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Asphalt Construction, Premature Rutting, and Surface Friction Courses

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Contents

Foreword

This Record contains information on compaction of asphalt pavements, segregation of mixtures, performance of seal coats, and case studies of pavement rutting. It should be of interest to state and local engineers of materials, construction, and maintenance as well as contractors and material producers.

Brown et al. observed the extent of segregation on a number of Georgia construction projects using larger maximum-size aggregates, developed a test program to quantify the problem, and concluded that most segregation can be prevented by using good practices in construction and quality control. Elton proposes an expert system solution, SEG, which interactively interviews the nonexpert user and suggests changes in the construction operation to eliminate segregation. Kennedy et al. report on the effectiveness of a thin-lift nuclear gauge for estimating core densities on new asphalt pavements. Linden et al. studied the influence of air voids on the performance of dense asphalt concrete pavement surfaces using a literature search, a survey of 48 state highway agencies, and performance data from the Washington State Pavement Management System. Anani et al. executed a laboratory program to study the effects of varying the filler content and compactive effort on the properties of asphalt mixes. A rational method of mix design is suggested. Selim discusses the use of antistripping agents to enhance the bond between aggregate and emulsion, the most feasible way of incorporating the agents, and a mathematical model to predict friction values of a seal coat at any time after construction. Abdul-Malak et al. investigated the effects of aggregate characteristics, construction variables, traffic volume, and environment on the field frictional resistance of seal-coat surfaces. Tam et al. summarize the experience of a field trial placed on Ontario Highway 401 that included a comprehensive range of asphalt surface-course mixes, monitored for 11 years. Flowers et al. describe rutting problems on two sections of asphalt concrete overlay on the Interstate highways in Arkansas, and corrective measures taken to avoid future problems. Hensley and Leahy document three case studies of premature rutting and its causes.

Investigation of Segregation of Asphalt Mixtures in the State of Georgia

E. R. BROWN, RONALD COLLINS, AND J. R. BROWNFIELD

Using large maximum-size aggregates produced segregation of aggregate in asphalt mixtures in the state of Georgia. This report summarizes a study of the problem by Auburn University for the Georgia Department of Transportation. Researchers observed the extent of segregation on a number of construction projects and developed a test plan to quantify the problem. The results of this study suggest that most segregation can be prevented by following good construction practices and paying close attention to quality control. An associated laboratory study shows that the properties of an asphalt mixture can be significantly changed when segregation occurs.

The Georgia Department of Transportation (GDOT) typically uses relatively large maximum-size aggregates to ensure that base and binder course mixtures are resistant to rutting. Although such mixtures do minimize rutting, they tend to segregate during the production, hauling, and/or laydown operation. Previous GDOT-funded projects have examined procedures for minimizing aggregate segregation and recommended specific steps to alleviate the problem. Because these steps have not completely solved the problem, however, this project was undertaken to evaluate the aggregate segregation problem in Georgia and to recommend further steps to minimize the problem.

The proposed work began with a review of literature on aggregate segregation. Next, several ongoing construction projects were observed to identify any aggregate segregation problems and to evaluate existing construction procedures in the state of Georgia. In addition, a sampling and testing plan was developed to evaluate pavements with segregation problems and to compare samples of the segregated mixture with random samples. The GDOT Materials Laboratory and various division laboratories tested all the samples to determine gradation, asphalt content, and voids in total mix; the resulting data were analyzed to compare the gradation and asphalt content of mix from segregated areas to that of material from other areas. Finally, a series of laboratory tests conducted at Auburn University measured properties of asphalt mixture representing the segregated and nonsegregated areas.

PREVIOUS RESEARCH

Aggregate segregation in asphalt pavements occurs when coarse aggregate congregates at one spot in the pavement. The coarse spots exhibit open textures and low densities, which often result in areas of high permeability susceptible to raveling, cracking, and moisture damage (*1*). Previous research on aggregate segregation can be divided into three parts: sources of segregation, diagnosis of segregation, and prevention of segregation.

Sources of Segregation

The National Asphalt Paving Association (NAPA) suggested in 1987 (*2,3*) that stockpiling "single-sized" aggregate minimizes segregation in the stockpiles. Also, stockpiling in horizontal layers reduces segregation because the aggregate cannot roll down long slopes. In addition, NAPA recommended improved cold bin openings that allow unrestricted flow. Conventional bin openings may become partially plugged by bridging aggregate, but a trapezoidal bin opening, with the calibration belt flowing away from the wider end of the opening, allows more uniform flow out of the bin.

Kennedy et al. (*1*) state that a segregated stockpile creates special problems in a drum mix plant because there is no internal gradation check. They recommend using at least three stockpiles—more if there is a large variation in aggregate size—and up to five or six stockpiles to effectively minimize segregation. They state that loaders should not scoop from the side of a stockpile, but should instead ram the side of the stockpile and rotate the bucket after coming to a stop. The material should then be dumped directly into the center of the cold bins; if the aggregates in the cold bins intermingle, bulkheads should be used (*1*).

Conveyor Belts and Drums

The gradation of the aggregate is not usually altered on the conveyor belt that leads from the cold feeds to the drum. Segregation may occur in the drum mixer, however. NAPA (*3*) states that good asphalt coating of large particles will reduce this segregation, and recommends that the mixing dwell time be increased or the asphalt cement (AC) be introduced earlier in the drum. To achieve increased dwell time, NAPA suggests that donuts be welded to the inside of the drum or that the slope of the drum be decreased; either method would let the aggregate be coated with asphalt longer. NAPA also found that a mix can become segregated when it is deposited on the belt from drums that allow fines to fall on one side of the belt and coarse particles on the other.

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Kennedy et al. (*1*) consider conveyor belt speed to be a possible reason for segregation: if the belt is run too fast, large particles will be thrown to the far side of the silo, eventually creating a coarse strip on one side of the mat.

Batchers, Rotating Chutes, and Silos

In their 1967 study on the effects of hot storage on an asphalt concrete mix (*4*), Middleton et al. concluded that storage silos had little or no effect on aggregate gradation. Their study concerned asphalt concrete mixtures with fine aggregate gradations (½-in. maximum size), which are not very susceptible to segregation.

In 1970, Foster (*5*), using a gradation with a maximum size of 1½ in., noted that considerable segregation can take place in the silo. He showed the segregation pattern to be large aggregates around the edge of the silo with finer aggregates in the center. He also established that the gradation of unloaded material differed from the gradation of the material that was being loaded into the silo.

Foster suggests keeping the material one silo diameter above the top of the cone to force the material to recombine as it is loaded into the trucks. He mentions the use of gob hoppers (batchers) at the top of the silo, but there are no data to show how much the hoppers could reduce segregation.

Dan Houston (*6*) in the same year reported that certain bin geometry combinations yield less segregation. The combination that made for the least segregation was a circular silo with a 1-ft by 4-ft opening at the bottom of the silo and a rotating spout at the top of the silo. The second least segregated mix came from a circular bin with a 1-ft by 4-ft opening at the bottom of the silo and a batcher at the top of the silo.

In 1974, Zdeb and Brown (*7*) reported that gradation variability increased with storage, as indicated by a more than twofold increase in standard deviation of percent passing most sieves.

In 1987, NAPA (*3*) considered storage silos the most sensitive place for segregation to occur. The report stated that batchers and rotating chutes are effective only as long as they are operated properly. Rotating chutes must rotate, and batchers must be filled sufficiently before dumping and never emptied until the end of the daily operation. NAPA says that emptying the silo below the cone, or operating the silo at maximum capacity, will also result in segregation. The report suggests that trucks be loaded in three separate drops instead of one large one. The first drop should be behind the cab, the second in front of the tailgate, and the third between the first two.

Kennedy et al. (*1*) also considered improper use of storage silos to be the most important cause of segregation. They stated, as did NAPA, that operating a silo at 25 to 75 percent capacity would produce the most consistent mix.

Pavers

Until recently, pavers have not been considered serious areas of segregation. However, NAPA now recognizes that poor paver operation can cause segregation (*3*). NAPA recommendations for paver operators are as follows:

- Do not empty hopper.
- Do not dump wings unless absolutely necessary.
- Flood the hopper.
- Adjust gates so that augers run continuously.
- Adjust paver speed to match the rate of production of the hot mix asphalt (HMA) plant.

Kennedy et al. (*1*) suggested modifications to the paver itself. They recommend welding a beveled bottom on the wings to promote a more continuous flow of HMA from the wings to the drag slats and placing fillets in the corners of the wings to hinder the collection of coarse material on the outside of the wings.

Diagnosis of Segregation

Nady, in a paper published in 1984 (*8*), reported that eight cores within a 20-ft section of roadway could not be removed intact due to a lack of fines in the mixture. He noted that there was no visible segregation in this area.

Two years later, Lackey, in a study directed toward segregation in Kansas (*9*), stated that a big problem with segregation is that it is often unnoticeable when the pavement is placed, but, after a year of traffic, segregated spots appear. The problem is, as Lackey says, "You can't cure them if you can't see them."

Lackey's concern about nonvisibility was shared by Kennedy et al., who suggested that wet pavements and a low angle of sunlight would make segregation more visible (*1*). Lackey approached the problem by measuring density profiles with nuclear density meters. As stated earlier, segregated areas of pavements have open textures and low densities; hence, low-density spots on profiles may well be the result of segregation. These profiles can indicate segregated spots shortly after the pavement is placed.

Plentiful information exists to help diagnose the cause of segregation that is visible behind the paver. Acott and Dunmire prepared a paper on hot mix construction (*2*) in which field and laboratory experience was rendered in a format that could readily assist field diagnosis of mat deficiencies, one of which was segregation. Their table points to possible causes of segregation and helps identify what may appear to be segregation but is in fact not detrimental to the performance of the HMA pavement. In another paper (*3*), NAPA presented much of the same data in a flowchart. Kennedy et al. (*1*) also prepared a checklist that can help pinpoint the source of a segregation problem.

Prevention of Segregation

In his 1984 paper, Nady (*8*) referred to a case study of several paving jobs in which all variables (e.g., aggregate blend, asphalt course, HMA facility, paver crew, and so on) were held constant except asphalt content. From this study, he concluded that segregation could be reduced by increasing the asphalt content.

Kennedy et al. (*1*) reported that a 0.2 percent increase in asphalt content often would eliminate segregation problems. They stated that a mix with an asphalt content significantly less than the one that produced the minimum voids in mineral aggregate (VMA) tended to have more segregation problems than the mix whose asphalt content was near the one that produced the minimum VMA. They also recognized that mixes with large or coarse-graded aggregate are more prone to segregate than fine-graded mixes; likewise, gap-graded mixes tend to segregate more than well-graded mixes.

NAPA (*3*) said that proper mix design could eliminate segregation without changing the asphalt content. NAPA cautioned, however, that segregation in the stockpiles had to be eliminated to produce a mix near the design gradation.

TEST PLAN FOR CURRENT RESEARCH

The first step in developing a test plan to collect data was inspecting ongoing projects to identify what problems needed to be studied. Completed pavements were also inspected to evaluate the extent of segregation.

Next, the causes of segregation on a number of projects were evaluated. Three types of GDOT mixes were evaluated, B, base, and E mixtures. Because the base mix was coarser than the other mixes, it tended to segregate more. Mixes produced with various types of plant and equipment were evaluated, as shown in Table 1. Projects under way did not encompass all of the combinations shown in Table 1, but as many combinations as possible were evaluated.

A sampling plan and series of tests were specified for each block evaluated. The sampling plan shown in Figure 1 was followed at each location. Additional samples were taken within each test area at observed segregated areas. Data were also obtained from quality control tests during construction. By using this approach, the aggregate gradation and variability could be followed from start to finish. Tests for gradation, asphalt content, density, and theoretical maximum density were run on all of the samples.

After data were obtained and analyzed, laboratory evaluation of material representative of that in the field began. One aggregate type was selected and mixes were prepared with gradation varying from slightly finer than the job mix formula to a gradation representative of a badly segregated mix. (When a mixture segregates, the asphalt content is normally higher for the finer material and lower for the coarser material, a fact taken into account when the laboratory samples were prepared.) The mixes were prepared in a Gyratory Testing Machine (GTM) using 120 psi pressure, 30 revolutions, and 1-degree angle. These samples were tested after being compacted to determine density, voids in total mix, stability, flow, indirect tensile strength, and permeability.

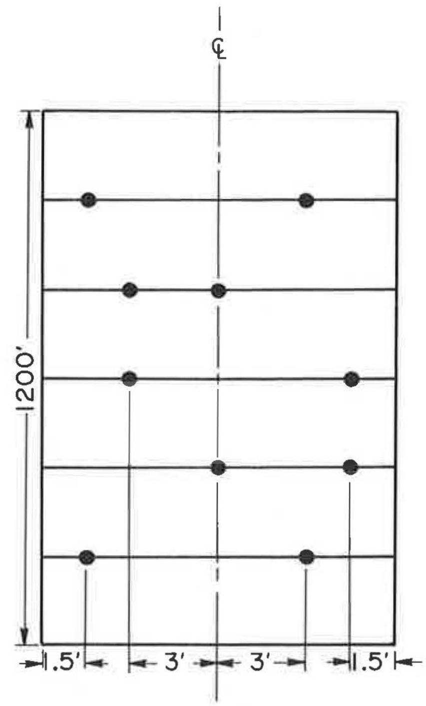

FIGURE 1 Layout of random sampling plan.

ANALYSIS OF TEST RESULTS TO EVALUATE AGGREGATE SEGREGATION

A test plan was developed to evaluate segregation of asphalt mixtures being produced and placed in the summer of 1987. The plan involved sampling a number of projects immediately after construction (see Figure 1) to evaluate bulk density, theoretical maximum density, gradation, asphalt content, and voids in total mixture. Cores were also taken in noticeably segregated areas within the 1,200-ft test layout and subjected to the same tests as the random cores. Data on mix design and testing during plant production were also obtained for comparison with the in-place properties.

The test results for the 19 projects evaluated for the random

TABLE 1 TYPES OF MIXES AND EQUIPMENT EVALUATED FOR SEGREGATION

	Batch Plant				Drum Mix Plant			
	Silo Used		Silo Not Used		Coater		No Coater	
	Separate Stockpiles[a]	Combined Stockpiles[b]	Separate Stockpiles	Combined Stockpiles	Separate Stockpiles	Combined Stockpiles	Separate Stockpiles	Combined Stockpiles
B mix	—	—	—	4[c]	—	2	—	3
Base mix	1	1	—	—	—	2	1	2
E mix	—	—	—	—	—	—	1	2

[a]Coarse aggregate stockpiles separated into individual sizes.
[b]Coarse aggregate stockpiles containing combined stockpiles.
[c]Number of projects evaluated.

TABLE 2 SUMMARY OF DATA FOR THE 19 PROJECTS EVALUATED

		Plant Mixes				Random Samples						Segregated Samples					
		Passing No. 8		Asphalt Content		Passing No. 8		Asphalt Content		Voids		Passing No. 8		Asphalt Content		Voids	
Project Number	Mix* Type	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ
1	1	36	2.3	4.4	0.26	34	1.2	4.3	0.20	3.0	1.2	-	-	-	-	-	-
2	2	38	3.1	4.6	0.13	37	3.1	4.6	0.27	3.5	0.5	35	5.4	4.5	0.24	4.1	1.5
3	10	37	0.0	4.9	0.01	42	2.1	5.3	0.40	3.7	0.8	39	2.4	5.0	0.23	5.0	0.9
4	10	36	1.2	4.9	0.09	34	1.9	4.8	0.25	6.3	3.6	-	-	-	-	-	-
5	10	36	1.4	5.2	0.12	37	2.7	5.4	0.30	-	-	35	4.2	5.4	0.71	-	-
6	10	38	1.5	5.0	0.01	38	1.3	5.0	0.20	6.4	2.1	31	5.3	4.4	0.57	10.5	1.1
7	14	-	-	-	-	37	2.2	4.5	0.24	3.6	0.6	30	2.4	4.4	0.49	5.8	1.0
8	14	33	2.5	4.8	0.29	34	3.2	4.0	0.46	7.5	1.8	32	3.6	4.0	0.42	7.8	1.3
9	16	37	1.2	4.5	0.20	35	2.2	4.0	0.58	4.1	1.8	28	1.8	3.6	0.10	4.4	1.9
10	16	37	2.7	4.5	0.22	39	1.5	4.6	0.14	6.1	0.9	36	1.3	4.5	0.30	7.1	0.3
11	19	36	2.1	4.2	0.23	34	4.1	4.1	0.84	7.3	1.7	30	2.0	4.6	1.21	2.6	-
12	20	36	2.5	4.4	0.21	38	3.4	4.4	0.57	7.1	1.8	26	3.3	3.2	0.42	9.6	2.5
13	20	34	1.7	4.0	0.20	37	3.0	4.1	0.16	5.5	3.8	28	0.7	3.2	0.35	2.8	1.3
14	22	40	1.0	5.1	0.09	42	4.8	4.8	0.37	7.3	1.5	28	7.3	3.6	0.63	9.5	1.1
15	22	39	2.0	5.0	0.16	40	2.0	5.0	0.19	4.4	1.2	30	1.1	4.3	0.23	6.4	1.9
16	22	28	7.7	3.7	0.65	36	2.9	4.4	0.29	4.5	1.1	28	2.5	3.6	0.33	8.8	0.8
17	23	48	2.0	5.5	0.22	48	0.8	5.6	0.2	5.5	0.7	-	-	-	-	-	-
18	24	43	2.7	5.4	0.16	42	1.6	5.4	0.16	5.2	0.9	42	2.5	5.7	0.18	4.2	1.0
19	24	47	4.7	5.3	0.10	50	3.3	5.8	0.51	8.6	1.2	48	5.6	5.9	0.52	8.6	0.7

* Mix Type 1 - Batch Plant with Silo, Base mix, Single Size Coarse Aggregate
2 - Batch Plant with Silo, Base mix, Combined Size Coarse Aggregate
10 - Batch Plant without Silo, B Mix, Combined Size Coarse Aggregate
14 - Drum Mix with Coater, Base Mix, Combined Size Coarse Aggregate
16 - Drum Mix with Coater, B Mix, Combined Size Coarse Aggregate
19 - Drum Mix without Coater, Base Mix, Single Size Coarse Aggregate
20 - Drum Mix without Coater, Base Mix, Combined Size Coarse Aggregate
22 - Drum Mix without Coater, B Mix, Combined Size Coarse Aggregate
23 - Drum Mix without Coater, E Mix, Single Size Coarse Aggregate
24 - Drum Mix without Coater, E Mix, Combined Size Coarse Aggregate

samples, segregated samples, and plant samples are shown in Table 2. The results cover percent passing the No. 8 sieve, asphalt content, and voids in total mix. These data were analyzed to evaluate the extent of segregation in the paving projects and to identify sources of segregation.

In many cases the plant mix tests were made over several days of operation, on material placed in the test area and on other material as well. Using plant mix samples beyond the test area was necessary to obtain sufficient plant samples for analysis. Also, although the random samples taken in the field were truly random, the plant samples were not. The plant samples did come from material taken from the back of trucks, however, and truck samples are normally representative of the batch being sampled.

After the random samples were obtained, the test area was inspected to locate any segregated areas. Of the 19 sections sampled, 16 contained segregated areas, which were tested for comparison with the random samples. The No. 8 sieve, common to all of the projects, was used to compare the various mixtures.

The test results were evaluated to better understand the difference between random and segregated samples. Test results indicate that the difference in the percent passing the No. 8 sieve for the random and segregated samples measures the degree of segregation. Figure 2 shows significant correlation between percent passing the No. 8 sieve and asphalt content, which suggests that high variability in asphalt content may be caused not by high variations in asphalt being added to the mixture nor by the variability of the extraction test, but by segregation of the mixture before sampling.

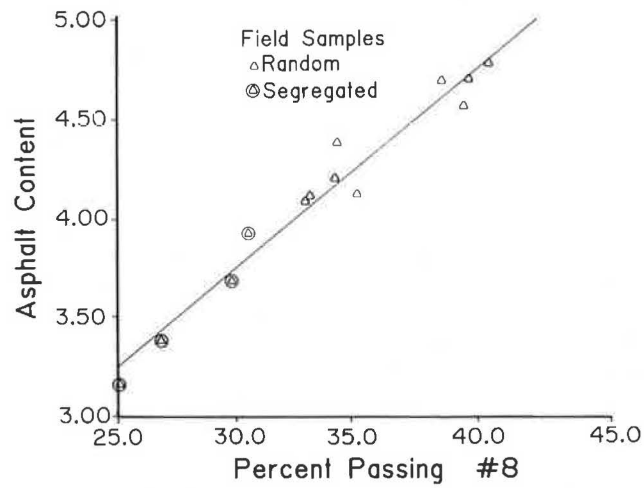

FIGURE 2 Typical relationship between asphalt content in field samples and percent passing no. 8 sieve (project 16).

The diversity of project sample origin precluded a statistical evaluation of the effects of plant type, silo, and mix type on segregation, but analysis of test results at least suggests trends. Three independent measurements of gradation and asphalt content could, however, be used to compare the various projects. These measurements are variability of random samples,

TABLE 3 NUMERICAL COMPARISON OF VARIOUS PROJECTS

Project	Mix Type	Rating for Random Samples	Rating for Plant Samples	Rating for Random Minus Segregated Samples	Overall Score	Overall Rating
1	1	2	9	1	4	2
2	2	13	13	2	9.3	12
3	10	8	1	3	4	2
4	10	6	2	1	3	1
5	10	10	3	2	5	3
6	10	3	4	5	4	2
7	14	9	-	5	7	7
8	14	14	10	2	8.7	11
9	16	9	2	5	5.3	4
10	16	4	12	3	6.3	6
11	19	4	12	3	7.7	9
12	20	16	10	9	11.7	15
13	20	12	5	7	8	10
14	22	18	6	10	11.3	14
15	22	7	7	8	7.3	8
16	22	11	11	6	9.3	12
17	23	1	7	1	3	1
18	24	5	12	1	6	5
19	24	15	14	2	10.3	13

TABLE 4 COMPARISON OF AVERAGE RESULTS FOR BATCH AND DRUM MIX PLANTS

Plant Type	Standard Deviation Plant Mixes: Percent Passing No. 8 Sieve	Standard Deviation Plant Mixes: Asphalt Content	Standard Deviation of Random Samples: Percent Passing No. 8 Sieve	Standard Deviation of Random Samples: Asphalt Content	Differences in Percent Passing No. 8 Sieve for Random and Segregated Samples
Batch	1.7	0.10	2.0	0.27	3.5
Drum Mix	2.7	0.25	2.8	0.38	7.6

variability of plant samples, and difference between random samples and segregated samples. A comparison of the various projects is shown in Table 3. The higher the variability of the project, the higher its numerical rating. The overall rating of a particular project was determined by averaging the three individual ratings. Table 3 clearly shows that the overall rating of the batch plant projects exceeds that of the drum mix projects. It also demonstrates that the overall best five projects (excepting E mixes, which tend not to segregate) from the standpoint of variability were constructed with batch plants. The worst performing projects were those with mix types 20 and 22, which included drum mix plants without coaters using combined-size coarse aggregate. Table 4 compares differences in random and segregated samples from drum mix and batch plants. On the average, control of mixes produced with a batch plant was much better than for those produced with drum mix plants.

LABORATORY INVESTIGATION OF PROPERTIES OF SEGREGATED MIXES

Shortly after placement of the binder layers of four different asphalt pavements in Georgia, ten 6-in. cores were drilled according to the pattern shown in Figure 1. Cores were also drilled at any apparent segregated spots in the 1,200-ft test section. Extraction tests were performed on all cores to reveal aggregate gradation and AC content. The results of these gradation tests were used to design a laboratory study to evaluate the effect of segregation on properties of asphalt mixtures.

Two facts emerged from the gradation test results. First, the severity of segregation varies widely from one project to another. Second, gradation curves apparently run approximately parallel for the random and segregated samples of the mixes investigated.

Six different gradations were used in the laboratory investigation (Table 5). These gradations ranged from the fine side of the mix design to the coarse side. The field data were also used to establish AC contents for the laboratory-prepared samples. The average asphalt content along with the average percent passing each sieve (for the random samples only) were used to determine the mix design film thickness. Working backwards from film thickness, the AC content was calculated for each aggregate gradation evaluated in the laboratory. These calculated AC contents compared favorably with those measured during extraction tests.

Each sample was compacted with a GTM set at 30 revo-

TABLE 5 RANGE OF AGGREGATE GRADATION FOR MIXTURES EVALUATED IN THE LABORATORY

Sieve Size	Mix 1	Mix Design Mix 2	Mix 3	Mix 4	Mix 5	Mix 6
1"	100	100	100	100	100	100
3/4"	100	98.1	96.6	95.0	93.4	91.9
1/2"	88.2	80.6	73.6	66.7	59.8	52.8
3/8"	77.6	67.0	59.6	52.3	44.9	37.6
4	60.1	51.5	44.0	36.6	29.2	21.8
8	45.8	39.8	34.3	28.8	23.3	17.8
16	31.4	27.8	24.5	21.2	17.9	14.6
30	24.0	21.2	18.8	16.5	14.2	12.1
50	16.7	14.7	13.3	11.9	10.5	9.1
100	10.7	9.1	8.2	7.4	6.6	5.7
200	6.4	5.4	4.8	4.3	3.8	3.2
% AC	5.75	5.04	4.57	4.09	3.61	3.13

lutions, 120 psi, and 1-degree angle. After the sample was compacted, it was removed from the mold and allowed to cool for at least 2 hr before handling.

Information on the voids in total mix for each of the six mixes is presented in Figure 3. The total voids increased dramatically as the degree of segregation increased. Since segregation leads to high voids, even the smallest amount of segregation is unacceptable.

A falling head permeability test was set up to more directly approach the problem presented by high voids. Tests were performed only after 30 mm of water had drained through the sample. If there was no noticeable drop (1 mm) in the water level in 5 min, the sample was considered impermeable.

The permeability data are plotted in Figure 4. Note that the samples are impermeable for gradations 1, 2, and 3, and that permeability increases dramatically from gradation 4 to gradation 6. Most AC pavement layers are designed and constructed to be impermeable. If segregation results in a permeable layer, then the AC will allow water to seep through, potentially causing weakening of the subgrade or stripping.

Samples of asphalt mixture with each aggregate gradation were used to determine how tensile strength changed with an increasing degree of segregation. The samples were tested in indirect tension using a constant deformation rate of 2 in. per minute. The data from the indirect tensile test are shown in Figure 5. The graph indicates that tensile strength decreased rapidly with an increase in degree of segregation. Decreased tensile strength may lead to excessive cracking in the pavement or to raveling of the mixture. Inspection of pavements that had segregation problems verified that raveling was a problem if the segregated areas had not been overlaid.

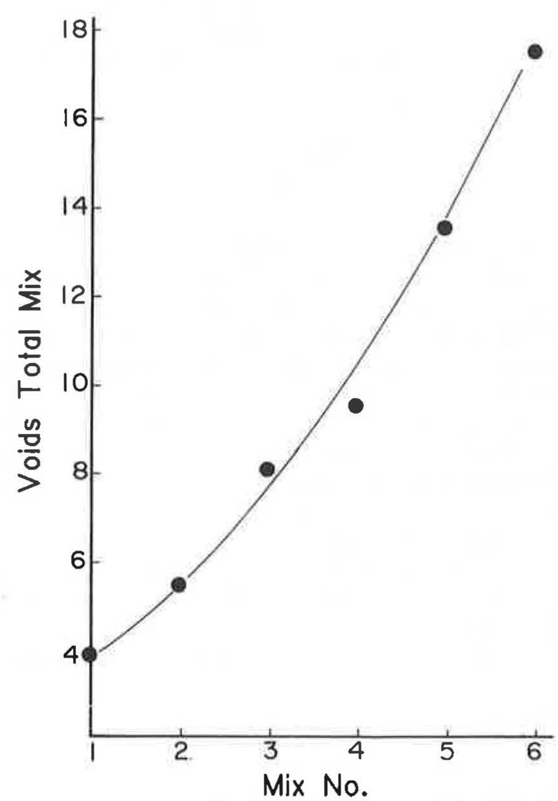

FIGURE 3 Voids in total mix versus degree of segregation.

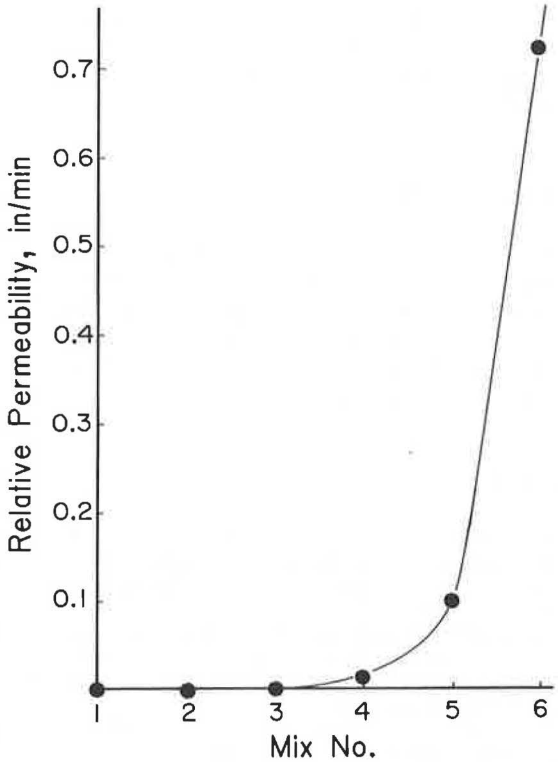

FIGURE 4 Permeability versus degree of segregation.

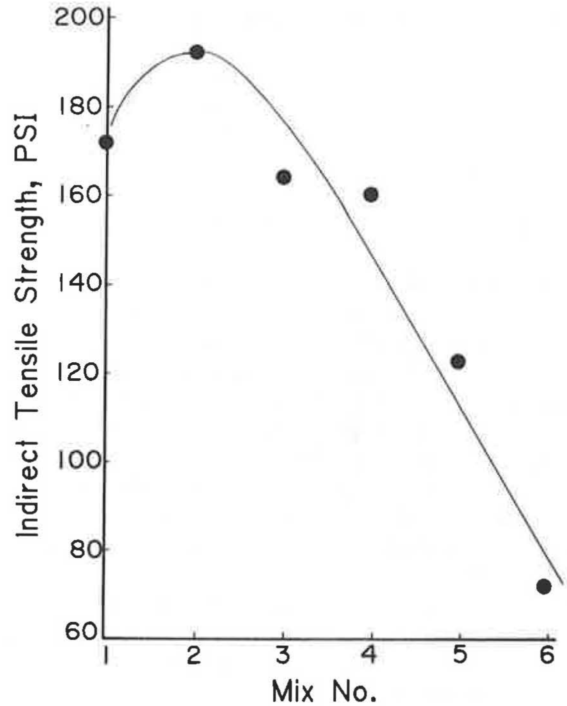

FIGURE 5 Indirect tensile strength versus degree of segregation.

REVIEW OF CONSTRUCTION PROCEDURES

Several site visits were made during construction of projects to observe construction procedures. Construction operations were observed at both the asphalt plants and laydown sites.

Most of the asphalt plants visited were drum mix plants, which were then becoming popular because of their portability, low initial cost, and production capacity. The stockpiling operations in some locations were satisfactory but less than desirable at other locations. In many instances, the contractor did not follow GDOT guidelines. For example, the contractor did not maintain separate stockpiles for different sizes of aggregate, especially for drum mix plants. The contractor often used one coarse aggregate containing several aggregate sizes, making it difficult to control aggregate gradation and minimize aggregate segregation. In a few cases, the contractor used crusher-run material graded from coarse to fine, yet segregation undoubtedly occurs when crusher-run material is used with a drum mix plant.

Many segregation problems in the field seemed to stem from the storage silo. All drum mix plants have some sort of storage silo and most batch plants do, as well. Segregation in some cases was caused by nonsymmetrically loading the conveyor belt carrying material to the silo. In at least one case, the batcher at the top of the silo was not functioning correctly, resulting in segregation. Again, following the comprehensive GDOT guidelines would have deflected these problems. However, on many projects segregation problems are either not seen as they occur or nothing is done to correct them during construction. Many contractors will not correct a problem if they are allowed to operate with the problem; it is the state that usually has to take the initiative to get a problem solved.

In some cases, the asphalt mixture segregated as it was loaded from the storage silo onto the truck. One reason for this segregation is the length of time the gate at the bottom of the silo remained open. Ideally, the material should be dropped in batches to minimize segregation caused by the large aggregate rolling down the side of the asphalt mixture to the edge of the truck bed. Most contractors used the suggested method of loading trucks (front first, back second, and middle last), but some did not.

Regardless of its source, segregation was most apparent when material was unloaded from trucks. If segregation could be eliminated at this juncture, segregation would no longer be a major concern. However, end-of-load segregation has many causes. On one project, the contractor used dump trucks with long bodies, which seemed to exacerbate segregation. On another, the asphalt paver was traveling too fast, allowing the material being fed to the screed to run low. The auger was turning rapidly to feed the two ends of the screed, consequently throwing coarse aggregate to the outside plate. Several steps have been taken to reduce end-of-load segregation. One method that has been partially successful is the use of Flo-Boys to haul asphalt mixtures; the mixture is extruded from the Flo-Boys and hence separation of materials is minimized.

In general, field observations validate that the contractor and GDOT personnel were paying close attention to segregation problems. Nonetheless, segregation is difficult to spot during construction and it is not always a simple matter to correct even when it is seen.

CONCLUSIONS AND RECOMMENDATIONS

Based on observation of paving projects during construction and on analyses of tests conducted by GDOT and further tests conducted at Auburn University, the following conclusions are warranted.

- Segregated areas that are not overlaid tend to ravel under traffic.
- The loss of desirable mixture properties is significant when the gradation of the segregated mixture is approximately 10 percent coarser than the job mix formula on the No. 8 sieve.
- Quality control is very important in reducing segregation. Either type of asphalt plant can be controlled to produce a good product; uncontrolled, either can produce a bad product. Generally, the batch plant produces a more consistent product (one with less segregation) than the drum mix plant. Data show that a drum mix plant with a coater produces a more consistent mixture with less segregation than a drum mix plant without a coater.
- Segregated areas are generally 8 to 15 percent coarser than nonsegregated areas on the No. 8 sieve; the voids are typically 3 to 5 percent higher; and the asphalt content is often 1 to 2 percent lower.
- There is no correlation between the variability of plant sample gradations and the amount of segregation. There is a general correlation for random in-place gradation and segregation.

Recommendations concerning segregation are as follows.

- The best approach for minimizing segregation is to use a batch plant without silo and to use good stockpiling techniques (separate horizontally layered stockpiles for different aggregate sizes). If a drum mix plant is used, a coater is preferred; good stockpiling techniques are a necessity. (Even a well-controlled mixture can segregate if it is improperly placed in the storage silo or when it is removed from the silo.)
- Since normal quality control tests cannot be used to predict segregation, some other method must be. Test results from this study show that visually locating segregated areas is difficult. Therefore, a nuclear gauge might be considered for use in identifying segregated areas since one will likely already be on the project for density measurement. Based on the results of this study, any segregated area with a density 4–5 pcf lower than the adjacent nonsegregated material will have a significant reduction in mix properties and should therefore be removed and replaced.

ACKNOWLEDGMENT

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Evaluation of a Thin-Lift Nuclear Density Gauge

THOMAS W. KENNEDY, MAGHSOUD TAHMORESSI, AND MANSOUR SOLAIMANIAN

This paper describes the results of a research study to determine the effectiveness of the Troxler Model 4640 thin-lift nuclear density gauge in estimating core densities. The study consisted of obtaining density measurements using cores and the nuclear gauge on seven construction projects and comparing the nuclear to core density readings. The projects were either newly constructed or under construction when the tests were performed. Correlation coefficients were determined to indicate the degree of correlation between core and nuclear densities. Linear regression was used to investigate how well the core densities could be predicted from nuclear densities. Using statistical analysis, the ranges of differences between core and nuclear measurements were established for specified confidence levels. Analysis of the data shows that the accuracy of the nuclear gauge is highly material-dependent: The gauge produced acceptable results with limestone mixtures, but it did not perform satisfactorily with mixtures containing siliceous aggregate. The data presented in this paper indicate that the gauge could be used as a quality control tool, provided calibration lines are developed for each project; calibration lines can be developed using simple linear regression.

