Performance of 18 Bituminous Test Sections on a Major Urban Freeway During 11 Years of Service

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This paper summarizes the results of a field trial conducted on one of the most heavily trafficked roads in North America, Highway 401, which carried some 250,000 vehicles in average daily traffic (ADT) in 1985. The trial involved 18 test sections representing a comprehensive range of asphalt surface course mixes. Mix designs ranged from sand mix to high-stone-content mixes of various materials and compositions. The conditions of the test sections were monitored periodically for 11 yr, during which samples were obtained for laboratory testing and analysis. Although the primary objective of this study was to find out which mix designs would most improve the frictional characteristics of urban freeways, other parameters relating to material properties and performance of the mix designs, and their various relationships, were also evaluated. The evaluation of these test sections led to formation of policies regarding the use of high-quality asphalt mixes on Ontario highways. All the test sections performed better than expected for single-course thin overlays on concrete pavement under heavy traffic. Open and dense mixes performed equally well. It was found that, for optimum friction characteristics for high-speed roads, a surface must possess sufficient macrotexture for bulk surface water drainage and sufficient microtexture for penetrating the remaining thin-water film in the contact area. To design mixtures with these properties, the coarse aggregate content should be set above 50 and 60 percent for dense and open friction-course mixes, respectively. Good-quality, polish-resistant aggregate should be used for both the coarse and fine aggregates.

Increased understanding of the mechanism of frictional behavior of the tire-to-road surface interface in the late 1960s has led to the search for and development of methods to design asphalt mixes with better surface friction characteristics. In Ontario, the catalyst for the search for techniques to improve the highway, as well as for a hot-mix surfacing system that produces long-wearing and good friction properties, was the need to rehabilitate and upgrade a section of the Highway 401 Toronto Bypass. In response to such demands, an evaluation program to determine the most suitable designs to provide the desirable driving qualities for freeways under anticipated heavy traffic was initiated in 1972. In the summer of 1974, 18 different designs of bituminous-overlay test sections were constructed. This paper describes the evaluation and performance of the test sections over 11 yr of service.

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LOCATION OF TEST SECTIONS

The site selected for the test sections was the westbound core lane between the Allen Expressway and Jane Street on the Toronto Bypass. The pavement structure consisted of 230 mm (9 in.) of reinforced concrete overlaying 305 mm (12 in.) of granular base. The concrete pavement was selected for this highway because it was expected to have long service life under the anticipated heavy traffic conditions. However, within 10 yr of construction, the original burlap drag-and-broom textured concrete pavement surface was polished by the heavy traffic. The rapid reduction of surface friction had prematurely shortened the serviceability of the pavement. The existing concrete pavements were grooved to improve frictional properties. This technique, although somewhat effective for a while, created excessive tire noise.

The annual average daily traffic (AADT) for this section of the Toronto Bypass was 179,550 in 1974 and 260,600 in 1985 (Table 1). The allowable truck weight and axle loadings were 63.5 tons and 10.6 tons, respectively. Truck traffic was about 17 percent for both directions. The percent-traffic split of the three westbound core lanes is about one-third for each lane (Table 2). The relative proportions of the traffic split remained the same from 1975–1985. However, the driving lane carried about 65 percent of the truck traffic, as compared with 33 percent and 2 percent for the center and passing lanes, respectively.

The weather around Toronto is moderated by the Great Lakes with alternate flows of warm, humid air from the Gulf of Mexico and cold, dry air from the Arctic. The southern air masses are as influential as the northern ones, so winters are milder than in midcontinental areas of the same latitudes.

TABLE 1 TRAFFIC VOLUME AND TRUCK WEIGHT

YEAR	AADT	GVW (tonnes)	LEGAL AXLE WEIGHT (tonnes)
1963	77,000	32.0	8.6
1974	179,550	63.5	10.6
1985	260,600	63.5	10.6

TABLE 2 WESTBOUND CORE LANE TRAFFIC SPLIT DATA

YEAR	DRIVING LANE		CENTRE LANE		PASSING LANE	
	AADT	% TRUCK	AADT	% TRUCK	AADT	% TRUCK
1975	12,900 (29)	65	17,300 (39)	33	14,600 (32)	2
1985	19,431 (35)	N/A	20,382 (31)	N/A	17,887 (31)	N/A

Notes: ( ) = % of total core lane traffic
N/A = Data not available

A mean daily maximum temperature for the summer is about 27°C (81°F); minimum for the winter is −8.5°C (17°F). The degree-days below 0°C is about 600 per year, whereas the degree-days above 18°C is 300. The mean annual precipitation is 765 mm.

The 18 test sections were all 137 m (448 ft) long and 11.3 m (37 ft) wide. Sections 1–10 and 17 were 38-mm (1.5-in.) thick overlays, whereas Sections 11–16 were only 25 mm (1 in.) and Section 18 consisted of two lifts of 38-mm (1.5-in.) overlay.

MIX DESIGNS AND MATERIALS

Eighteen different trial sections were placed to examine the various mix design factors affecting the frictional properties of road surfaces. These mix design features are

1. Type of aggregate,
2. Coarse aggregate content,
3. Fine aggregate content,
4. Different blends of fine aggregates, and
5. Use of asbestos fiber and mineral filler.

In recognizing the contributions of macrotexture and microtexture to surface friction characteristics, mixtures of varying ratios of coarse-to-fine aggregate as well as blends of different aggregate types were selected. The selection of mix designs was also based on experience from previous small pilot projects and a review of mixes used by other jurisdictions. The test sections included both dense- and open-graded mixtures with a variety of coarse and fine aggregate types, including traprock, steel slag, and blast furnace slag, as shown by Tam and Lynch (*1*, Table 2). A mastic mix was also included in the trial.

Test sections 1–10 and 18 consisted of HL-1 mixes in which the coarse-aggregate content was progressively increased to obtain a greater density of stone particles at the surface. The HL-1 mix is a standard, dense-graded surface course mix used on main highways in Ontario; it generally consists of either crushed traprock or slag coarse aggregate and locally available fine aggregates. Different types of fine aggregate ranging from traprock screenings to natural sand were used in these 10 sections, however.

Mixes in Sections 11 and 12 were sand asphalt mixes using traprock screenings as fine aggregate. Both mixes contained a small percentage of oversized particles in the form of 6-mm traprock chips and asbestos fiber filler.

Sections 13 and 14 consisted of open-graded mixes designed to have high permeability characteristics that facilitate rapid drainage of surface water into the surface course layer. The mixes used a large proportion (67 percent) of single-size coarse aggregate (9.5 to 4.75 mm) and a small amount of washed fine aggregate. Mixes in Sections 15 and 16 had only 30 percent coarse aggregate and the same washed, fine aggregate as for mixes in Sections 13 and 14.

Section 17 consisted of a mix called "Mastiphalt," which is a kind of mastic asphalt derived from the German *Gussasphalt* technology. The modification was made so that the material could be mixed and laid by conventional equipment.

The composition of the mix designs was shown elsewhere (*1*, Table 2). Except for mixes 11, 13, 14, and 17, the percentages retained on the 4.75-mm (No. 4) sieve are quite close to the target proportions (column 8 versus column 3, respectively). The coarse aggregate proportions of mixes 11, 13, 14, and 17 are about 5–10 percent finer. The gradations of the mixtures are shown in Figure 1 (see also the discussion of material properties, below).

The high-quality coarse aggregates used were traprock, blast furnace slag, and steel slag, possessing good polished-stone values (PSV) of 43, 52, and 56, respectively. The fine aggregates were either a combination of natural sand and limestone screenings or screenings from the same source as the coarse aggregates. The bulk relative density (BRD) values of the aggregates are listed below. The grade of asphalt cement (AC) used for all the mixes was 85/100 penetration except for the mix in Section 17, where a 60/70 penetration grade AC was used.

