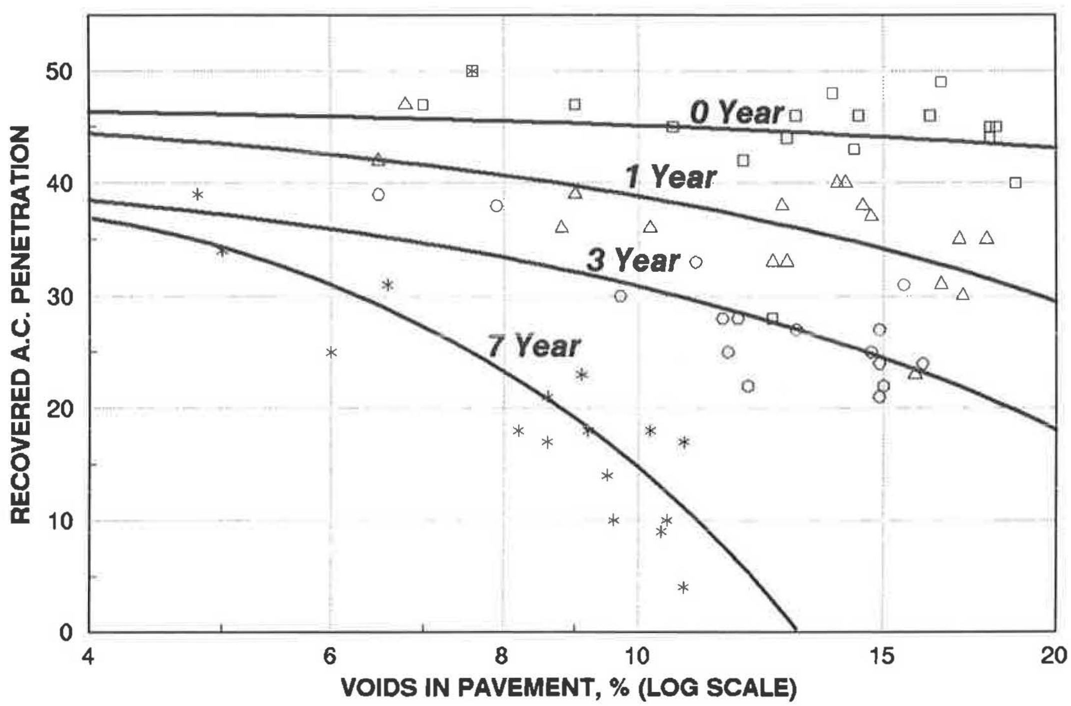

FIGURE 5 Penetration of recovered AC versus air voids content in pavement.