Density is one of the most important factors affecting the performance of hot-mix asphalt concrete pavements; many highway agencies use it as a quality control parameter. In-place density has traditionally been estimated by measuring the density of cores drilled from the pavement or by using nuclear gauges. Yet the core density technique is destructive, and results are seldom available fast enough to permit effective quality control. Traditional nuclear density gauges have shortcomings that make them inaccurate for layers under 2 in. Therefore, the need is strong for a density measurement technique that can accurately and quickly measure the density of thin lifts of pavement.

The Troxler Model 4640 thin-lift nuclear density gauge was specifically designed to measure in-place density of thin pavement layers. The Texas State Department of Highways and Public Transportation, as part of its Cooperative Highway Research Program and an ongoing research project to determine density of hot-mix asphalt concrete pavements, wanted an evaluation of the Troxler 4640 gauge. The specific purpose of this study was to find out whether the gauge could accurately determine the in-place density of hot-mix asphalt concrete pavements. To this end, nuclear densities were obtained from highway sections that were under construction or were newly paved and cores were then taken from each location. The relationships between core and nuclear densities were analyzed.

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EXPERIMENTAL PROGRAM

The objective of this study was to compare measured core densities with nuclear densities obtained with the Troxler 4640 gauge. Regression analyses were used to establish the relationships between the two methods, and the differences between the core and nuclear densities for each project were analyzed. The purpose of these analyses was to establish the accuracy with which the nuclear density gauge could estimate the core density.

The experimental program consisted of measuring in-place density of several highway sections during construction or shortly after construction had been completed by both methods.

TEST SITES AND METHODS

Seven construction projects at various locations throughout Texas were selected for field tests. Four projects used limestone as the primary aggregate source. The remaining three projects used siliceous aggregates.

The mixtures used in all projects were dense-graded hot-mix asphalt concrete placed on heavily trafficked roads. All projects were overlays on existing pavement surfaces; the overlay thickness ranged from 1 to 2 in.

Nuclear Density Measurements

To use the Troxler 4640 gauge, the thickness of the top layer of thin lifts of hot-mix asphalt concrete pavements must be entered in the gauge; thickness may range from 1 to 2.5 in. The gauge operates on a backsatter mode and uses an 8-mCi cesium 137 source, which emits gamma (GM) radiation, and two GM radiation detector tubes. Placing the two GM tubes at different distances from the source allows the top layer density to be mathematically determined (*1*).

According to the manufacturer, the gauge's accuracy increases as the thickness of the top layer increases, and the best accuracy is obtained with a 4-min reading time. Reading times as low as 30 sec may be used, but accuracy is lower (*2*). For this study, 1-min readings were taken. For 1-min readings, the accuracy ranges from ± 0.76 to ± 1.25 pcf, depending on the thickness of the layer (*2*).

For each project in this study, nuclear density measurements were taken with the Troxler 4640 gauge at 15 to 25 different locations on the wheel path at intervals of 100 to 500 ft. The following briefly describes the technique used for taking the nuclear density measurement.

- A 4-min count was taken and used for each project as a baseline against which to measure other readings.
- Four 1-min nuclear density readings were taken for each core location. The gauge was rotated 90 degrees between consecutive readings. If one of the four readings significantly differed from the other three, another reading was taken without moving the gauge, when possible. Inconsistent readings appeared to occur at random and without apparent cause.
- To minimize the effects of surface voids, a very thin layer of sand was spread on the surface. Care was taken to use only as much sand as necessary. The sand passed 100 percent through the No. 40 sieve and was retained on the No. 80 sieve.
- The gauge was moved from place to place until it could be seated flat on the pavement surface. Past experience has proven that improper seating of the gauge will result in extremely low nuclear density readings.
- The gauge was approximately 50 ft from any vehicles when the readings were taken because interference from large objects could cause measurement errors.
- The thickness entered in the gauge for each location was the estimated overlay thickness.

Core Density Measurements

At each location, cores were drilled immediately after the nuclear density readings were taken. The cores were labeled and transferred to the laboratory where they were cut to the same thickness that was entered into the gauge.

All cores were dried to constant weight at room temperature before their densities were measured. Densities were measured using ASTM method D2726 (*3*).

DATA PRESENTATION

The results of the density measurements are shown in scatter plots in Figures 1 through 7. The difference between core density and nuclear measurements—the important parameter to be statistically analyzed—is shown in Figure 8 for projects containing limestone aggregate and in Figure 9 for projects containing siliceous aggregate. Figures 10 and 11 illustrate the relationship of the differences between density measurements to the density of the layer.

DATA ANALYSIS

The data graphically presented in Figures 1 through 7 indicate that there is better agreement between core and nuclear density measurements for mixtures containing limestone aggregates (Figures 1 through 4) than for mixtures containing siliceous aggregates (Figures 5 through 7). The bar graphs in Figures 8 and 9 demonstrate the same trend.

Statistical Analysis

The primary objective of this study was to determine the accuracy of the nuclear density gauge in estimating in-place density. Core density is commonly used to estimate in-place density, so the difference between core and nuclear densities was statistically analyzed. The larger the difference, the lower is the accuracy of the nuclear gauge. It should be noted, however, that there are measurement errors associated with determination of core density (see ASTM D2726 for the bias statement for core density measurement). The error in mea-

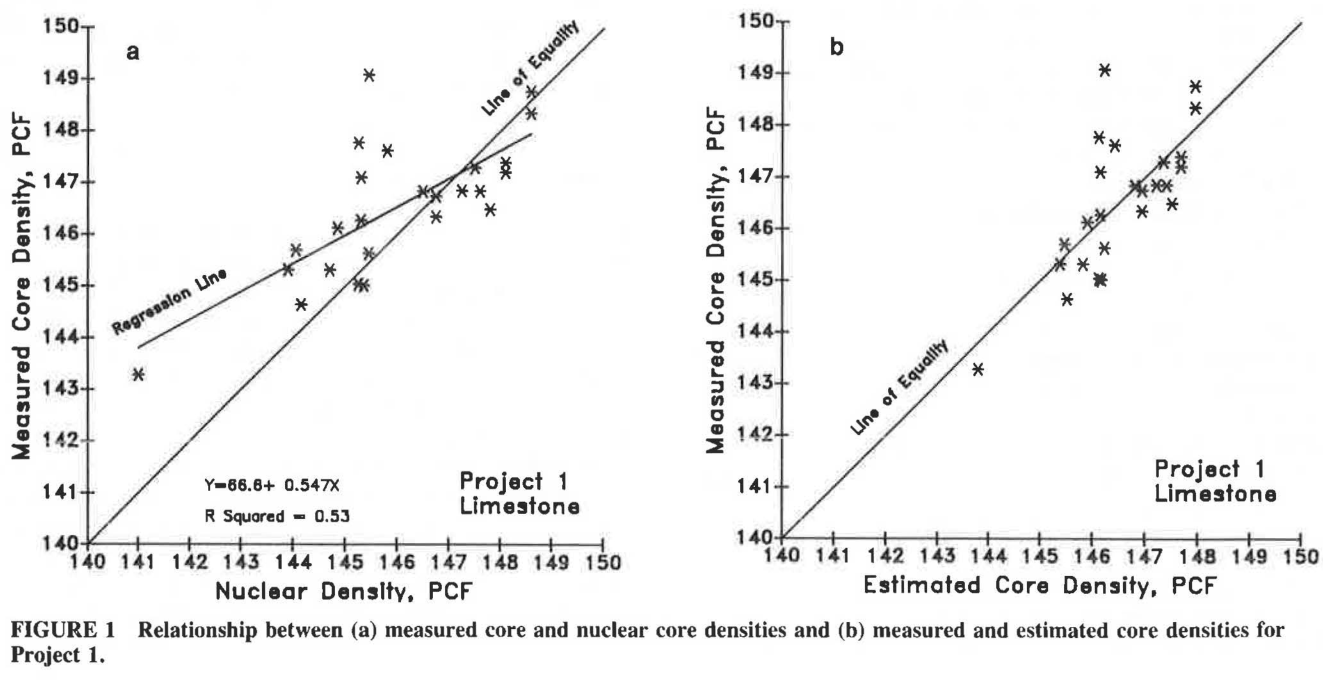

FIGURE 1 Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 1.

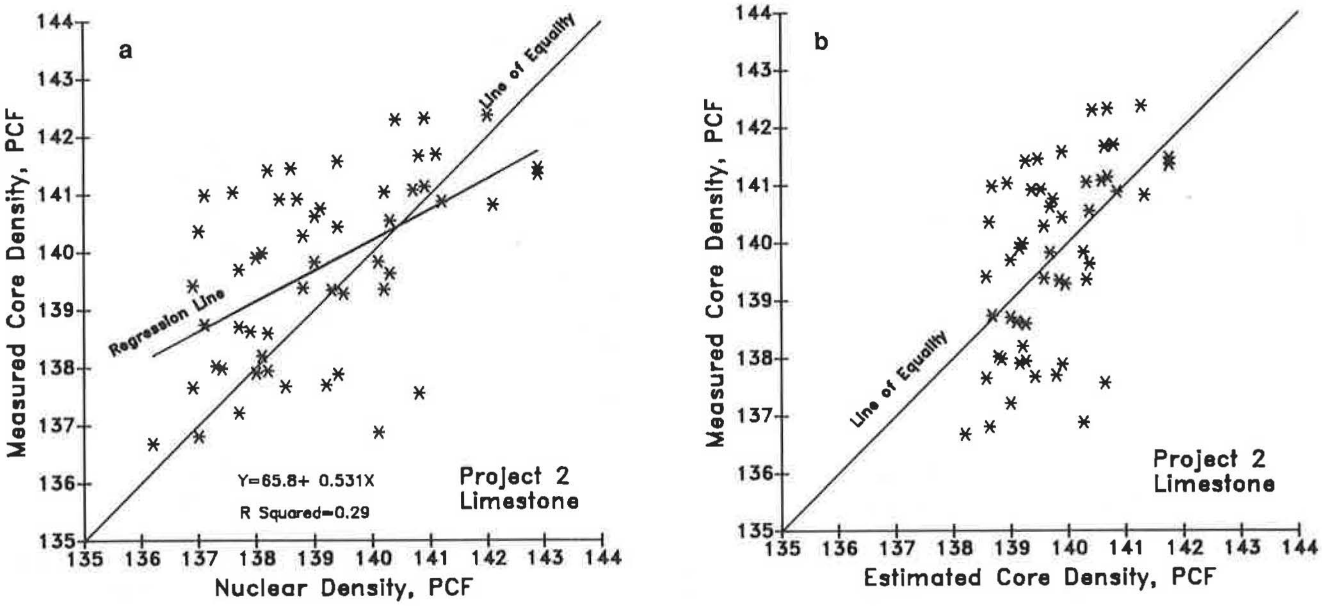

FIGURE 2 **Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 2.**

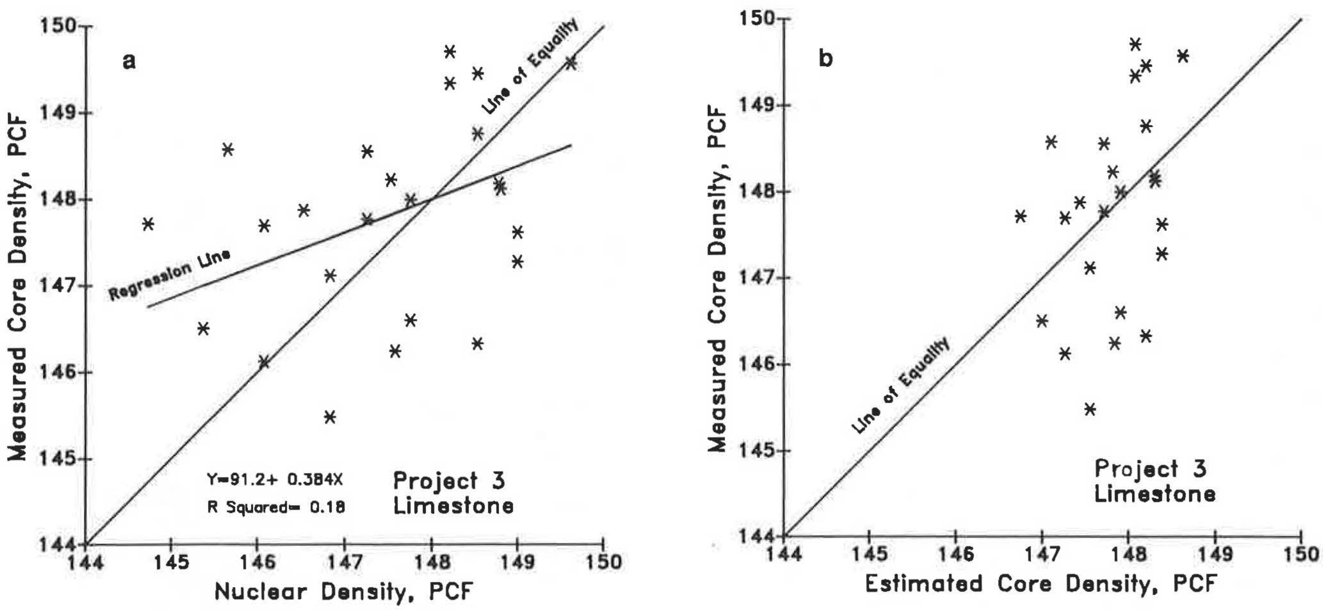

FIGURE 3 **Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 3.**

surement of core density was not incorporated in the analysis because the authors intended to determine how well the nuclear density could estimate the core density, not the true pavement density.

Confidence levels and linear regression analysis were used to analyze the differences between core and nuclear densities (*4*).

Regression Analysis

A regression analysis was performed to determine how much the results could be improved if the core densities were estimated from nuclear densities based on a regression equation. Regression lines and their corresponding equations were established based on the least-square method. The relationship between the estimated core and nuclear densities is presented by $Y = aX + b$ where X and Y are nuclear and estimated core densities, respectively. The coefficients obtained from regression are a and b. Figures 1 through 7 show the values of measured core densities and estimated values of the core densities from regression as well as the difference between the two values. Scatter plots of measured core densities versus estimated core densities are given in Figures 1b through 7b.

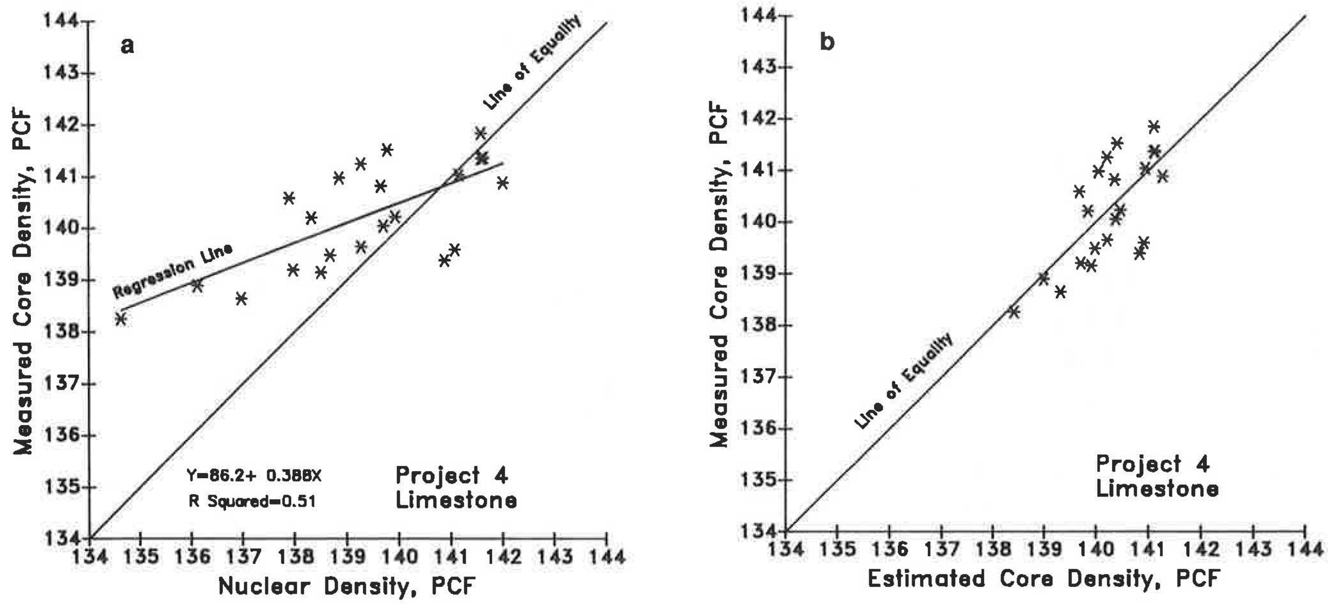

FIGURE 4 Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 4.

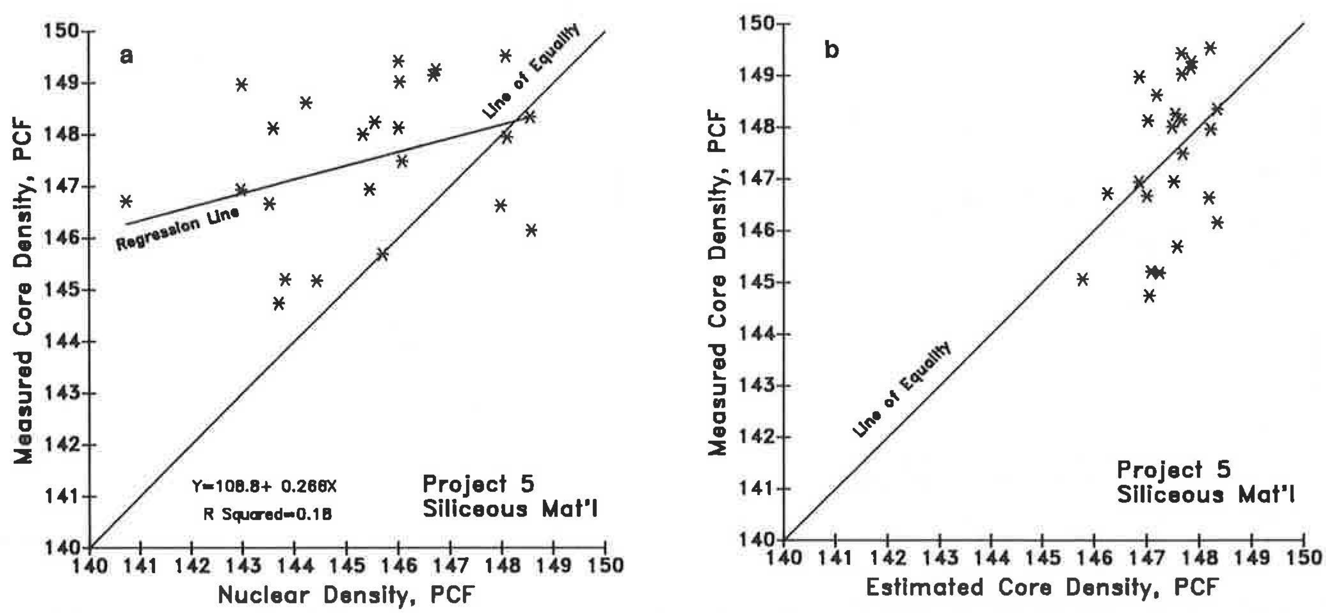

FIGURE 5 Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 5.

Confidence Levels

Ranges of differences between core and nuclear measurements were established for certain confidence levels. The probabilities (confidence levels) used to determine these ranges were 80, 90, and 95 percent. For example, for a 95 percent confidence probability, a random nuclear measurement will fall within the established range of differences with a probability of error of 5 percent (i.e., the difference between nuclear and core densities will be beyond the range for 5 percent of the paired measurements). The *t*-distribution was used to determine the desired ranges for various confidence probabilities. (Normal distribution was not used because true population mean and standard deviation were not available, but the mean and standard deviation could be estimated based on the number of observations and existing sample size.) However, because using *t*-distribution requires that the sample be drawn from a normal population, the normality of the

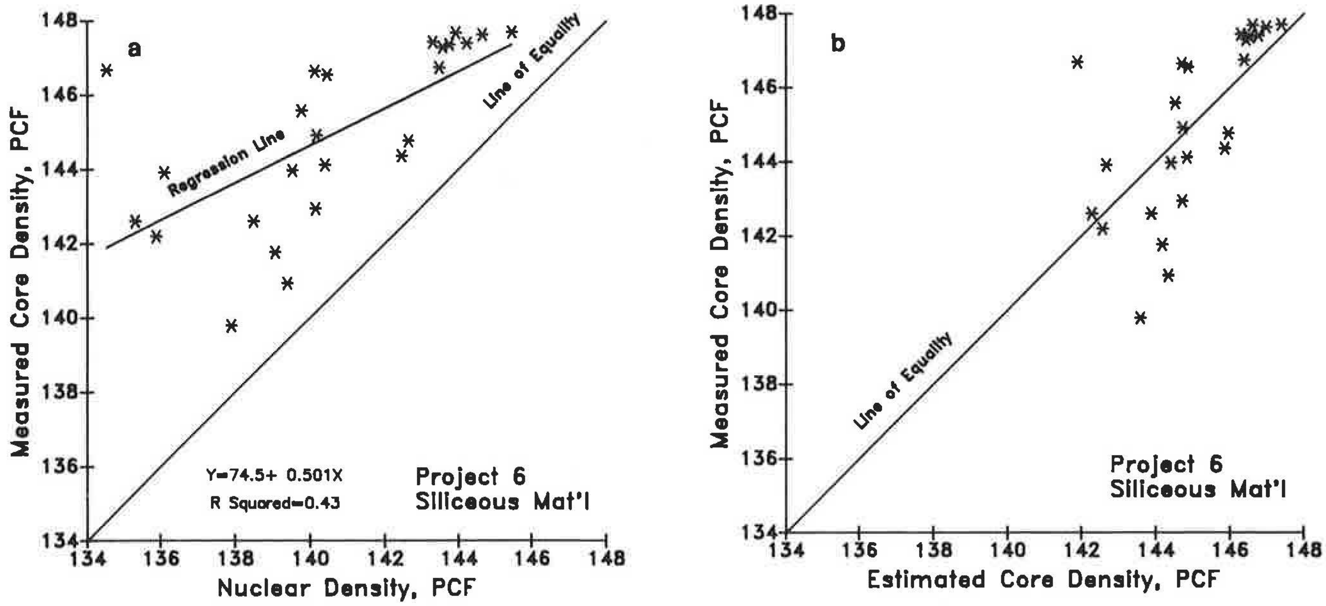

FIGURE 6 Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 6.

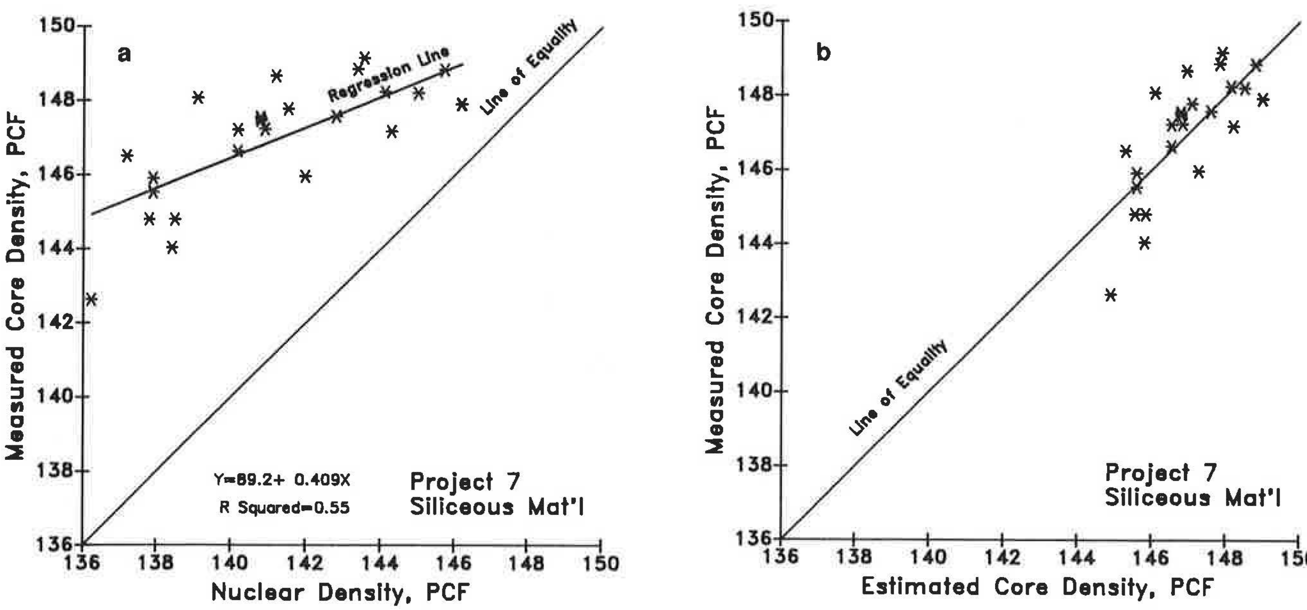

FIGURE 7 Relationship between (a) measured core and nuclear core densities and (b) measured and estimated core densities for Project 7.

sample data had to be checked by plotting the frequency histogram of the data. The typical histogram shown in Figure 12 does closely follow a normal distribution.

The following formulas show how the desired ranges were established:

$$d = X - Y$$

$$\bar{d} = \frac{\Sigma d}{n}$$

$$S_d = \left[\frac{\Sigma (d - \bar{d})^2}{n - 1}\right]^{1/2}$$

$$\nu = n - 1$$

$$R_L = \bar{d} - S_d \cdot t_\nu$$

and

$$R_U = \bar{d} + S_d \cdot t_\nu$$

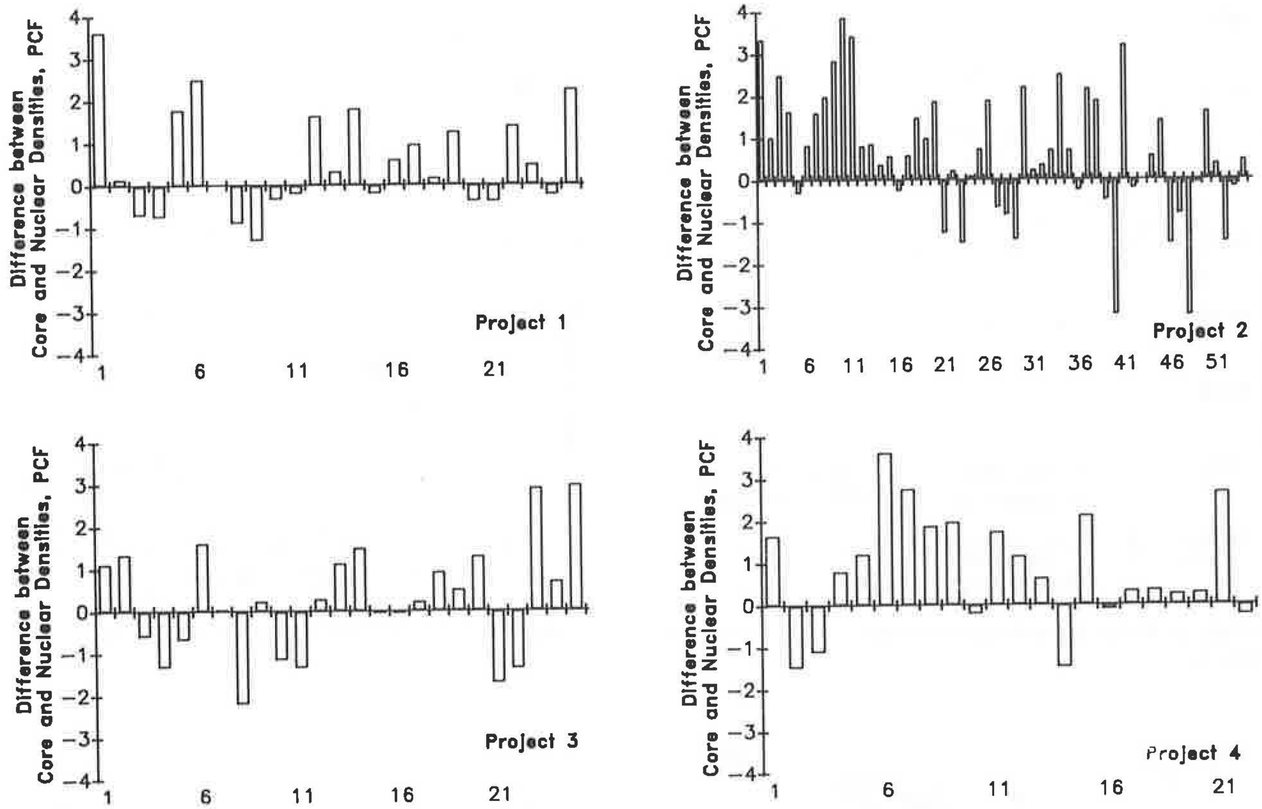

FIGURE 8 Differences between core and nuclear densities for projects containing limestone aggregate.

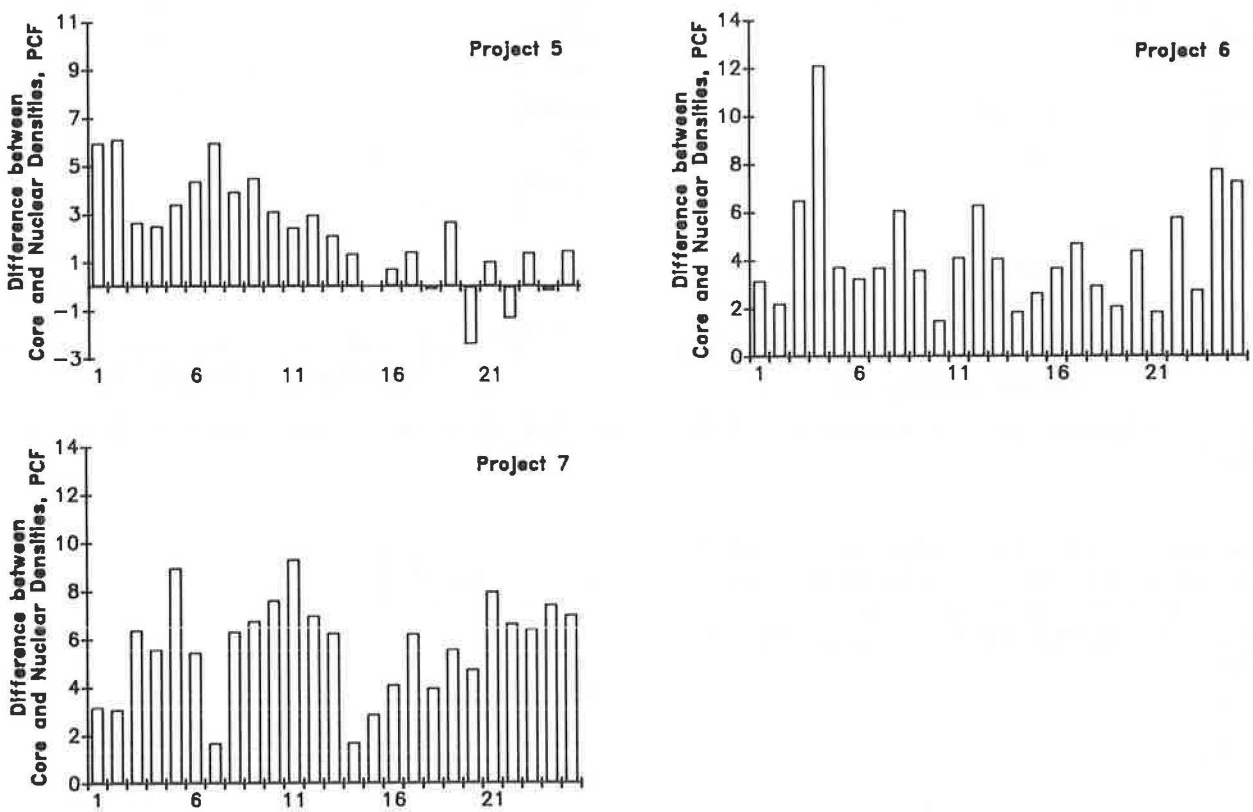

FIGURE 9 Differences between core and nuclear densities for projects containing siliceous aggregate.

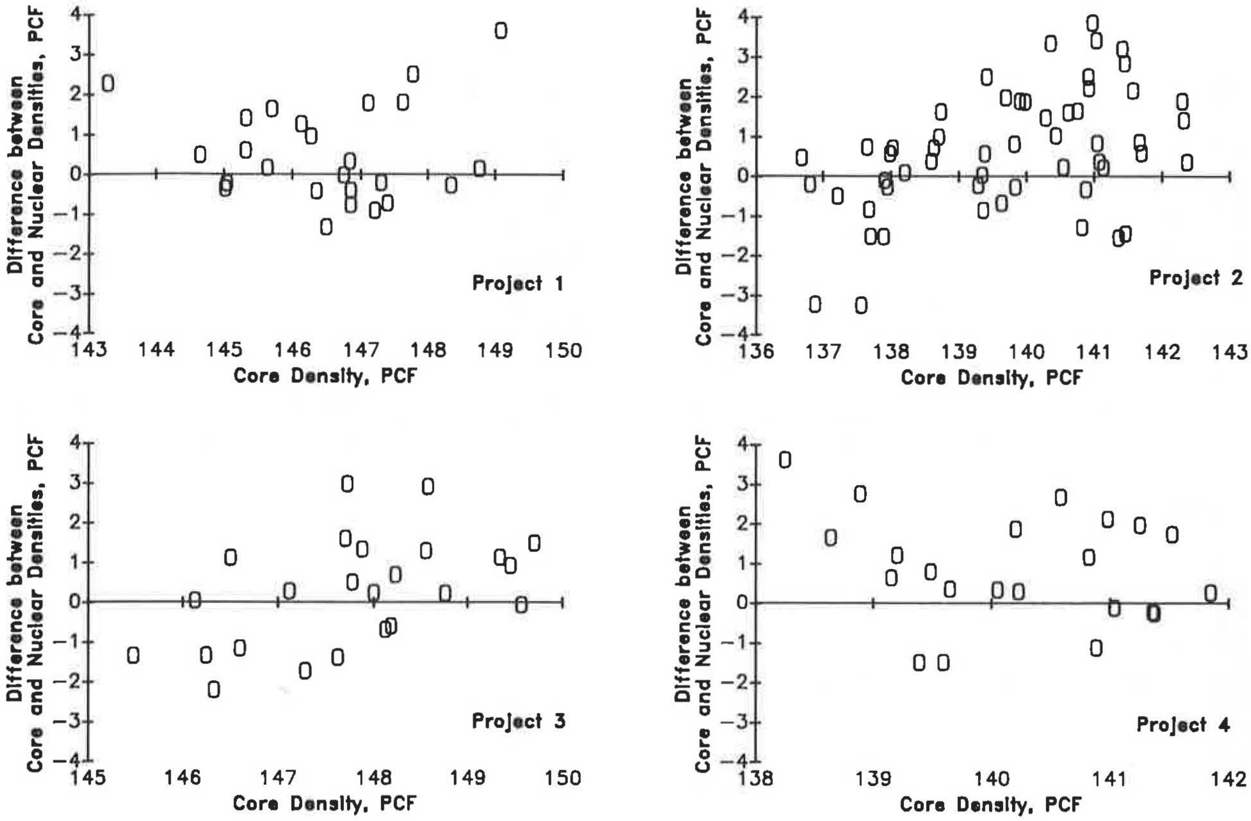

FIGURE 10 Relationship between core density and difference between core and nuclear densities for projects containing limestone aggregate.

where

X and Y = measured core and nuclear densities, respectively;
d = difference between the measurements;
$\bar{d}$ and S_d = estimates of the difference mean and standard deviation, respectively;
n and ν = sample size (number of paired observations) and degrees of freedom, respectively;
t_ν = t-value corresponding to a specified confidence probability and degree of freedom, found from t-distribution tables; and
R_L and R_U = lower and upper limits of the range of differences, respectively.