Aggregate	*BRD*
Coarse	
Traprock	3.21
Steel slag	3.42
Blast furnace slag	2.58

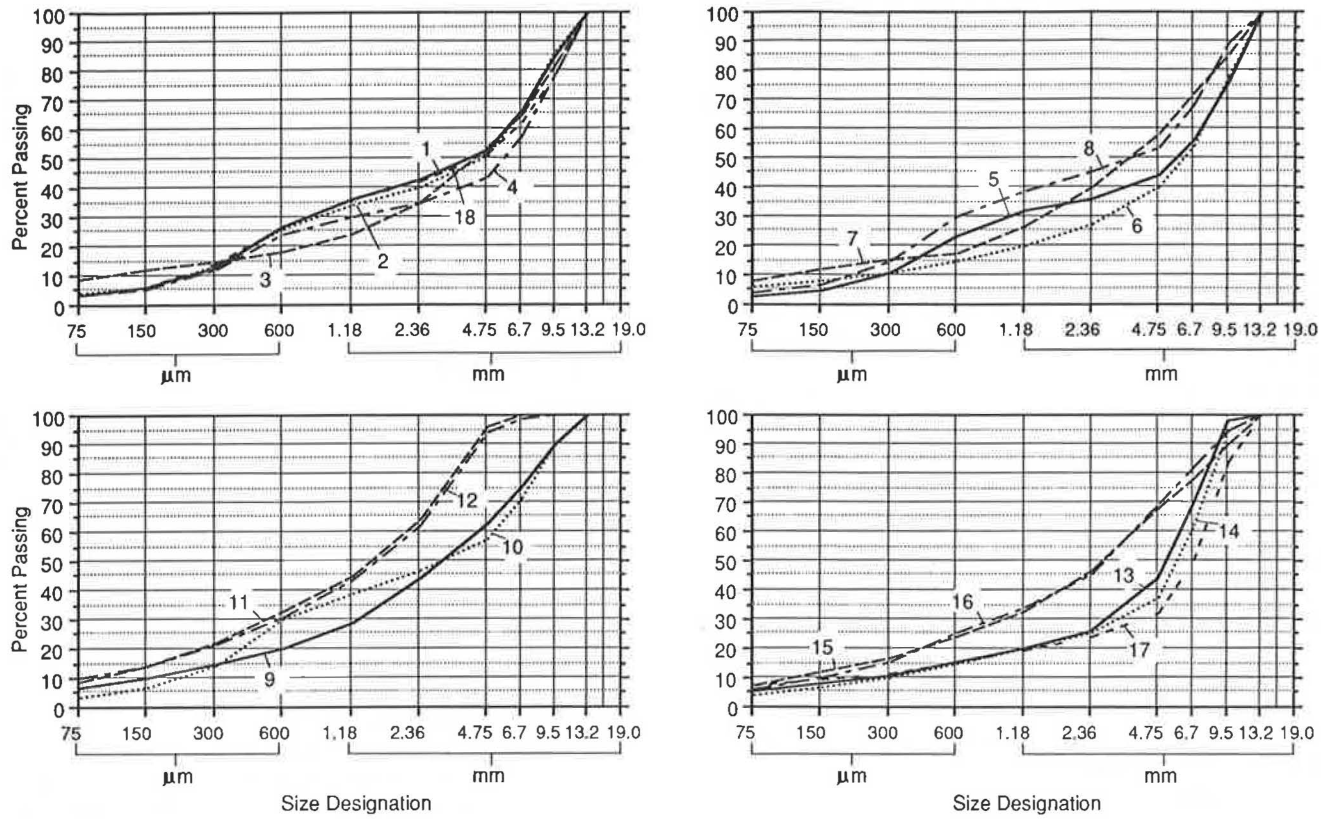

FIGURE 1 Aggregate gradation from cores taken at 11 years (average of cores from the three lanes).

Aggregate	*BRD*
Fine	
Traprock screenings	3.19
Steel slag screenings	3.48
Blast furnace slag screenings	2.93
Limestone screenings	2.68
Natural sand	2.73
Asbestos	2.69
Limestone filler	2.70

CONSTRUCTION

The mixing plant was a Madsen 3630-kg (8,000-lb) batch plant, situated about 40 km (25 mi) southwest of the test sections. The plant produced one or two batches of the desired mix, samples from the mix were tested for compliance with the mix design requirements, and any necessary adjustments to the blend or batch weight were made. The asbestos and/or mineral filler was added by using a vertical bucket elevator and belt conveyor to the weigh hopper. A normal mixing time (40 sec) was extended for those mixes with asbestos because of lumps appearing in the mixes (e.g., for the mix in Section 17, which contained 9 percent mineral filler and 2 percent asbestos, a total mixing time of 70 sec was necessary). The construction commenced on July 22, 1974, and was completed a month later (*2*).

Because of heavy traffic, only night paving was permitted. The lighting system consisted of one telescopic tower with double floodlight and four spotlights accompanying the spreader. The asphalt overlays were placed with a conventional Cedarapids tracked paver and compacted with a 9072-kg steel-wheeled breakdown roller, an 8165-kg rubber-tired intermediate roller, and a 7258-kg steel-wheeled finishing roller. The rubber-tired roller could not be used for compacting the mixes containing asbestos filler because the tires picked up too much of the mix. The other mixes did not present this problem. The shoulders were paved with a Pavemaster driveway spreader since working space on the shoulder was limited.

To reduce any reflection cracking in the overlay, joints were sawn and sealed with hot-poured rubberized asphalt sealer directly over the existing joints in the underlying concrete pavement.

PERIODIC MONITORING AND FINAL EVALUATION

Because the main thrust of the project was to develop surface course mixes that would provide adequate and long-lasting friction properties for heavy-traffic sites, monitoring efforts were devoted primarily to obtaining comprehensive data on the frictional characteristics of the test sections. At the final evaluation, however, extra samples were taken from the test sections so that additional data on material performance could be analyzed.

Initial Evaluation and Policy on Friction Course

Initial findings suggested that mixtures that provide good friction value are (a) dense graded, with both coarse and fine aggregates consisting of traprock, steel slag, or blast furnace slag; or (b) open graded, with high stone content using traprock coarse and fine aggregates (*3*).

On the basis of the performance of these test mixes, the Ministry of Transportation in Ottawa introduced a policy in 1978 governing the selection of surface course mixes for main highway facilities. The policy specified the use of open friction course (OFC) mixes (as in Sections 13 and 14) as the surface layer for urban freeways and dense friction course (DFC) mixes (similar to Sections 3, 7, and 9) for other heavily trafficked main highways carrying an AADT of more than 5,000 vehicles per lane.

Field Evaluation

In addition to the initial evaluation performed during the first 3 yr, the sections were monitored at different stages of the service life up to the end of the eleventh year in 1985, when the whole test area was completely rehabilitated. Traffic control was a major undertaking for full testing of all the sections across the three lanes. Hence, evaluation of in-between years was selective, done only on certain sections (notably Sections 1, 3, 7, 9, and 13). However, friction testing by the ASTM E-274 brake force trailer was performed on the driving lane in the spring or fall, or both, of most of the years.

Laboratory Testing

Cores were periodically taken from the driving lane for laboratory testing. For the final evaluation, 15 cores were taken from each of the test sections across the three lanes, totaling 270 cores from the 18 sections. The different types of field and laboratory tests performed were either standard laboratory methods or common ASTM tests; they fall into the following three catagories:

1. Mixture analysis
   - Marshall properties of recompacted mix
   - gradation
   - resilient modulus (MR), ASTM D-4123 (*4*)
   - indirect tensile strength (S_t), ASTM D-4123 (*4*)
2. Pavement property
   - pavement compaction
   - pavement voids (%)
   - sand patch texture depth
   - air permeability, ASTM D-3637
   - photointerpretation, ASTM E-770
3. Recovered AC
   - penetration (25°C)
   - viscosity (135°C)
   - softening point (R&B)

RESULTS

The numerous results obtained from the monitoring of the 18 test sections over 11 yr are discussed under the following headings.

- Pavement performance
- Surface friction characteristics
- Material properties
- Summary of findings

Pavement Performance

In general, the pavement performed well early in the trial. However, since 1982, some local repair has been required on a few of the sections, especially Sections 9 and 14. The deterioration of these two sections was rapid, and, within a period of about 3 yr, complete rehabilitation of the sections had to be carried out. Because it was not practical or economical to resurface only the two sections, all 18 sections were rehabilitated in 1985. The serviceability of most of the sections was still considered acceptable at that time. A complete condition survey of the sections was made just before the rehabilitation work. Observations were made on the types of distress and their severity and frequency of occurrence. The distresses observed during the survey were transverse and longitudinal cracking, raveling of surface aggregates, delamination, joints at deteriorated areas, and rutting in the wheel tracks.

On the basis of the principles outlined in the *Manual for Condition Rating of Flexible Pavements* (*5*), the weighted distresses were totaled, to arrive at the overall distress rating (Table 3). A high rating indicates poor performance, a low rating good performance.