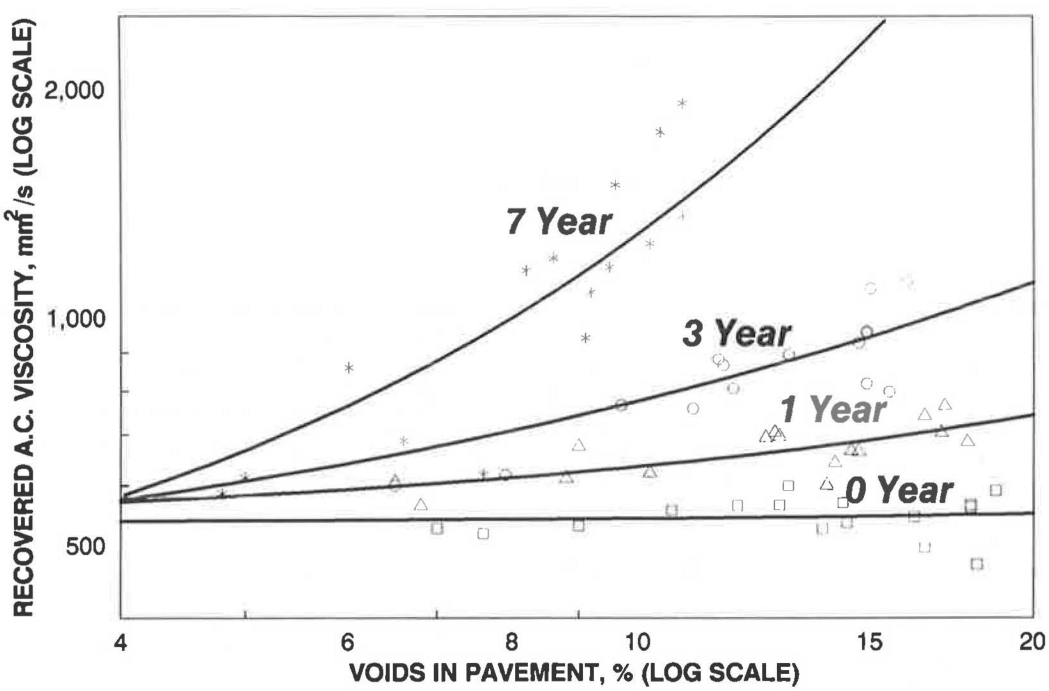

FIGURE 6 Viscosity of recovered AC versus air voids content in pavement.

pacted mixes are summarized in Table 5. In an effort to determine the optimum mixes for durability and friction, factors such as voids in the mineral aggregate (VMA), stability, AC content, CA content, interrelation between Marshall stability, voids in mineral aggregate, and the optimum AC content are examined.

The recompacted air void content of the mixes increases relative to the original voids at construction by an average of 3 percent. The increase is slightly higher for OFC mixes owing to more hardening of the AC (Figures 5 and 6). The control mixes (15 and 17) were the least susceptible to weathering and they had the lowest increase in voids at the seventh year.

From the results, a steady increase in stability was observed during the first 3 years. However, some of the mixes (Nos. 2, 3, 5, 7, 8, 15, and 17) had lower values at the seventh year because of the changes in gradation and variations in sampling locations. Consequently, some of the Marshall stiffnesses dropped below the initial levels. The DELUGRIP mix became stiffer by approximately 126 percent. These changes reflect the poorer potential of the mix to resist cracking and subsequent deterioration.

It was found that the relationship between Marshall design stability and the void content in mineral aggregate (VMA) can be used as an indicator for frictional performance of surface courses. As illustrated in Figure 7, there is a good correlation between the VMA and mix design stability values (correlation coefficient $r = .87$). This is an indication that within the range analyzed, statistically about 75 percent of the changes in stability can be attributed to the change in percent VMA. Figure 7 shows that mixes with $FN_{80} > 30$ are located above the line drawn for the graph between stability and VMA. The equation is

TABLE 5 ASPHALT MIX CHARACTERISTICS SHORTLY AFTER PLACEMENT AND AFTER 7 YEARS OF PAVEMENT SERVICE

Test Section No.	Years	RECOMPACTED MIX Voids in Mix %	MARSHALL Flow mm	MARSHALL Stability N	Stiffness (Quotient) N/mm	Voids in Pavement %	Pavement Compaction %
1	0	5.0	3.9	10050	2584	16.5	87.6
	7	6.9	6.0	17004	2848	8.6	98.1
2	0	5.7	3.6	10890	3059	18.1	86.5
	7	7.4	6.4	14345	2259	10.2	97.0
3	0	4.6	4.1	10350	2518	16.2	87.7
	7	7.2	6.4	13195	2078	9.5	97.5
4	0	5.2	3.5	11960	3437	17.9	86.6
	7	10.1	5.1	17374	3400	10.8	99.1
5	0	5.6	3.6	10550	2963	17.9	87.1
	7	9.8	4.1	15679	3787	10.8	98.9
6	0	5.4	3.9	10765	2753	18.7	85.9
	7	9.1	4.7	15790	3324	10.4	98.5
7	0	2.3	3.9	12554	3211	9.0	93.2
	7	4.0	3.7	14892	3961	6.6	97.3
8	0	2.6	4.2	13341	3199	10.6	91.6
	7	4.5	4.0	16436	4271	6.0	97.0
9	0	4.1	3.7	14247	3820	12.8	90.7
	7	6.3	4.2	19583	4820	9.1	97.1
10	0	4.3	4.3	13446	3113	14.3	90.0
	7	8.1	5.6	18895	3381	9.6	98.4
11	0	3.2	4.5	12693	2808	13.8	89.0
	7	5.6	6.0	17503	2897	9.2	96.6
12	0	2.8	5.3	13612	2588	14.4	88.1
	7	5.6	5.2	20127	3880	8.6	96.8
13	0	3.2	4.8	12843	2687	13.0	89.9
	7	5.8	5.3	18270	3398	8.2	97.4
14	0	2.1	5.1	11877	2315	11.9	90.0
	7	5.9	5.3	18549	3566	7.6	98.3
15	0	2.2	3.6	12596	3518	7.6	94.5
	7	3.1	3.4	16569	4820	5.0	99.3
16	0	3.7	5.3	11159	2094	12.5	90.4
	7	8.7	4.2	20018	5047	10.5	98.1
17	0	2.0	3.8	14019	3709	7.0	94.9
	7	3.1	3.9	17129	4347	4.8	98.3