The ranges determined using t-distribution are shown in Table 1 for specified confidence levels and for different projects. The same type of analysis was performed on the data after the linear regression was applied; the results of this analysis are shown in Table 2.

DISCUSSION OF RESULTS

The scatter plots of measured core densities versus nuclear density measurements (Figures 1a through 4a) for Projects 1 through 4, which used limestone, indicate that the data are scattered on both sides of the line of equality. In some cases, the nuclear densities are higher than the core densities; in others, the opposite is true. The same trend is also evident from the bar plots in Figure 8; both negative and positive differences show up in this figure. However, for Projects 5, 6, and 7, which used siliceous material, nuclear densities tend to be consistently lower than the measured core densities (see the scatter plots in Figures 5a through 7a and bar plots in Figure 9). Moreover, the difference between core and nuclear densities is significantly higher for siliceous materials than for limestone.

The correlation coefficient for projects involving limestone varies between 0.43 and 0.73 (R squared between 0.19 and 0.53) and between 0.42 and 0.75 (R squared between 0.18 and 0.56) for those involving siliceous material. A comparison of correlation coefficients indicates that the correlation between core and nuclear densities is probably not material-dependent, whereas the nuclear density measurement itself is.

After regression equations were applied to the data to estimate core densities, the results were significantly improved. Figures 1b through 7b show that the data are considerably less scattered about the line of equality and that the differences between the measured and estimated core densities are significantly lower after applying the regression equation.

Results of the statistical analysis for confidence intervals are presented in Table 1. The table shows that, for Project 1, there is a 95 percent chance that the difference between the core and nuclear density measurements will not exceed 3

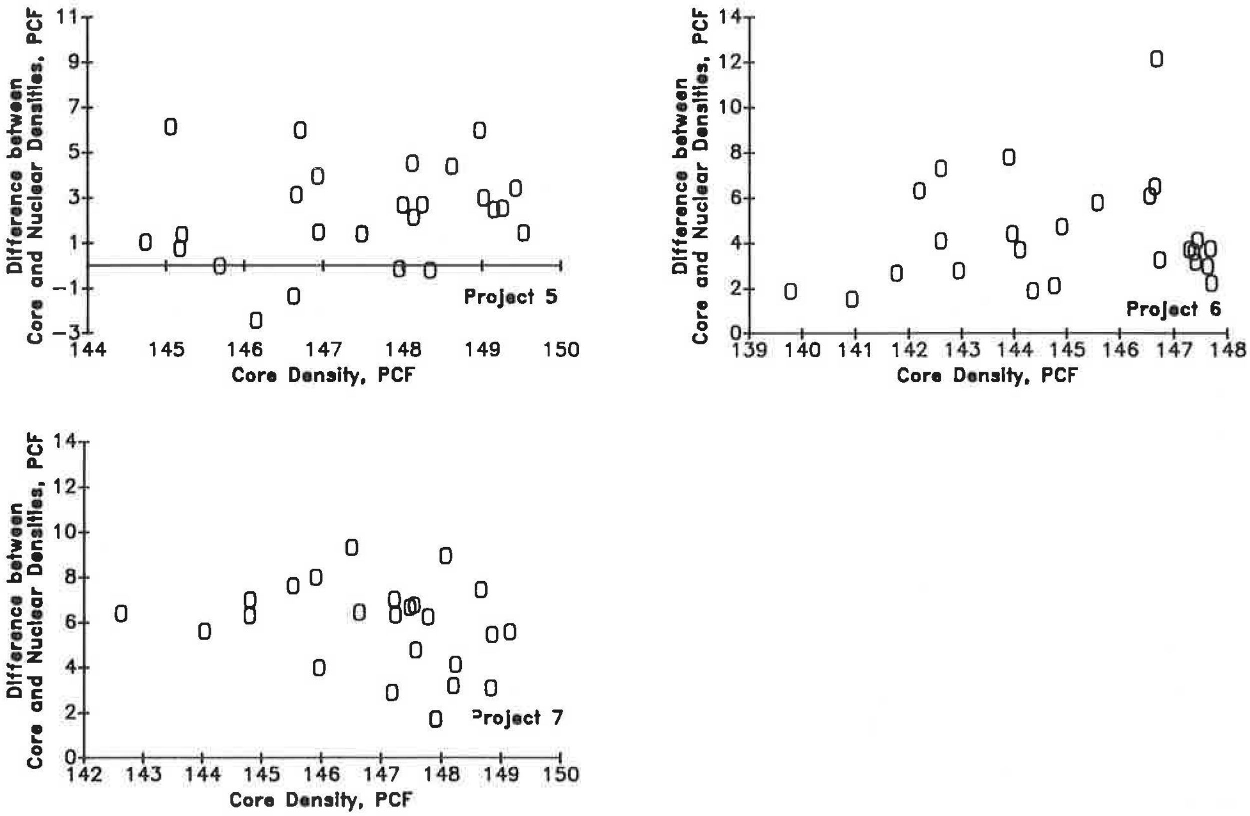

FIGURE 11 Relationship between core density and difference between core and nuclear densities for projects containing siliceous aggregate.

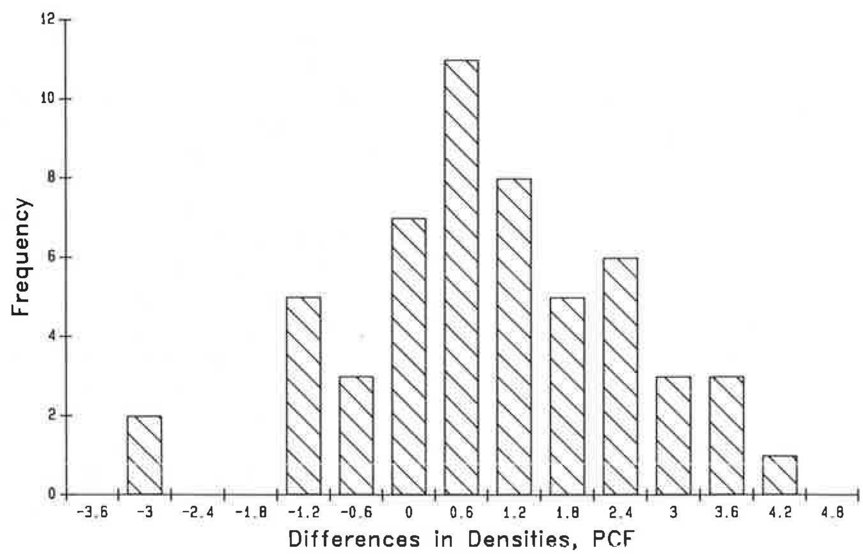

FIGURE 12 Typical histogram of the difference between core and nuclear densities, Project 2.

TABLE 1 RANGE OF DIFFERENCES BETWEEN MEASURED CORE DENSITY AND NUCLEAR DENSITY FOR VARIOUS CONFIDENCE INTERVALS BEFORE APPLYING REGRESSION ANALYSIS

PROJECT	COUNT	MEAN OF DIFF. (PCF)	STAND. DEV. (PCF)	STAD. DEV. OF MEAN	CONFID. LEVEL (%)	t-VALUE	LOWER LIMIT (PCF)	UPPER LIMIT (PCF)
1	25	0.5	1.2	0.240	80	1.318	-1.08	2.08
1	25	0.5	1.2	0.240	90	1.711	-1.55	2.55
1	25	0.5	1.2	0.240	95	2.064	-1.98	2.98
2	54	0.6	1.5	0.204	80	1.298	-1.35	2.55
2	54	0.6	1.5	0.204	90	1.674	-1.91	3.11
2	54	0.6	1.5	0.204	95	2.006	-2.41	3.61
3	25	0.3	1.33	0.266	80	1.318	-1.45	2.05
3	25	0.3	1.33	0.266	90	1.711	-1.98	2.58
3	25	0.3	1.33	0.266	95	2.064	-2.45	3.05
4	22	0.9	1.34	0.286	80	1.323	-0.87	2.67
4	22	0.9	1.34	0.286	90	1.721	-1.41	3.21
4	22	0.9	1.34	0.286	95	2.080	-1.89	3.69
5	25	2.3	2.16	0.432	80	1.318	-0.55	5.15
5	25	2.3	2.16	0.432	90	1.711	-1.40	6.00
5	25	2.3	2.16	0.432	95	2.064	-2.16	6.76
6	25	4.3	2.3	0.460	80	1.318	1.27	7.33
6	25	4.3	2.3	0.460	90	1.711	0.36	8.24
6	25	4.3	2.3	0.460	95	2.064	-0.45	9.05
7	25	5.7	2.02	0.404	80	1.318	3.04	8.36
7	25	5.7	2.02	0.404	90	1.711	2.24	9.16
7	25	5.7	2.02	0.404	95	2.064	1.53	9.87

pcf. Similar conclusions can be drawn for other confidence probabilities and other projects. This table also shows clearly that the results are better for projects involving limestone material than for those involving siliceous material.

The results of a similar type of analysis after applying regression are shown in Table 2. The ranges of differences before and after regression are compared in Figure 13, which shows confidence intervals for all projects at 95 percent probability. Obviously, regression equations can significantly improve the nuclear density results.

Therefore, it seems possible to establish an appropriate regression line based on a reasonable number of core and nuclear density measurements and to use that line to estimate core densities from nuclear measurements. However, the degree to which a regression line will improve the estimate is not well established. Moreover, analyses indicate that the accuracy of the Troxler 4640 nuclear density gauge depends on the mixture which is being measured.

SPECIAL CALIBRATION

Troxler Electronics has suggested a calibration method for the 4640 thin-lift density gauge. The procedure requires obtaining 5 cores from the roadway for a density estimate and 20 nuclear density readings for reestablishing density parameters for each project. The procedure is based on the fact that nuclear density readings are influenced by three parameters, A, B, and C, as used in the following equation:

$$D = (1/B) \times \ln[A/(CR + C)]$$

where

D = nuclear density,
CR = count ratio,
A and C = parameters that depend on gauge geometry, and
B = parameter that depends on material property.

TABLE 2 RANGE OF DIFFERENCES BETWEEN MEASURED CORE DENSITY AND NUCLEAR DENSITY FOR VARIOUS CONFIDENCE INTERVALS AFTER APPLYING REGRESSION ANALYSIS

PROJECT	COUNT	MEAN OF DIFF. (PCF)	STAND. DEV. (PCF)	STAD. DEV. OF MEAN	CONFID. LEVEL (%)	t-VALUE	LOWER LIMIT (PCF)	UPPER LIMIT (PCF)
1	25	0.0	0.9	0.180	80	1.318	-1.19	1.19
1	25	0.0	0.9	0.180	90	1.711	-1.54	1.54
1	25	0.0	0.9	0.180	95	2.064	-1.86	1.86
2	54	0.0	1.3	0.177	80	1.298	-1.69	1.69
2	54	0.0	1.3	0.177	90	1.674	-2.18	2.18
2	54	0.0	1.3	0.177	95	2.006	-2.61	2.61
3	25	0.0	1.06	0.212	80	1.318	-1.40	1.40
3	25	0.0	1.06	0.212	90	1.711	-1.81	1.81
3	25	0.0	1.06	0.212	95	2.064	-2.19	2.19
4	22	0.0	0.71	0.151	80	1.323	-0.94	0.94
4	22	0.0	0.71	0.151	90	1.721	-1.22	1.22
4	22	0.0	0.71	0.151	95	2.080	-1.48	1.48
5	25	0.0	1.33	0.266	80	1.318	-1.75	1.75
5	25	0.0	1.33	0.266	90	1.711	-2.28	2.28
5	25	0.0	1.33	0.266	95	2.064	-2.75	2.75
6	25	0.0	1.8	0.360	80	1.318	-2.37	2.37
6	25	0.0	1.8	0.360	90	1.711	-3.08	3.08
6	25	0.0	1.8	0.360	95	2.064	-3.72	3.72
7	25	0.0	2.02	0.404	80	1.318	-2.66	2.66
7	25	0.0	2.02	0.404	90	1.711	-3.46	3.46
7	25	0.0	2.02	0.404	95	2.064	-4.17	4.17

Based on the manufacturer's information, *A*, *B*, and *C* can be established for each project by using the recommended calibration technique, but that technique needs to be tried in the field to test its validity.

CONCLUSIONS

Based on research for this study, it appears that the accuracy of the Troxler 4640 thin-lift nuclear density gauge varies from project to project and depends on the materials involved. Better accuracy was observed for mixtures containing limestone than for mixtures containing siliceous aggregates.

The gauge proved to be very sensitive to improper seating; it must therefore be used by experienced and knowledgeable personnel to minimize measurement errors. The use of calibration lines through regression analyses significantly improved the prediction of core densities from nuclear measurements. However, even with calibration lines, the gauge's results must be treated cautiously: an acceptable range of difference between core and nuclear density measurements must be clearly specified, as well as an acceptable risk of error.

ACKNOWLEDGMENT

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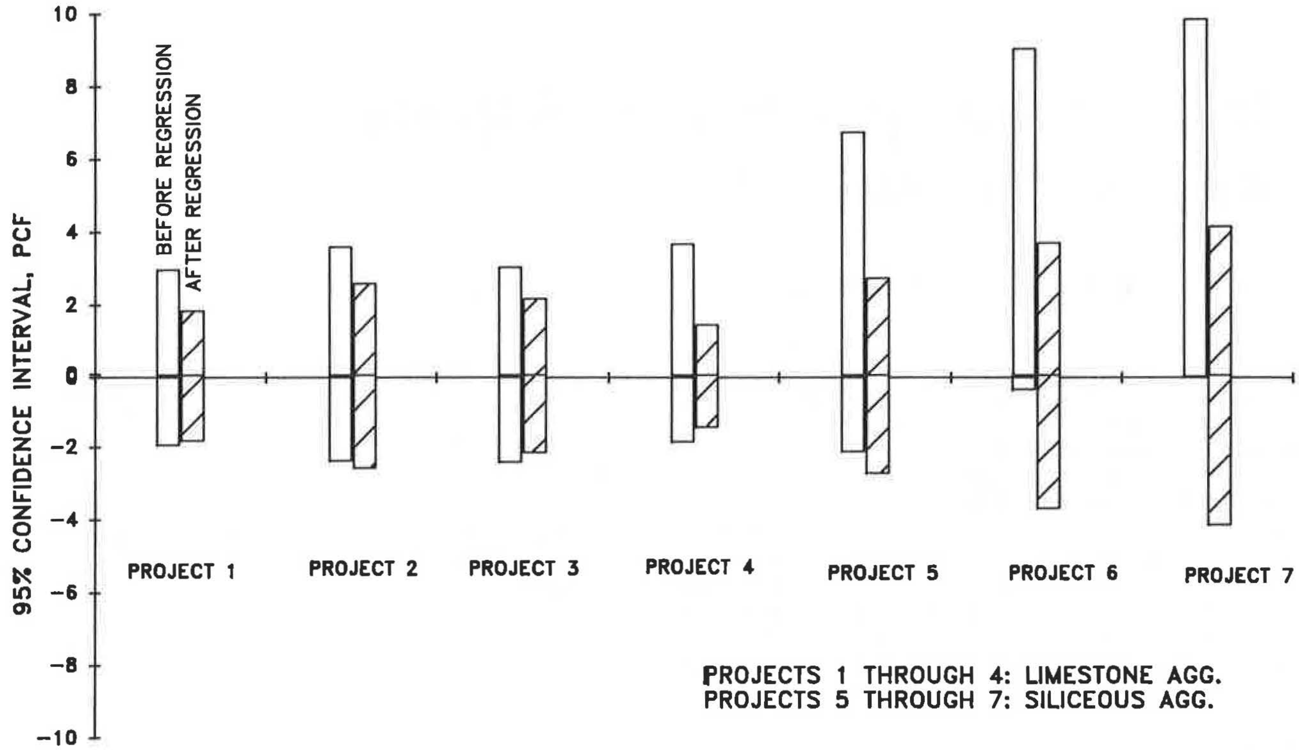

FIGURE 13 Ninety-five-percent confidence intervals for differences between core and nuclear densities before and after regression for different projects.

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Effect of Compaction on Asphalt Concrete Performance

ROBERT N. LINDEN, JOE P. MAHONEY, AND NEWTON C. JACKSON

This paper presents information on how compaction (specifically, air voids) influences the performance of dense asphalt concrete pavement surfaces. The information is based on three separate sources: the existing literature on the subject, a questionnaire survey of 48 state highway agencies on compaction practice, and performance data from the Washington State Pavement Management System. All three information sources show some correlation between the degree of compaction and the performance of asphalt concrete pavement. Overall, a 1 percent increase in air voids (over the base air-void level of 7 percent) tends to produce *about* a 10 percent loss in pavement life.

This paper is the result of research on how compaction—specifically, air voids—affects the performance of dense asphalt concrete pavement. Three information sources support the findings: a literature review; a survey of state highway agencies (SHAs); and data from the Washington State Pavement Management System (WSPMS).

PREVIOUS RESEARCH

Other researchers have found that asphalt concrete performance is in part a function of compaction, and hence air voids, in dense mixtures. Two frequently used terms indicative of performance are fatigue cracking and aging.

Fatigue Cracking

Fatigue cracking (or "alligator cracking") usually describes cracked pavement that has been repeatedly bent by heavy traffic. Researchers might reasonably ask how air voids affect asphalt concrete fatigue life.

Several authors (*1–3*) have shown that the fatigue life (time from original construction to substantial fatigue cracking) of asphalt concrete is reduced from 10 to 30 percent for each 1 percent increase over normal in air voids. For example, if 7 percent air voids are achievable for a mix yet 11 percent air voids resulted from construction, one could expect about a 40 percent reduction in pavement surfacing life (11 percent minus 7 percent equals 4, times 10 percent).

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Another way to approach air voids is to ask how they affect the thickness of asphalt concrete. As demonstrated by Finn and Epps (*1*), the effective thickness of asphalt concrete layers decreases as air voids increase. Finn and Epps evaluated two thicknesses of asphalt concrete, 4 and 6 in., at a starting point of 7 percent air voids (7 percent is generally considered achievable in normal paving construction). The following tabulation summarizes their findings.

Percent Air Voids in Asphalt Concrete	*Effective Thickness of Asphalt Concrete (in.)*	
	Example 1	*Example 2*
7	4.0	6.0
8	3.5	5.0
9	3.0	4.5
10	2.5	4.0
12	2.0	4.0

Thus, if the air voids are increased from a desirable level of 7 percent to a very poor compaction level of 12 percent, a 4-in.-thick asphalt concrete layer effectively lasts only as long as a 2-in. layer; a 6-in. layer is reduced to the effectiveness of a 4-in.-thick layer.

Aging

Aging of asphalt concrete can be evaluated in many ways. One way pertinent to this research is to judge aging by considering asphalt penetration and determining how air voids affect that property.

Goode and Owings (*4*) showed that, for asphalt concrete mixtures 4 years after construction, asphalt penetration is reduced about 6 percent for each 1 percent increase in air voids. They demonstrated, for example, that mixes retain about 75 percent of their original (immediately after construction) asphalt penetration at an air void level of 6 percent. If the air voids immediately after construction are 12 percent, however, the asphalt penetration is only 30 percent of its original level. The lower the asphalt penetration, the more susceptible the mixture is to cracking, as established by Hubbard and Gollomb (*5*), who showed that asphalt penetration of 30 or less (at 77°F) generally leads to distressed asphalt concrete. High-air-void pavements, as Goode and Owings (*4*) conclude, can be expected to result in low penetration levels for asphalt cement whose penetration level before mixing was at 85–100.

Air Void Levels

The work of the authors discussed above corroborates the findings of Finn et al. (*6*) concerning the maximum air void levels desirable for asphalt concrete construction. Those levels are summarized below.

Asphalt Concrete Layer	*Maximum Air Voids (%)* *Light Traffic*	*Moderate to Heavy Traffic*
Upper 1½–2 in. of asphalt concrete pavement	8	7
Asphalt concrete pavement deeper than 2 in.	7	6

Results of the SHA survey also support the levels established by Finn et al.

QUESTIONNAIRE

A questionnaire (*7*) was prepared and sent in 1987 to the materials engineer in each of the 50 SHAs. The questionnaire's purpose was to review practice and gather opinions of the SHAs on the control of air voids in asphalt concrete pavements. Each agency was asked to respond to eight questions on methods of mix design, construction compaction control and tests, field density limits and average asphalt content, pavement air voids, primary mode of pavement failure, and, finally, the effect of increasing air voids on pavement life. Forty-eight agencies responded to the questionnaire.

It should be noted that some questions are subjective; answers may be based on the respondent's opinion rather than on data.

A sample completed questionnaire is shown in Figure 1. Each question and a summary of agency responses follow. (Percentage figures for number of agencies responding have been rounded to the nearest whole percentage point.)

Question:

1. Asphalt concrete mix design procedure
   (a) Marshall method
   (b) Hveem method
   (c) Other (please specify)

Analysis:

Responses are summarized in the following table. Two agencies use both Marshall and Hveem methods. Other methods used are aggregate gradation and AASHTO T-167.

Method	*No. of Agencies Using Method*	*Percent of 48 Respondents*
Marshall	34	71
Hveem	10	21
Both	2	4
Other	2	4

Question:

2. Are field results used to verify the adequacy of asphalt concrete mix designs? Yes _____ No _____ . If yes, what types of field data are considered?

Analysis:

Yes—39 agencies (81 percent)
No—9 agencies (19 percent)
Agencies that responded yes typically use field results to verify air voids, aggregate gradation, asphalt content, and so on.

Question:

3. Construction compaction requirement
   (i) Percent of laboratory-compacted density
   (ii) Percent of theoretical maximum density (i.e., zero air voids)
   (iii) Percent of control strip density (e.g., target = 95% of control strip)
   (iv) Other (please specify)

Analysis:

(i) Percent of laboratory compacted density—18 agencies (38 percent)
(ii) Percent of theoretical maximum density (TMD) (AASHTO T209-82)—21 agencies (44 percent)
(iii) Percent of control strip density—6 agencies (12 percent)
(iv) Other—3 agencies (6 percent), which reported percent of field Marshall density

Question:

4. Construction compaction control tests
   (a) Nuclear densimeter
   (b) Core samples
   (c) Other (please specify)

Analysis:

Eighteen agencies indicated use of both nuclear gauge and core samples. The core samples are most likely used to calibrate the nuclear gauge when both methods are specified.
(a) Nuclear gauge — 18 agencies (38 percent)
(b) Core samples — 12 agencies (25 percent)
(c) Other — 18 agencies (38 percent), which reported using both methods

QUESTIONNAIRE ON EFFECT OF AIR VOIDS
ON ASPHALT CONCRETE PERFORMANCE

Please answer the following questions with respect to your agency's procedures or requirements for construction control of asphalt concrete pavement. In this questionnaire "air voids" are defined as the air spaces that occur between the coated aggregate particles in the final compacted asphalt concrete mixture.

1. Asphalt concrete mix design procedure
   (a) Marshall method ______
   (b) Hveem method ___X___
   (c) Other (please specify) ______

2. Are field results used to verify the adequacy of asphalt concrete mix designs? Yes ___X___ No ______. If yes, what types of field data are considered?

3 (a) Construction compaction requirement
   (i) Percent of laboratory compacted density ______
   (ii) Percent of theoretical maximum density ___X___ (i.e., zero air voids)
   (iii) Percent of control strip density ______ (e.g. target = 95% of control strip)
   (iv) Other (please specify) X-Roller passes for peak density (non main line)
  (b) Compaction specifications - Please include a copy of your standard specification for asphalt concrete compaction with the questionnaire.

4. Construction compaction control tests
   (a) Nuclear densimeter ___X___
   (b) Core samples ______
   (c) Other (please specify) ______

5. Construction compaction control criteria
   (a) Does the Agency have a maximum field density limit?
   Yes ______ No ___X___
   If yes, what is the limit (percent) ______%
   (b) What do you normally do if a contractor exceeds your maximum compaction requirements?

   (c) Does the Agency have a minimum field density limit?
   Yes ___X___ No ______
   If yes, what is the limit? (percent) ___92___% of Rice Density
   (d) What do you normally do if a contractor does not meet your minimum compaction requirements?
   Price Adjustment

   (e) Has the Agency recently changed or is it considering a change in its compaction requirements?
   Yes ______ No ___X___ Year ______

FIGURE 1 Sample completed questionnaire.

(f) At what minimum compacted course thickness does the agency require compaction control? 1-1/4 (inches)

6. What is the average asphalt cement content you use in your normal asphalt concrete surfacing mixes (% by weight of total mix)? 5.5

7. What is the range and average of field air voids in pavements constructed in the past five years?
Maximum 10
Minimum 5
Average 7

8. (a) What is the typical "life" of an asphalt concrete surfacing in your state? 12-1/2 years. ("Life" is defined as the time between construction and the time when the next overlay or rehabilitation is needed.)
(b) What is the principal mode of failure at the end of an asphalt concrete surfacing life (i.e. fatigue cracking, rutting, etc.) fatigue
(c) What is your experience or opinion of the effect of field air voids on asphalt pavement life?
Directly affects rutting, raveling and fatigue cracking
(d) In your opinion, what is the effect of increasing air voids on asphalt pavement life, expressed as a percentage of design life, for the following field (as constructed) air void contents: 4% 100% 5% 100%
6% 100% 7% 100% 8% 90% 9% 80%
10% 70% 11% 60% 12% 50%
(Normally, 4 to 6% air voids would constitute 100% of design life)

9. Please make any additional comments you wish to add on this subject.

The results of this questionnaire will be summarized.. Please indicate if you wish to receive a copy of this summary.
Yes X No ____

Person answering this questionnaire:

Name ____
Title ____
Agency ____
Address ____
Telephone ( ) ____

FIGURE 1 *continued*

Question:

5. (a) Construction compaction control criteria
Does the Agency have a maximum field density limit?
Yes ____ No ____ If yes, what is the limit (percent) ____

Analysis:

Yes — 12 (25 percent)
No — 36 (75 percent)
The following table shows the limits for maximum field density specified by the 12 agencies that responded yes.

Specification Limit	*No. of Agencies*
96% of TMD	2
97% of TMD	6
98% of TMD	1
101% of Marshall	1
102% of control strip	1
105% of control strip	1

Question:

5. (b) What do you normally do if a contractor exceeds your maximum compaction requirements?

Analysis:

The 12 agencies that reported taking action did the following if the maximum density limit was exceeded.

Action	*No. of Agencies*
Price adjustment	2
Price adjustment or remove and replace	2
Penalty system	2
Removal if severe	1
No incentive payment	1
Adjust job-mix formula	1
Adjust rolling procedure	1
New control strip	1
No answer	1

Question:

5. (c) Does the Agency have a minimum field density limit?
Yes ____ No ____ If yes, what is the limit (percent) ____

Analysis:

All 48 responding agencies have a minimum density limit, summarized as follows:

Specification Limit	*No. of Agencies*
90% of TMD	1
91% of TMD	1
92% of TMD	12
92.5% of TMD	2
93% of TMD	5
93% of Marshall density	1
95% of Marshall density	9
96% of Marshall density	2
97% of Marshall density	1
95% of field Marshall density	2
98% of field Marshall density	1
95% of control strip density	1
97.5% of control strip density	1
98% of control strip density	4
95% of Hveem density	1
95% of other lab density	1
98% of other lab density	1
No answer	2

Question:

5. (d) What do you normally do if a contractor does not meet your minimum compaction requirements?

Analysis:

The agencies reported the following actions when the minimum density limit was not achieved.

Action	*No. of Agencies*
Price adjustment	17
Price adjustment or remove and replace	13
Penalty system	5
Reevaluate compaction procedure or mix design	3
Require additional compaction	5
Reject below 92% of TMD	1
Assess liquidated damages	1
No answer	3

Question:

5. (e) Has the Agency recently changed or is it considering a change in its compaction requirements? Yes ____
No ____ Year ____

Analysis:

Yes—23 agencies (48 percent)
No—25 agencies (52 percent)

Question:

5. (f) At what minimum compacted course thickness does the Agency require compaction control? ______(inches)

Analysis:

The 48 responses are summarized as follows:

Minimum Thickness (in.)	*No. of Agencies*
¾	5
1	13
1⅛	1
1¼	5
1½	10
2	2
No minimum	8
No answer	4

Question:

6. What is the average asphalt cement content you use in your normal asphalt concrete surfacing mixes (percent by weight of total mix)?

Analysis:

The range of asphalt content was 4.6 to 6.7 percent, with an average of 5.7 percent. While the spread of asphalt content is 2.1 percent, it is noteworthy that 25 agencies (52 percent) reported an average asphalt content between 5.5 percent and 6.0 percent. Within this band, about as many of the SHAs use the Marshall method as use the Hveem method to determine optimum asphalt content.

Percent by Weight of Total Mix	*No. of Agencies*
4.6	1
5.0	4
5.1	2
5.2	1
5.3	3
5.4	1
5.5	6
5.6	4
5.7	3
5.8	3
6.0	9
6.1	2
6.2	2
6.3	1
6.4	3
6.5	2
6.7	1

Question:

7. What is the range and average of field air voids in pavements constructed in the past 5 years?
Maximum _____ Minimum _____ Average _____

Analysis:

This question is somewhat subjective, and it was not uniformly interpreted. Some agencies listed the range of field air voids and others listed only the average. Seven agencies did not answer the question at all for a variety of reasons: "Data unknown"; "Don't know without a lot of analysis"; "Do not test for air void content in the field"; and "Figures are not available."

The averages and ranges that were reported are shown below.

	Pavement Air Void Content (%)	
	Average	*Range*
Maximum	9.9	5–15
Minimum	3.5	1–6
Average	6.5	2.8–10

Question:

8. (a) What is the typical "life" of an asphalt concrete surfacing in your state? _____ years. ("Life" is defined as the time between construction and the time when the next overlay or rehabilitation is needed.)

Analysis:

Of the 46 responding agencies, 34 (74 percent) reported an average pavement life of 10 to 15 years. Six agencies reported longer life and six reported shorter life. The overall average for all responding agencies was 12.5 years. Naturally, this question is a difficult one in that most SHAs have only an approximate idea of the "typical" life of asphalt concrete.

Average Pavement Life (years)	*No. of Agencies*
7	1
8	2
9	3
10	10
10.5	1
11	1
12	6
12.5	5
13	1
13.5	3
15	7
17.5	3
20	2
21	1

Question:

8. (b) What is the principal mode of failure at the end of an asphalt concrete surfacing life (i.e., fatigue cracking, rutting, etc.)?

Analysis:

In responding to this question, some agencies reported more than just the principal mode of failure. The several causes of failure mentioned are summarized below, along with the number of agencies reporting them.

Mode	*No. of Agencies*
Fatigue cracking	20
Rutting	14
Cracking (non-specific)	12
Thermal cracking	6
Stripping	5
Weathering	4
Raveling	3
Reflective cracking	2
Base failure	2
Shrinkage cracking	1
Wear	1
Variable modes	5
No response	1

Question:

8. (c) What is your experience or opinion of the effect of field air voids on asphalt pavement life?

Analysis:

Forty-six agencies responded to this question. Comments are grouped in three categories.

Air Void Significance

All 46 respondents said that air voids play a significant role in the performance and life of asphalt concrete pavement. Fourteen (30 percent) described the role as

- Critical [to have an acceptable range]
- Significantly influencing life of asphalt pavement
- Playing a tremendous part in performance
- Very critical (four agencies used this description)
- All-important
- Very important
- Having a dramatic effect
- The most important item relative to life
- The single most important property affecting durability
- One of the most important criteria
- Extremely important

Minimum Air Voids

Twenty of the respondents (44 percent) commented that too few air voids cause a reduction in pavement life due to rutting, shoving, and bleeding. Eight of the 20 respondents indicated a minimum field-air-void content to avoid this distress. The specified minimum air-void level and the number of agencies reporting that level are listed below.

1–2 percent—1 agency
2 percent—2 agencies
3 percent—3 agencies
4 percent—2 agencies

One agency commented that low air voids in the surface mix are more likely than raveling to cause pavements to fail.

Maximum Air Voids

Of the 46 respondents, 44 indicated that increasing or excessive air voids adversely affect pavement performance and life. Opinions ranged widely, however, on the level of air voids at which performance and life begin to be affected. Fourteen agencies (30 percent) reported the following levels.

3 percent—1 agency
4 percent—1 agency
5 percent—1 agency
6 percent—5 agencies
7 percent—1 agency
8 percent—3 agencies
10 percent—1 agency
11 percent—1 agency

Question:

8. (d) In your opinion, what is the effect of increasing air voids on asphalt pavement life, expressed as a percentage of design life, for the following field (as constructed) air void contents: 4%____ 5%____ 6%____ 7%____ 8%____ 9%____ 10%____ 11%____ 12%____
(Normally, 4 to 6 percent air voids would constitute 100 percent of design life.)

Analysis:

Twenty-eight respondents (only 58 percent of the whole sample) addressed this question, the most subjective one in the questionnaire. The opinions of those agencies that did respond varied widely, but suggest that air void levels above about 6 percent will decrease asphalt concrete life by about 7 percent for each 1 percent increase in air voids.

	Percent of Design Life	
Air Void Content (%)	*Range*	*Average*
4	20–120	97
5	30–120	97
6	70–120	98
7	50–100	93
8	40–100	87
9	30–100	79
10	20–100	73
11	10–95	62
12	0–90	54

AIR VOID EFFECTS ON PAVEMENTS IN WASHINGTON STATE

In Washington State, data from WSPMS support the results of the questionnaire survey. The Washington State Department of Transportation (WSDOT) has conducted visual condition surveys of the entire state system every 2 years since

TABLE 1 SUMMARY OF CONTRACT DATA

State Route	Contract No.	Construction Year	Life to PCR = 40 (years)	Percent Loss
2	8602	1970	7.0	42
82	8672	1971	8.5	29
395	8004	1968	5.0	58
			6.8 (avg)	43 (avg)

1969. Specific performance curves are developed from this data for over 2,600 project-length segments of the complete 7,000-mi centerline system. These segments usually match with past paving projects which represent areas of unique and consistent performance. The survey-and-distress deduct values are weighted toward fatigue cracking. The Pavement Management System thus tracks over time the general fatigue performance of all past paving projects for the entire state.

In addition, the WSDOT conducts a reasonably detailed resurfacing investigation on all resurfacing or reconstruction projects as part of the normal design process. This investigation, which examines the performance of well over 100 projects a year, demonstrates that a combination of factors contributes to the particular performance of each project. Almost never can shortened service life be attributed to a single cause. But of the several factors that can cause reduced performance, air voids is consistently one of the most significant, probably because it is so difficult to achieve the optimal air void levels during construction. The effects of high air voids are well documented in project files, but the data have not been rigorously culled to show clear cause and effect.

The three projects discussed below are typical of those in which high-air-void contents in the wearing and leveling courses caused early fatigue failure. The pavements in eastern Washington were originally constructed in the late 1960s and early 1970s, and the wearing and/or leveling courses were hot-recycled in the early 1980s to correct the problems associated with early fatigue failure. Eastern Washington has a dry, freeze environment, which is more prone to performance problems associated with void content and moisture sensitivity than is a wet, no-freeze environment, such as that in western Washington. In all three cases, the void content of the original pavements was in the 11- to 12-percent range for the wearing or leveling courses. (When the pavements were put down, there was little construction control of air voids and plant production increases were not marked by compaction equipment increases behind the paver.) All three projects were built to similar specifications, shown below.

1.8 in.	Class B ACP (wearing course)
1.2 in.	Class B ACP (leveling course)
4.2 in.	Class E ACP (base course)
3.0 in.	Crushed surfacing top course (unstabilized)
6.0 in.	Gravel base (unstabilized)
16.2 in.	Total

The Class B and Class E ACP are normal, dense-graded asphalt concrete pavement mixes. Class B's maximum size aggregate is ⅝ in. and Class E's is 1¼ in.