Transverse and Longitudinal Cracking

Cracks were observed on both sides of the transverse joints that were formed over the existing joints of the underlying concrete pavement. Some cracks were caused by saw cuts (not directly over the concrete joints) and intermediate cracks were reflected through the overlay from the existing concrete slabs. For the open mixes, there were small areas of material lost in the joints and cracks. High crack counts were recorded for Sections 5, 11, 12, 16, and 17, but the results do not suggest any difference in crack generation between dense and open types of mixes. This is because reflection cracking is a function of the support conditions of the underlying concrete slabs. Nonetheless, the five mixes that contained 2-percent asbestos appeared to have a higher transverse cracking rating, which suggests that the cracks might be related to the use of asbestos. The asbestos could soak up the binder and reduce the "free" binder available for coating the aggregate, thus reducing the film thickness and flexibility of the mixes.

A review of the crack maps on transverse and longitudinal joint repairs shows that Sections 3 and 6 appeared to have the highest total length of joint repaired (see Table 3). These two sections, which contain mixes using traprock coarse and fine aggregates, are similar to what is now called dense friction

TABLE 3 PAVEMENT CONDITION RATING OF TEST SECTIONS AFTER 11 YEARS OF SERVICE

SECTION	TYPE OF DISTRESS							JOINT REPAIRS(m)				
								CRACKS			SHOULDER	
NO.	TRANS. CRACKS	LONG. CRACKS	RAVEL--LING	PATCH	DELAMI--NATION	RUTTING	OVERALL RATING	TRANS.	LONG.	TOTAL	LEFT	RIGHT
1	3.5	44.0	6.0	.0	.0	3.0	56.5	41	8	49	0	0
2	3.5	3.0	12.0	.0	.0	18.0	36.5	53	2	55	58	0
3	.0	1.0	18.0	6.0	4.0	3.0	32.0	87	4	90	116	0
4	11.5	3.7	6.0	.0	.3	3.0	24.5	47	0	47	15	0
5	44.5	7.0	18.0	4.5	2.8	.0	76.8	49	26	75	122	0
6	3.0	.0	12.0	7.5	5.5	.0	28.0	51	60	111	128	0
7	16.0	2.0	6.0	.8	1.0	3.0	28.8	40	6	45	137	0
8	21.5	7.8	.0	.0	.2	4.5	34.0	--	--	--	--	--
9	5.0	7.0	18.0	5.5	14.0	4.5	54.0	26	15	42	0	0
10	10.5	6.5	6.0	.0	3.5	4.5	31.0	55	0	55	0	0
11	33.5	.0	.0	.0	2.0	.0	35.5	57	4	61	0	0
12	48.0	5.8	6.0	.0	1.2	.0	61.0	17	4	21	0	27
13	6.0	2.0	6.0	.0	4.5	.0	18.5	19	4	23	137	8
14	4.0	1.0	18.0	15.0	3.0	.0	41.0	0	8	8	137	0
15	19.5	2.0	6.0	.0	.2	.0	27.7	0	0	0	137	0
16	39.5	17.5	6.0	.0	4.0	.0	67.0	17	0	17	137	0
17	25.5	5.0	18.0	1.5	1.5	.0	51.5	4	0	4	73	0
18	3.8	4.0	.0	.0	1.0	6.0	14.8	0	4	4	0	0

Note: -- = Data not available

course (DFC) aggregate. It seems that the change of coarse aggregate content from 60 percent to 45 percent (namely, in Sections 6 and 3, respectively) could have slightly reduced the length of joint repairs, but the use of asbestos in Section 6 tends to confuse the issue. It seems also that the use of different types of fine aggregate (e.g., natural sand and limestone screenings for the mix in Section 5) could have the effect of reducing the length of joint repairs from 111 m (as for Section 6) to 75 m (Section 5). However, the results did not suggest that the open friction course (OFC) type of mixes (Sections 13 to 16) required any more joint repairs than the DFCs or the dense-graded mixes, although the OFCs contained a higher stone content. In effect, Figure 2a reveals that by increasing the coarse aggregate content to as high as 65 percent, the cracking potential is reduced significantly.

The discussion above shows that the key factor in reducing joint repairs is not reducing the stone content but ensuring that there is sufficient binder-film thickness around the aggregates, as in the case of the OFC mixes.

Raveling

The highest raveling rating of 18 is found in Sections 3, 5, 9, 14, and 17. Although there is a slight tendency of more raveling with high-stone-content mixes (Figure 2b), the fact that mixes 4 and 13 yielded ratings of 3 or less could be due to their thicker binder film. On the other hand, the raveling of the mixes mentioned above (i.e., Sections 3, 5, 9, 14, and 17) is a result of the poor cohesion caused by either low AC content (Section 3) or the reduction of effective binder content with the use of absorptive aggregates (e.g., the blast furnace slag in Section 9) and asbestos (Sections 5, 14, and 17). Raveling of most of the sections occurred along the longitudinal joints, except in Sections 3 and 9 where raveling was observed randomly across the three lanes.

Delamination

Quite extensive delamination was observed in Section 9, which contained the blast furnace slag aggregate. This mix looked dry and raveled in places to the extent that water was able to penetrate into the surface course and the pavement interface. The delamination in Section 14 could be caused by the use of asbestos, as discussed before. In both of these sections, the delamination occurred mainly in the driving lane. The weak bonding between the concrete pavement and the thin overlays has led to poor durability and overall performance. The construction staff recalled that tack coating on the concrete pavement was "spotting" and "scanty," which did not promote good interfacial bonding. In addition, a dense shoulder mix (HL-3) was used next to the open mixes (namely, in Sections 9 and 14), which created a drainage problem because water draining through the open mixes was held up by the dense shoulder mix, thus keeping the open mixes moist. The prolonged soaking in turn undermined the bonding between the overlay and the concrete surface by stripping. Signs of strip-

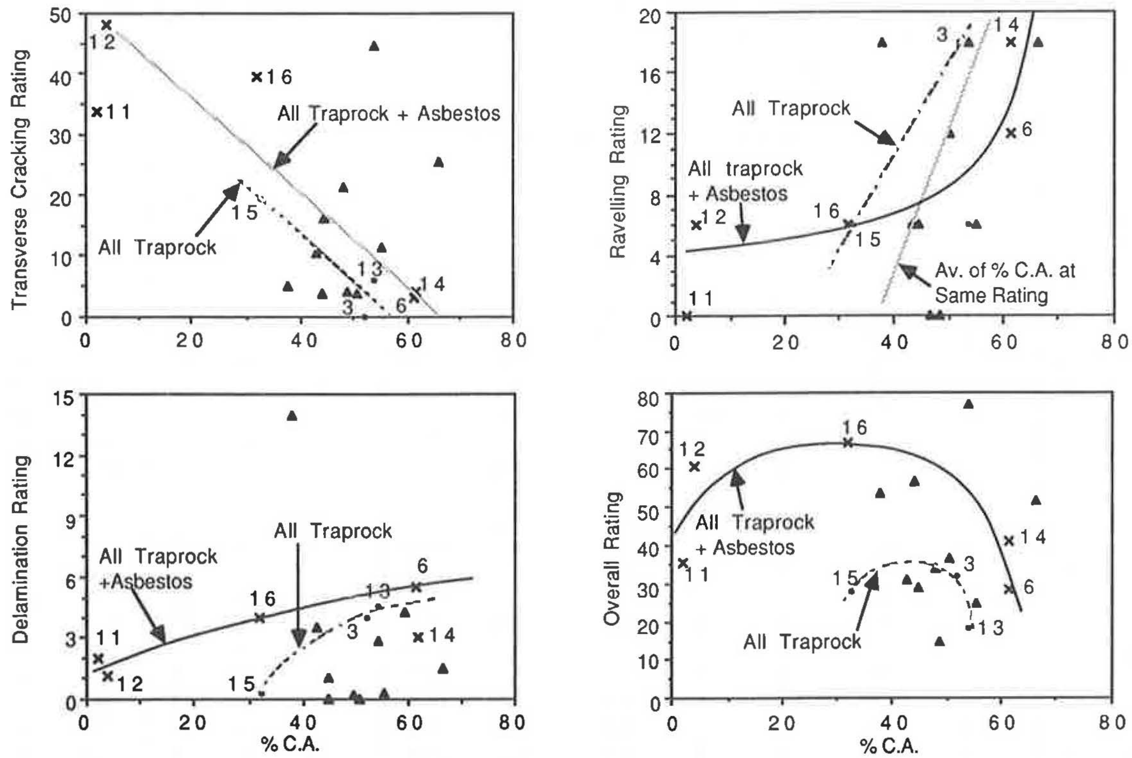

FIGURE 2 Influence of coarse aggregate content (% CA) on the different performance parameters (number = section no.; ▲ = other mixes).

ping were indeed observed on cores taken from the test sections. Also, open mixtures tend to retain more water and keep the pavement wet longer than dense mixes do. Hence more delamination was observed in the mixes with higher stone content, as shown in Figure 2c.