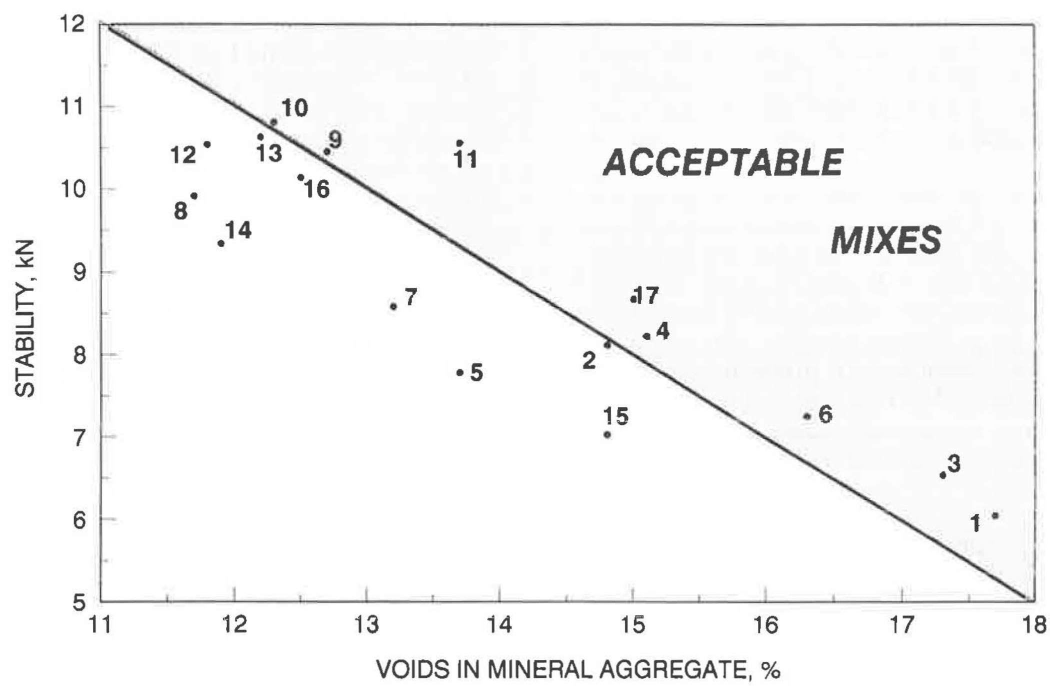

FIGURE 7 Mix design stability versus VMA.

Stability (kN) = 23 − VMA (%)

It must be stressed that the relationship is developed based on the results obtained from mixes used in this trial only. This equation or a similar one may also be applicable to other mixes.

FRICTIONAL CHARACTERISTICS

Friction Number and Mix Type

Further to the initial measurement at 1 month after construction, testing of frictional properties was performed after the second and sixth year of pavement service. The Breaking Force Trailer was used, and the FN values were taken at 80 km/h only (Table 6). What follows shows the changes from the initial FN values for the different types of mixes:

Mix Type	*At 2 years*	*At 6 years*
OFC (1–6)	−1.15	+2.62
DFC (7–14)	−2.26	+0.99
HL-3 (15)	−4.90	−1.90
HL-1 (17)	−0.20	+6.40
DELUGRIP (16)	−4.30	+1.25

OFC mixes had an average FN value of about 1.6 units higher than DFC mixes at the sixth year but lower by 1.1 units in the second year. The results show that friction level of FN_{80} > 30 can be obtained from both OFC mixes (1 and 6) and DFC mixes (11, 12, 17) (Figure 8).

Friction Number Versus Coarse Aggregate in the Mix

Friction values obtained varied with different mix compositions. For the two periods monitored (second and sixth year) the greatest overall decrease in FN, ranging from one to five units, took place on test sections Nos. 2, 5, 7, 8, 14, and 15, where limestone coarse aggregate and limestone and/or sand fine aggregate were used (except for mix No. 2). The highest relative increase in FN value over these years occurred in test sections Nos. 4, 6 and 17. All of them contained hard coarse aggregate and relatively soft fines. There was no significant change in frictional values for other test sections.