A summary of the pertinent data for each contract is shown in Table 1. The first three columns are self-explanatory. The fourth column shows the time it took for each project to reach a pavement condition rating (PCR) of 40, which represents about 10 percent fatigue cracking, the level at which rehabilitation projects are programmed in the WSPMS. The percent-loss column represents the percent reduction in service life (to PCR = 40), as compared to 12½ years, the average service life of such pavement in Washington State.

For all three projects, fatigue cracking was confined to the wearing and/or leveling courses. No fatigue cracking was found in the Class E base course, which had air void contents in the 6- to 9-percent range. The lower air-void contents in Class E is likely explained by its greater lift thicknesses, which retain heat longer and permit more compaction. To account for greater thickness, contractors often provided more or heavier compaction equipment for the base lift. During the recycling mix design process, the asphalt recovered from the wearing and leveling courses was found to be quite stiff. Tests on the recovered asphalt showed penetrations in the range of 7–16 and 140°F absolute viscosities of 50,000 to 250,000 poise. The asphalt used in the earliest project was 85–100 penetration grade, while the asphalt from later projects was AR 4000w, which registers a penetration range of about 100–115. Personal observation suggested that the effects of the high air voids increased the rate of hardening of the asphalt cement binder and decreased the fatigue resistance of the pavement. State Route 82 experienced the best service life and evidenced the least hardening of the binder, but this project was sand-sealed shortly after construction to reduce the effects of the high air voids on pavement performance.

The three pavements have been in service now for 4 to 6 years after recycling with a more normal void content; the latest condition survey, conducted in the spring of 1988, revealed no signs of any deterioration.

The effects of air voids on pavement life were also studied but not reported in a small FHWA-funded activity on pavement performance equations (*8*). The 70 projects studied, equally divided between eastern and western Washington, were initially selected based without regard to void content. Enough data on air void content were gathered, however, to allow analysis of the effects of air voids on pavement life (PCR = 40).

Void Content (%)	*Loss in Service Life (%)*
7	0
8	2
9	6
10	17
12	36

The data are based on only a small sample (less than 5 percent of the state system) and are perhaps somewhat skewed because western Washington's climate is mild, which ameliorates the detrimental effects of high air voids. Nonetheless,

TABLE 2 EFFECT OF COMPACTION ON PAVEMENT PERFORMANCE

	Pavement Life Reduction (%)		
Air Voids (%)	Literature[a]	SHA Survey[b]	WSPMS
7	0	7	0
8	10	13	2
9	20	21	6
10	30	27	17
11	40	38	—
12	50	46	36

[a]Lower bound of range.
[b]Average.

the data confirm the findings of the previously described SHA survey.

FINDINGS

Table 2 summarizes the findings on the effect of compaction (specifically, air voids) on asphalt concrete pavement performance.

All three sources of information confirm that air void content affects pavement performance. The rule-of-thumb that emerges is that each 1 percent increase in air voids (over a base air void level of 7 percent) results in about a 10 percent loss in pavement life (or about 1 year less).

Of course, the performance of asphalt concrete is affected by more than just air void content. Although the primary goal of the SHA survey was to estimate the effects of air voids, the summary of the responses necessarily addresses current mix design procedures, compaction specifications, and field practice, all of which play a part in performance outcome.

CONCLUSIONS

Results of an examination of the literature, a survey on compaction practice, and performance data from WSPMS lead to the following conclusions.

1. An analysis of the available literature and WSPMS data suggest that both fatigue cracking and age-hardening of dense asphalt concrete mixtures is exacerbated by poor compaction (high air voids). Further, pavement life is reduced about 10 percent for each 1 percent increase in air voids over a base level of 7 percent. The SHA survey on compaction practice tended to confirm this finding.
2. The survey of the SHAs reveals that compaction specifications, field practice, and asphalt concrete performance vary widely from state to state. Although some questions in the survey were subjective, the responses still tend to show a definite correlation between compaction and pavement life.

ACKNOWLEDGMENT

The authors greatly appreciate the response to the survey from the state departments of transportation.

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Effects of Field Control of Filler Contents and Compaction on Asphalt Mix Properties

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The objective of this study is to determine the effects on asphalt mixes of inadequate field supervision over filler content and compaction. Inadequate field control frequently leads to asphalt mixes with more fines in the mix production stage and less compaction than is desirable. An extensive laboratory program was executed in which these effects were studied. Varying the levels of compactive effort and filler content produced appreciable effects on the properties of asphalt mixes, but the effects were not consistent with the degree of variance. Voids are affected more by changes in the filler content than by compactive effort. The variation of modulus of resilience with changes in void content was established and suggests a definite relationship. Finally, a rational mix design procedure is suggested based on the results of the laboratory analysis.

Pavement structures in Saudi Arabia are beset by a multitude of severe environmental conditions, such as high temperature and humidity. Compounding the environmental factors are problems related to mix design and construction techniques. Due to the nature of fragile aggregates in most areas of the Kingdom, especially in the central and eastern regions, asphalt pavements usually contain larger percentages of fine materials than originally planned. Local aggregates in general expand when soaked, leading to fracture of the aggregate, which means an increase in the fines content of the mix. The resulting increase in fines content is believed to affect field compaction results. One major step in ensuring better performance of roads starts with better understanding of the effects of these mix and field variables.

Kallas et al. (*1*) found that by changing the concentration of filler up to a certain value, stability increases and optimum asphalt content decreases. As filler is added beyond that certain point, however, stability decreases. Tunnicliff (*2*), too, has demonstrated that as filler concentration increases, internal stability increases.

Kallas and Puzinauskas (*3*) found that sensitivity of mixes can be decreased by keeping the ratio of filler to asphalt at a sufficiently low level. This can be achieved either by increasing the asphalt content or by decreasing the filler concentration in the mixture, within limits that allow satisfactory air voids, stabilities, and flow values.

The interaction between compactive effort and filler content is not well established. The precise correlation between binder viscosity and the compactive effort required to densify a paving mixture needs to be found. Some research shows, however, that fillers increase the amount of effort necessary for compacting specimens to the same volume or air void content (*4*). In other research, Bissada (*5*) concluded that resistance to compaction is significantly affected by mix variables such as filler content.

The objectives of this study are as follows:

- To study the effects of field variations in filler content and degree of compaction on Marshall mix design criteria.
- To establish the effect of those variations on mechanistic properties, such as modulus of resilience.
- To suggest a rational mix design procedure that takes into account those effects.

MATERIAL CHARACTERIZATION

The aggregate used in this research was brought from Al-Mahdiyah, which is approximately 30 km west of Riyadh. This aggregate is widely used in the Riyadh area for asphalt concrete mixes. The other material used, asphalt, was obtained from Petromin Riyadh Refinery.

Aggregate

Several tests were conducted to characterize the aggregates to ascertain their conformity with both ASTM and AASHTO specifications, as applicable. Table 1 gives the results of quality tests performed on the aggregates.

Asphalt

The results of quality tests performed on the asphalt from Riyadh Refinery are given in Table 2.

DESIGN OF EXPERIMENT

The steps outlined in Figure 1 describe the procedure for determining how varying filler content and compactive effort affects the properties and performance of asphalt mixes.

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TABLE 1 RESULTS OF QUALITY TESTS ON AGGREGATES

Test	Designation of Method	Result	Specification
Sand equivalent (%)	AASHTO T-176	59.0	Min 45
Plasticity index	T-90	Nonplastic	Max 3
Clay lumps and friable particles (%)	T-112	0.00	Max 1
Los Angeles abrasion (%)	T-96		
Grade B		25.2	
Grade C		25.5	
Grade D		25.3	Max 40
Specific gravity of combined aggregate	T-84		
Bulk specific gravity over dry	T-85	2.540	
Apparent specific gravity		2.681	
Absorption (%)		2.314	
Specific gravity of filler material	C-854	2.754	
Crushed aggregate (%)			
Particles passing No. 4 retained on No. 8	MOC[a]	94	Min 85
Particles retained on No. 4	MOC[a]	97	Min 90
Thin and elongated aggregate (%)	MOR[b]	2	Max 10
Soundness of aggregate sodium sulfate (%)	T-104		
Fine aggregate		4.9	Max 10
Coarse aggregate		8.4	

[a]Ministry of Communications, Saudi Arabia.
[b]Municipality of Riyadh, Saudi Arabia.

TABLE 2 RESULTS OF QUALITY TESTS ON ASPHALT

Test	Designation of Method	Result	MOC[a] Specification
Penetration at 25°C, 100 gm, 5 sec	D5-78	68.0	60–70
Viscosity at 135°C kinematic (Cst)	D2170-81	383.0	
Flash point (°C)	ASTM D92-78	336.0	232.2 min
Ductility at 25°C	D13-79	100.0+	100.0 min
Thin-film oven test	T-179		
Penetration, percent of original	D5-78	60.3	52.0 min
Percent solubility in organic solvents	D2042-81	100.0	99.5 min
Specific gravity	D70-76	1.031	

[a]Ministry of Communications, Saudi Arabia.

The initial design set 4.8 percent as the optimum asphalt content, with 7 percent filler content. (In this study, filler is defined as limestone dust passing the No. 200 sieve.) Mixes were then prepared with filler contents ranging between 0 and 12 percent, which represents the usual range of filler content in locally produced asphalt mixes. The change in the filler content from the optimum mix prepared with 7 percent filler was offset by proportionally decreasing or increasing the other aggregates. The asphalt content was not changed because the optimum asphalt content is dictated by the design; it is the amount of filler that varies, for the reasons explained earlier.

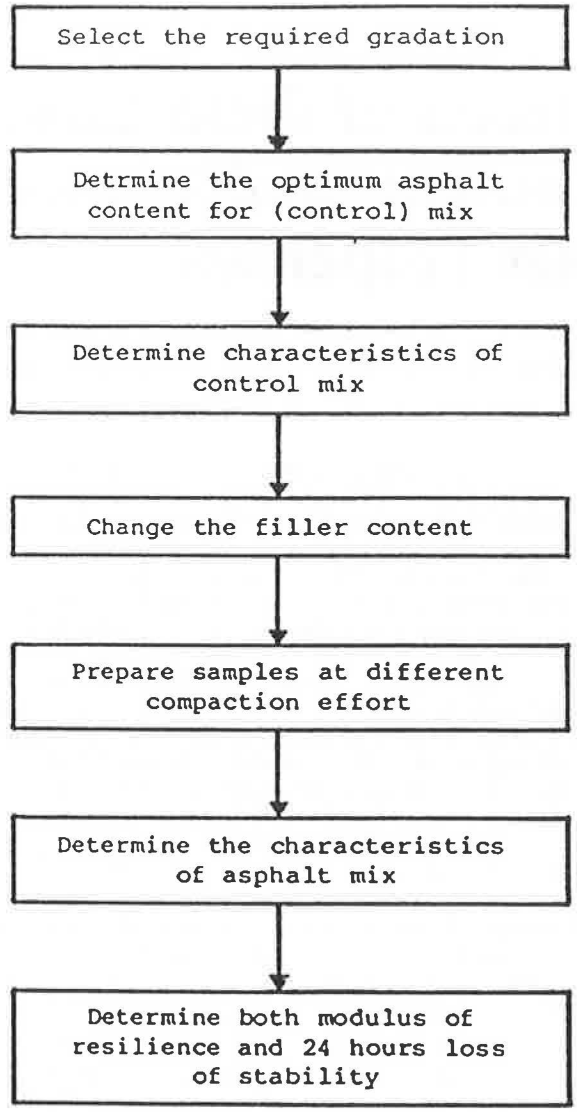

FIGURE 1 Steps for determining sensitivity of asphalt mixes to changes in filler content and compactive effort.

Table 3 shows the gradation of mixes used for various filler contents. These mixes were all prepared using 75 blows. In order to assess the effect of compactive effort, the procedure was repeated, but this time the mixes were prepared using between 30 and 90 blows. All mixes were prepared at a mixing temperature of 300°F and a compaction temperature of 280°F. Figure 2 gives the matrix of tests performed.

EFFECTS OF CONSTRUCTION VARIATION ON MARSHALL MIX DESIGN CRITERIA

Several tests were conducted to determine the effect of construction variations represented by changes in both filler content and compactive effort on mix design variables. The aim of the tests was to find possible combinations of variations that could satisfy design requirements.

TABLE 3 AGGREGATE GRADATIONS USED FOR VARIOUS FILLER CONTENTS

	Percent Passing			
Sieve No.	JMF for 0.0% Filler	JMF for 3.0% Filler	JMF for 7.0% Filler	JMF for 12.0% Filler
¾	100.0	100.0	100.0	100.0
½	89.2	89.6	90.0	90.5
⅜	79.5	80.2	81.0	82.0
4	56.9	58.3	60.0	62.1
10	34.9	36.9	39.5	42.7
40	15.0	17.6	21.6	25.2
80	7.5	10.3	14.0	18.6
200	0.0	3.0	7.0	12.0

No. of blows	% of filler content 0	3	7	12
30	X	X	X	X
45	X	X	X	X
60	X	X	X	X
75	X	X	X	X
90	X	X	X	X

(X) Four samples were prepared for each cell

FIGURE 2 Matrix of tests for determining effects of filler contents and compactive efforts.

Effect on Mix Stability

Figure 3 shows Marshall stability values for varying filler contents and at various compactive efforts. At the lowest compactive effort (30 blows), stability increases almost linearly with the increase in the percentage of filler used. However, for compactive efforts of 45 and 60 blows, stability increases up to a point as the filler increases, and then decreases. At higher compactive efforts (75 blows and above), stability generally continues to increase with the increase in the filler content of the mix.

As the filler-to-asphalt ratio increases, the viscosity of the mortar also increases, requiring more compaction energy to produce a uniformly compacted mix. At a very high compactive effort, the increased viscosity's resistance to compaction can be overcome. At lesser compactive effort, a mix with a high filler-to-asphalt ratio cannot be densely compacted and it will consequently possess a lower stability value.

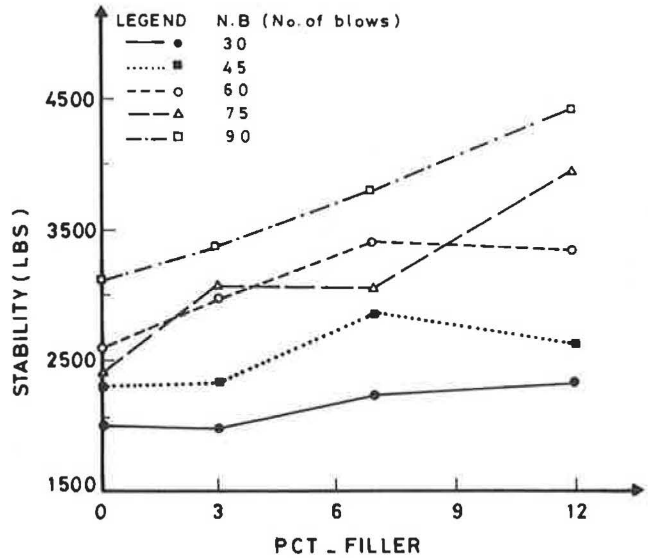

FIGURE 3 Effect of percentage of filler in the mix on Marshall stability.

Effect on Mix Voids

Changing both the filler content and the compactive effort affects the amount of air voids in the asphalt mix. As Figure 4 indicates, air voids in the mix are more sensitive to changes in the percentage of filler at higher compactive efforts (60 blows and above). It also appears that, for all compactive levels, raising the percentage of filler content yields a smaller percentage of air voids up to an optimum percentage of filler. Filler contents higher than this optimum tend to lead to an increased percentage of air voids as the mix becomes more resistant to compaction.

It is important to establish how air voids in the mix affect other mix properties such as bulk specific gravity and stability.

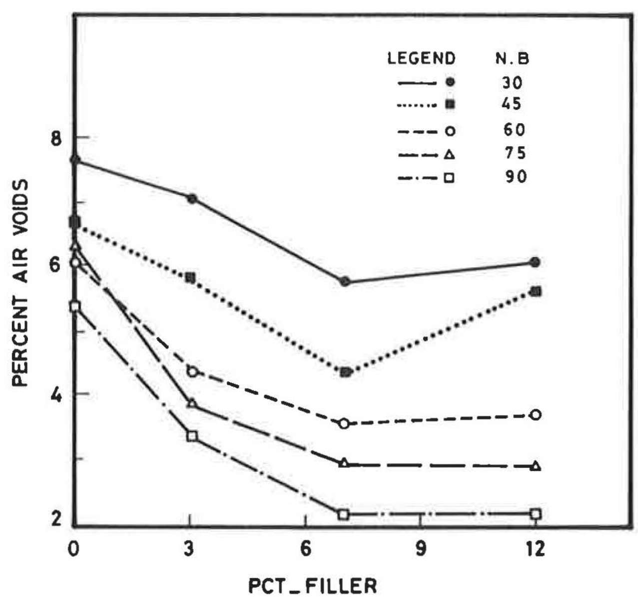

FIGURE 4 Effect of increasing percentage of filler on air voids for various compactive efforts.

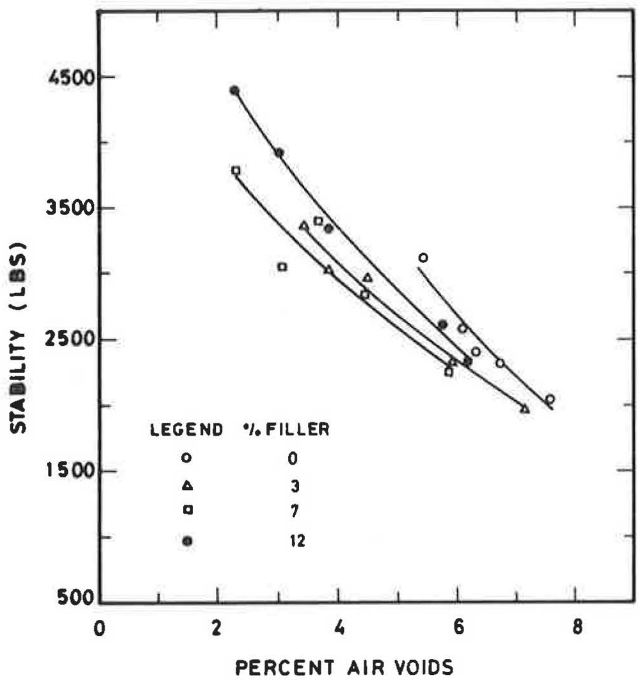

FIGURE 5 Variation in mix stability with air voids for various filler contents.

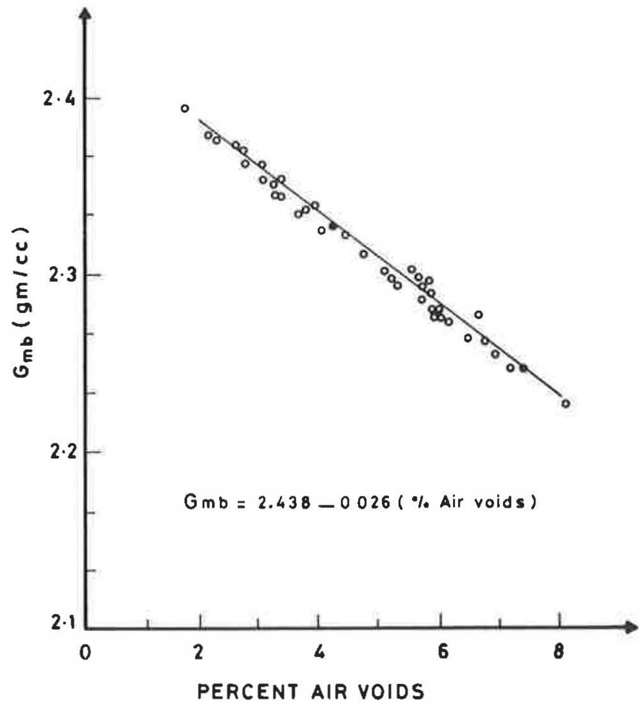

FIGURE 6 Relationship between mix air voids and bulk specific gravity.

The voids for each test, which corresponded to either a specified filler content or the compactive effort, were therefore plotted against the resulting Marshall stability and specific gravity; the results are shown in Figures 5 and 6, respectively.

Figure 5 shows that for all mixes, with different filler contents, stability decreases as the percentage of air voids increases. The effect of filler content is more pronounced at lower air voids (that is, at higher compactive efforts). Also notable is that for a selected stability value, e.g., 3,000 lb, an increase in filler content from 0 to 7 percent is associated with a reduction in air voids from about 5.5 percent to about 3.9 percent. However, any further increase in filler content results in higher air voids (about 4.8 percent) at the selected stability value. This can be explained by the sharp increase in the viscosity of the filler/asphalt mortar when the filler content exceeds 7 percent (filler-to-asphalt ratio of 1.46), which was found with a sliding plate microviscometer to vary as follows:

Filler-to-Asphalt Ratio	*Absolute Viscosity at 25°C (poises × 10^6)*
0.00	1.31
0.63	2.98
1.46	10.32
2.50	26.21

The increase in stability caused by the sharp increase in the stiffness of the mortar apparently more than counteracts the reduction in stability associated with the increase in air voids resulting from higher resistance to compaction.

Similarly, Figure 6 shows that the bulk specific gravity of these mixes decreases when air voids increase, no matter what causes the void increase. The maximum theoretical specific gravity corresponding to a voidless mix is 2.438.

Effects on Other Mix Properties

The effects of compactive effort and percentage of filler on other mix design properties (voids in the mineral aggregate (VMA), voids filled with bitumen (VFB), and flow) are shown in Figure 7. The variation in VMA is, as expected, similar to that shown earlier in Figure 4 for air voids because asphalt cement content was the same for all mixes.

The percentage of VFB, which is used as a mix design criterion in Saudi Arabia, was calculated as follows:

$$VFB\ (\%) = \frac{VMA - \text{air voids}}{VMA} \times 100$$

Figure 7(b) shows VFB variation with changes in percentage of filler and compactive effort.

Figure 7(c) shows that reduction in filler content beyond 7 percent—the level at which optimum asphalt content was determined—results in a larger decrease in flow than when filler content rises above the 7 percent value. However, the relationship between the rate of reduction in flow and the compactive effort cannot be well defined. In general, test results were erratic and the precise effect of both variables on flow could not be established.

Effect on Loss of Marshall Stability

The effect of field control procedures on loss of Marshall stability holds pronounced local interest because existing asphalt pavements exhibit poor water resistance. Figure 8 shows the percent loss of Marshall stability after specimens were soaked

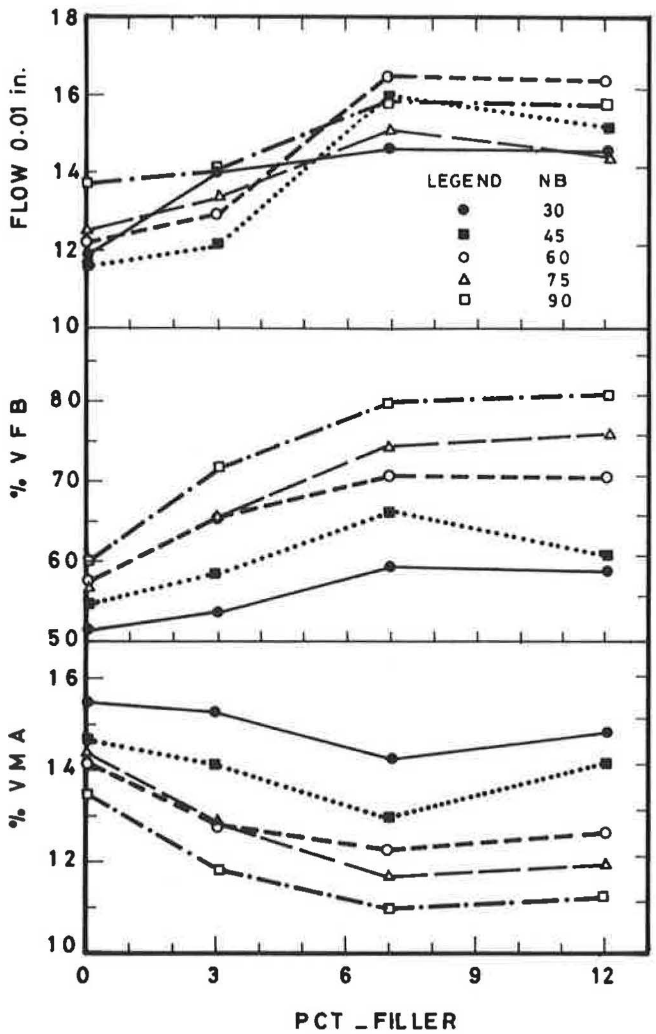

FIGURE 7 Effect of percentage of filler on percent VMA, percent VFB, and flow for various compactive efforts.

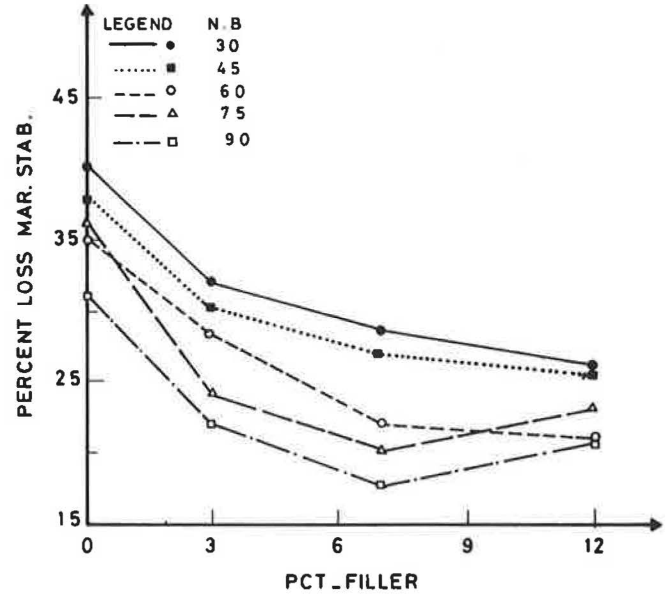

FIGURE 8 Effect of increasing percentage of filler content on loss of Marshall stability for various compactive efforts.

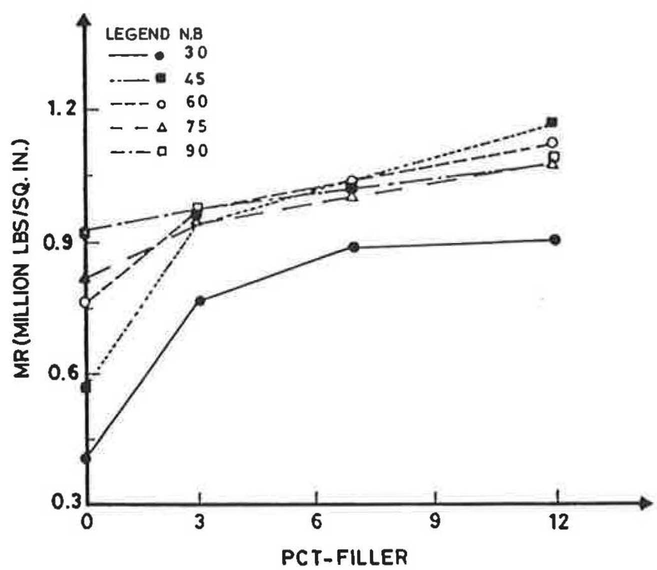

FIGURE 9 Effect of increasing percentage of filler content on modulus of resilience for various compactive efforts.

in water for 24 hr at 60°C. Varying filler content affects the percent loss of Marshall stability according to the compactive effort applied. At lower compactive efforts (30, 45, and 60 blows), any increase in filler content (within the range studied) causes a reduction in the percent loss of Marshall stability. The rate of the reduction decreases with the increase in filler content. At higher compactive efforts (75 and 90 blows), however, percent loss of Marshall stability decreases only until the optimum filler content (i.e., 7 percent) is reached.

EFFECTS OF CONSTRUCTION VARIABLES ON MECHANISTIC PROPERTIES

In order to study the effects of field control on the mechanistic response of the mix [i.e., the modulus of resilience (M_R)], tests were carried out on samples prepared according to the matrix in Figure 2. As Figure 9 shows, test results indicate that M_R values are not sensitive to variations in filler content at high compactive efforts (60, 75, and 90 blows). The results also indicate that the reduction in M_R values resulting from a drop in filler content can be offset by increasing the compactive effort.

The effect of air voids in the mix on the M_R was also established. Figure 10 shows test results, which indicate that M_R drops drastically at air void contents around 6–8 percent. Test results also indicate that, for mixes with 3 percent or more filler content, the reduction in air voids achieved by higher compactive efforts has relatively little effect on M_R values where the variation of M_R values is within 10–20 percent for the different levels of compactive effort used. For mixes with 0 percent filler content, however, the variation in M_R values is greater (above 50 percent), pointing to the sensitivity of

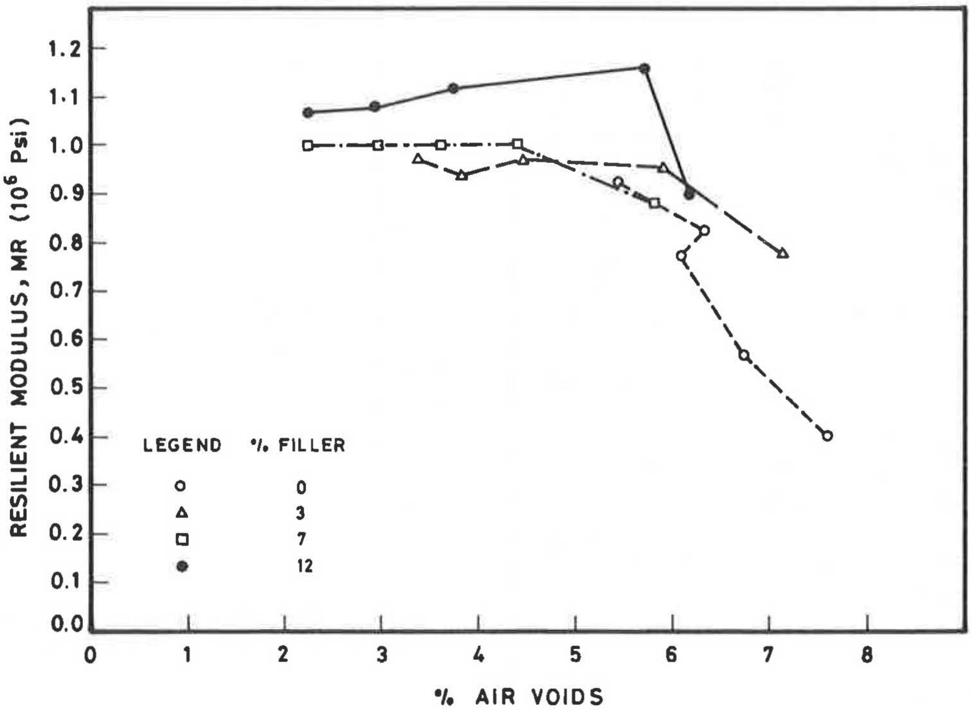

FIGURE 10 Variation of resilient modulus with air voids for various filler contents.

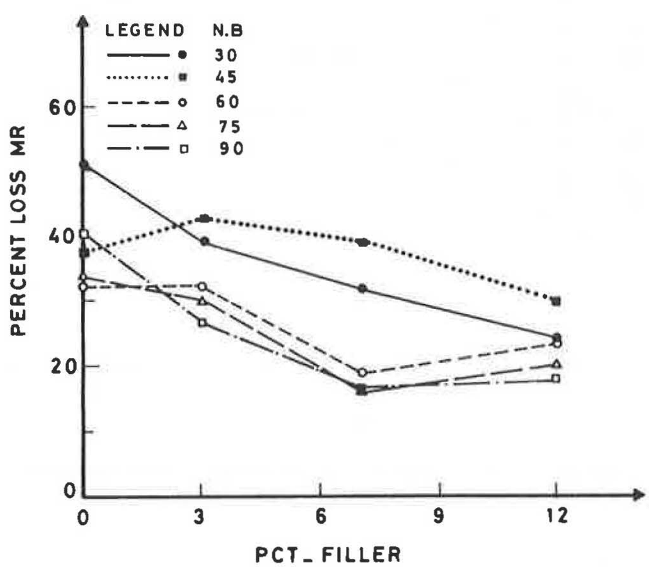

FIGURE 11 Effect of increasing percentage of filler content on M_R loss for various compactive efforts.

the mixes to air void variation. Because mixes with low filler content are associated with high air voids, it is especially important that the compactive effort for such mixes be strictly controlled.

The effect of variations in filler content and compactive effort on the moisture susceptibility of the mixes was also evaluated using the M_R test. Specimens were immersed in water for 24 hr at 60°C and M_R values for those specimens were then determined at 25°C. The percentage loss of M_R caused by immersion was established; test results are in Figure 11. The percentage loss in M_R values generally decreases as the percentage of filler in the mix increases. When filler content is increased above 7 percent, resistance to moisture damage is improved only for mixes with lower compactive efforts (30 and 45 blows), probably because these mixes have high air void values.

RATIONAL MIX DESIGN

The aim of the proposed mix design is to determine optimum asphalt content of the mix by taking into account the effects of filler content and compactive effort in the process. In addition, more criteria are specified in order to arrive at the optimum asphalt content. Table 4 shows the properties and values that went into this analysis. Besides the criteria in Table 4, a minimum VMA value of 14 percent was selected based on an aggregate nominal maximum particle size of ¾ in.

The mix design draws on the various relationships estab-

TABLE 4 SPECIFICATIONS FOR DETERMINING THE OPTIMUM ASPHALT CONTENT

Mix Property	Specification
Stability (lb)	Min 1,550
Flow (0.01 in.)	9.4–15.7
Percent loss of stability	Max 25
Percent air voids (AV)	3–5
Percent voids filled with bitumen (VFB)	70–80

NOTE: Based on specifications for the Ministry of Communications, Saudi Arabia.

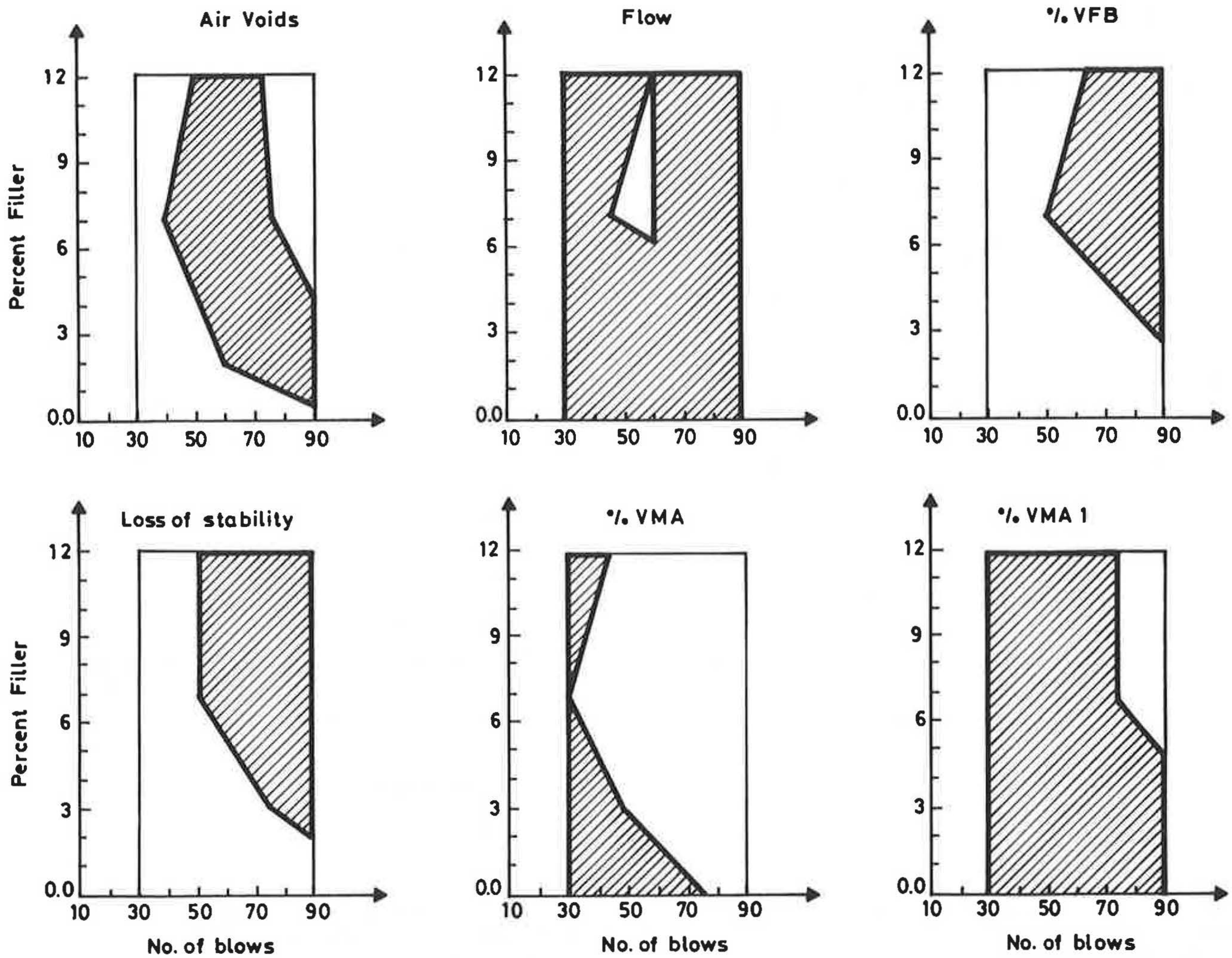

FIGURE 12 Ranges of compactive effort and filler contents that satisfy mix design criteria.

lished earlier between mix properties and changes in both percentage of filler used and the number of compaction blows administered. For each variable at the specified limit, the ranges for number of compaction blows and percentage of filler contents that satisfy these limits were established; the results are shown in Figure 12. For example, the dashed range for percent-VFB was set by interpolating the possible combinations of filler content and compactive effort (see Figure 7-b) that render VFB values between 70 and 80 percent (as specified in Table 4). The plot for flow contains within it an area of unacceptable combinations of compactive effort and filler content, probably because the test results for flow, as shown in Figure 7, were so erratic. Stability is not addressed at all in Figure 12 since it is satisfied for all mixes.