Another aspect that affected the delamination of the mixes along the joints in the driving lane was pumping. Pumping was observed on some of the joints where water was continuously pumped to the surface by moving traffic on the unstable concrete slabs in wet weather. This condition too promotes stripping and debonding.

Rutting

Rutting was observed on Section 2. An average rut depth of 15 mm was recorded on the heavily trafficked driving lane, and an insignificant amount of rutting was measured on all other sections. Replacing a small percentage of fine aggregate, 14 percent of the limestone screenings, with traprock screenings did not improve the stability of the mix in Section 2. In effect, the use of traprock screenings had reduced the voids in mineral aggregate (VMA) and voids in the mix, which caused the rutting. The small percentage of traprock was not sufficient to provide the firm particle interlock needed for rutting resistance.

The important points on rutting performance that emerged from these test sections are as follows:

- The marginal standard mix (e.g., HL-1, as in Section 2) is not stable enough for the volume of traffic on Highway 401.
- The sand mix, consisting of crushed aggregate, does not rut under the same traffic, because the crushed aggregates in the mix (Sections 11 and 12) exhibit high particle interlocking properties.
- Asphalt mixes with proper gradation and aggregate selection can be designed to withstand the heaviest traffic experienced on Ontario highways.

General

In general, for good durability with minimum patching and good resistance to raveling and delamination, overlays on concrete pavement should consist of a dense mix or at least a dense binder course if an open mix is used, as demonstrated by the performance of Section 18. The excellent overall rating of this mix can be attributed to the use of a dense binder course, which produced a good bond between the overlay and the concrete pavement. Also, the performance results show that existing concrete surfaces must be properly tack-coated before resurfacing. The use of asbestos did not seem to contribute to good performance. Regardless of mix designs, observations confirm that good binder-film thickness results in good durability.

However, the limited results relating the coarse aggregate content (percent CA) with the general performance of the asphalt mixes (Figure 2d) suggest that the percent CA should be either above 45 percent or below 35 percent.

TABLE 4 EQUILIBRIUM FRICTION NUMBER (EFN) AT 50 AND 100 KM/H OF THE 18 TEST SECTIONS

TEST SEC-TION NO.	DRIVING LANE		CENTRE LANE		PASSING LANE	
	FN50	FN100	FN50	FN100	FN50	FN100
1	32	23	34	23	36	26
2	33	23	36	25	38	29
3	36	29	38	29	43	34
4	33	26	35	27	40	30
5	34	26	35	27	39	30
6	36	29	37	29	41	33
7	45	34	45	34	50	39
8	38	26	38	27	39	26
9	44	33	44	36	47	39
10	36	26	39	29	42	32
11	40	29	42	30	45	33
12	40	29	42	30	45	31
13	35	30	37	30	42	34
14	35	30	36	30	40	33
15	37	26	38	28	44	32
16	37	28	39	29	46	33
17	34	25	35	25	37	26
18	31	22	33	23	37	27

Surface Friction Characteristics

The friction characteristics of the pavement were evaluated mainly by the ASTM brake force trailer at 50 and 100 km/hr. Other methods, such as photointerpretation (ASTM E-770), British Pendulum Tester (BPN), and sand-patch texture depth (TD), were also used. However, photointerpretation and the BPN results are inconclusive and are not discussed here. The following describes the different friction properties and performance of the mixes.

Development of Friction with Time

An equilibrium friction number (EFN, Table 4) is used to evaluate the different mixes; it is obtained by averaging the FN values beyond the third year, 1977. All of the mixes experienced a sharp decrease in FN within the first 2 to 4 yr; the FN then fluctuated from year to year depending on the season or the month in which the test was performed. Averaging FN values therefore appeared to be a reasonable way to assess long-term surface frictional performance. It seems that mixes with high initial friction values took longer to arrive at their equilibrium (e.g., Section 7 versus Section 1, Figure 3).

After 11 yr of service, a reasonably good level of friction, of EFN above 30, was obtained for mixes in Sections 7, 9, 13, and 14; a medium level of friction between 26 and 30 was reached for mixes in Sections 3, 6, 11, 12, and 16; and a fair level of 23 to 26 for mixes in Sections 1, 2, 4, 5, 8, 10, 15, 17, and 18. The steel-slag DFC (Section 7) consistently held the highest FN throughout the pavement life. It was followed by Section 13, an OFC using traprock, and Section 3, a DFC using traprock aggregate. The standard HL-1 mix reached its EFN within the first year, whereas the other mixes (i.e., Sections 3, 7, and 13) took about 3 yr. The HL-1 mix had the lowest FN among the test sections. Differences are accounted for by the following:

- The good sections (7 and 9) were DFC mixes consisting of all-slag aggregates.
- The reasonably high EFN 50 obtained for Sections 11 and 12 is due, first, to the use of all-crushed traprock aggregate and, second, to the close surface texture that provides the high contact area with the tire at low speeds (50 km/hr) where surface water drainage is not a problem. However, the rapid drop in FN at higher speeds (i.e., 100 km/hr) shows that macrotexture was inadequate.
- Medium EFN 100 mixes were those using all-traprock aggregate and having close macrotexture due to the embedment and flattening of stone particles on the surface.
- Mixes containing fine aggregates with natural sand and limestone screenings produced low EFN 100 results. Apparently the 12 percent limestone screenings in the mix of Section 8 reduced the EFN 100 to 27, as compared with a value of 34 obtained for Section 7, whose mix contained only steel-slag fine aggregate. The low EFN (at both 50 and 100 km/hr) obtained for Section 17 is a result of high AC content and the use of mineral filler and asbestos in the mastic mix.

The EFNs at both test speeds for the center lane and the passing lane were higher than the values obtained for the driving lane. This is because traffic volume is lower in the center