Mixes containing more of the crushed aggregates performed better in respect to frictional properties. A correlation coefficient of 0.71 (Figure 5) was obtained between FN and percent crushed CA. This shows that more than 50 percent ($r^2 \times 100$) change in FN_{80} can be directly related to the crushed coarse aggregate content in the mixes. In general terms and within limits the correlation means that to increase the FN_{80} by 1 unit (at sixth year) an increase of crushed coarse aggregate content by about 1.4 percent is required.

All of the mixes with good quality crushed coarse aggregate at content of >50 percent had FN_{80} > 36 after 6 years of service. Mixes containing limestone coarse aggregate with low content of crushed particles (i.e., <40%) and natural sand had FN_{80} value <30. Figure 8 illustrates a strong dependence of friction level at both second and sixth year of service on the content of hard crushed gravel in the coarse fraction of the mix. Figure 8 also shows that to provide friction level of FN_{80} > 30 throughout the 6-year period monitored, the percentage of crushed igneous coarse aggregate content in a mix should be at least 25 percent.

CONCLUSIONS

- Imported premium quality aggregates (e.g., Maple Ridge or Havelock) can significantly improve the performance of

TABLE 6 FRICTION NUMBER VALUES AT 80 KM/H

Test Section No.	LANE					Average of Both Lanes	
	Westbound			Eastbound			
	Years After Construction						
	0	2	6	0	6	0	6
1	37	36	38	36	38	37	38
2	35	30	33	34	34	35	33
3	36	34	38	35	37	35	38
4	33	33	38	30	36	32	37
5	31	28	32	30	31	31	32
6	33	37	41	35	40	34	40
7	30	26	29	32	28	31	28
8	29	26	29	30	27	29	28
9	30	26	32	31	33	30	32
10	33	29	34	35	37	34	36
11	37	36	41	37	39	37	40
12	35	37	39	35	38	35	38
13	33	32	37	33	33	33	35
14	28	25	30	30	29	29	29
15	29	24	27	27	25	28	26
16	38	34	39	37	38	37	39
17	34	33	40	33	39	17	40

surface friction courses. For moderate traffic roads, the coarse aggregate should contain at least 25 percent of hard igneous aggregate in the total aggregate mix.

• Sands and limestone available locally (as those used in the experiment) in southern Ontario are not considered suitable for use alone in asphalt mixes for moderate traffic roads to provide satisfactory frictional characteristics.

• Open-graded mixes employing 100 percent of local aggregate (limestone, natural sand) did not perform satisfactorily either in terms of friction or durability. However, open mixes generally achieved slightly higher friction values than dense mixes using the same local aggregates.

• OFC mixes containing about 65 percent of coarse aggregate in total aggregate mix, at least 25 percent of high quality coarse aggregate in total aggregate mix, and washed screenings performed the best among the 17 mixes.

• Both open and dense friction course mixes can be designed to provide satisfactory level of friction over a long period of time. Mix No. 11 (dense, without limestone) is an example of a good dense friction course.

• Control mix (HL-1), composed of traprock coarse aggregate and natural sand, performed well both in terms of frictional properties and durability.

• DELUGRIP mix, with blends of hard and soft coarse aggregates and screenings as fine aggregate, performed satisfactorily initially but cracked severely at the end of the 7-year period. The friction values remained relatively high.

RECOMMENDATIONS

Mixes with the best achievable frictional properties could be costly owing to the need for importing high quality aggregates. It has been found elsewhere (*6*) that wet accident rate at AADT in the range of 5,000–10,000, on rural (80 km/h speed) highways, remains relatively insensitive to FN values. In this context, the frictional properties of asphalt mixes should not be considered as a major priority in mix designs (except for accident black spots). Instead, friction property should be considered as equally important as other factors such as durability.