The overall results of the analysis are shown in Figure 13. The range of allowable filler content and compactive efforts should satisfy all the criteria established in Table 4. Ideally, a "common" block would satisfy all criteria simultaneously, yet obviously there is no "zero" common block in this case; no mix satisfies all requirements.

One important reason is that all void criteria cannot simultaneously be met. Figure 14 illustrates the theoretical relationship for voids while the dashed area indicates the range that satisfies the requirements of Table 4 for voids (air voids and VFB). If the VMA requirement is also introduced, it is obvious that the size of the dashed area will decrease as the minimum VMA required increases. For the type of aggregate used in this study, an acceptable range of compactive efforts and filler contents that could satisfy all three void criteria simultaneously could not be found.

To address that problem, a new set of voids in mineral aggregates (hereafter VMA1) was established, based on the apparent specific gravity (2.681) rather than on the bulk specific gravity of the aggregates (2.540). Using apparent specific gravity allowed an increase in the calculated VMA; consequently, an acceptable range could be set for filler content and compactive effort that satisfy mix-design requirements (see Figure 15). The range of acceptable filler content is between 3 and 12 percent if the compactive effort used to produce the mix ranges between about 60 and 90 blows. This range indicates that the mix is more sensitive to variations in compactive effort than to variations in filler content.

Figure 15 also shows that if the filler content of the mix exceeds the design value (7 percent), strict control on com-

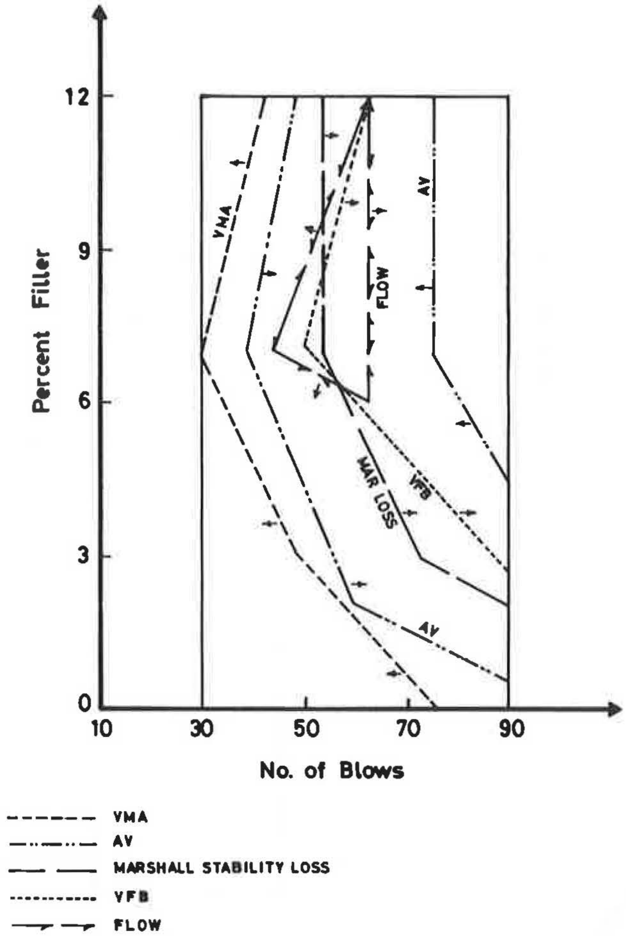

FIGURE 13 "Zero" common block boundary of compactive effort and filler content that satisfies mix variables (VMA based on bulk specific gravity of aggregate).

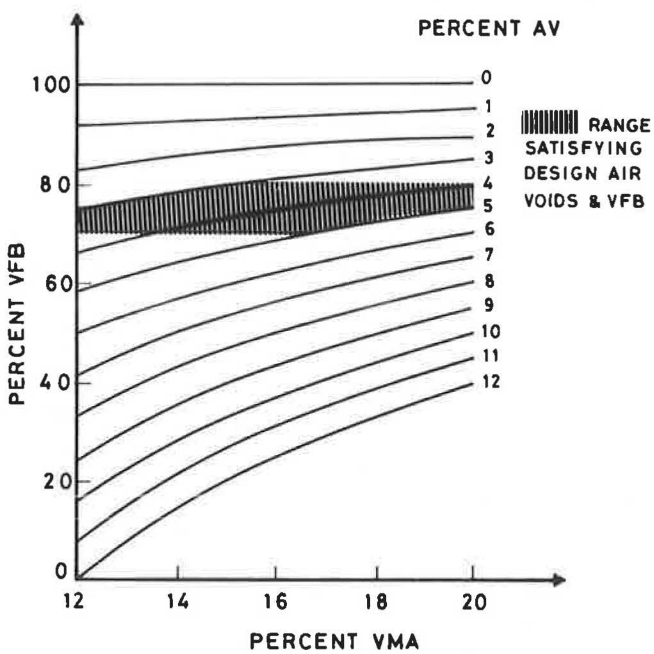

FIGURE 14 Theoretical relationship between VFB and VMA for various air voids.

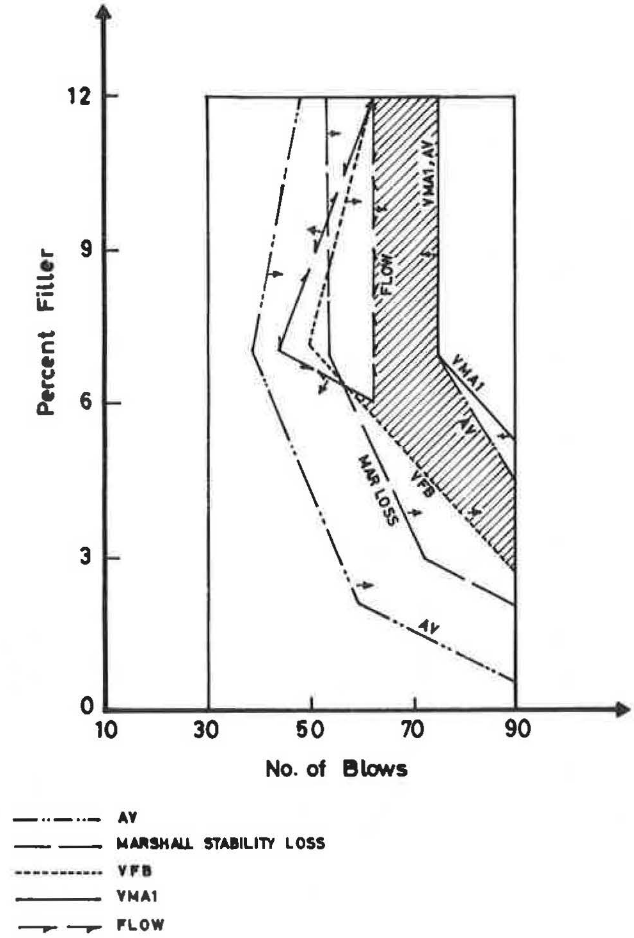

FIGURE 15 "Common Block" boundary of compactive effort and filler content "dashed area" that satisfies mix variables (VMA based on apparent specific gravity of aggregate).

paction must be applied to ensure that the mix will satisfy design criteria. On the other hand, overcompacting this mix above 100 percent (corresponding to 75 blows) will produce unacceptably low air voids. Nevertheless, air voids could be satisfactory at the high compactive effort if filler content is reduced beyond the design value.

SUMMARY AND CONCLUSIONS

The results of this study have demonstrated that the effect of field control on properties of mix design is appreciable. Most important is the effect on mix voids which, in turn, directly affects the M_R of the mix and the water-resistance of asphalt mixes. Incorporation of the effects of field variables in a rational mix design procedure was illustrated, and the difficulty of simultaneously satisfying all void criteria was apparent. Nonetheless, the field compaction and filler contents can vary in the field and still produce acceptable mixes.

The results of the study provide the basis for setting better specifications on both degree of compaction and percentage of filler contents. Please note, however, that the results of this study are based on only one type of aggregate and filler.

The conclusions of this study can be summarized as follows:

- The degree of compaction and filler content can vary and still produce acceptable mixes, which points up the need for setting specifications on both degree of compaction and percentage of filler content.
- Within the range of filler contents studied, mix design criteria are more sensitive to decreasing the filler content than to increasing it above the design value.
- The effect of decreasing air voids on modulus of resilience is more pronounced at high air voids (above 6 percent), whereas the effect of decreasing air voids on Marshall stability is more pronounced at lower air voids.

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Expert System for Diagnosing Hot-Mix Asphalt Segregation

DAVID J. ELTON

Asphalt pavement segregation is a continuing, costly problem in the paving industry. The problem is complicated by the many sources of segregation found in typical paving jobs. This paper reviews the origins and remedies of segregation, emphasizing the complex nature of the problem. Because of the complexity of the problem and its remedies, an expert-system solution, SEG, is proposed. SEG is a computer program founded on a knowledge base taken from the literature, combined with current expert input. The system interactively interviews the nonexpert user, who answers questions about the paving operation exhibiting segregation. On the basis of the user's replies, the system suggests changes in the operation to eliminate segregation. The system is rapid, simple, and portable—it can be used in the field with a lap-top computer. Expert systems hold great potential for changing the way transportation engineering problems are solved. The introduction of SEG as a practical tool for analysis is expected to help reduce segregation, and consequently improve pavement quality.

Random segregation of aggregate in asphalt pavements is a problem of much concern to the paving industry (*1,2*). Segregation of hot-mix asphalt (HMA) mixtures occurs when large aggregate sizes accumulate in one area of the paving mat while smaller ones accumulate in other areas of the mat. This effect leads to uneven distribution of density and asphalt contents in the mat, which can lead to premature distress in the pavement. Cracking, moisture-induced damage, raveling, and other durability-related damage may ensue. Pavement wear is increased by segregation (*3*). Moreover, segregation may reduce the extracted asphalt content in coarse areas, which may result in severe penalties for the pavement contractor (*4–6*). Consequently, it is important to identify conditions conducive to segregation and to transmit that knowledge to those responsible for the paving operation.

Because there are many sources of segregation, detecting the cause or causes on any particular job can be very difficult (*2*). A knowledge-based expert system is well suited to assist in the detection (*7,8*). Expert systems are a form of artificial intelligence, which can be used to help solve complex, nonnumerical problems (that is, problems that contain subjective elements). Expert systems have already found applications in civil engineering (*9*), with several applications in transportation (*9–12*).

This paper presents origins of and remedies for asphalt pavement segregation and then introduces an expert system, SEG, that captures that knowledge in a form that can easily and efficiently be presented to the field engineer. The expert system is a computer program that interviews the user and, based on the user's responses to queries from the program, suggests solutions to the segregation problem. In the past, the engineer needed to know the causes of segregation. With the help of the expert system, he need now only observe the various operations—the program will suggest the causes of segregation. Expert systems programs differ from conventional programming in that they use a heuristic set of rules and conclusions (which form the knowledge base) to make decisions.

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ORIGINS AND REMEDIES OF SEGREGATION

Random segregation of hot-mix asphalt can occur at any or all of several stages of the paving process—during mix design, mix production, transportation, or laydown operations. The problems associated with each are described below.

Mix Design

The mix design has a significant effect on segregation (*2, 13–15*). The best design is a well-graded aggregate that falls just above or below the maximum density line (A-line) shown in Figure 1, which is a plot of sieve size raised to the 0.45 power versus percent passing that sieve size on a linear scale. The slope of the A-line is determined from 100 divided by $M^{0.45}$, where M is the maximum-size aggregate in the mix. This line represents the particle-size distribution yielding the maximum density (minimum voids) for that maximum particle size (*16*). The maximum particle size is the sieve size on which no material is retained. Gradations in close proximity of this line are not likely to have enough void space to allow adequate asphalt coverage of the particles, resulting in a dry, friable mix. Preferable is a smooth curve lying just above or below the A-line. Such curves retain the well-graded characteristic needed to reduce segregation and provide sufficient void space for asphalt. Brock (*2*) suggests 2–4 percentage points above or below the A-line. Gradations above the A-line result in a finer mix, below the line in a coarser mix. Grain-size distribution curves that cross the A-line indicate gap grading—the absence of a range of particle sizes. Gap-graded curves are quite prone to segregation (*2*) because different aggregate sizes tend to interlock. If some sizes are missing, less interlocking occurs, and the particles are freer to move and segregate. Excessively large-size aggregate also accentuates segregation (*17,18*). The mix design engineer should be alert for segregation in any

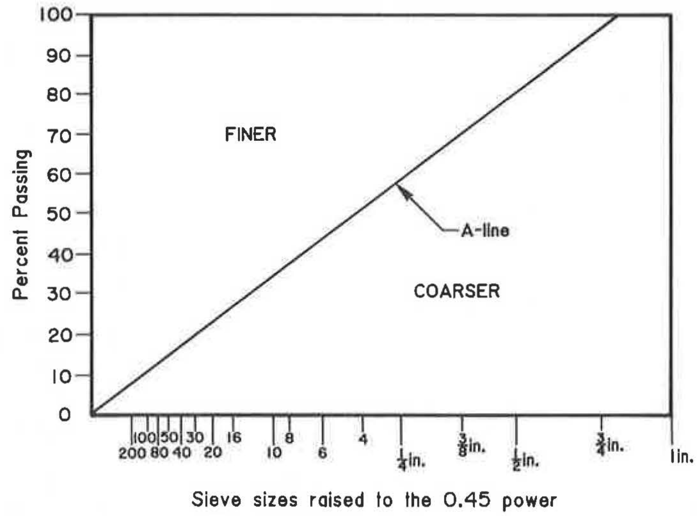

FIGURE 1 Gradation plot showing maximum density line (A-line).

mix, but special caution should be exercised for larger sized aggregate blends, which are used more and more now to minimize rutting potential.

The asphalt content also affects the segregation potential of the mix. Low asphalt content leads to segregation and high asphalt content deters it, since additional asphalt increases cohesion. Although increasing the asphalt content may be costly, penalties for placement of segregated pavements may be even more expensive. Brock (*2*) and Brock and May (*19*) suggest that changes in the asphalt content as small as 0.2 percent may significantly reduce segregation problems. When calculating asphalt content, mix designers should keep in mind the aggregate's absorptive capacity, which reduces the asphalt's ability to increase cohesion and reduce segregation.

Last, differing specific gravities of the aggregates can cause segregation. Particles with higher specific gravity are denser and tend to settle to the bottom, given the opportunity, during handling. Moreover, denser particles are more likely to roll when stockpiled, increasing segregation.

Mix Production

Segregation can occur at many different places in the asphalt mix production cycle. The operator must be alert for segregation at all the points described below, as well as at any facilities or operations peculiar to his plant. In general, segregation occurs when the aggregate is in motion.

Segregation may occur in the stockpiling operation (*2,13,15*). The aggregate may already be segregated when it is delivered to the HMA facility. If so, it must be integrated before asphalt is added. Batch plants do this with the screens above the hot-feed bins. Drums or continuous mix plants may require extra screening before such aggregate can be used. Segregation may also take place when the stockpile is created. The greatest segregation occurs when the material is placed in a large pile from a conveyor that throws the aggregate outward as it is placed. The larger the pile, the greater the amount of segregated large particles that accumulate around the base. The conveyor casts the larger particles a greater distance than it does the smaller ones, abetting segregation. Thus, if conveyor-fed stockpiles are used, a vertical chute ought to be added to the unloading end of the conveyor and stockpiles should be kept small. Still better would be to use a bulldozer to spread out the conveyor-deposited material into a large, flat area, thereby creating a layered structure. The conveyor would continue to deposit onto this structure while the bulldozer continued to spread the material out. Care should be taken not to push the material over the edge of the pile, which would allow larger particles to roll and segregate at the base. The slopes of the bulldozer-created pile should be less than the angle of repose of the aggregate.

Stockpiles may be created by end-dumping trucks. In general, the fewer times the aggregate is handled, and the shorter time it is handled and in free fall, the less the segregation will be (*15*). Trucks should discharge their loads as rapidly as possible; allowing the material to trickle or stream from the truck abets the segregation process. Trucks should minimize the height of fall of material by depositing each load to the side of the previous load, which will prevent large amounts of segregated material from accumulating from adjacent piles. (A larger pile can be created, if necessary, by bulldozing the smaller truckloads into a wedge-shaped pile. The bulldozer pushes the material up the wedge in successive layers, without shoving it over the back of the pile where it could roll and segregate (*2,20*). All slopes of this pile should be less than the angle of repose.)

All of the suggestions above apply to stockpiles of multiple-sized aggregate. Single-size aggregate does not segregate. However, care should be taken to prevent different sizes from mingling. Separately stockpiling different-sized aggregate is the safest way to deter segregation at this stage of the operation.

Segregation can be induced when the material is moved from the stockpile to cold-feed bins. The front-end loader should push the bucket directly into the pile rather than scooping up along the face of the pile (*15*), which would likely gather just larger particles in the bucket. With drum plants, it is especially important to ensure that the bin-loading procedure preserve the gradation that the mix design was based on. The cold-feed bins should be loaded by vertically dropping the material into the center of the bins. Placing the material against the sides of the bin or adding a horizontal component of trajectory increases segregation. Again, the more quickly the material is placed, the less chance of segregation.

Details of bin construction can reduce the potential for segregation. If bins are adjacent, vertical barriers between the top of the bins serve to keep material from spilling out of one into another. Kennedy et al. (*15*) and Brock (*2*) recommend that the bin discharge opening be trapezoidal rather than rectangular. If the wide end of the trapezoid is in the downstream edge of the opening, the material will flow more freely from all areas of the bins. Otherwise the material tends to discharge solely from the center, inducing segregation on the edges. The trapezoidal shape helps keep the aggregate from arching over the discharge opening.

Certain details of batch plants must be observed. The screens above the hot-feed bins must be kept in good repair. Holes in the screens will lead to an improper mix, which may segregate easily. The finest sized bin is most likely to develop segregation because it has the largest range of sizes—often a few orders of magnitude.

Material must be placed in the middle of the bin rather than along the side, as often happens. A baffleplate placed below the screens can direct the falling material from the edge of the bin into the center. This is especially important if the fines from the baghouse are returned here. Since the fines pass through the screens almost immediately, they accumulate on one side of the bin. Moreover, they may attach themselves to the side of the bin and later fall off in large chunks. A vibrating plate on the bin wall eliminates the latter problem, and a baffleplate attached to the side reduces the former (*2*). Adding a baffleplate may reduce bin capacity, but the benefits of doing so may well offset this disadvantage.

Particular components of continuous mix plants can contribute to segregation. Since sufficient asphalt content is an important parameter in reducing segregation, it is important that the aggregate stay in the drum mixer long enough to become coated with asphalt. If this is not occurring, several corrections are possible: introduce more asphalt, add the asphalt to the drum sooner, or increase the mixing time in the drum. Mix time can be increased by kickback flights in the drum, dams, or longer drum length. The increased time in the drum, however, also drives off more moisture, which Rice (*21*) indicated aids in segregation.

As the material rotates in the drum, it tends to segregate, with the coarser particles staying on the bottom of the drum and the finer particles going higher up the sides. As the material exits the rotating drum, the coarser material tends to fall to one side of the conveyor and the finer material to the other (*2*). If the discharge conveyor is parallel to the discharge direction, segregation is accentuated as the coarse material consistently accumulates on one side of the conveyor. This one-sided segregation is carried through the other plant operations and eventually manifests itself as a stripe of segregated pavement. However, if the discharge conveyor is perpendicular to the discharge direction, this effect is mitigated since the material is spent out evenly over the conveyor.

After the asphalt is mixed with the aggregate, it is either discharged to a truck or to a storage silo. The storage silo operation presents several opportunities for segregation. When the mix leaves the mixer, it is transported to the storage silo, usually by a drag conveyor. As Higgins (*18*) notes, drag conveyors are superior to screw augers, skips, or hoists in this phase of the operation. However, if the face of the drag conveyor is cool, fine parts of the mix may adhere to the surface, causing the drags to hydroplane, which in turn allows the coarser particles to skim over the tops of the drags, increasing segregation (*2*). In this situation, the conveyor should be equipped with heated bottoms and floating hold-downs (idler wheels) to eliminate the problem. This problem is particularly exacerbated at start-up. If the drag conveyor is running much faster than the mix is being produced, the drags will not be full, which leads to segregation because the particles can move around (*14*). Similarly, if the conveyor is running too slowly, the drags may become overloaded and overflow as the material is taken up towards the silo. This condition, too, leads to segregation, with the larger particles rolling back down the conveyor. Care should be taken to match the bin-batcher opening rate with the drag conveyor speed; if the batcher delays opening, it overflows, causing the mix to spill down the conveyor (*2*). The mix segregates when this happens.

The mix must be placed properly in the storage silo to avoid segregation. Rather than place the mix directly from the drag conveyor into the silo, the mix placement should be buffered by either a bin batcher (*5,18*) or a rotating chute (*15*). The bin batcher, which should hold at least 4,000 lb of mix (*2,13*), is simply a bin above the storage silo. The batcher is placed above the storage silo, and it is filled by the drag conveyor. After the batcher is filled, it should be emptied in one quick motion. The plug of mix should fall into the storage silo and splatter over the mix already there (*18*). The batcher should be emptied only when it is full, not on some arbitrarily decided time schedule. The batcher doors must be closed before all the mix has exited, lest any mix from the drag conveyor be placed directly in the storage silo. The height of fall into the storage silo should be sufficient that the material splatters on impact. Thus, the silo must not be kept too full or the splatter effect will be foiled. Similarly, if only small amounts are discharged from the batcher, there may not be enough redistribution of the material on impact (*13,15*). The bin batcher should be loaded by a vertical fall of the mix from the drag conveyor (*13*), because a horizontal trajectory would lead to segregation.

A rotating chute at the top of the storage silo may be used in lieu of the bin batcher to reduce segregation (*2,15*). The chute's effect is twofold. Primarily, the mix is distributed around the interior of the silo rather than landing in a pile in the center where larger particles may fall to the edges. Moreover, the properly operating chute directs the material straight down so that it does not land on the silo wall, producing segregation when some particles rebound more than others.

Other devices have been used to minimize storage-silo segregation. Middleton et al. (*6*) describe a horizontal, rotating-screw auger arrangement that fits on top of the storage silo, raining the material into the silo in a sheet instead of a stream. Foster (*5*) reports a storage silo with a pugmill on top. The pugmill removes the effects of previous segregation by mixing. The mix then dumps into a series of splitting hoppers before coming to rest in the silo. Kennedy et al. (*22*) note that while this method reduces segregation, it does not always eliminate it. Using an inverted (upward-pointing) cone in the silo exit causes some remixing as the mix departs, reducing segregation (*5*).

The storage silo details are important. The discharge-end of the silo should be a steep cone (greater than 55 degrees (*15*)) so that the material is pulled evenly from the bin and not just from the center. If the sides of the cone are not steep enough, an inverted cone forms in the mix as the silo empties and the aggregate segregates as it rolls down the center of the inverted cone (*14*). The silo cone should be insulated to improve the smoothness of the mix flow from the bin (*13*). Although it appears that the discharge-gate details are not extremely influential, the gates should be large enough so that the material does not stream out but instead emerges as a large mass (*2,14*).

The silo may be emptied entirely with little effect on segregation if the mix is well graded (*2,19*). However, as Kennedy et al. (*15*) recommend, the silo should be filled above the top of the cone at all times when using gap-graded mixes.

Transportation

Segregation can take place when the mix is transported from the HMA facility to the job site. Truck loading and unloading are the primary problem times. Trucks should be loaded with three small deposits rather than one large one (*2*). The truck bed's front, rear, and middle should receive separate loadings, in that order, to reduce the size of the piles and overall segregation (*14*). Moreover, segregation is further reduced because the length of the front and rear piles is reduced and particles cannot roll as far as they could if they were in one pile. This is especially important in reducing the end-of-load/beginning-of-load segregation overlap that occurs when the trucks discharge into the paver.

The location of the piston that raises the truck bed can cause segregation. If the piston intrudes upon the bed, typically at the front of the bed, the asphalt mix may segregate when it falls into the cavity left by the piston well when the truck is unloaded. The solution is to block off the parts of the bed adjacent to the well so that no cavity is formed when the mix exits the truck bed.

Rough or bumpy truck beds assist segregation. Beds should be kept smooth and clean so that the mix slides instead of rolls out, which enhances segregation. Last, the mix should be deposited in the paver in a deluge. Hensley (*14*) and Kennedy et al. (*15*) recommend raising the truck bed before opening the tailgate to make the mix move faster and aid in deluging the paver hopper. The rapid movement reduces segregation at the start of the dumping operation.

Laydown Operation

Segregation can be caused by incorrect operation of the paver. The variables affecting segregation in the paver are:

- Operation of the paver wings,
- Speed of drag slats,
- Paver throat opening, and
- Auger condition and speed.

Perhaps the most common cause of segregation in paving operations is the periodic emptying of the paver wings into the paver. If the hopper is allowed to empty, some coarser material will accumulate in the wings, and when they are dumped, a lateral band of segregated material is laid down. This problem can be moderated if the hopper is not allowed to become less than one-quarter empty (*14,15*).

Keeping the hopper at least partially full has other advantages. The drag slats should not be starved for mix, or they will segregate the mix. If the augers are starved for mix (because of small throat openings, slow drag slats, or absence of mix in the hopper), segregation will occur (*14*). The throat openings in the paver should be kept wide open (*2,14,22*). Restricting the flow from the hopper to the auger can result in starving the auger for mix, which causes the larger particles to move to the outside of the mat, leading to segregation at the edges of the mat. Therefore, it is important to keep the throats wide open and to adjust the speed of the auger to keep the mix spread the full width of the paver.

The same effect results if the auger speed is too fast for the amount of mix being sent to the auger. The larger particles are thrown to the outside, yielding side-to-side segregation. If the auger speed is too slow for the amount of mix going to it, the outside edges of the paving mat exhibit segregation (*14,15*).

The auger must be kept in good condition. Larger particles may congregate wherever the auger is worn or broken. Reverse baffles should be placed at the center of the auger to push the mix under the center of the auger gearbox. If the reverse baffles are not working properly, coarse material will accumulate, causing linear segregation in the center of the mat.

Finally, after the mat has been laid, some careful manual finishing operation must be done to join the mat to an adjacent mat. The mix should be placed over the joint carefully, and not scattered across the pavement, for higher joint density and less segregation. Generally, curtailing manual work results in higher quality pavement (*15*).

EXPERT SYSTEM

An expert system, SEG, which incorporates the information on segregation discussed above, was constructed to aid hot-mix industry personnel in attenuating random segregation problems. The expert system is not a conventional computer program with a specific algorithm for solving problems. Rather, it uses a knowledge-based processor (an inference engine) to determine when to apply the expert knowledge contained in the program. SEG is an interactive microcomputer-based program that gathers information from the program user. The user-provided information is compared to information in the program, resulting in suggestions on possible sources of segregation. The program requires a personal computer, either a conventional desktop or a portable lap-top machine. The portability of a lap-top model may especially suit it for field evaluations.

SEG has the potential to reach several conclusions, based on user input. The general categories of conclusions for SEG are as follows:

- There is a problem with the mix design.
- There is a problem with the stockpiling operation.
- There is a problem in the HMA plant per se.
- There is a problem in transporting the mix.
- There is a problem in the laydown operation.

Within each of these statements are more specific sublevels that help pinpoint sources of segregation and recommend remedies. SEG does not stop once a source of segregation is found; it continues to delve until enough information is gathered to make a decision in all the areas outlined above.

In order to implement the system, the user must be able to recognize both random segregation in the completed mat and segregation of the mix at various points in the HMA plant. He must have knowledge of plant machinery and be able to visually inspect the aggregate and the mix at different locations in the operation. The program will ask the user what plant type is in operation and the details of particular equipment, including the storage silo, the conveyor leading to the silo, the hot-mix bins, and truck and paver. Depending on the user responses, the expert system continues to ask questions and display possible solutions to the problem. Because it is unlikely that the segregation problem has only one source, the program continues to interview the user until all sources of random segregation have been investigated.

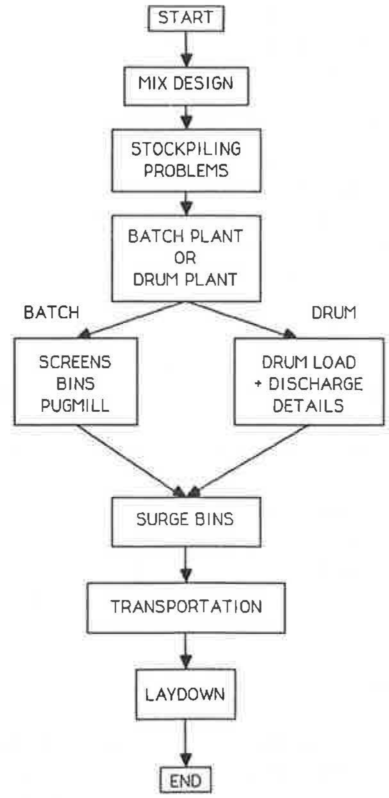

FIGURE 2 General structure of the knowledge base of SEG.

Knowledge Base

The information in the program on sources of segregation represents the knowledge of the experts. This information forms what is called the knowledge base of the program. For this study, the knowledge base was taken from the literature (*2,5,6,13–15,19–21*) and experts at the National Center for Asphalt Technology. The knowledge is entered in the program in a series of logical rules:

IF a certain condition exists (is true)
THEN a certain conclusion is reached
AND some action is initiated.

The action initiated is either the reaching of a conclusion with attendant suggestions for improving the operation or the firing of another rule to gather more information from the user. These rules are networked so that each rule leads to the execution of another rule and/or the display of a conclusion. The results of one rule are remembered by the program and lead to the selection of the next rule to be executed. The order of succession of the rules reflects the thought process of the experts used to analyze the problem. The rules are entered into the knowledge base in the English language, making programming relatively easy.

The general structure of the knowledge base is shown in Figure 2. Figure 3 shows the broad rules corresponding to Figure 2. The RULE statement indicates the topic that the rule concerns. The IF statement is the true/false question that the program asks the user. If the user replies *true*, then the THEN statement is executed, as well as any AND statements that follow. If the user replies *false*, the THEN statement is not fired, and the program proceeds to the ELSE statement. DISPLAY statements indicate that the program will give suggestions on how to attenuate the problem at hand. The CHAIN statements indicate that the program will branch to another set of subrules or conclusions before returning to the main rules shown in Figure 3.

Program Operation

Expert system operation is straightforward. The program asks the user a series of true/false questions about the particular hot-mix production/transportation/laydown cycle. A sample question is as follows:

Is it true that the cold-feed bin is allowed to empty entirely?
True *False*

This simple true/false format was chosen over more complex interfaces because it fits the problem and because the conclusion had to be found quickly. Most segregation problems are encountered only after the paving operation is under way, making rapid solution of the problem a high priority. While the user needs to understand the question, he need not understand the *significance* of the question (that is, he need not be an expert). However, the program does understand significance and it explains what problems may arise from allowing the cold-feed bin to empty entirely. Herein lies the usefulness of the expert system—an untrained observer can give his

RULE Check material for gap-grading.
IF The material as received was gap graded.
THEN DISPLAY gapgrade
AND CHAIN agmixtype.prl
ELSE CHAIN agmixtype.prl

RULE Check if a single mix being used.
IF Is a single mix type being used.
THEN CHAIN pileseg.prl
ELSE CHAIN surgebin.prl

RULE Check if operation is a batch or drum plant.
IF Are you operating a batch plant?
THEN CHAIN batch.prl
ELSE CHAIN drum.prl

RULE Check if mix is segregating in the surge bin.
IF Is the mix segregating in the surge bin?
THEN DISPLAY surgebin
AND CHAIN full-empty.prl
ELSE CHAIN hydroplane.prl

RULE Check if transportation is causing segregation.
IF Is the mix unloaded by the truck segregated.
THEN DISPLAY trucking
AND CHAIN laydown.prl
ELSE CHAIN drum.prl

RULE Check if paver is causing segregation.
IF Is the mat segregated.
THEN DISPLAY paver

END

FIGURE 3 General rules for inference network shown in Figure 2.

response to the system and receive the significance of his observations from the knowledge base in the program: The computer becomes the "expert," based on the expertise incorporated into the knowledge base; the personnel responsible for the hot-mix job need not have an expert on hand to interpret the observations for him—the expert system does it, based on user observations and responses to program queries.

SEG was written using the expert system shell program INSIGHT2+ (*23*). The shell provides a language and compiler that allow the creation of rules that constitute the knowledge about segregation. The compiler provides for an orderly execution of the rules so that the user progresses through the knowledge base in systematic fashion. For example, if the user replies TRUE to the query on whether he is running a batch operation (as opposed to a continuous operation), the program ceases to make inquiries related solely to continuous operations. The shell thus provides an efficient way to search the knowledge base. SEG was written on an IBM personal computer and operates on PC-compatibles.

The information stored in the program is called a knowledge base, which is a set of rules consisting of questions asked of the user followed by branching statements based on the user's responses. The branches lead to conclusions about where segregation may be occurring in the operation and to suggested cures. Some of the rules contain information explaining the significance of the question, as well as information on the potential for segregation based on the reply to the question.

For example, the user might be asked if the cold-feed bins are adjacent (*true/false*). If they are adjacent (*true*), the program generates the following display before branching to the next rule:

> *Segregation can originate in the mixing of different size aggregates in the cold feed bins. Since the bins are adjacent, check to be sure that the aggregates are not spilling over from one bin to another. If so, provide bin dividers to prevent mixing. Instruct loader operator to be careful not to mix sizes.*

The display can be sent to the printer to create a permanent record of the session. Given the arrangement of rules, the program can reach many different conclusions before the session is completed by chaining together several knowledge bases (each with its own conclusions). The program, having reached one conclusion, proceeds to the next knowledge base rather than ending the session with the first conclusion reached. Chaining also allows a very large number of rules to be incorporated into the program, limited only by the extent of the computer memory.

One of the advantages of an expert system is its ability to quickly enact several scenarios to test the effect of different plant changes on segregation. A typical session with SEG takes less than 15 min, allowing the user to make different trials quickly. Moreover, the trials could be run *before* job operation begins to locate potential causes of segregation.