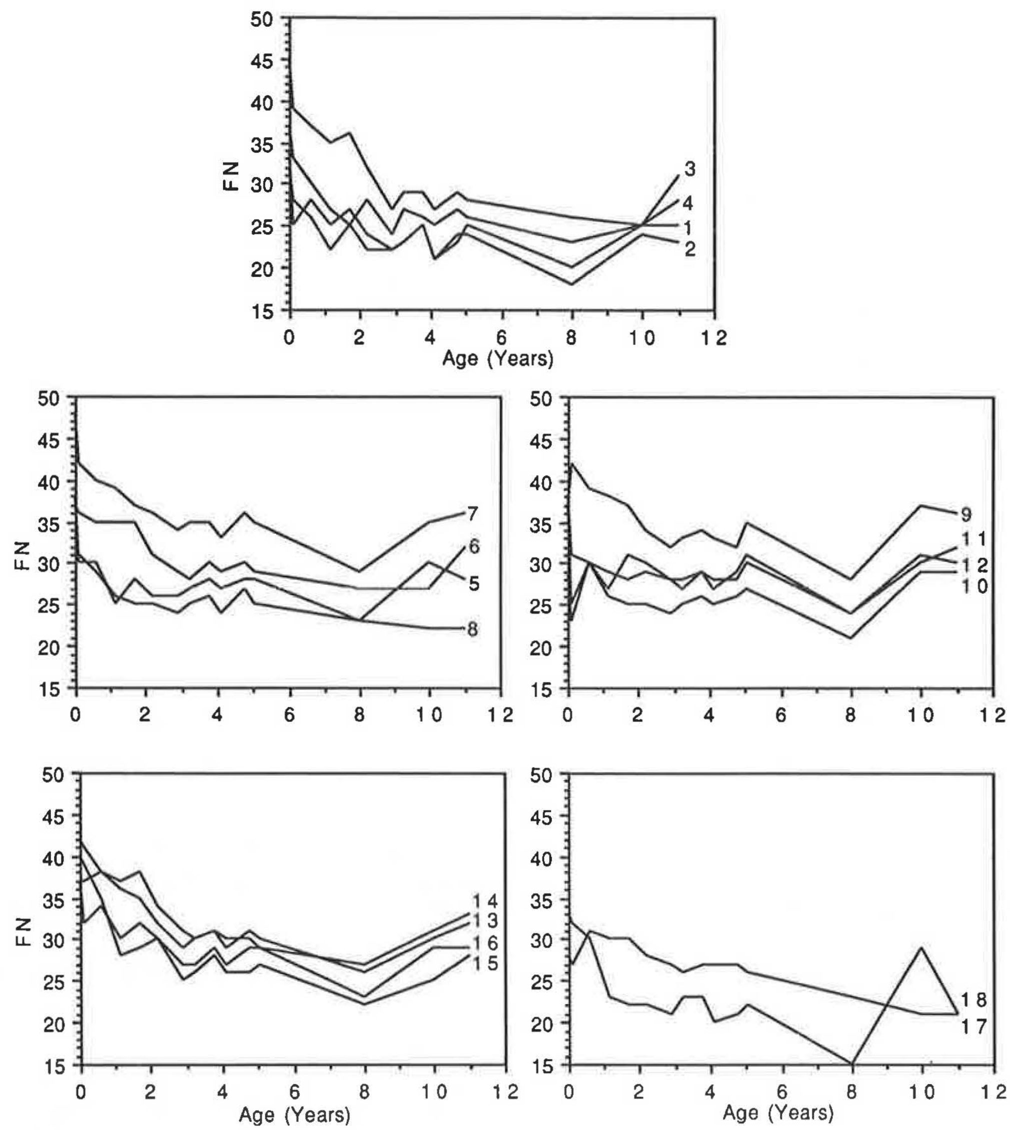

FIGURE 3 Friction number (FN) at 100 km/h at different age of the pavement in the driving lane.

and passing lanes, resulting in less polishing of the road surface and closing up of the macrotexture. Again, the relative EFN level of the mixes is similar to that of the driving lane.

Friction Number Versus Speed

The FN/speed gradient is commonly used to evaluate the textural properties of a surface with change in speed. The mixes were analyzed in groups having (1) increasing CA content, (2) a similar coarse-to-fine aggregate ratio, and (3) natural sand as a fine aggregate.

The FN/speed gradient seemed to be more affected by the CA content than by the type of fine aggregate used. A progressive increase in percent CA tended to increase the macrotexture and reduce the gradient of the FN/speed curve, as illustrated in Figure 4, for the mixes in Sections 11, 15, 3, 6, and 13. The other mixes, which clearly showed the differences in gradient to be a result of the change in texture, were Sections 12 and 14, which consisted of 9 and 67 percent of CA, respectively.

The benefit of the coarse macrotexture of Sections 13 and 14 (OFC mixes) in facilitating drainage of surface water was realized when their EFN values were maintained at 100 km/hr relative to the values at 50 km/hr.

Mixes with similar coarse-to-fine aggregate ratios (Sections 1, 3, and 7) tended to produce similar gradients (see Figure 4). However, the level of FN at any speed under consideration is also a function of the quality of the aggregate used.

The FN/speed relationship of the mixes in the center and passing lanes was relatively the same except that the gradients were more gradual and at a higher FN value. This is explained by reduced compaction and polishing of the surface textures by traffic.

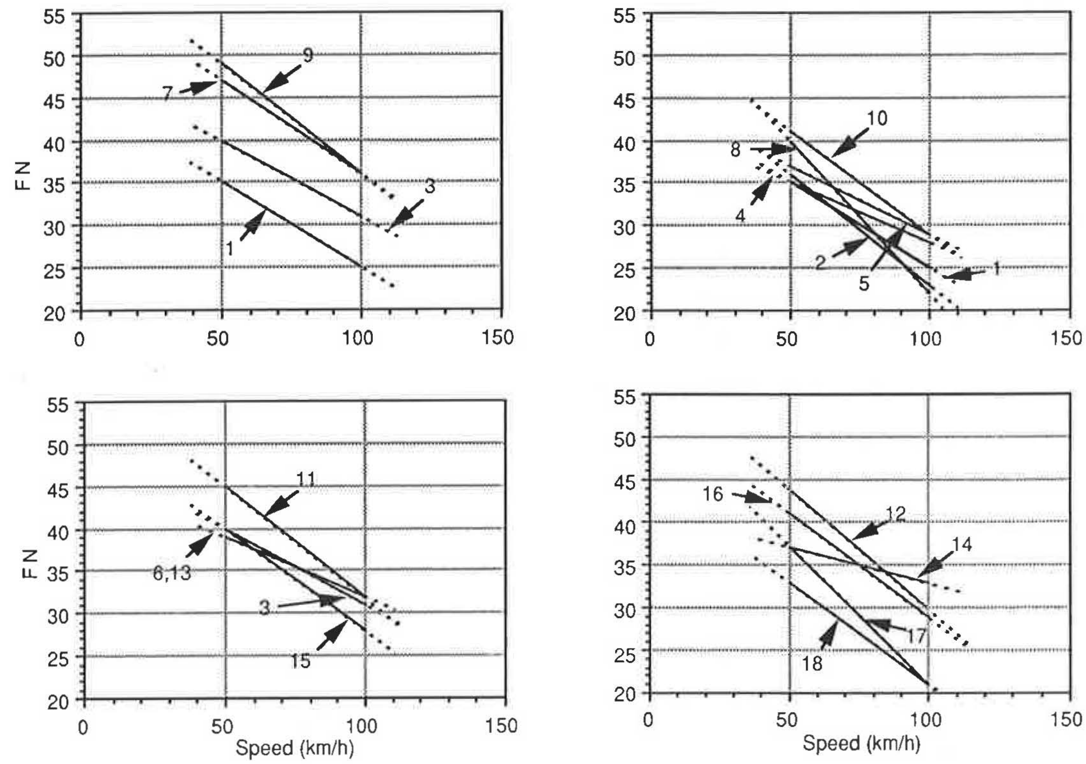

FIGURE 4 Influence of speed on the friction number (FN) for different mixes with various degrees of macrotexture (after 11 years of service).

Sand-Patch Texture Depth

Because of constraints such as time and traffic control, the sand-patch texture depth (TD) was measured in the laboratory on 6-in. diameter cores from the field. The results (Table 5) show that the average wheel track TD was lower for the driving lane (average = 0.46 mm) than for the center lane (0.55 mm) or the passing lane (0.62 mm), because there is more traffic compaction in the driving lane. The TD was lower between than on the wheel tracks because oil spills from the engines soften and compact the texture of the asphalt surfaces.

The wheel track TDs for Sections 11, 3, and 6 (0.28, 0.67, and 0.74 mm, respectively) at the driving lane show that an increase in CA content generally results in an increase in the TD of a mix.

Mix Composition and Friction

Mix composition is one of the most significant factors affecting the frictional characteristics of a mixture. In particular, the CA content directly affects the availability and durability of the macrotexture. This is shown in the relationship of percent CA and the difference in FN at 50 and 100 km/hr (Figure 5).

Figure 5 reveals that a clear trend existed; with an increase in percent CA, the drop in FN with an increase in speed is reduced. This relationship was maintained across the three lanes and appeared to be nonlinear. The trend shows that for all-traprock mixes (Sections 11, 15, 6, and 13, with increasing CA content of 14, 30, 60, and 67 percent, respectively), the optimum CA content for maximum friction characteristics is around 63 percent, which approximates the CA content in the present specification for OFC. In other words, this finding suggests that mix designs for high-speed traffic roads should have a coarse aggregate content of about 63 percent. Also, an improvement of about 3 FN units over the FN at 50 km/hr can be achieved by increasing the CA content from 0–45 percent. But more significantly, the same improvement was obtained by increasing the percent CA from 45 to 55 percent.

A similar trend in the change in FN 100 and FN 50 with a decrease in the CA content was observed for the center and passing lanes.

Material Properties

Three aspects of material properties were examined: pavement characteristics, asphalt cement properties, and strength characteristics. Because samples were taken and tested only at the eleventh year of service (in 1985) and the AC properties and material strength data on the original materials at construction were not available, the analysis on the change of material properties was necessarily limited.