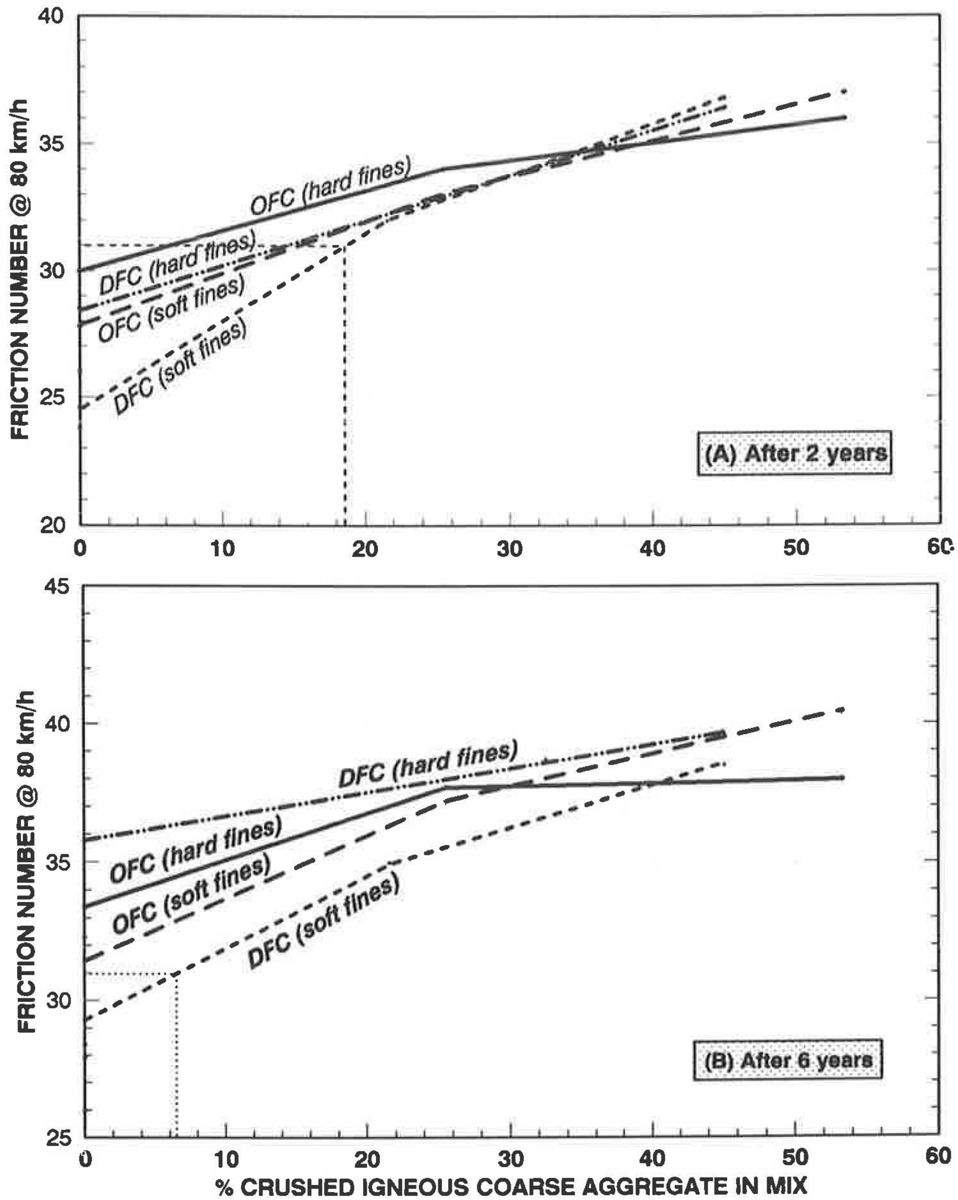

FIGURE 8 Relation of friction number at 80 km/h to proportion of crushed aggregate in mix.

In view of this and based on the results of the experiment, the following general guidelines are recommended for the design of friction mixes for moderate traffic highways:

- Avoid the use of all soft limestone aggregates in both the coarse and fine aggregates. However, if this is not possible, a mix should contain at least 25 percent of blended crushed hard-rock coarse aggregate (>4.75 mm) in the total mix in the coarse fraction.
- Continue the current practice of using a softer asphalt cement grade (e.g., 85/100 in southern and 150/200 in northern Ontario) to prevent the premature cracking and deterioration of a surface course, especially on a weak base.
- The following relationship can be used as a guide for selections of mixes (using an 85/100 AC grade) for frictional properties when other mix design criteria are met:

Marshall Stability (kN) > 23 − VMA (%)

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