Sample SEG Run

A sample run of SEG is presented to illustrate the system's application. The system gathers information from the user until it has sufficient data to indicate where segregation may be occurring in the HMA operation. When starting the system, the user is asked first about the mix design [the most likely parameter causing segregation (*2,15*)]:

> *Is it true that the material, as received, was gap-graded?*
> *True* *False*

If the user is unsure of the term "gap-graded," he may obtain further explanation by invoking the EXTEND function, which generates the following message:

> *A material is "gap-graded" if some sizes of aggregate are missing from the aggregate.*

If the user answers TRUE, the program generates the following information:

> *If the material is "gap-graded," i.e., some sizes of aggregate are missing from the aggregate, segregation could be occurring in the source pile of aggregate. Gap-graded grain size distribution curves have a stair-step appearance. The best cure is to blend sizes of aggregate to fill in the gaps. When grain sizes are plotted on the 0.45 power plot, the grain size distribution should lie slightly above or below the maximum density line (A-line), but never cross it.*

The system then gathers information on aggregate handling.

First, to determine whether the aggregate is becoming segregated in the stockpiling operation:

Is it true that the material is segregating in the stockpile?

True *False*

If true, remedies are presented to the user. To determine and explain problems in cold-feed bin operation, the program continues by asking:

Is it true that the material is segregating in the cold feed bin?

True *False*

At this point, the program follows different lines of inquiry depending on the type of plant. Until this point, the features of both batch and drum plants have been about the same. The user is asked next whether he is analyzing a batch or drum plant. The following question is:

Is it true that the mix is completely coated with asphalt?

True *False*

If the user had responded earlier that a drum plant was being analyzed, the remedies offered would be:

Low-asphalt content aids segregation. If the mix is not well coated, consider these improvements:

1. Increasing time in drum mixer by

- *Decreasing slope of the drum,*
- *Putting kickback flights in the drum,*
- *Putting a dam in the drum.*

2. Moving the asphalt nozzle further into the drum.
3. Increasing the asphalt content.

Had it been a batch operation, the reply would be:

If the mix is not coated completely, consider the following changes:

- *Increase mixing time in the pugmill.*
- *Inspect the tips and shanks of the mixer for wear.*
- *Increase the asphalt content.*

Next, the operation of the drag conveyor at the drum exit is examined.

Is it true that the drag conveyor is perpendicular to the drum exit?

True *False*

Is it true that the drag conveyor to the surge bin is hydroplaning?

True *False*

These questions, if answered affirmatively, generate displays that show the user the proper orientation of the drag conveyor to the drum. If hydroplaning is a problem, two possible remedies are suggested.

Hydroplaning results in segregation as the smaller particles accumulate on the conveyor and pass under the drags, leaving the coarser mix to travel up the conveyor.

Two solutions are available:

1. Heat (or increase the heat on) the drag conveyor to reduce the possibility of asphalt accumulating on the conveyor.
2. Equip the drag with floating hold-downs.

The surge (or storage) silo is examined next. The first question determines whether there is a silo, and, if so, whether it might be causing a problem:

Is it true that there could be segregation occurring in the surge silo?

True *False*

If there is a surge silo, the system examines whether the fall of the mix from the drag conveyor into the bin is buffered. The details of the buffer device, whether it is a bin batcher or a rotating chute, are reviewed.

The last two parts of the system go over the trucking and paving operations. The program asks the user how trucks are loaded and unloaded, emphasizing the need to load the truck in several small batches rather than in one big one. If the user notes that unsegregated truck mix is segregated after it is placed in the paver bin, the following advice is given:

The truck should flood the paver bin with hot mix. Do not allow the mix to be dribbled into the bin through an opening in the tailgate or through the tailgate itself.

Tip the bed of the truck up before opening the tailgate to assist in the flooding of the paver bin.

If the truck bed has a well at the front for a hydraulic piston to lift the bed, beware of segregation around the cavity in the mix left by the piston well as the mix slides off the bed. Block off the parts of the bed adjacent to the well so that no cavity is formed as the mix exits the truck bed.

Truck beds should be kept smooth and clean so that the mix slides out rather than exits with any rolling action, which enhances segregation.

Last, the paver is examined. The system queries:

Is it true that unsegregated mix is coming out segregated from the paver?

True *False*

If the answer is *true*, the ensuing text is shown.

Check the following items:

- *Drag slats are covered with mix at all times (bin is always at least 25 percent full).*
- *Throat openings are wide open at all times.*
- *Auger is not broken or worn.*
- *Auger speed matches slat speed, so mix is not slung to the sides (too fast) or mix is not driven to the ends of the auger (too slow).*
- *Asphalt content is not too low.*

- *Paver wings are being emptied.*
- *Auger gearbox baffles are in good repair.*
- *Manual dressing of the joint is done properly (pack the mix in the joint—don't cast it over the mat).*

Thus SEG completes the survey of the HMA facility, transportation, and laydown operations. The user can then either go through another session with SEG or exit the system. Simply by recognizing segregation, the user can run the expert system and receive all the information he needs to remedy the problems causing segregation.

CONCLUSIONS

Asphalt pavement segregation is a continuing, costly problem in the paving industry. Segregation's manifold sources further compound the problem—no single procedure will eliminate all instances of segregation. This study reviewed segregation's origins and remedies, emphasizing the complex nature of the problem.

Because the problem and its possible remedies are so complex, an expert system solution has been proposed. The expert system, SEG, was created with the shell program INSIGHT2+; SEG's knowledge base is taken from the literature, combined with current expert input. The system interactively interviews the nonexpert user, who answers questions about the paving operation exhibiting segregation. Based on the user's replies, the system suggests changes in the operation to eliminate segregation. The system is rapid, simple to use, and lends itself to field application with a portable lap-top computer.

Expert systems are a new technology with great potential for changing the way transportation engineering problems are solved. The emergence of expert systems in engineering is relatively recent, even though these systems have already found routine application in such diverse fields as health care and computer system selection (*8*). The introduction of SEG as a practical tool for analysis is expected to aid in the reduction of segregation. The knowledge base and further updates for SEG are available from the author.

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Enhancing the Bond of Emulsion-Based Seal Coats with Antistripping Agents

ALI A. SELIM

Seal coats are highly regarded as a pavement maintenance tool because, among other functions, they enhance the friction value of existing road surfaces. Asphalt emulsions are commonly used in seal coat construction for their adaptability to environmental and handling conditions, and the better the bond between emulsion and aggregate, the better the friction value of the seal coat. This study seeks to find out whether antistripping agents enhance the bond between aggregate and emulsion, consequently improving the friction value of the seal coat; to determine the best way of including the antistripping agent in the seal coat; and to establish a mathematical model to predict friction value of a particular seal coat at any time after construction. To those ends, several seal-coat test sections were prepared, subjected to traffic for several months, and then evaluated. The outcome of the research reveals that (a) including the antistripping agent in a seal coat improves the bond between aggregate and asphalt and reduces the tendency of aggregate particles to rotate under horizontal drag forces, both of which led to higher friction values; (b) mixing the antistripping agent with the emulsion instead of applying it to the aggregate surface improved the total bond and led to higher frictional values; and (c) quadratic models were better than exponential models at predicting friction values of seal coats at any time after construction.

Maintenance officials consider seal coats to be a reliable method of preserving the integrity of road surfaces. One of the cardinal functions of seal coats is to restore the friction value of the road surface to its original construction level.

The success of any seal coat depends on the type of aggregate used, the type of binder, the compatibility of aggregate and binder (how well the two materials bond), and quality control during construction. In this research, crushed quartzite, which exists in abundance in the southeastern part of South Dakota and the southwestern part of Minnesota, was used. The asphalt emulsion CRS-2 was chosen for this study because of its suitability to the type of aggregate used and to the ambient temperature at the time of construction. Two different types of antistripping agents were also used to prevent debonding between the aggregate and the emulsion.

A review of the literature on asphalt pavement debonding revealed that most of the research was geared toward hot mixes; virtually no work addressed cold mixes or seal coats, except for a study by Selim (*1*) on a new testing method for evaluating seal coat debonding when asphalt emulsion is used as a binder.

To enhance the bond between the aggregate and the binder in a seal coat, antistripping additives are sometimes used. The chemical industry has introduced several antistripping agents designed to be added to the asphalt mix at a dose of 0.5 to 1.5 percent of the weight of residual asphalt. Adding antistripping agents to asphalt hot mixes has been well documented, but no well-recognized method exists for including the antistripping agent in cold applications, such as seal coats made with asphalt emulsions. In this study two types of antistripping agents were employed. The first type, REDICOTE 82-S, is 100 percent active, heat stable, and when added to the mix produces a water-resistant film of asphalt. The second type, HIB7178, is also heat stable, fluid at ambient summer temperature, and recommended for use in all seasons. It makes the aggregate surface wettable and allows the asphalt to deposit an intact coat on the aggregate surface.

Seal coats are more vulnerable to debonding than hot mixes because

- All uncoated aggregate surface is exposed to moisture.
- The relatively thin layer of seal coat mass (aggregate and binder) is subjected to the abuses of weather and traffic.
- Once debonding at any aggregate particle begins, it can accelerate when the particle rotates under traffic load, braking action, or both.

Clearly, seal coats have as much need for antistripping additives as some hot mixes do.

The term *total bond* will be used throughout this paper to mean the original bond between the aggregate and binder (without the use of any antistripping agents) plus the additional bond produced when the antistripping agent was added. The term *friction value* will also be cited frequently instead of other terms such as *skid resistance*, *friction resistance*, and so on.

RESEARCH OBJECTIVES

This research was carried out to shed some light on the total bond of seal coats made with asphalt emulsion as the binder. The objectives of the research were threefold:

- To determine whether the antistripping agents produced

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added bonding between the aggregate and the emulsion's residual asphalt.

- To find out whether antistripping agents, if they are helpful, should be mixed with the emulsion first or should be used to coat the aggregate surface before the construction of the seal coat.
- To examine the trend of friction value loss over time with a reliable mathematical model.

TOTAL BOND AND FRICTION VALUE IN A SEAL COAT

Because the original bond between aggregate and residual asphalt and the bond added when an antistripping agent is present are difficult to measure, this research uses total friction value to indicate effective bonding. It should be pointed out that friction value is indirectly affected by the presence of antistripping agents in the residual asphalt. Although tires and the asphalt matrix do not make direct contact (unless bleeding occurs), the friction value will be affected by how much rotation the aggregate particles yield under drag forces. Aggregate rotation is vulnerable to residual asphalt stiffness and the degree of bond between aggregate and residual asphalt. Hence, the presence of antistripping agents in the residual asphalt matrix should enhance the bond and reduce aggregate particle rotation, which would consequently improve the friction value as measured by a device such as the British Pendulum Tester (BPT).

Seal coats do not exhibit their friction value characteristics in the same way as some heterogeneous solid material, such as asphalt hot mixes used for wearing courses. In a hot-mix wearing course, friction value depends highly on the type of aggregate and its surface texture. Aggregate particles are totally embedded in the asphalt matrix. When steel rollers are used during compaction, aggregate particles in the soft hot mix have a good chance to rotate and lay flat. Those aggregate particles at or near the surface are well situated and totally surrounded by other aggregate particles; the binder will ultimately provide the friction value of the wearing course surface. Pneumatic rollers are used only to seal the surface and make it tight.

In seal coats, pneumatic tire rollers are the recommended compaction device for embedding aggregate particles into the very thin layer of asphalt binder. Aggregate particles cannot rotate freely and lay flat because the aggregate on the surface of the seal coat interlocks, as shown in Figure 1. The extra, loose aggregate is usually shoved away to the side by traffic or removed with power brooms a few days after the seal coat is constructed. This extra aggregate is in excess of what the board test recommends (*2*); its purpose is to ensure sufficient cover over the embedded aggregate and to prevent possible tracking of asphalt binder by the pneumatic roller tires.

Total friction value is the total resistance force encountered by an object sliding over a pavement surface. This object can be a locked wheel tire or the rubber unit of the BPT. Two factors go into friction:

1. The surface texture and roughness of the exposed portion of embedded aggregate, as well as the conditions of the wheel tire (drag force); and

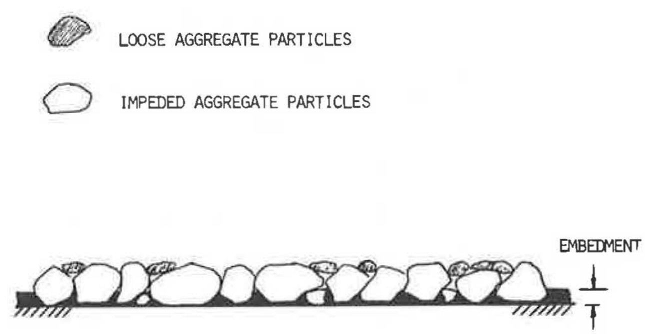

FIGURE 1 Arrangement of aggregate particles after compaction of a newly constructed seal coat.

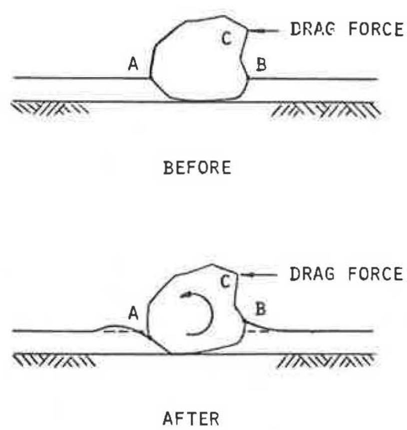

FIGURE 2 Rotation of aggregate particles due to drag force during service.

2. The ability of asphalt binder to prevent the aggregate particle from rotating under the horizontal drag forces produced by braking tires (see Figure 2). Resistance to rotation depends on the binder's consistency and the degree of bond between the binder and the aggregate. Rotation depends greatly on the residual asphalt stiffness and its ability to resist deformation—the less the rotation, the higher the friction value, and the more the rotation, the less the friction value. Tension zones, as depicted in Figure 2, can foster the separation of aggregate and asphalt binder once debonding begins.

For a seal coat to do the job it should, the second element—ensuring no particle rotation—must exist before the first element—appropriate surface texture and aggregate surface roughness—can contribute fully to total friction value.

When only one type of aggregate is used (for instance, crushed quartzite, as in this study), any improvement in total friction value should be attributed to the additional bond between quartzite and residual asphalt that was prompted by the antistripping agent. This additional bond hinders aggregate particle rotation, which leads to higher friction value.

PREPARATION OF MATERIALS

Although CRS-2 emulsion is known to contain an antistripping chemical (diamene salt), this research investigated the

TABLE 1 TREATMENT IDENTIFICATION

TREATMENT	DESIGNATION	DESCRIPTION
A	Q + E	Quartzite and Emulsion
B	Q + (E * 82-S)	Quartzite and Treated Emulsion with REDICOTE-82S
C	Q + (E * HIB-7187)	Quartzite and Treated Emulsion with HIB-7178
D	(Q * 82-S) + E	Treated Quartzite with REDICOTE 82S and Emulsion
E	(Q * HIB-7178) + E	Treated Quartzite with HIB-7178 and Emulsion

benefits of using a commercial antistripping agent to further improve the bonding between quartzite aggregate and CRS-2 emulsion. Quartzite is a hydrophilic rock known to have stripping problems. Two commercial antistripping agents were chosen, REDICOTE 82-S and HIB-7178. Either antistripping agent must be added to the seal coat by mixing it with the emulsion or spraying it over the aggregate in a thin coat. Because available literature did not stipulate the appropriate way of including antistripping agents in seal coats, both methods were used. A professional laboratory mixed a predetermined amount of the antistripping agent with the emulsion during the asphalt phase of the project; for the coating method, the antistripping agent was diluted and sprayed so that a predetermined amount of the agent was deposited on the quartzite surface. The predetermined amount was about 1 percent of the amount of base asphalt in the emulsion (*1*).

Enough quantities of the following materials were prepared to construct the field seal-coat strips:

- Untreated quartzite
- Untreated CRS-2 emulsion
- Treated quartzite with REDICOTE 82-S
- Treated quartzite with HIB-7178
- Treated emulsion with REDICOTE 82-S
- Treated emulsion with HIB-7178

FIGURE 3 Spreading emulsion on test strip.

CONSTRUCTION OF TEST SECTIONS

To meet the objectives of this research, a total of five treatments were chosen (see Table 1); for each treatment, two identical seal-coat strips were constructed across a traffic lane. Each strip measured 3 ft in width and 11 ft in length (the width of a traffic lane).

Test sections were constructed in the following steps.

1. The road surface was cleaned and a strip 3-ft wide was chalked on the pavement.
2. The emulsion (plain or treated) was kept at 110°F for 2 hr before it was spread by hand and leveled by a special tool, as shown in Figure 3. The rate of application was controlled at 0.3 gal/yd^2 (the residual asphalt rate was about 0.22 gal/yd^2).
3. When the emulsion started turning black from its original brownish color, aggregate was added manually at a rate of 30 lb/yd^2 and spread evenly over the emulsion.
4. A pneumatic tire roller with 50 psi tire pressure provided the required compaction through four passes.

All 10 test strips were constructed about 25 to 30 ft apart on the right lane of a major arterial (10,000 vehicles per day). Having all test strips within one very long block ensured that each strip was exposed to the same traffic.

DATA COLLECTION

Test strips were cured for about 6 hr before traffic was allowed on them. Test strips also received additional compaction by traffic for one more week, which allowed aggregate particles to get situated in the seal coat under traffic and weather action. The portable British skid tester was used to measure friction values (friction number) for the test strips (treatments). Tests were taken along the wheel path near the middle of the strip, about 3 ft away from the right pavement edge.

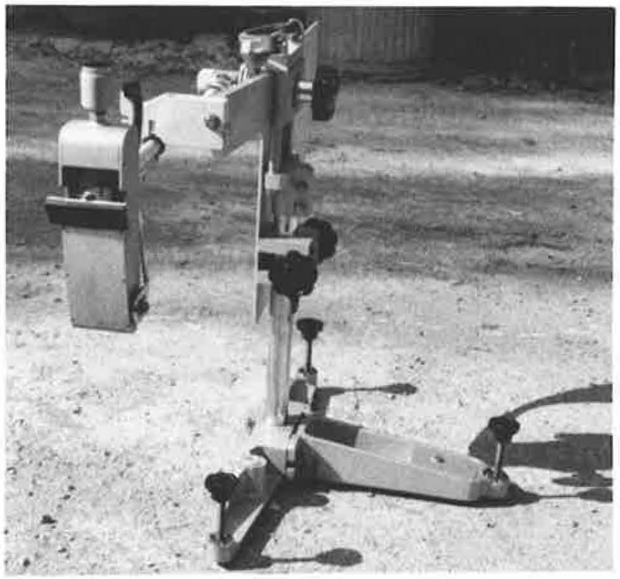

FIGURE 4 The British Pendulum Tester used for measuring friction values.

To guarantee that the exact same spot was used to measure the friction value at different times, paint markers were put on the pavement where the BPT's legs were placed when readings were taken. Figure 4 shows the BPT. Three friction values were obtained for each test spot, according to the recommended methodology established in ASTM E-303, and values modified to compensate for temperature variation were recorded. Friction values could not be measured every 2 or 3 months, as they would have to be to observe a significant decline in friction values due to traffic action, because weather conditions in South Dakota are too severe for data collection between November and March.

Final construction of all test strips was completed in the middle of August and the first set of readings was taken a week later. Further data were collected in August and October 1986 and in April, July, and October 1987. Table 2 summarizes all the data collected.

DATA ANALYSIS

To examine the rate at which friction values declined with time, several models were tried, but it soon became evident that the quadratic model and the exponential model were superior to the others for this research. Only the results from these two models are presented here in detail. The Statistical Analysis System package was employed for data analysis and model development.

Quadratic Model

Table 3 and Figure 5 are self-explanatory; they represent the results of the quadratic equation.

TABLE 2 FRICTION VALUES

Treatment \ Date	Aug. 86	Oct. 86	Apr. 87	July 87	Oct. 87
A-1	65,65,67	65,64,64	54,56,54	53,52,52	51,51,51
A-2	58,58,59	58,58,57	53,52,52	54,53,53	50,52,51
B-1	70,72,74	71,68,68	59,59,57	54,55,54	55,54,54
B-2	71,72,72	71,69,69	55,55,55	55,54,54	55,54,54
C-1	70,70,71	67,66,66	57,56,56	56,56,56	56,56,56
C-2	74,74,72	69,65,68	59,57,59	58,60,58	58,56,57
D-1	70,70,70	66,66,66	60,59,59	58,57,57	58,56,55
D-2	67,68,68	62,61,60	57,57,58	57,57,56	56,56,56
E-1	66,66,67	67,67,64	54,53,52	54,54,52	52,52,52
E-2	69,69,68	65,66,65	56,54,54	56,55,54	52,52,53

TABLE 3 QUADRATIC EQUATION MODELS

TREATMENT	R^2	MODEL*
A	0.75	$S = 63.9 - 1.39\ T + 0.035\ T^2$
B	0.96	$S = 75.8 - 2.97\ T + 0.101\ T^2$
C	0.95	$S = 74.4 - 2.83\ T + 0.110\ T^2$
D	0.89	$S = 69.9 - 1.97\ T + 0.071\ T^2$
E	0.94	$S = 70.8 - 2.45\ T + 0.082\ T^2$

*S = $a + a_1T + a_2T^2$, where

S = Predicted Friction Value

a = Intercept

a_1, a_2 = Coefficients

T = time in months from construction date

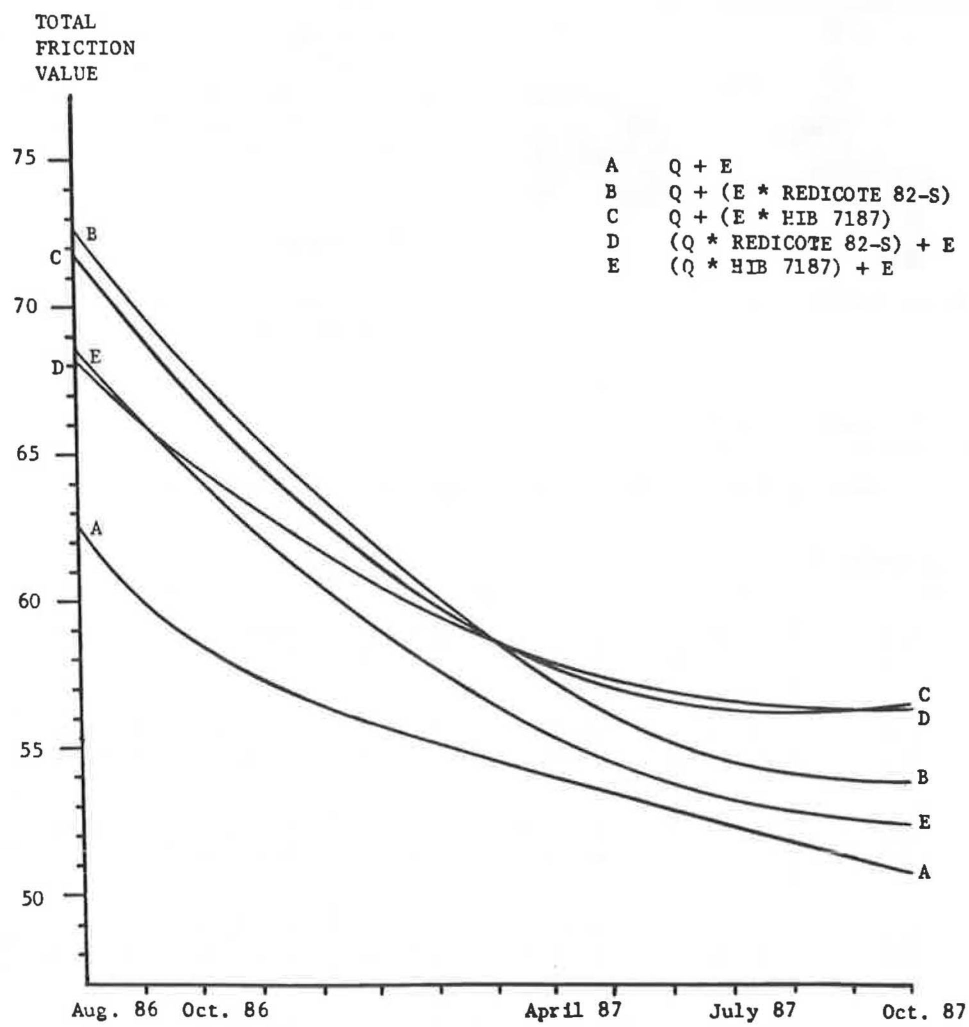

FIGURE 5 Friction values as predicted by the quadratic equation.

TABLE 4 EXPONENTIAL EQUATION MODELS

TREATMENT	R^2	MODEL*
A	0.84	$S = 50 + 14.98e^{-0.160\,T}$
B	0.86	$S = 54 + 25\,e^{-0.249\,T}$
C	0.81	$S = 56 + 17.55\,e^{-0.188\,T}$
D	0.81	$S = 56 + 13.5\,e^{-0.195\,T}$
E	0.85	$S = 52 + 20.05\,e^{-0.200\,T}$

*S = S_{min} + be $^{-cT}$

S = Predicted Friction Value

S_{min} = Minimum friction value as predicted by the quadratic model

b,c = coefficients

T = Time in months from construction date

Exponential Model

The analysis outlined below led to the exponential model.

The general form of the model is

$$S = S_{min} + be^{cT} \quad (1)$$

where

S = friction value at any time T;
S_{min} = stabilized value of friction;
b, c = coefficients; and
T = time in months from construction date.

To determine the value of S_{min}, it was necessary to use the quadratic model, which reads

$$S = a + a_1T + a_2T^2 \quad (2)$$

where

S = friction value at any time T;
a = intercept;
a_1, a_2 = coefficients; and
T = time in months from construction date.

By taking the first derivative and equating it to zero, the time it takes the treatment to stabilize can be determined (T'). The term T' can be substituted in the model to solve for the lowest friction value (S_{min}), as follows:

$$dS/dT = a_1 + 2a_2T = 0 \quad (3)$$

$$T' = -a_1/2a_2 \quad (4)$$

$$S_{min} = a + a_1\,(-a_1/2a_2) + a_2\,(-a_1/2a_2)^2 \quad (5)$$

$$S_{min} = a - a_1^2/4a_2$$

Now that the stabilized value of friction is known (S_{min}) for each treatment at time T', the following analysis can complement the development of the exponential model. From Equation 1,

$$S - S_{min} = be^{cT}$$

$$\ln(S - S_{min}) = \ln b + cT \quad (6)$$

This equation actually represents a linear model with intercept equal to ($\ln b$) and slope equal to c. Once the linear model is developed, the values of b and c can be determined, and finally the exponential model will read

$$S = S_{min} + be^{cT} \quad (7)$$

Table 4 and Figure 6 summarize the models for each treatment and the correlation coefficients.

CONCLUSIONS AND RECOMMENDATIONS

The use of antistripping additives should not be limited to hot mixes. This research demonstrates that antistripping additives can also improve the field performance of seal coats by improving the coat's total bond and friction value. Friction value is determined by both aggregate surface texture and aggregate particles' resistance to rotation when a horizontal drag force is applied. The main conclusions drawn from this research are as follows:

1. Antistripping additives, whether added to the aggregate or to the emulsion, can enhance the total friction value of a seal coat. The following observations arise from examination of the total friction values in Table 2.

- Treatments B, C, D, and E, where quartzite and antistripping agents were part of the seal coat, yielded higher total friction values than the control Treatment A, which contained quartzite but no antistripping agent. The average gain was about 8.6 percent in total friction value.
- When the emulsion was treated with the antistripping

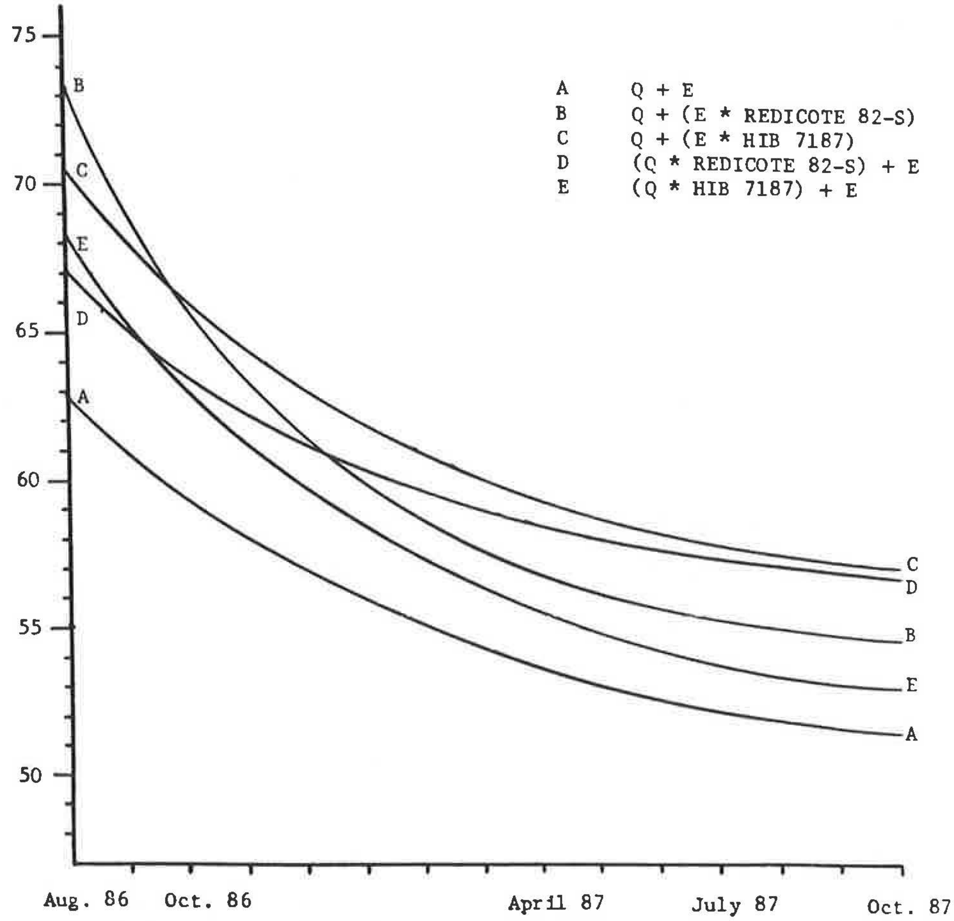

FIGURE 6 Friction values as predicted by the exponential equation.

agent (Treatments B and C), the average gain in total friction value over that of control Treatment A was about 10.3 percent.

- When quartzite was treated with the antistripping agent (Treatments D and E), the average gain in total friction value over that of control Treatment A was only 6.9 percent.

2. Antistripping agents should be added to the emulsion instead of applied to the aggregate surface for the following reasons.

- The treated emulsion (Treatments B and C) yielded higher friction values.
- In a massive production of seal coats where either emulsion or aggregate need to be treated with an antistripping agent, it is much easier to add the agent when the emulsion is manufactured (during the asphalt phase). Doing so ensures better disbursement of the agent with a minimum amount of work. Moreover, if the agent is added to the aggregate, special equipment is needed to dilute the agent and apply it to the aggregate surface, a relatively complex and expensive operation.

3. Quadratic models can successfully predict the friction value of seal coats. Correlation coefficients for quadratic models were generally higher than their counterparts for exponential models.

ACKNOWLEDGMENTS

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Research Program for Predicting the Frictional Characteristics of Seal-Coat Pavement Surfaces

MOHAMED-ASEM U. ABDUL-MALAK, A. H. MEYER, AND D. W. FOWLER

Numerous factors, including aggregate characteristics, construction variables, traffic volume, and environment, are believed to affect the frictional performance of highway pavements. In this research, the effects of these factors on the field frictional resistance of seal-coat surfaces are investigated. The investigation reported here involved establishing seal-coat test sections in different climatic regions in the state of Texas with various aggregate types and sources and under different traffic volumes. Samples of the aggregates used were examined in the laboratory to determine their physical properties, polish and wear characteristics, resistance to weathering, resistance to impact and abrasion, and petrographical and mineralogical qualities. Field tests for measuring friction and texture are being performed on the surface of test sections twice a year at random intervals. Probabilistic prediction models resulting from the study will provide an engineering solution whereby the frictional life of a seal-coat surface can be predicted or the characteristics of the aggregate required to maintain a given level of frictional resistance can be determined during the design phase.

The lack of skid resistance of highway pavements, particularly wet pavements, is a serious problem of increasing concern to highway engineers and researchers. As traffic speeds and average daily traffic (ADT) continue to rise, the chances of skidding accidents and their attendant consequences are growing at an alarming rate with each passing year (*1,2*).

Many variables contribute to skid resistance. These include pavement surface friction, pavement microtexture and macrotexture, construction variables, drainage properties of the surface, traffic volume, environment, highway geometrics, vehicle speed and load, tire-tread depth and inflation pressure, driver experience, and rainfall intensity.

Lack of pavement surface friction has long been recognized as the primary factor in skidding (*3–5*). The use of polish-resistant coarse aggregates or other aggregates with good frictional performance has always been considered a useful remedy. The Materials and Tests Division (D-9) of the Texas State Department of Highways and Public Transportation (SDHPT) employs the polish-value (PV) test (*6*), whereby an aggregate undergoes accelerated polishing to establish the polish susceptibility of coarse aggregates incorporated in pavement work. The skid resistance test (*7*) is used by D-9 to measure the frictional resistance of pavement surfaces, expressed as the skid number (referred to in this paper as the friction number (FN)). Minimum laboratory PVs of coarse aggregates have been established and used in Texas for years for the purpose of providing acceptable pavement friction. Normally, high-traffic-volume roads require aggregates with high resistance to polish and wear, whereas low-traffic-volume roads may operate with lower polish-resistant aggregates. The current PV requirements based on ADT are as follows:

ADT	*PV*
Greater than 5,000	32
5,000 to 2,000	30
2,000 to 750	28
Less than 750	None

OBJECTIVES OF THE STUDY

This research is one phase of a study whose overall objective is to investigate and develop design criteria to provide and maintain adequate pavement friction. Specifically, these objectives are

- To develop a comprehensive, long-range strategic research plan which addresses all aspects of pavement friction, and
- To investigate the relationship between laboratory frictional properties of coarse aggregates (i.e., PV) and the frictional resistance (FN) of roads built with these aggregates.

Implied in the second objective is an investigation of the factors that help predict the friction number; the PV test by itself, or with other laboratory tests performed on the coarse aggregate, may predict the FN with a certain confidence. Investigation of the effects of traffic, environment, and other factors on any possible relationships is also included in the scope of the second objective.

SCOPE OF THIS RESEARCH

In general, providing skid-resistant surfaces for highway pavements involves developing guidelines for skid resistance and incorporating the guidelines into the design of new pavements or into the process of maintaining and rehabilitating existing

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pavements. These research efforts should be directed more toward improving the frictional resistance of existing pavements since a huge highway network already exists in Texas.

Many pavement rehabilitation methods (*8–13*), including seal-coat and hot-mix asphalt concrete (HMAC) overlays, have been used in Texas to improve the frictional resistance and other surface characteristics of the highways. In this phase of the study, the frictional resistance of seal-coat overlays is being investigated; that of HMAC will be investigated in a later phase. A seal-coat overlay is a rehabilitation method in which asphalt and aggregate are applied to a roadway surface in a layer usually under 1 in. thick; seal coats can be put on pavements of all classes, from low-volume roads to Interstate highways, but they are used mostly on rural highways.

This investigation included gathering and assimilating the pertinent literature, surveying nine selected districts in Texas, establishing seal-coat test sections with various coarse aggregates and traffic volumes, performing laboratory tests on the obtained samples and field tests on the test sections, and designing the layout of the analysis to be performed on the data.