Gradation

The mix gradation at 11 yr (Figure 1) was obtained from extraction of pavement cores. Original gradation data are not

TABLE 5 TEXTURE DEPTH BY THE SAND-PATCH METHOD TESTED ON 6-IN. DIAMETER CORES AT 11 YEARS

TEST SEC-TION NO.	DRIVING LANE			CENTRE LANE			PASSING LANE		
	LWT	ML	RWT	LWT	ML	RWT	LWT	ML	RWT
1	0.54	0.28	0.43	0.33	0.32	0.21	0.62	0.31	0.38
2	0.29	0.27	0.30	0.56	0.38	0.27	0.57	0.38	0.47
3	0.59	0.40	0.74	0.90	0.44	0.67	0.80	0.48	0.61
4	0.30	0.45	0.52	0.58	0.44	0.53	0.63	0.43	0.42
5	0.53	0.39	0.55	0.76	0.34	0.50	0.89	1.05	0.52
6	0.65	0.40	0.83	1.07	0.51	0.68	1.39	1.17	0.67
7	0.37	0.32	0.30	0.54	0.31	0.53	0.62	0.67	0.57
8	0.33	0.32	0.21	0.42	0.28	0.30	--	--	--
9	0.45	0.31	0.46	0.71	0.40	1.10	0.89	0.44	0.60
10	0.36	0.22	0.28	0.33	0.25	0.48	0.54	0.25	0.46
11	0.25	0.22	0.31	0.32	0.32	0.34	0.60	0.26	0.27
12	0.26	0.22	0.25	0.34	0.30	0.33	0.37	0.16	0.22
13	0.60	--	0.90	0.69	0.61	0.84	1.04	0.64	0.85
14	0.77	0.64	0.93	1.01	0.55	1.03	1.57	0.79	0.96
15	0.20	0.12	0.39	0.40	0.25	0.25	0.47	0.25	0.25
16	0.33	0.29	0.41	0.47	0.31	0.36	0.47	0.36	0.33
17	0.62	0.38	0.66	0.64	0.53	0.56	0.76	0.64	0.54
18	0.30	0.26	0.25	0.28	0.31	0.29	0.53	0.33	0.34

Notes:
1. LWT = Left Wheel-Track ML = Center of the lane
RWT = Right Wheel-Track -- = Result not available
2. Results related to wheel-tracks are averages of 2 cores.

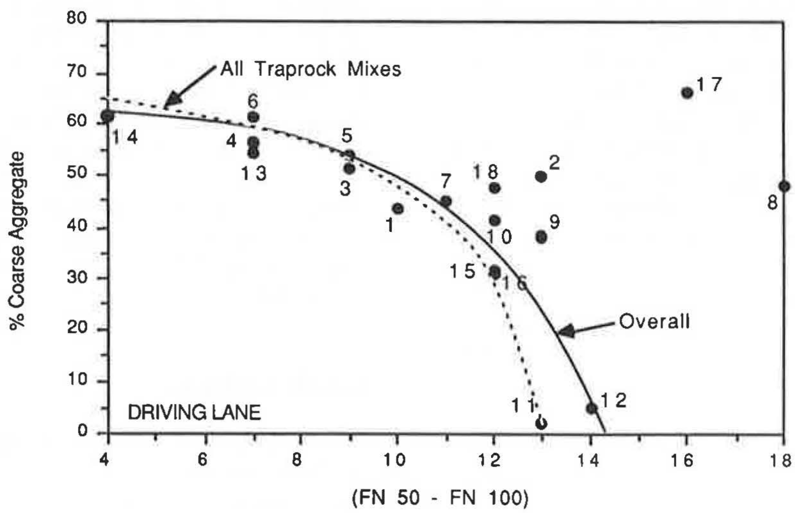

FIGURE 5 Relationship of coarse aggregate content (% CA) and difference in friction number at 50 and 100 km/h.

available for comparison, but the coarse-aggregate content is slightly lower than it was at construction, which may be a result of the loss of stones in the form of raveling in some of the open mixes. However, the percentage passing the 75-μm (No. 200) sieve increased (by 0.5 to 4.4 percent) because of dirt accumulation in the voids of the open mixes (Table 6).

Pavement Voids

In all cases, the pavement void content in the driving lane was lower than in the center and passing lanes because of more traffic compaction (Table 7). The dense friction course mix (Section 3) had voids close to or slightly higher than the

TABLE 6 COMPOSITION OF AGGREGATE MIXES AT DIFFERENT YEARS OF SERVICE

TEST SEC-TION NO.	COARSE AGGREGATE CONTENT (RETAINED ON 4.75 MM SIEVE, % BY MASS)					PASSING .075 MM SIEVE (% BY MASS)				
	YEAR 0	YEAR 7	CHANGE (7-0)	YEAR 11	CHANGE (11-0)	YEAR 0	YEAR 7	CHANGE (7-0)	YEAR 11	CHANGE (11-0)
1	43.8	43.1	-0.7	47.1	3.3	2.4	3.5	1.1	3.4	1.0
2	48.2	48.3	0.1	49.1	0.9	3.3	4.4	1.1	4.0	0.7
3	47.5	43.9	-3.6	46.9	-0.6	7.8	10.3	2.5	8.6	0.8
4	54.1	54.6	0.5	56.5	2.4	2.6	3.5	0.9	3.5	0.9
5	58.1	54.3	-3.8	55.2	-1.9	1.5	1.8	0.3	2.6	1.1
6	62.3	56.6	-5.7	50.1	-2.2	3.4	4.0	0.6	5.7	2.3
7	46.8	43.8	-3.0	41.9	-4.9	3.7	6.4	2.7	8.1	4.4
8	47.1	47.0	-0.1	46.8	-0.3	2.8	3.7	0.9	3.8	1.0
9	43.2	34.3	-8.9	37.4	-5.8	3.2	6.5	2.3	6.7	3.5
10	40.5	38.7	-1.8	42.5	2.0	2.3	3.8	1.5	3.4	1.1
11	5.4	4.1	-1.3	3.9	-1.5	6.3	8.2	1.9	8.5	2.2
12	6.9	4.5	-2.4	6.2	-0.7	8.5	9.7	1.1	9.7	1.1
13	60.5	51.0	-9.5	55.2	-4.3	2.4	5.8	3.4	5.1	2.7
14	71.7	61.2	-10.5	62.5	-9.2	1.9	2.4	0.5	3.9	2.0
15	29.3	29.4	0.1	32.2	2.9	7.1	8.0	0.9	7.6	0.5
16	31.4	32.5	1.2	30.7	-0.7	3.8	5.6	1.8	6.2	2.4
17	75.2	64.0	-11.2	57.7	-7.5	4.5	4.4	-0.2	7.5	2.9
18	47.4	50.0	2.5	48.9	1.5	2.7	4.3	1.6	3.6	0.9

TABLE 7 ASPHALT MIX DENSITY AND VOID CONTENT DATA FOR THE 18 TEST SECTIONS

TEST SEC-TION NO.	CONSTRUCTION CONTROL BRIQUETTES			PAVEMENT CORES AT 11 YEARS: DRIVING LANE			CENTRE LANE			PASSING LANE		
	MRD	BRD	VIP	MRD	BRD	VIP	MRD	BRD	VIP	MRD	BRD	VIP
1	2.65	2.55	3.8	2.66	2.60	2.2	2.71	2.65	2.4	2.70	2.57	4.7
2	2.71	2.69	0.7	2.75	2.66	3.3	2.74	2.68	2.3	2.75	2.69	2.7
3	2.93	2.85	2.7	2.95	2.73	7.7	2.97	2.65	10.9	3.00	2.67	11.3
4	2.73	2.69	1.5	2.79	2.70	3.2	2.77	2.59	3.1	2.76	2.61	5.5
5	2.69	2.67	0.7	2.70	2.59	4.3	2.71	2.58	4.5	2.71	2.54	6.3
6	2.84	2.76	2.8	2.84	2.57	9.5	2.89	2.56	11.3	2.89	2.49	14.0
7	3.08	2.96	3.9	3.10	2.76	11.1	3.11	2.69	13.4	3.11	2.62	15.8
8	2.75	2.66	3.3	2.72	2.62	3.5	2.79	2.67	4.1	--	--	--
9	2.41	2.19	9.1	2.45	2.18	10.9	2.46	2.10	14.7	2.44	2.09	14.3
10	2.43	2.28	5.2	2.46	2.27	7.7	2.44	2.25	7.7	2.46	2.21	10.3
11	2.77	2.76	0.4	2.80	2.62	6.3	2.79	2.57	7.9	2.77	2.43	13.2
12	2.77	2.74	1.1	2.77	2.73	1.3	2.80	2.58	7.8	2.78	2.58	7.4
13	2.81	2.69	4.3	2.86	2.60	8.9	2.84	2.56	9.9	2.83	2.51	8.8
14	2.80	2.69	3.9	2.82	2.49	11.8	2.81	2.47	12.0	2.84	2.45	13.9
15	2.82	2.81	0.4	2.88	2.82	2.2	2.86	2.79	2.6	2.87	2.60	9.5
16	2.78	2.76	0.7	--	2.59	--	--	2.65	--	--	2.66	--
17	2.70	2.68	0.7	2.75	2.56	6.8	2.72	2.61	4.0	2.70	2.60	3.7
18	2.57	2.57	0.0	2.75	2.59	2.6	2.71	2.64	2.7	2.72	2.65	2.8

Notes:

MRD = Maximum Relative Density of a mix

BRD = Bulk Relative Density of cores

VIP = Voids in Pavement (%)

-- = Result not available

open friction course in Sections 13 and 14. The OFC void content was therefore unusually low in this case (about 9 percent in the driving lane and 11 percent in the passing lane), as compared with the void contents obtained from mixes in current use (about 18 percent). As the core samples from the driving lane wheel tracks showed, the surface texture of the DFC mixes (Sections 3 and 6) was closed up by traffic, preventing the debris from getting into the open matrix and keeping the pavement voids high. This explains why high percent voids were recorded for some of the cores from the dense mixes. The all-slag mixes (Sections 7 and 9) had the highest void content among the test mixes (about 13 percent).

The influence of void content on the durability of asphalt mixtures was examined in terms of the permeability and aging of AC properties. These two properties are closely related since higher voids allow more accessibility to air and water and hence engender more rapid aging of the AC. These two factors are discussed below.

Permeability

Permeability tests were performed initially on the driving-lane wheel tracks of each of the test sections using the Johns Manville outflow permeameter [as described by Ryell et al. (*3*)]. However, at the eleventh yr, the wheel track surface texture of all the sections was closed up to the extent that it required much longer (more than 1 hr) to obtain a water permeability reading. Cores were therefore taken from the test sections and tested in the laboratory with the air permeability method.

The open-graded mixes (e.g., OFC as in Sections 13 and 14) had a very high initial permeability (> 140 ml/min), but the voids closed up after 7 yr of traffic compaction, reducing the permeability to about 10 ml/min. For the dense mixes (e.g., DFC as in Sections 3 and 9), the values were reduced to 0 from a value of 70 ml/min within the same period. The effectiveness of the OFC in draining water from the surface was reduced to the level of a new DFC after 2 yr of traffic (*1*).

However, the air permeability of the OFC at the eleventh yr on the driving lane is still relatively higher than that for the other mixes (10^{-8} cm versus $< 10^{-9}$ cm, respectively). The lower air permeability in the driving lane confirms the belief that traffic compaction in the driving-lane wheel tracks is greater than in the passing lane tracks because of the heavier traffic volume (dashed line, Figure 6). In general, an increase in pavement voids will result in higher permeability in the pavements (solid line).

The influence of percentage of coarse aggregate on permeability and percent voids in pavement was also examined. It seems that the increase in percent CA (as in Sections 11, 6, and 14) increases the permeability and voids in a pavement, as shown by the dashed line in Figure 6.

Asphalt Cement Aging

This experiment provided a good opportunity to examine the aging properties of the asphalt cement under the same service and environmental conditions for the variety of mixes placed. One important piece of information derived from this experiment concerned the influence of air voids in the pavement on the rate of AC hardening after 11 yr of aging in the same service environment. Assuming that the original level of an 85/100 penetration grade AC was 92 (except for Section 17, which was assumed to be at 65), the retained penetration—obtained by dividing the recovered penetration at the eleventh yr by the original AC penetration level (see Figure 7)—confirms the general belief that increase in air voids in the pavements will lead to more rapid oxidation and hardening of the AC in a mix and that the relationship is nonlinear. It seems that a pavement with a higher void content of, say, 10 percent will have 10 percent less retained penetration than a mix with 5 percent at the eleventh yr.

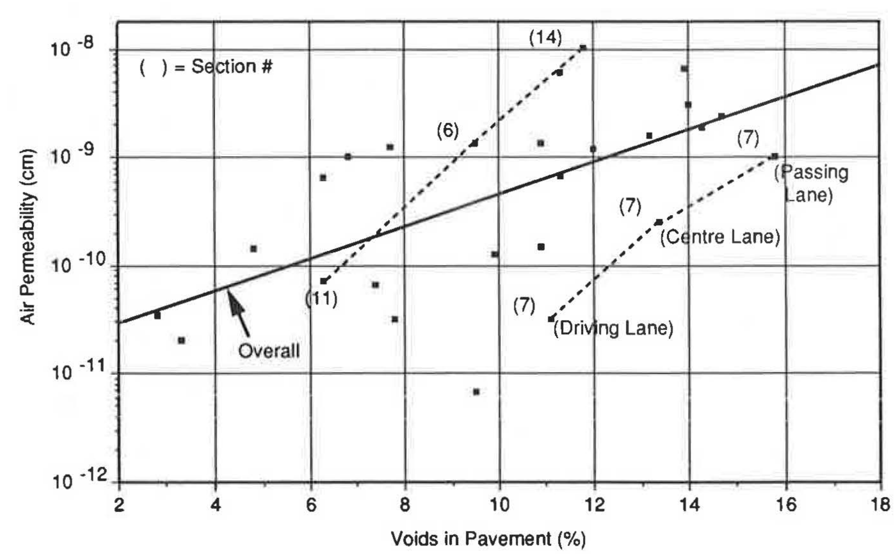

FIGURE 6 Influence of air void content on the air permeability of asphalt pavement materials.

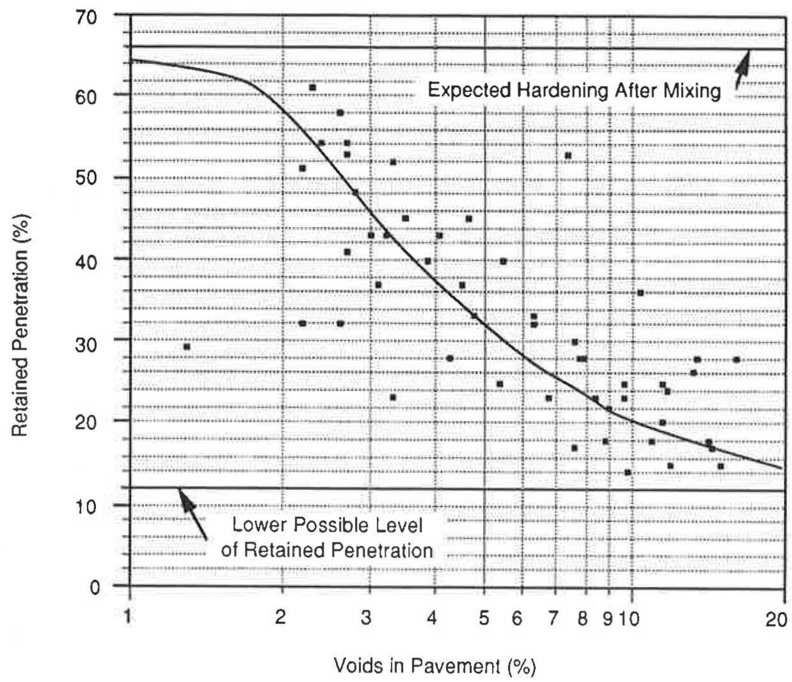

FIGURE 7 Influence of air void content (%) on the retained penetration (%, 11 yr/original value*100) of recovered asphalt cement from the test sections after 11 years of service.

Recompacted Marshall Properties

Cores taken from the pavement were reheated and recompacted into Marshall briquettes for testing. The results showed that the stability of samples from the driving lane was generally lower than for the center or the passing lane (see Figure 8a). This is due to the increased hardening of AC in the center and passing lanes where higher voids prevail.

Tensile Strength and Modulus Characteristics

The resilient modulus (M_R) results were obtained by testing the core samples at 23°C (see Figure 8b). The mixes with the highest M_R were Sections 5, 3, 12, and 9, and the lowest ones were Sections 17, 10, and 6.

The relationship of M_R to coarse aggregate content (percent CA) was examined. Mixes using the same types of aggregate but at various proportions showed some optimum percent CA at which the highest M_R occurred. Different types of aggregate combinations had different peaks, ranging between 45 and 55 percent CA content. The mixes with all-traprock aggregate seemed to have the highest maximum M_R at about 50-percent CA.

Similarly, indirect tensile strength (S_t) analysis was carried out (Figure 8c). Again, there seemed to be a maximum S_t for each of the groups of mixes using the same aggregate type and combination of various CA contents. The peaks ranged from 35–45 percent CA content.

The M_R and S_t relationships to the percent-CA confirm that the conventional type of dense mixes with about 45 percent of retained 4.75-mm sieve provide better strength properties for structural purposes than the open mixes.