Survey of Texas Districts

Nine Texas districts (Districts 2, 3, 4, 5, 15, 16, 18, 23, and 25) were surveyed to find out what laboratory evaluations of coarse aggregates and aggregate frictional performance were made and whether any problems were encountered. Although this research is directed toward investigating the frictional resistance of seal-coat surfaces only, the district surveys covered the use of aggregates in HMAC as well as seal-coat surfaces and the frictional resistance of the aggregates. The researchers believed that they could better rank and refine the study objectives with this broader understanding of testing policies and problems. Information sought included requirements for the PVs of aggregates used, other methods for laboratory evaluation of coarse aggregates, FNs obtained, correlation between PV and FN where applicable, visual inspection of typical sections for high and low polish values and high and low frictional resistance, and personal observations.

Findings

The following information was gathered.

- All surveyed districts use the PV test to evaluate the polish susceptibility of coarse aggregates in pavements. In seven of the surveyed districts, the four-cycle magnesium soundness test is used along with the PV test (and is preferred to the PV test in Districts 5 and 25). The minimum PV requirements in most of the surveyed districts were 32 for high-volume roads and 28 for low-volume roads. The maximum allowable loss in the magnesium soundness test (MSS) was 30 percent.
- The soundness of the aggregates is an important characteristic affecting the frictional resistance of pavement surfaces. Aggregates that had high PVs but were inadequate in soundness did not have good frictional performance on the roads.
- Districts 5, 15, and 25 allow the use of aggregates that do not meet the PV requirements only if the aggregates have good frictional performance history. District 2 preserves friction data for many seal-coat and HMAC projects, along with laboratory information on the aggregates used in those projects.
- Districts 15, 16, and 18 have set up seal-coat and HMAC test sections for investigating frictional resistance. A study conducted years ago in District 25 revealed little correlation between PV and FN but did find some correlation between sand equivalency of the surface texture and FN.
- Districts 2 and 23 do not like to use aggregate blends; District 18 does. District 2 personnel believe that although the initial FN is improved for blends, the FN eventually drops and tends to decrease to the PV of the poorer material. District 23 personnel prefer to use the low-PV aggregate and then apply a sealant of lightweight aggregate (high PV) when the FN drops below acceptable limits. However, District 18 personnel believe that blending aggregates with different PVs (e.g., 30 and 34) gives better performance than using one aggregate with a PV of 32.
- District 3 reported that an aggregate need not meet a high PV requirement because, usually, the road is resurfaced for other rehabilitation purposes before the FN drops below the acceptable limit. However, for low-volume roads, where low-PV aggregates can be used, the relationship between the drop in FN and accumulated traffic was reported to be of value.
- Factors mentioned as important for frictional resistance were stripping of aggregates in the wheel paths (District 4), flushing of asphalt in the wheel paths (District 5), and slipperiness of pavement surfaces right after rainfall (District 5). District 18 reported that the outside lanes have lower FNs than the inside lanes because traffic is heavier on the outside lanes.
- Finally, only District 2 incorporated the FN in its pavement management rehabilitation system. However, District 5 personnel believe that safety regulations will eventually call for the inclusion of FN in such a system.

OBSERVATIONS FROM OBTAINED FRICTION DATA

Friction data, collected over the past 6 to 8 yr, were obtained from several districts supplemental to those surveyed. Data from three districts were combined to yield information for four selected sources of aggregates used in HMAC surfaces with various traffic volumes. Besides FN, the data included laboratory information on PV and MSS. Graphs of the FN versus accumulated traffic per lane (*14*) are plotted in Figure 1. The aggregate types and the laboratory information are shown in Table 1.

Obviously there are performance differences among the aggregates. First, the overall performance of the sandstone aggregate, which had a high PV of 47 and an MSS in the range of 9 to 14 percent, was markedly better than that of the limestone aggregates. A comparison between the PV and

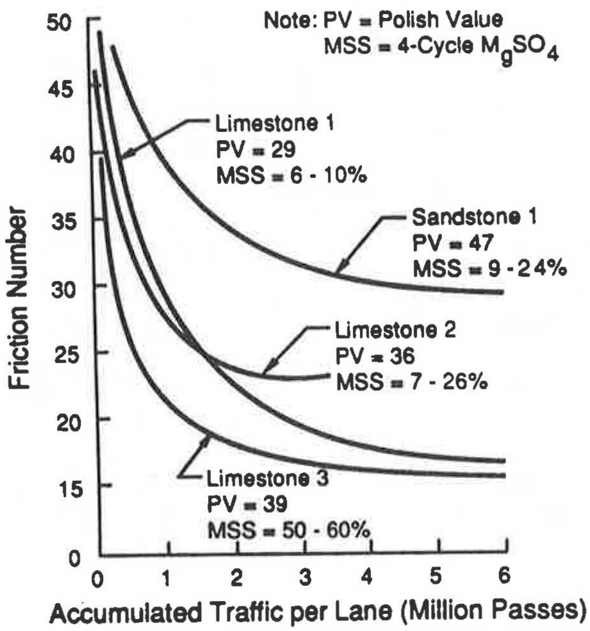

FIGURE 1 Frictional resistance of HMAC surfaces constructed with four aggregates in three Texas districts (*14*).

TABLE 1 AGGREGATES CONSIDERED IN THE ANALYSIS OF OBTAINED FRICTION DATA

	Aggregate Properties	
Aggregate Type	PV	MSS, %
Sandstone	47	9-24
Limestone 1	29	6-10
Limestone 2	36	7-26
Limestone 3	39	50-60

MSS values for the sandstone and Limestone 1 aggregates suggests that, because the MSS is less for the limestone aggregate, the sandstone aggregate's markedly better performance is most likely due to its higher PV of 47. Second, up to 2 million passes, the limestone aggregates exhibited a dramatic decrease in FN compared with the rather flattened decrease exhibited by the sandstone aggregate. Third, Limestone 1, which had the lowest MSS, maintained an FN higher than those of the other two limestone aggregates up to about 2 million accumulated passes, after which the FN of Limestone 2, with a PV of 36 and an MSS value in the range of 7 to 26 percent, flattened and remained constant. Fourth, Limestone 3, in spite of its good PV of 39, had the worst performance throughout the life of the road because it was inadequate in soundness (MSS = 52 percent). Last, the terminal FNs of Limestone 1 and Limestone 3 were about the same at 6 million passes. Yet, had the roads been resurfaced when the FN dropped below 20, Limestone 1 could have sustained twice as much traffic. In summary, good polishing and soundness characteristics both are essential to good frictional performance.

Please note that these observations must be interpreted cautiously because they are tentative and based on plots that represent best-fit curves. Although the trend of the decrease in FN was traceable, the data of each curve suffered such large, unexplained variations that overlapping between some of the data resulted. These variations could be attributable to factors unknown to the researchers.

RESEARCH METHODOLOGY

Many coarse aggregate types and sources have been used for placing seal coats on Texas highways. The major types, or categories, include crushed limestone, crushed sandstone, and crushed siliceous gravel. Other types include lightweight aggregate, limestone rock asphalt, traprock, granite, and rhyolite. Differences in field frictional resistance of these aggregates have been observed over the years (*15–21*). Numerous factors, including aggregate characteristics, construction variables, traffic volume, and environment, are believed to be major contributors to these performance differences. The objective of this phase of the study was to investigate the effects of these factors on the field frictional resistance of coarse aggregates used in seal-coat surfaces.

The methodology followed in this investigation involved preparing seal-coat test sections in different climatic regions of Texas, using as many as possible of the affordable aggregates predominant in the areas. A construction survey was made for each test section. The survey included construction variables such as design application rates of asphalt and aggregate, asphalt and aggregate type, weather condition, type and condition of existing pavement, and type of construction forces. Aggregate samples were obtained from the job sites and examined in the laboratory to determine physical properties, polish and wear characteristics, resistance to weathering, resistance to impact and abrasion, and petrographical and mineralogical qualities. Field tests continue to be made on the test sections twice a year at random intervals. Testing involves measuring surface friction and texture. Finally, annual and periodic data on average temperature, total precipitation in inches, and total inches of snow are gathered for each test section.

All information is being stored in the database being created on an IBM PC-AT. In-depth statistical analysis of the data will lead to probabilistic models that incorporate the effects of the involved variables on the friction of seal-coat surfaces.

TEST SECTIONS

Environmental Considerations

Figure 2 is a map that shows the six different climatic regions of the United States and the environmental characteristics associated with each (*22*). Texas lies within four of these regions (I, II, IV, and V), as shown in Figure 3. The environmental characteristics of each respective region are wet and no freeze, wet and freeze-thaw cycling, dry and no freeze, and dry and freeze-thaw cycling.

Seal-coat test sections have been established in all four climatic regions. Ideally, one source of each major aggregate category would have been used in all four regions so that the

REGION	CHARACTERISTICS
I	Wet and no freeze
II	Wet and freeze-thaw cycling
III	Wet, hard freeze and spring thaw
IV	Dry and no freeze
V	Dry and freeze-thaw cycling
VI	Dry, hard freeze and spring thaw

FIGURE 2 The six climatic regions of the United States (*22*).

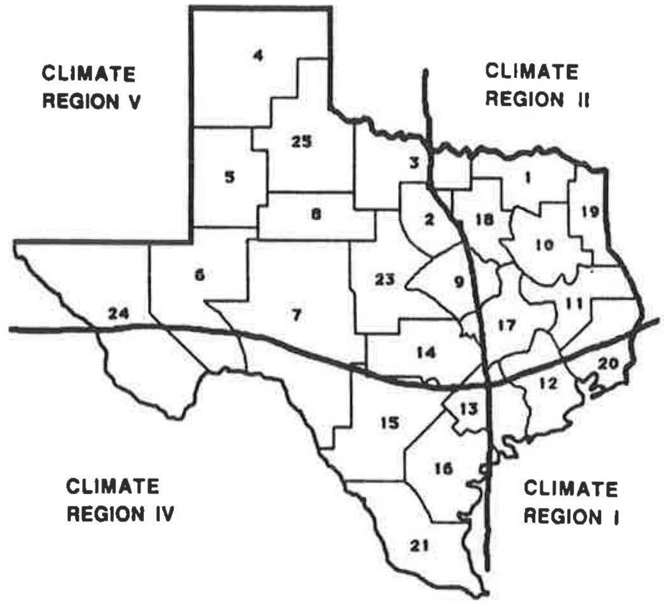

FIGURE 3 Location of the four climatic regions in Texas (*22*).

effect of climate on that aggregate category could be evaluated. No one aggregate is currently used in all four regions, however, and hauling an aggregate to distant locations is neither feasible nor practical. Instead, and to make the experimental design representative of current practice, sections were constructed using different qualities of the main aggregate categories that were locally available.

Statistical Considerations

To make the experimental design statistically sound, some requirements were established.

- Sections must be at least 1,000 ft long, to allow five friction or texture values to be measured.
- For each major aggregate type, as many as four sections with aggregates obtained from four different sources are to be constructed in each environmental region. However, the total number of test sections must be kept to a level that allows effective handling.
- Replications of sections should be established wherever possible to test for a constant variation in field responses (friction and texture) under various experimental conditions. Expected variations can result from the following:
  - Time. The quality of an aggregate pit can change over time.
  - Traffic count. The ADT is provided for a divided highway as a total figure for both directions. Replications built in both directions may clarify whether traffic is divided equally. Also, in the case of more than one lane per direction, replications are built in all lanes to account for different traffic volume in different lanes.
  - Less importantly, construction practices. Two sections are placed a few miles apart on the same lane to evaluate variations caused by construction equipment, such as changes in application rates of asphalt or aggregate.

Please note, however, that replications were constructed for only a few sections, where circumstances permitted.

Criteria for Selection

Test sections were chosen according to the following criteria.

- Have all sections be tangent sections.
- Consider only sections with a minimal slope, up to 2 percent.
- Have no major intersections within or between sections.
- Ideally, have as many sections end-to-end as the number of different aggregates used.
- Ideally, select two sections for each aggregate, one constructed by the maintenance forces and the other by a contractor.
- In the case of divided highways, have sections in only one direction, preferably the direction of heavier traffic. Where possible, make replications, as discussed earlier.

Selected Test Sections

Sixty-two seal-coat test sections were established in nine districts of the four environmental regions; fourteen of them are replications. Various aggregate types and sources were used, as shown here.

Aggregate Type	*No. of Sources*
Crushed limestone	10
Limestone rock asphalt	1
Crushed sandstone	4
Crushed siliceous gravel	8
Lightweight	5
Traprock	1
Rhyolite	2

More than 50 percent of the aggregates were non-coated; the rest were precoated. Different aggregate grades were also considered, in view of the effect gradation may have on pavement surface texture and friction.

The region, district, county, and highway designation where each aggregate was placed as well as the ADT to which the aggregate is exposed, the aggregate type, material, grade, producer, and pit were documented.

DATA COLLECTION

Field, laboratory, and weather data are being collected. The field data and the weather data are related to the established test sections and their geographical locations, respectively. The laboratory data involve the aggregates used in establishing the test sections.

Field Data

The field data constitute information obtained by monitoring the sections since the date of construction. Specifically, the data for each test section consist of a survey of construction variables performed at the construction sites, results of field testing obtained twice a year, and evaluations of a visual condition survey done concurrently with field testing.

Construction Survey Data

The construction survey consists of information on the location of each section, condition and type of existing pavement, personnel contacted at the site, type of construction forces, coarse aggregate material and asphalt type used, design application rates of aggregate and asphalt, weather conditions, and traffic volume.

Field Testing Data

The skid resistance test (*7*) and the British pendulum test (*23*) for measuring friction and the sand patch test (*24*) for measuring macrotexture are being performed twice a year on the surfaces of test sections.

The original plan, based on the literature (*25–28*), was to conduct the tests after long periods of dryness, when the pavement surface is expected to show the minimum frictional resistance and texture depth. Later, in order to detect and understand the effects of long-term seasonal variations caused by long periods of wetness or dryness, tests were performed on a random basis. The season in which the tests are undertaken is now viewed as a variable that can help explain variations in the obtained measurements.

Visual Condition Survey Data

The visual condition survey is made to determine the condition of a test section at the time of field testing. Three types of distress affect the friction and texture measurements of seal coat surfaces: poor aggregate retention, inadequate aggregate embedment, and bleeding or flushing of asphalt (*29,30*). If any of these types of distress is observed to be severe, the results of field testing will be questionable in that they might not properly represent the frictional properties of the surface aggregate. Only if a test section displays a differential or discontinuous type of distress along a wheel path may field testing still be considered, and then only on parts of the wheel path where the distress is minimal. Otherwise, monitoring of the distressed section must be terminated.

Laboratory Data

Two types of laboratory data are being collected on the aggregate samples obtained from the job site of test sections. The first type concerns the aggregates' physical properties, which are determined from the results of numerous tests performed at the engineering laboratories. The other type deals with aggregate mineralogy and petrographic characteristics gathered from examinations at the geology laboratories.

Data of Aggregate Physical Properties

Four groups of tests were found applicable for measuring the degree of deterioration an aggregate may exhibit when placed in field service. Most of the selected tests are performed on prepared samples in conformity with the test methods described in the *Manual of Test Methods* of the SDHPT. Most of these methods are modifications of standard ASTM test methods; some are procedures identical to those prescribed by ASTM. Two of the selected tests, the insoluble residue test and the aggregate durability index, are performed in accordance with ASTM standards.

Group One: Testing for Basic Properties

Sieve Analysis The test is used to determine the particle distribution, that is, gradation, of the obtained aggregate samples (31). The gradation of an aggregate is the main determinant of the texture of a seal-coat surface and thus affects surface friction.

Specific Gravity and Absorption Test The test is performed (a) according to Test Method Tex-403-A (*32*) to determine the saturated-surface/dry specific gravity and water absorption of natural aggregates and (b) according to Test Method Tex-433-A (*33*) to determine the dry bulk specific gravity and absorption of lightweight aggregates. Specific gravity and absorption together indicate the porosity of an aggregate. Both characteristics importantly influence aggregate frictional properties.

Decantation Test This test is performed in conformity with Test Method Tex-406-A (*34*). During the test, the amount of aggregate material finer than the No. 200 sieve is removed by washing and its percentage by weight is calculated. The amount of material removed may be related to the relative stability of aggregate particles in seal-coat surfaces (adhesion between aggregate particles and asphalt) and to the amount of asphalt needed to ensure a desired stability. This measure, along with some of the construction variables, may explain some of the problems pertaining to surface texture, particularly to dislodgement or loss of aggregates from seal-coat surfaces.

Group Two: Testing for Polish and Wear Characteristics

Accelerated Polish Test Test Method Tex-438-A (*6*) is employed. An aggregate is subjected to accelerated polishing to evaluate its polish susceptibility when incorporated in pavement work. The PVs of aggregates could be a helpful tool in predicting the frictional characteristics of aggregates placed in field service. The idea is based on the concept that the limiting PV of an aggregate, which is reached after 9 hr of polishing, may match or correlate well with the terminal frictional resistance of a roadway after exposure to a certain volume of traffic.

Insoluble Residue Test for Carbonate Aggregates This test is conducted in accordance with ASTM D-3042 (*35*). It estimates the amount of noncarbonate (insoluble) material in carbonate aggregate and involves a grain-size distribution of these insoluble particles. The theory is based on the concept that the frictional resistance of carbonate aggregates is related to the differential hardness of the minerals that make up the structure of the aggregate (*26*). According to this concept, when a carbonate aggregate is subjected to polish, the softer minerals will wear away at a faster rate than the harder ones. The result will leave the wearing surface of the aggregate with a rough, uneven texture, which increases or maintains the friction properties of the carbonate aggregate.

Crushed Particles in Gravel Aggregates This test is performed in accordance with Test Method Tex-413-A (*36*) and is used to determine the percentage by weight of crushed particles in aggregates. This characteristic is of interest because the asperities of the texture of crushed particles, as opposed to the smooth texture of noncrushed particles, greatly affect an aggregate's polish susceptibility.

Group Three: Testing for Resistance to Disintegration Due to Weathering Action

Four-Cycle Sodium or Magnesium Sulfate Soundness Test In this test, the aggregates are examined according to Test Method Tex-411-A (*37*) to estimate their soundness when subjected to weathering action. The aggregate samples are repeatedly immersed in saturated solutions of sodium or magnesium sulfate, which is followed by oven-drying to partially or completely dehydrate the salt precipitated in permeable pore spaces. The dehydration of salt upon reimmersion causes internal expansive forces, which simulate the expansion of water on freezing.

Coarse Aggregate Freeze-Thaw Test As in the four-cycle soundness test, the aggregates are tested to judge their soundness when subjected to weathering action. This is accomplished, as Test Method Tex-432-A (*38*) dictates, by subjecting the aggregates to 50 cycles of freezing and thawing in the presence of water. The internal expansive forces created by repeated freezing of water in the pore spaces cause the aggregate to disintegrate. This action is supposed to simulate what happens to the aggregates when they are placed in regions characterized by freeze-thaw cycling.

The soundness and freeze-thaw losses are believed to be indicative of the strength and hardness of the cementing material that holds the crystal grains of aggregate particles together.

Group Four: Testing for Resistance to Degradation Due to Abrasion, Impact, and Grinding

Los Angeles Abrasion Test The Los Angeles test provides a measure of degradation resulting from a combination of actions, including abrasion or attrition, impact, and grinding. This is done, in accordance with Test Method Tex-410-A (*39*), by placing the aggregate in a rotating steel drum with a specified number of steel spheres, the number depending upon the gradation of the test sample.

Aggregate Durability Index This test is performed in compliance with ASTM D-3744 (*40*). The test establishes the durability index—an empirical value indicative of the aggregate's relative resistance to generating detrimental claylike fines when subjected to mechanical degradation by agitation for 10 min in a mechanical washing vessel containing water.

Aggregate Degradation Test This test was developed as a part of Center for Transportation Research Research Project 3-9-85-438 for the SDHPT (*14*). Its purpose is to determine the resistance of aggregates to degradation in HMAC and seal-coat surfaces. The procedure is to subject an aggregate to mechanical degradation by agitation in the wet ball-mill apparatus in the presence of water. Degradation ensues from interparticle impact, abrading, and grinding actions.

The mechanical degradation an aggregate undergoes in this group of tests is expected to simulate (a) the impact of axle loads on aggregate particles in the wearing surface of a roadway and (b) the abrasive and grinding actions created when fines and grit accumulated on the roadway surface come between rubber tires and aggregate particles.

Data of Aggregate Mineralogy and Petrographic Examinations

The feasibility of performing petrographic analyses on the obtained aggregate samples is being investigated. The following discussion outlines the purposes, preliminary procedures, and significance of these analyses.

Purposes Petrographic examinations would be made

- To determine by petrographic methods the physical properties of an aggregate that have a bearing on the performance of the aggregate in seal-coat surfaces;
- To identify, describe, and classify the constituents of the aggregate sample; and
- To determine the relative amounts of the constituents when they differ significantly in a property, such as hardness, that may influence the frictional behavior of the aggregate when it is used in pavement surfaces.

Summary of Procedure A systematic petrographic examination of each aggregate is made under a polarizing microscope to determine the percentages of mineral constituents. The percentages are then used to determine the approximate percentage of hard mineral content (i.e., minerals harder than 5 on Moh's scale) in each aggregate. The results suggest the relationship between aggregate performance and mineralogy.

The microscopic examination also reveals information on the size and shape of crystal grains, the ground mass formation, and the grain distribution of different minerals. This information is supported by photomicrographs taken for later comparison of the aggregates.

In addition, other relevant features of the aggregate are described during the examination. These include particle surface texture and particle shape. Particle surface texture is assessed with a binocular microscope to determine the degree of roundness or grittiness the aggregate particles possess. Particle shape is evaluated by roundness and sphericity of particles. Roundness is concerned with the curvature of the corners of a particle; six classes, from very angular to well rounded, are distinguished. Sphericity is a measure of how closely the particle shape approaches that of a sphere.

Significance of Findings The results can be used in many different ways. First, by correlating or regressing the percentages of hard mineral content with the respective FNs of the aggregates, a conclusion might be reached as to whether a relationship between the two exists. Another possible finding could be what the optimum compositional proportion of hard to soft minerals should be for an aggregate to have highly favorable skid resistance. Aggregates having mixed composition of hard and soft minerals are expected to have higher skid resistance than do aggregates consisting predominantly of minerals of the same type or of the same hardness (*41,42*). The concept is that the soft ground mass wears away relatively quickly, exposing the hard grains and providing a sandpaper-like surface. Before the asperities of these hard grains lead to enough wearing action to cause them to polish, the matrix has been worn down to where it can no longer hold the hard particles, allowing them to be dislodged to expose fresh, unpolished particles. This continuous renewal of the pavement surface is believed to yield highly favorable skid resistance properties. It should be noted, however, that the influence of the compositional proportion might be modified by the effects of other features, such as size, shape, and distribution of the hard grains.

Second, the photomicrographs of two aggregates grouped in the same classification (e.g., sandstone) and with approximately the same percentages of hard mineral may reveal markedly different grain sizes. The more angular and the larger the mineral grains or crystals in individual aggregate particles, the higher the expected skid resistance of aggregate particles incorporated in pavement surfaces. Also, the coarser and more angular the hard mineral grains, and the more uniform their distribution in the softer mineral matrix, the higher the expected skid resistance.

Finally, the results of particle surface texture and particle shape may turn out to be valuable indications of micro- and macrotexture of the surfaces where aggregates are placed.

Weather Data

Climatological data from many recording weather stations in Texas are published. The primary components of the climatic description furnished by the majority of these stations include precipitation, snowfall, snow on ground, temperature, evaporation, and wind.

Annual averages of the climatic components are being compiled for each test section from the publications of the nearest recording weather station. Detailed climatological data for the periods before and during field testing are also sought. Specifically, the data include the length of the last rainfall period, the number of days between the last rainfall that occurred in that period and the day of field testing, and the total inches that fell in that period. This information will help explain the effects of short-term weather variations, caused mainly by localized showers, on the frictional properties of roadway surfaces (*43,44*).

The season, dry or wet, in which field testing is undertaken is a variable that affects the obtained field measurements and may account for long-term seasonal variations in pavement surface frictional properties caused by long periods of dryness or wetness. To properly define this weather-caused variable, detailed information on climatic, precipitation-related patterns in the state of Texas was sought (*45,46*). The state was found to have 10 climatic subdivisions formed by blocks of counties with similar amounts of rainfall. Monthly precipitation data based on the averages for the 30-year period 1951–1980 were obtained for the 10 climatic subdivisions. After manipulation, the data suggested the appropriate segmentation of a year into wet and dry seasons.

DATABASE AND STATISTICAL METHODS

All data are being stored and manipulated in the database being created on an IBM PC-AT using the Statistical Analysis System (SAS) program. In-depth statistical analysis will be performed on the data to formulate multivariable probabilistic models for predicting the frictional resistance or performance of seal-coat surfaces. The literature review revealed that under the effects of the long-term seasonal changes, the magnitude of which depends on traffic volume and aggregate type (*28*), the curves of frictional performance in numerous studies showed no consistent upward or downward trend for the annual minimum levels after about 2 yr of exposure to traffic (*21,44, 47–52*). Accordingly, a preliminary analysis may be performed on the data after the 2-yr friction measurements are obtained. The analysis will involve analysis of covariance (ANCOVA), which combines the analysis of variance (ANOVA) and regression analysis with multivariable regression analysis.

STATISTICAL ANALYSIS

The friction and texture measurements or performance responses of test sections are the dependent variables—the criterion variables. Performance is measured by FN, British Pendulum Number (BPN), and average texture depth (ATD), all of which are quantitative dependent variables. These responses will be dealt with, one by one, to evaluate how much their variations can be explained by independent variables (construction, laboratory, weather, and traffic variables). All of the performance measurements obtained for each test section and their associated accumulative traffic volumes will be used when an analysis is performed. This will be done so that the effect of traffic on changes in friction and texture over time can be better estimated.

The normality and homoscedasticity assumptions will be tested using the five readings obtained for each response and the established replications, respectively. It may happen that one of these assumptions is not satisfied. For example, the response variances may not be homogeneous. This situation can sometimes be remedied by transforming the response measurements (*53–55*). That is, instead of using the original response measurements, their square roots, logarithms, or some other function of the response might be used. Similarly, transformation will be done if the normality assumption of the response measurements is not satisfied. In fact, transformations that tend to stabilize the variance of a response have been found to make the probability distribution of the transformed response more nearly normal (*56*).

Analysis of Covariance

Design of the Experiment for the Environmental Effect

The design of this experiment is aimed toward better understanding of the effect of environment on the frictional resistance of seal-coat surfaces. In this experiment, the field responses—FN, BPN, and ATD—are the criterion, or dependent, variables, while climate and aggregate type are the main predictors considered. The layout of this experiment is shown in Table 2. The number of sections built in each region with each of the aggregate categories is shown inside the cells. These numbers actually mean the numbers of observations obtained for each of the criterion variables, with each observation being a set of five readings.

It happened that test sections for some of the cells in the table could not be established. The researchers will attempt to employ some of the approaches suggested by Dodge (*57*) to solve this problem. However, if the designed experiment is found to lose balance and not all of the usual parametric functions can be estimated, testing for the climatic effects will be incorporated only in the multiple regression analyses. This will be done by obtaining the total annual freeze-thaw cycles and total annual precipitation recorded in the weather station closest to each test section. These two weather variables will then be considered as covariables in the regression analyses.

TABLE 2 DESIGN OF THE EXPERIMENT FOR THE ENVIRONMENTAL EFFECT

		Climatic Zones			
Aggregate Type		I	II	IV	V
Limestone	PV < 30	1[b]	-	1[b]	1
	PV > 33	-	2[a]	-	2[b]
Sandstone	PV ~ 40	-	4[a]	-	-
Siliceous Gravel	PV < 30	1	-	-	1
Limestone Rock Asphalt		4[c]	2[c]	3[c]	-
Lightweight		5[c]	-	-	-

[a] Traffic count and/or construction replication(s)
[b] Time or pit replication(s)
[c] Replications that comply with both "a" and "b"

Formulation of Model

The model generated by applying the ANCOVA technique has the following form:

$$Y = AGE + ADT + WTV + CSV + CLR + AGT + CLR * AGT + R(CLR\ AGT)$$

where the criterion variable Y could be Y_1 (FN), Y_2 (BPN), or Y_3 (ATD). The predictors are CLR (climatic regions) and AGT (aggregate type). $CLR * AGT$ is the interaction term between climatic regions and aggregate type. $R(CLR\ AGT)$ is a term used to account for the fact that replications exist for some aggregates within regions. The covariables are AGE (age of section), ADT (average daily traffic), WTV (weather variables), and CSV (construction variables).

Regression Analysis

Formulation of Prediction Model

In general, this analysis is intended to find the best general linear regression model of the type

$$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_k x_k + \varepsilon$$

to describe the relationship between the frictional performance of seal-coat surfaces (y) and all of the independent variables involved ($x_1, x_2, \ldots, x_k$). Since an explanation of causal effects of each independent variable is the primary thrust of this investigation, the stepwise regression procedure, an option in the SAS program, is used for building the prediction model.

Multicollinearity

Multicollinearity is a problem that arises when two or more of the independent variables are found to be highly correlated to one another. When such a problem is encountered, the respective individual contribution of the correlated variables to the reduction in the error sum of squares cannot be determined. If two variables contribute overlapping information, the first β parameter may be overestimated, whereas β_2 tends to be underestimated (*56*). In fact, multicollinearity may even cause the algebraic sign of one or more regression parameter estimates to be contrary to logic. Thus, if a multicollinearity problem arises, two approaches may be employed (*56,58*). The first is to drop one of the two correlated variables from the equation and to reestimate it; this can cause bias in the reestimated model, but it may be justified if the bias can be argued to be small. The second approach is to combine the two variables into an index variable by standardizing their effects; the variables should be conceptually and theoretically related for this approach to be used.

Measuring the Goodness-of-Fit of the Model

The term R^2, the multiple coefficient of correlation, provides a measure of the fit of the multivariable regression model. That is, R^2 gives the proportion of the total sum of squares that is explained by the predictor variables. The remainder is explained by the omission of important information-contributing variables from the model, an incorrect formulation of the model, and experimental error. R^2 takes values in the interval $0 \leq R^2 \leq 1$.

A small value of R^2 means that the predictor variables contribute very little information for the prediction of frictional performance; a value of R^2 near 1 means that the predictor variables provide almost all the information necessary for the prediction of frictional performance.

A relatively poor fit of the model (a small R^2) may result if the predictor variables are not entered properly into the model (perhaps interaction terms x_1x_2, x_1x_3, x_2x_3, etc., and quadratic terms x_1^2, x_2^2, x_3^2, etc., should be included), or perhaps frictional performance is a function of many other variables besides the ones already considered. Interaction terms are considered in the analysis for their ability to contribute information for the prediction of frictional performance, and residual analysis is performed to identify new variables that may improve the fit of the model.

Residual Analysis

Residual analysis, a capability of the SAS program, examines the degree to which the model satisfies the random-error assumption of multivariable regression analysis and thereby suggests the inclusion of additional variables that may improve the fit of the model (*56,58*). The analysis involves plotting the residuals against each independent variable. In some cases a residual plot might suggest the inclusion of a second-order term, say x_2^2, into the analysis as an additional independent variable. Another plot might depict a case where the variance in the response (frictional performance) increases proportionally to the independent variable x_3. Usually the addition of the variable $\log(x_3)$ will accommodate this problem (*53*).

Investigation of Other Relationships

Regression analysis is used to investigate the relationships that may exist among the field responses, FN, BPN, and ATD, in order to determine how microtexture and macrotexture, reflected by BPN and ATD respectively, influence the traditional friction measure, FN. Also, a friction curve for each aggregate source is developed that shows how friction decreases with accumulation of traffic. The curves are generated by regressing friction against accumulative traffic and weather-related variables.

USE AND SIGNIFICANCE OF FINDINGS

The prediction model will be of value in three ways:

1. It can be used to estimate the mean value of frictional performance for given values of the predictor variables.
2. It can be used to predict some future value of frictional performance for given values of the predictor variables.
3. If the predictor model provides a good fit to the set of

data (if R^2 is large) and the number of predictor variables is not too large, the model will help the engineer or researcher understand the relationship between the predicted value of frictional performance and the set of predictor variables. For example, if two predictor variables, say PV and four-cycle soundness, are found to be interacting, the model will show how the relationship between frictional performance and PV is dependent on the soundness loss.

The significance of the model lies in the fact that it will provide a method for nominating, at the planning or design phase of seal-coat projects, the physical properties and petrographic characteristics of the aggregate as well as the design values of the construction variables needed to provide or maintain a given acceptable frictional resistance under a certain projected traffic volume in a specific climatic region. In addition, the findings will likely have an immediate impact on specifications relating to aggregate laboratory preconditioning methods for the benefit of frictional resistance.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be drawn at this stage of the study:

- In Texas, the PV test is the most widely used method for evaluating the polish susceptibility of coarse aggregates used in pavements. The four-cycle soundness test is used along with the PV test in most of the surveyed districts.
- The soundness of the aggregates was found to be an important characteristic affecting the frictional resistance of highway surfaces. The findings of the survey of Texas districts and the analysis of obtained friction data indicated that aggregates with high PVs but inadequate soundness did not have good frictional performance.
- Petrographic tests may prove to be very useful in selection of aggregates if such tests or a combination of test results can be correlated with field performance.
- According to the literature, long-term and short-term seasonal changes are a major cause of variation in the frictional resistance of highway surfaces. In addition, in many studies the rejuvenating effects of wet periods appeared to offset the polishing effects of dry periods in that the curves of frictional performance showed no consistent upward or downward trend for the annual minimum levels after about only 2 yr of exposure to traffic. The magnitude of variation was found to be strongly associated with traffic volume and type of aggregate. Pennsylvania State University has developed models that treat these seasonal variations, but it has been suggested that those models be used only in the geographical area in which the investigation was conducted.
- There is not yet any relationship that can reliably predict field frictional resistance from aggregate properties. This is largely because no research has attempted to relate field friction to microtexture and macrotexture laboratory properties. The literature repeatedly stated that the inclusion of a field-measured macrotexture variable in predicting models would not serve any purpose in design (the current art of construction methods cannot assure a predesigned macrotexture). Another reason for the lack of reliable friction-predicting models is that the effects of seasonal variations have never been corrected.
- Considering all of the laboratory and petrographic tests relevant to determining the aggregate properties that hypothetically influence the frictional performance of seal coats may well suggest which tests (or properties) ought to be used to evaluate an aggregate.
- Because macrotexture's effect on frictional resistance is undisputed and including it in the formulation of prediction models decreases their design value, the methodology of this study introduced, instead, the factors that contribute to the formation of macrotexture (aggregate gradation and shape, application rates of asphalt and aggregate) and those believed to govern the rate of wear in such texture under traffic exposure (resistance to abrasion, soundness, petrographic properties, and others).
- The randomized selection of the seal-coat projects and aggregate sources for the construction of test sections points to the areas in which the results of this study may usefully be implemented.
- The construction of test sections end-to-end as number of different aggregates used is very convenient for performing field testing. Sections 1,000-ft long are of sufficient length to make five friction and texture measurements.
- The climatological data being collected and the tentative segmentation of a year into dry and wet periods are expected to help account for the variability in frictional resistance caused by short- and long-term seasonal variations, respectively.

Only a few recommendations can be made at this stage of the study.

- Until a reliable relationship is established between field frictional resistance and aggregate characteristics, the selection or evaluation of aggregates on the basis of the PV and soundness requirements, along with the frictional performance history (if available), should be continued.
- When considering whether to resurface an existing pavement, decision makers should rely upon friction measurements taken in the dry period or periods of the climatic division where the roadway is located.