Summary of Findings

The following summarizes the findings of this research:

Frictional Performance

- The analysis of the long-term frictional performance of the mixes confirms the findings reported previously by Ryell et al. (*3*) that (1) for high speed road overlays, good friction develops from surfaces with sufficient macrotexture and harsh microtexture; and (2) the dense mixes using slag or traprock fine aggregates provide better friction properties than limestone screenings or natural sand.
- The friction number for all mixes decreases within the first 2–3 yr before arriving at the equilibrium value. Mixes with high initial FNs tend to take longer to reach the equilibrium.
- Although the mixes in Sections 11 and 12 produced a reasonably high level of friction at a speed of 50 km/hr, the rapid drop in FN at a higher speed (i.e., 100 km/hr) shows that there was inadequate macrotexture. On the other hand, the benefit of the more open macrotexture mixes (Sections 13 and 14) was realized with the reduction in the drop of the FN and speed curve gradient.
- A good sand-patch texture depth of 0.6 mm was obtained after 11 yr of traffic in the driving lane from an OFC mix containing 67-percent coarse aggregate.
- For the best frictional properties, such as steady FN to speed gradient and good texture depth and density, the results confirm that coarse aggregate content should be set above 60 percent.

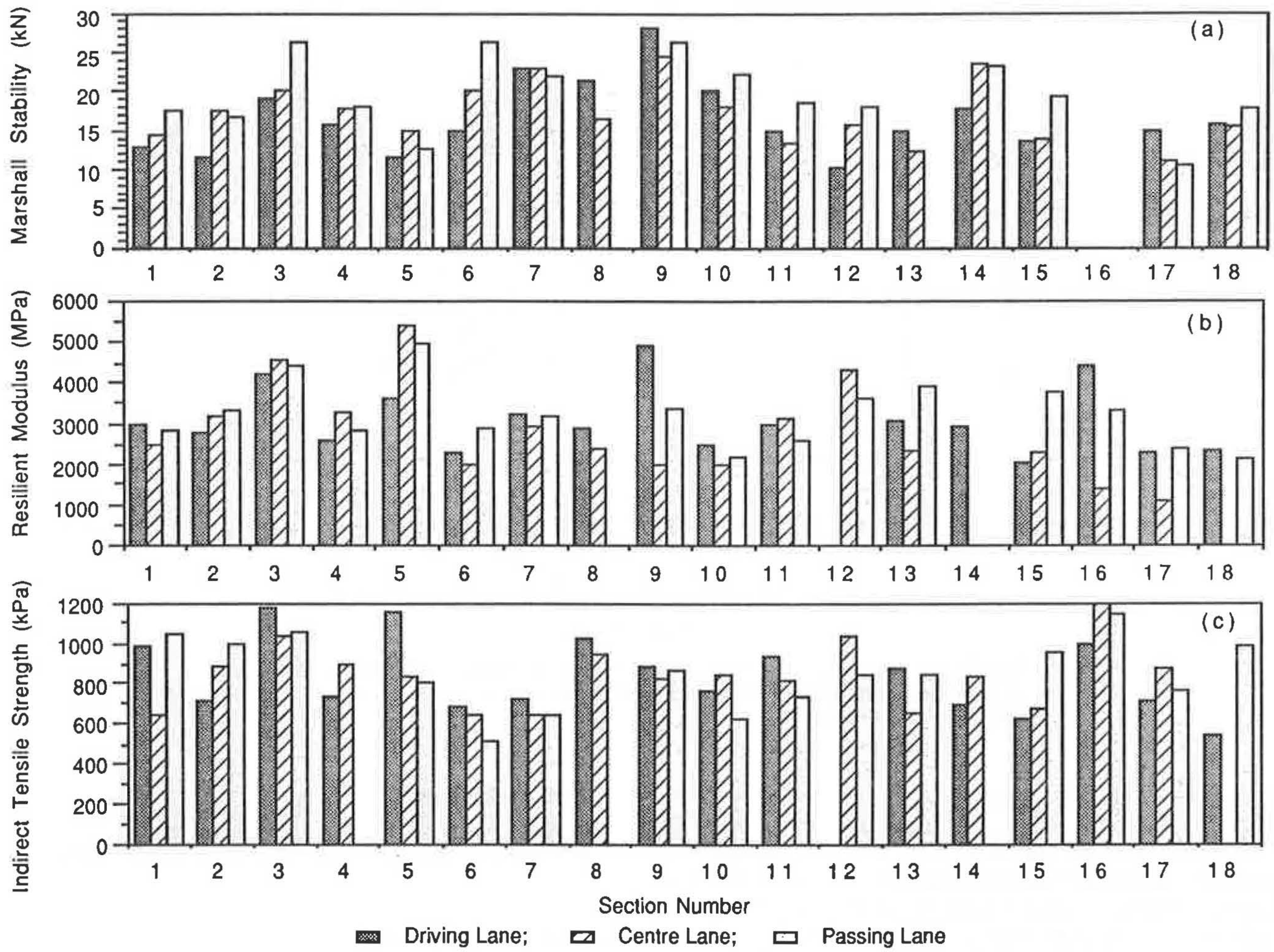

FIGURE 8 Marshall stability, resilient modulus, and indirect tensile strength of different test sections.

Durability Performance

- All of the 18 test sections performed better than expected for single-course thin overlays on concrete pavement.
- There is no apparent difference in crack generation and joint repair requirements between dense and open mixes, but high-stone-content open mixes do tend to have more raveling along the cracks.
- The use of asbestos may contribute to more cracking because it reduces binder-film thickness and flexibility. Increased binder-film thickness and coarse-aggregate content reduced the cracking potential of the asphalt mixtures tested.
- Poor bonding between the concrete pavement and the surface course mixes, as a result of inadequate tack coating and/or binder film, led to extensive delamination on the driving lanes of Sections 9 and 14. High-stone-content mixes tend to have increased delamination.
- The blast furnace slag mix experienced more raveling than nonabsorptive aggregate mixes.
- Standard mix with marginal stability rutted.
- Coarse-aggregate content had a definite effect on the overall durability performance of the asphalt mixes, particularly for such parameters as raveling, delamination, and transverse cracking. Higher CA-content mixes tend to have lower cracking but more raveling and delamination.

Material Properties

- After 11 yr of heavy traffic, the all-slag mixes had the highest void content (average 10 percent) among the test mixes.
- The OFC mixes had very high initial permeability, which was reduced to a level similar to that for dense mixes after 7 yr of traffic compaction. The permeability of the DFC mix was reduced to a level of a dense mix after only 2 yr of heavy traffic. As expected, an increase in the percent CA generally increased the permeability and voids of asphalt mixes.
- The results of this study confirm that an increase in air voids in the pavements led to more rapid oxidation and hardening of asphalt cement in asphalt mixes.
- For good structural properties as shown by M_R and S_t, the percent CA of dense asphalt mixes needs to be around 45 percent.

CONCLUSIONS AND RECOMMENDATIONS

All of the test sections performed better than expected for single-course thin overlays on concrete pavement. The open and dense mixes performed equally well under heavy traffic. In overlaying concrete pavements, in particular, good bonding created by either proper tack coating or the use of a dense binder course was found essential for promoting good durability.

To address the objective of this experiment, i.e., the search for bituminous mixes with good frictional performance, the most suitable mixtures for overlaying high-speed, heavily traveled highways appear to be those which contain

- steel slag or traprock aggregates for both the coarse and fine aggregates,
- coarse aggregate contents over 60 percent, and
- sufficient binder-film thickness and voids.

From the mix-design point of view and within the scope of this trial, the findings that are significant are the following:

- Coarse aggregate contents above 60 percent provided the optimum friction properties.
- Coarse aggregate content of 45 percent produced the highest resilient modulus and indirect tensile strength.
- Proper gradation design and aggregate selection seem to be the key to solving the rutting problem.
- A 10-percent decrease in retained penetration (i.e., more hardening) can be expected with an increase in void content of a mix in a pavement from 5 percent to 10 percent, as measured at the eleventh yr of service.

The findings of this experiment confirm that the existing policies of using OFC and DFC for treatment of highways requiring a friction surface course are sound and adequate. It is therefore recommended that

- The practice of using OFC and DFC mixes for heavily traveled pavements be continued;
- For OFC mixes, a coarse-aggregate minimum content of 65 percent be maintained; and
- For DFC mixes, a coarse-aggregate minimum content of 55 percent be maintained.

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