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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

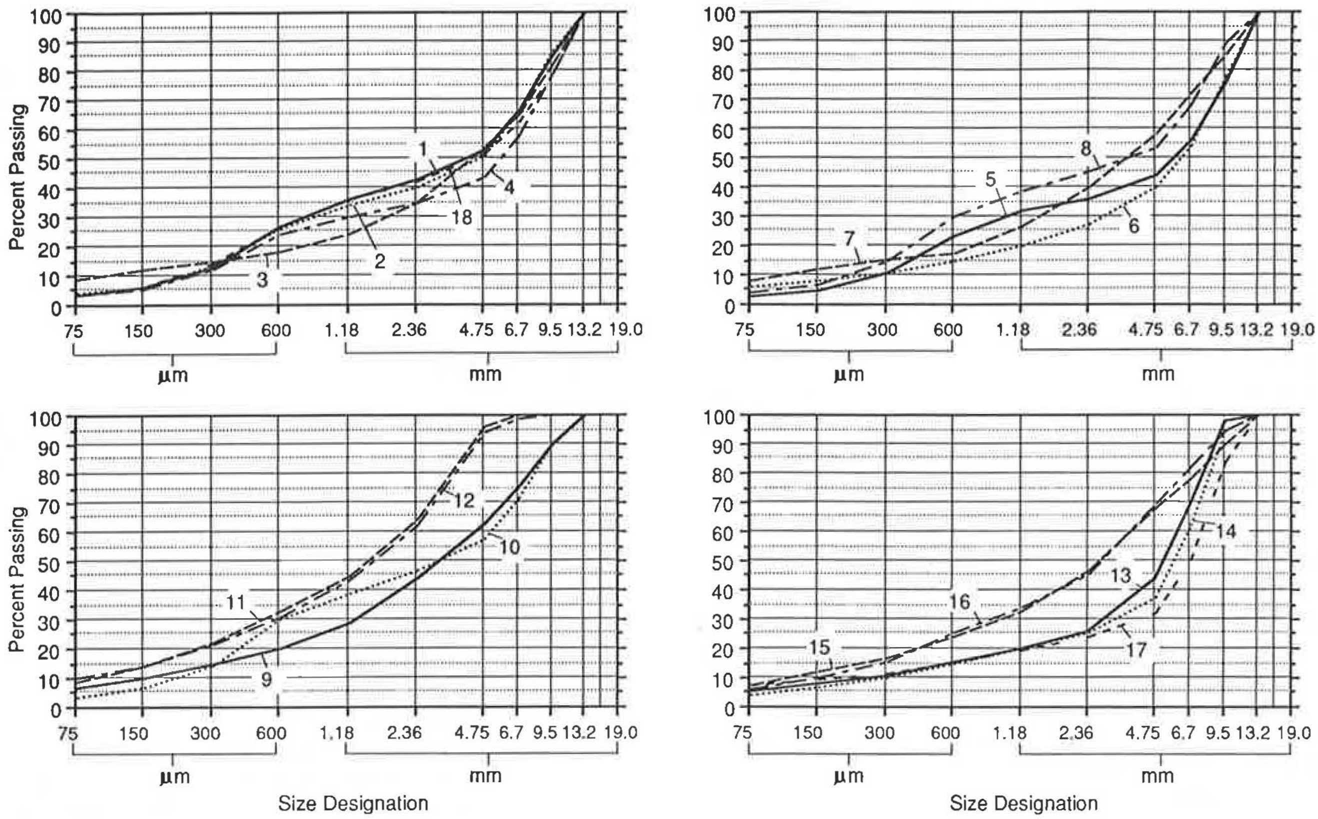

FIGURE 1 Aggregate gradation from cores taken at 11 years (average of cores from the three lanes).

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 NO.	TYPE OF DISTRESS							JOINT REPAIRS(m)				
	TRANS. CRACKS	LONG. CRACKS	RAVEL--LING	PATCH	DELAMI--NATION	RUTTING	OVERALL RATING	CRACKS TRANS.	CRACKS LONG.	CRACKS TOTAL	SHOULDER LEFT	SHOULDER 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-

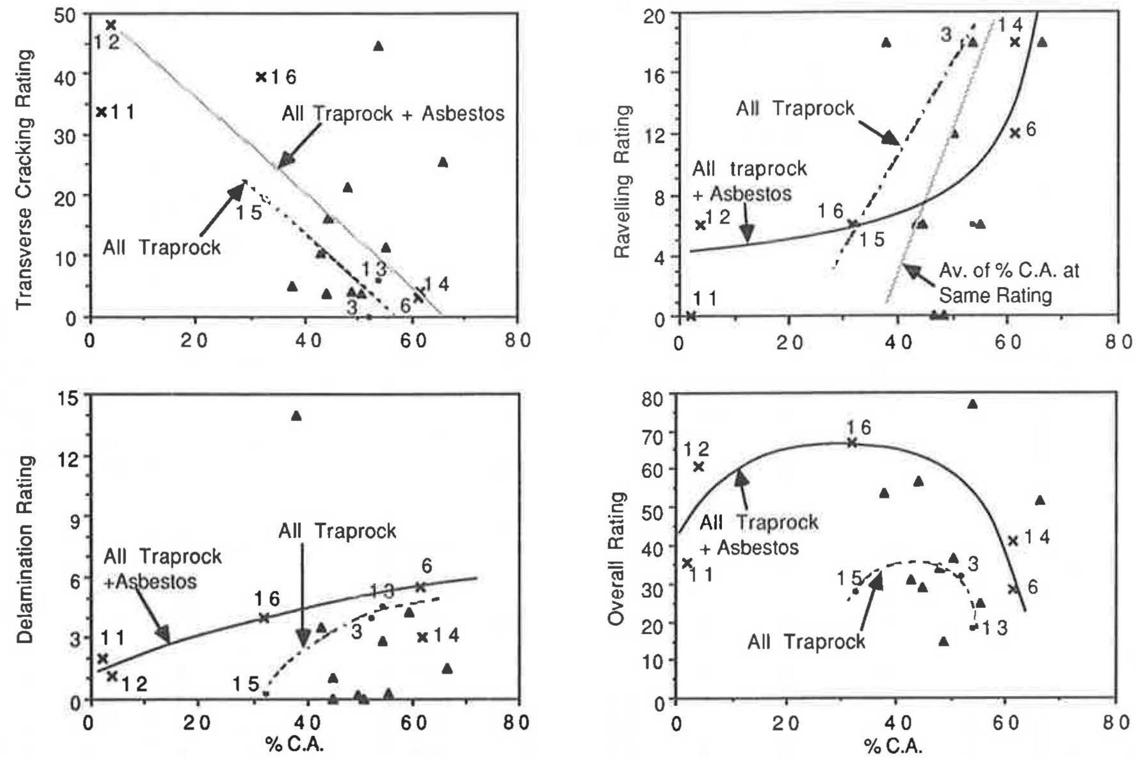

FIGURE 2 Influence of coarse aggregate content (% CA) on the different performance parameters (number = section no.; ▲ = other mixes).

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

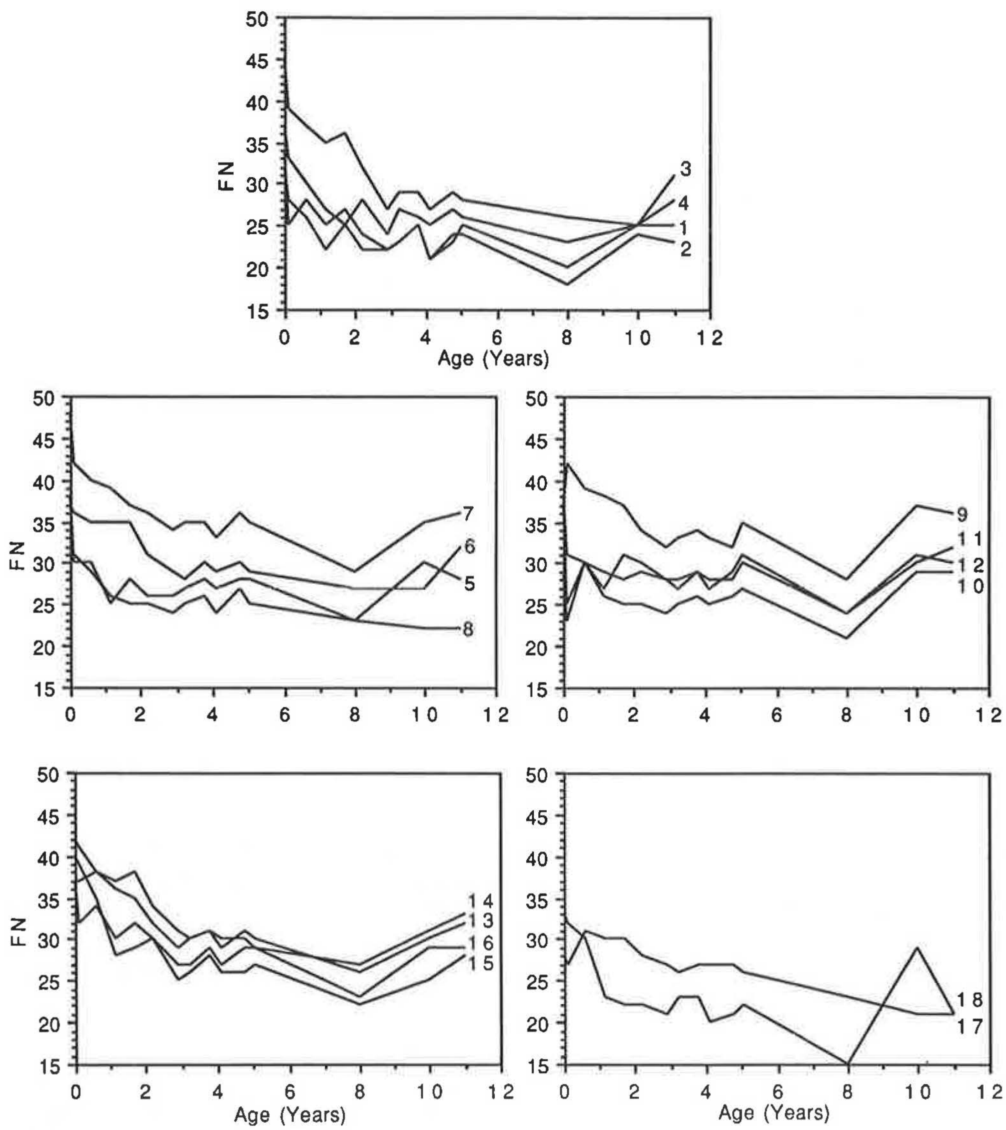

FIGURE 3 Friction number (FN) at 100 km/h at different age of the pavement in the driving lane.

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.

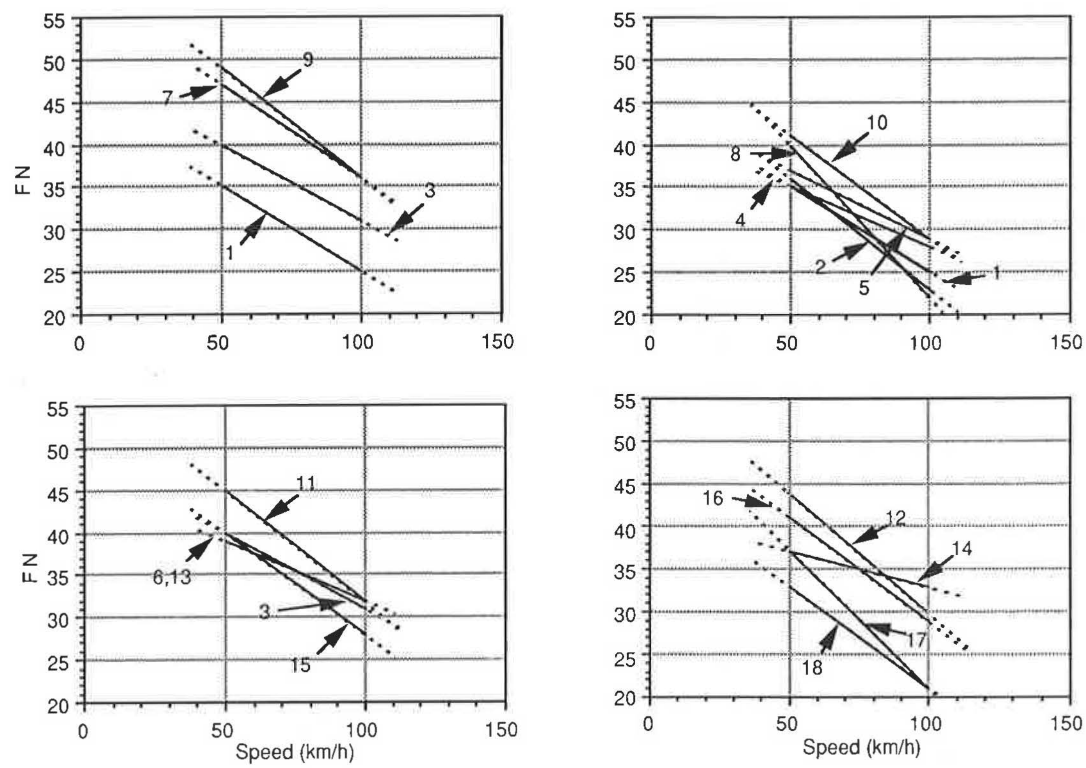

FIGURE 4 Influence of speed on the friction number (FN) for different mixes with various degrees of macrotexture (after 11 years of service).

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.

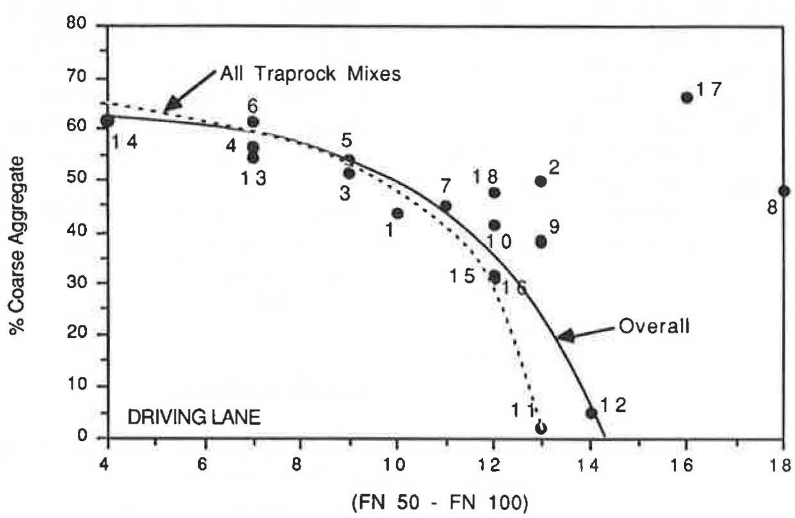

FIGURE 5 Relationship of coarse aggregate content (% CA) and difference in friction number at 50 and 100 km/h.

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.

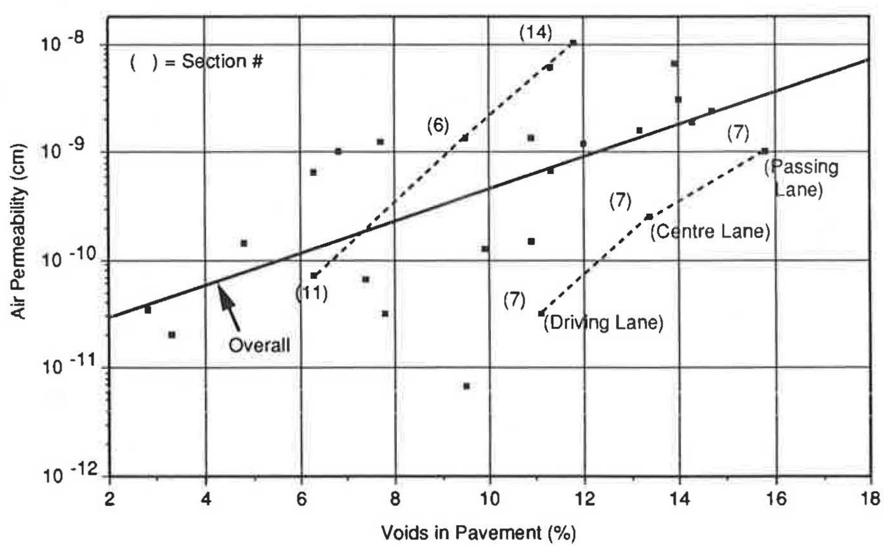

FIGURE 6 Influence of air void content on the air permeability of asphalt pavement materials.

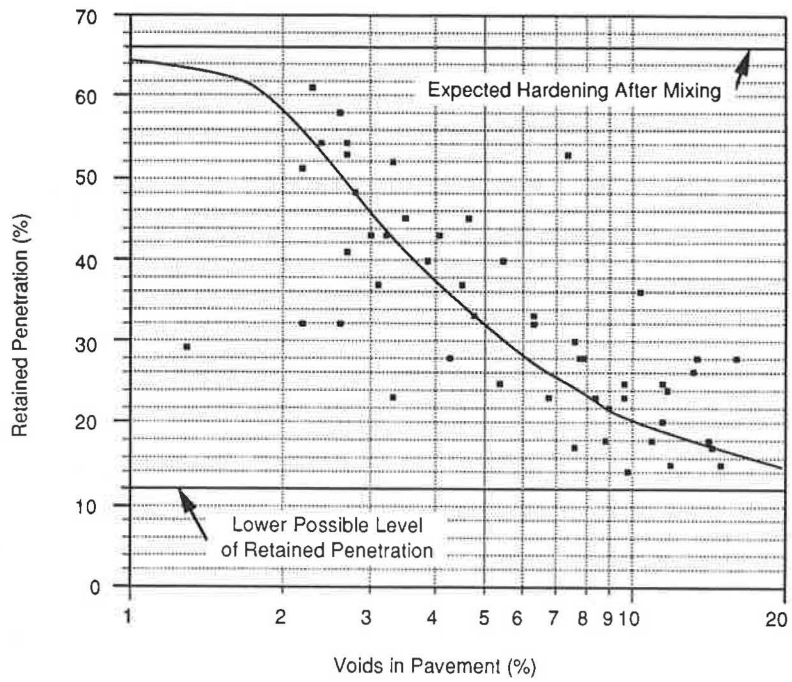

FIGURE 7 Influence of air void content (%) on the retained penetration (%, 11 yr/original value*100) of recovered asphalt cement from the test sections after 11 years of service.

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.

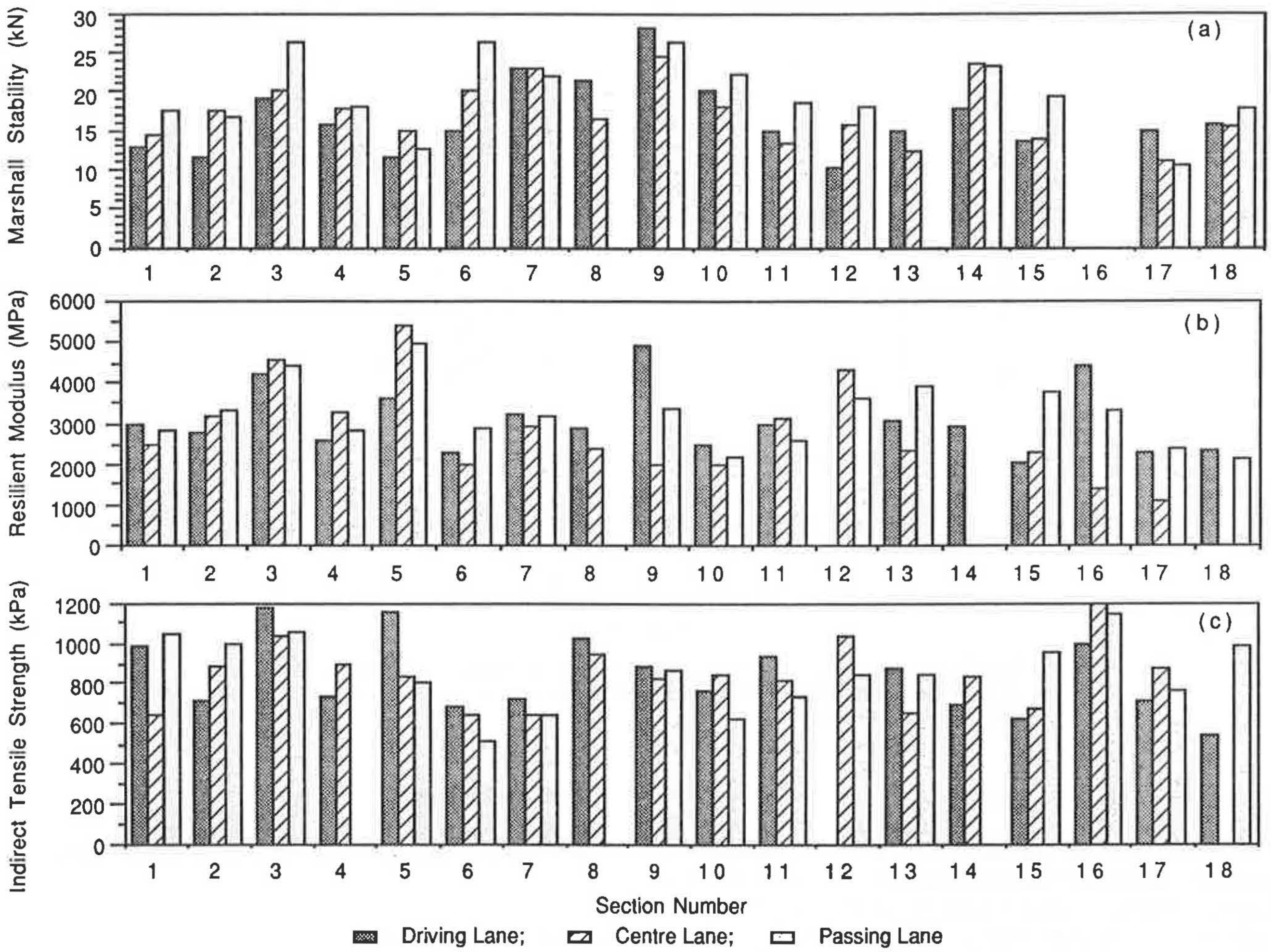

FIGURE 8 Marshall stability, resilient modulus, and indirect tensile strength of different test sections.

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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Asphalt Overlays of Concrete Pavements: Case Studies in Arkansas

DAN FLOWERS, JIM GEE, AND ALAN MEADORS

The rutting problems on two sections of Interstate highway in Arkansas are discussed in this paper. The original asphalt concrete overlays of concrete pavement placed in the late 1970s consisted of a total asphalt thickness of approximately 8½ in. Immediate rutting on one of the overlay jobs led to a reevaluation of Arkansas mix design practices. Modifications made in the mix designs included changing to a 75-blow Marshall design and following AASHTO T245 (Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus) with no alterations. The rutted overlay was milled and surfaced using the new mix design practice. After 6 yr of service, the new overlay is again performing poorly. The other asphalt concrete overlay in another part of the state, however, continues to perform well after 12 yr of service. Other factors not addressed by the Marshall design procedure must therefore have contributed to the difference in pavement performance between the two projects.

Arkansas Interstate highways were completed in their entirety using 8–10 in. of portland cement concrete pavement over various types of bases. Rehabilitation began in the mid-1970s and now 95 mi of the original 542 mi of concrete pavement have been overlaid with asphalt concrete hot mix (ACHM). Construction of the first asphalt overlay on the Interstate highway began in 1974. A typical section of that project consisted of 300 lb/yd² of crushed stone bituminous base course, otherwise known as the crack-relief layer, 415 lb/yd² of ACHM binder course, and 165 lb/yd² of ACHM surface course. This overlay began rutting shortly after construction was complete. The average rut depth was 9/16 in. While the cause of this rutting was under investigation, construction began on a similar overlay of 16 mi of Interstate 40 in East Arkansas. This is the project that will be discussed here.

The I-40 overlay contract was awarded in July 1975 for $6.6 million. Approximately 65 lane-mi of Interstate was overlaid in a four-lane section from Widener to Shearerville. In 1975 the truck volume at this site was 5,180 trucks/day. In 1988 the truck volume had grown to 8,170 trucks/day. While traffic was detoured to an adjacent highway, the westbound lanes were overlaid through the surface course and opened to traffic in July 1977. Work began on the eastbound lanes; while the binder and surface were being placed, the westbound lanes began to rut. By the summer of 1978, rutting had progressed in the westbound lanes to such a state that tractor-trailer rigs were hydroplaning during and after rain showers. After completion of the surface course in the eastbound lanes, the work was suspended. The final course, an open-graded friction course, was not laid.

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MIX DESIGN INVESTIGATION

Needless to say, the Arkansas Highway and Transportation Department (AHTD) was very concerned when these first two Interstate overlays began rutting. Work was undertaken to determine the cause of this problem and the changes necessary to prevent its recurrence. The Asphalt Institute and Federal Highway Administration worked closely with the department in reviewing the then-current Arkansas mix design and pavement construction practices. This review, coupled with research into the rutting problem, led to many modifications in design and construction practices.

Three types of rutting were occurring:

1. Excessive traffic consolidation in the upper ACHM course due to poor compaction during construction.
2. Plastic deformation due to insufficient mix stability and low void content.
3. Deformation due to instability, which was caused by stripping in the mix.

The asphalt surface course placed on Interstate 40 was composed of limestone coarse aggregate and natural sand. A modified Marshall 50-blow mix design of 22 percent natural sand, 5.3 percent asphalt cement, and 1.5 percent air voids was used for the westbound lanes; eastbound lanes used a mix of 30 percent natural sand, 5.0 percent asphalt cement, and 3.5 percent air voids. An initial look at the cores taken from the westbound lanes of Interstate 40 led to doubts about the stability of the crack-relief layer because some binder seemed to have penetrated that layer. However, the occurrence of binder in the crack-relief layer did not correlate well with rut depth. Furthermore, samples taken from the roadway showed that the binder and surface had less than 0.5 percent air voids in the wheelpaths and less than 1.5–2.0 percent air voids between the wheelpaths. This finding indicated that the binder and surface were responsible for this rutting. It was obvious that changes were needed in mix design procedures and construction practices before any more overlays were placed on the highway.

The mix design procedure used during the construction of the I-40 overlay was an Arkansas modified Marshall mix design procedure. The maximum theoretical specific gravity was a calculated effective specific gravity based upon bulk and apparent aggregate-specific gravities. The combination of the design procedure and the calculated effective specific gravity resulted in ACHMs that contained a high percentage of asphalt cement by weight of total mixture with questionable void content. These mixes are quite durable but can become plastic due to low voids, and rutting can result.

The aggregates that the contractor proposed to use were required to be hard, tough, durable, and 100 percent crushed particles (coarse aggregate), free from an excess of soft particles. In addition, the aggregates had to be uniformly well graded from coarse to fine and free from an adherent film of clay. When the proposed aggregate complied with these requirements, AHTD inspectors sampled the aggregate stockpiles and submitted samples along with stockpiled aggregate gradation to the central laboratory for design. (Historically, all asphalt concrete hot mixes have been designed in the central laboratory by the Materials and Research Division of the AHTD.) The aggregates were proportioned to develop a trial job-mix gradation, which was within the master range of the specifications. A modified Marshall mix design procedure was used to establish the optimum Marshall properties and, indirectly, the optimum asphalt cement content. This design procedure had been used since 1952.

Summary of Old Mix Design Procedures

- Except for mixes for North Arkansas, where an 85–100-penetration-grade asphalt was used, all mixes were designed using the trial mix gradation with a 60–70-penetration-grade asphalt cement from a standard source. The contractor was allowed (with proper documentation) to switch to another asphalt cement, without a mix redesign, so long as the new asphalt cement was a 60–70-penetration-grade from an approved source.
- Aggregated and asphalt cements were heated to mixing temperature (400°F and 300°F, respectively) on electric hotplates.
- Mixing was accomplished by hand.
- The Marshall specimen molds and Marshall compaction hammer were not heated.
- Air-void content in the compacted Marshall specimen was determined by comparing the specimen's bulk specific gravity with an effective maximum specific gravity of mixture, which was established by averaging the asphalt-cement specific gravity, +No. 4 aggregate bulk specific gravity, and −No. 4 aggregate apparent specific gravity.
- The ACHM mixture was compacted at a temperature above 300°F. The actual mix temperature at the time of compaction was not established.
- The Marshall specimens were extruded, whenever convenient, from the molds by using a hand-held Marshall hammer.
- Marshall stability and flow were obtained according to standard Marshall procedure using regular Marshall equipment.

Summary of Old Construction Procedures

- The asphalt cement was allowed to be heated to a maximum of 300°F before it was introduced into the plant.
- Aggregates were allowed to be heated to 350°F before being introduced into the plant.
- A minimum wet-mixing time of 40 sec was required. The mixture was to be discharged from the plant and delivered to the roadway at a temperature between 285°F and 325°F.
- The finished ACHM surface courses were to be compacted to 92 percent and maximum theoretical specific gravity, determined as described above.
- At least two rollers were required per project, one of which had to be a tandem roller. The rollers were required to weigh at least 8 tons, and at least one had to weigh 10 tons.
- For breakdown rolling of surface courses, a tandem steel wheel or three-wheel roller was required. Intermediate rolling was to be performed by a pneumatic roller, and final rolling by a tandem steel-wheel roller.

Implemented Changes in Mix Design and Construction Practice

Because traffic volume on Arkansas highways is increasing, AHTD developed a new specification in 1977, "Requirements for Asphalt Concrete Hot Mix Binder and Surface Courses," for highways with heavy traffic (especially heavy truck traffic). This new specification used the same gradation meter range as the regular (50-blow Marshall) surfaces and binders, but it further requires that these mixes be 75-blow Marshall and meet the design requirements shown in Table 1.

Aggregate requirements for this new specification remained the same. The stability and flow of the field mixture were to be checked daily by using the same type of Marshall equipment that is used in the central laboratory. Uniformity of the mixture was checked at least daily by performing extractions, in accordance with AASHTO T164.

A heat-stable antistrip was required in both the 75-blow binder and surface courses at a rate of application established in the laboratory by trial mixes. Before being used in ACHMs on highway projects, the effectiveness of each antistrip was to be determined by a boiling water test.

The trial job-mix gradation was established for these mixes as described earlier. The trial job-mix gradation was plotted on .45-power paper for comparison to the maximum density line of the particular ACHM course. Depending upon the relative position of the trial job-mix gradation to the maximum density line, the trial job-mix gradation was adjusted to provide sufficient voids in the mix. As a general rule, AHTD mixes are on the fine side of the maximum density line. Every effort is made to avoid paralleling the job mix gradation to the maximum density line to minimize the chances of producing a low-void, tender mix.

In 1980, a specification ("Special Requirements for Asphalt Concrete Hot Mix Binder and Surface Courses for Heavy Traffic") was developed to replace the 1977 specification. The specification has changed little since 1980 and is included in AHTD's latest standard specification for 1988. These design requirements are shown in Table 2.

Summary of 1980 Mix Design

- Coarse aggregate must be 100 percent crushed on the No. 10 sieve, and at least 70 percent of the particles on the No. 4 sieve must have a minimum of two fractured faces.
- All mix designs must accord with AASHTO T245, i.e.:
  - The ACHM binder and surface courses are 75-blow Marshall per side.
  - Mixing and compaction temperature ranges are established from the temperature-viscosity curve for the

TABLE 1 ARKANSAS ASPHALT MIX REQUIREMENTS, 1977

Test	Requirements for Surface	Binder
Mix Design		
Min. Stability (AASHTO T245)	1400	1400
Flow (0.1 inch)	8-14	8-14
Minimum VMA (%)	15	13
Air Voids (%)	3-5	3-5
Target Air Voids (%)	4	4
Field		
Min. Stability (lbs)	1200	1200
Flow (.01 inch)	8-14	8-14
Air Voids (%)	3-8	3-8

TABLE 2 ARKANSAS ASPHALT MIX REQUIREMENTS, 1980

Test	Requirements for Surface	Binder
Mix Design		
Min. Stability (AASHTO T245)	1500	1500
Flow (.01 inch)	8-16	8-16
Minimum VMA (%)	14	13
Air Voids (%)	3-5	3-6
Field		
Min. Stability (lbs)	1500	1500
Flow (.01 inch)	8-16	8-16
Maximum Air Voids (%)	8	8

asphalt cement to be used. These temperature ranges are reported on every Marshall mix design report that AHTD issues.

- Electronic digital thermometers are used to control the temperatures in the lab.
- Electronically controlled electric ovens are used to heat the aggregate, asphalt cement, and Marshall molds.
- Marshall compaction hammers are heated on hot plates.
- Aggregates and asphalt cement are mechanically mixed.
- Hydraulic jacks are used to extrude specimens from Marshall molds.
- Stability and flow values are electronically recorded by automatic-control Marshall equipment.

• Maximum theoretical specific gravity is determined by the Rice method (AASHTO T209).

• Bulk specific gravities of Marshall specimens are determined by AASHTO T166.

• Asphalt cements must meet the requirements of AASHTO M226, Table II.

• All mixes are checked to determine their sensitivity to moisture by AHTD Test Method 132, "Water Sensitivity for Compacted Mixtures."

• Antistrip agents must be prequalified by passing a boiling water coating test.

Summary of 1980 Construction Procedures

• Baghouse fines, if used, must be accurately metered into the mix.

• Heat-stable antistrip is added in a line at the hot-mix plant in a metered quantity.

• All fuel must be completely burned.

• The recommended temperature ranges shown on the mix design will yield the best results.

• Mixtures must contain no more than 0.75 percent moisture.

• Optimum rolling patterns are to be determined for each mix at the beginning of paving operations by making a 500-ft test strip.

- Field stability and flow values and target density for acceptance will be determined daily for each lot (a day's run) of mixture produced.
- Density requirements for the compacted pavement for each lot will be based on the average of five cores, which must equal or exceed 96 percent of laboratory density; no one density can be less than 91.0 percent of laboratory density.

ASPHALT MIX PERFORMANCE

Rutting progressed on the I-40 westbound lanes and in 1979, although the mix design procedures and specifications had not completely evolved, AHTD chose to mill and overlay. As an add-on to another overlay project, it was decided to mill 4/5 in., the depth of the ruts, and lay 165 lb/yd^2. This new surface was designed using the old procedure, except that the Marshall compactive effort was at 75 blows rather than 50 blows. After the overlay was complete, the original contract was resumed and completed except for the open-graded friction course in the eastbound lanes.

Two years later, in 1981, the eastbound lanes of the I-40 overlay had rutted enough to warrant rehabilitation. Using the revised 1980 design procedures and specifications, a contract was let to mill 1½ in. of the original surface and lay 165 lb/yd^2 of new surface mix as well as the open-graded friction course. This is the last rehabilitation work that has been done on this section of I-40.

In 1985, the performance of both eastbound and westbound lanes was checked. The westbound lanes, completed in 1979, had rutted to an average depth of ½ in. while the eastbound lanes averaged 5/16-in. rutting. Cores were taken from the westbound lanes to determine the reason for the recurrence of rutting. Cores taken in the wheelpaths showed that the air voids averaged 4.5 percent for the surface overlay, 0.9 percent for the original surface, and 0.2 percent for the binder (see Figure 1). Cores taken between the wheelpaths showed air void contents of 5.0 percent for the overlay, 2.7 percent for the original surface, and 0.7 percent for the binder course. The eastbound lanes were not cored. At this time, 6 yr after the 75-blow Marshall was used for the overlay, with air voids of 4.5 and 5.0 percent, it appeared that AHTD had been successful in its design and construction of an interstate highway overlay.

In 1988 the performance of both lanes was checked again. Because of continued rutting, the eastbound lanes had been milled and could not be measured. Ruts in the eastbound outside lane had an average depth of ½ in., a progression of 3/16 in. since the check in 1985.

Again, cores were taken from both the eastbound and westbound outside lanes. In the westbound lane, cores taken in the wheelpaths showed that the air voids averaged 2.4 percent for the surface overlay, 0.1 percent for the original surface, and 0.4 percent for the binder course (see Figure 2). Cores taken between the wheelpaths showed average air voids at 3.3 percent for the surface overlay, 0.5 percent for the original surface, and 0.5 percent for the binder course.

In the eastbound lane, average air voids in the wheelpaths were 1.2 percent for the overlay and 0.2 percent for the binder (see Figure 3). Between the wheelpaths, the average air voids were 1.6 percent for the overlay and 0.8 percent for the binder.

These rut depths and air void levels show that this section of I-40 now needs another rehabilitation project. The performance of the westbound lane for 9 yr and the eastbound lanes for 7 yr since their overlays in 1979 (westbound) and 1981 (eastbound) indicates that simply milling the ruts and

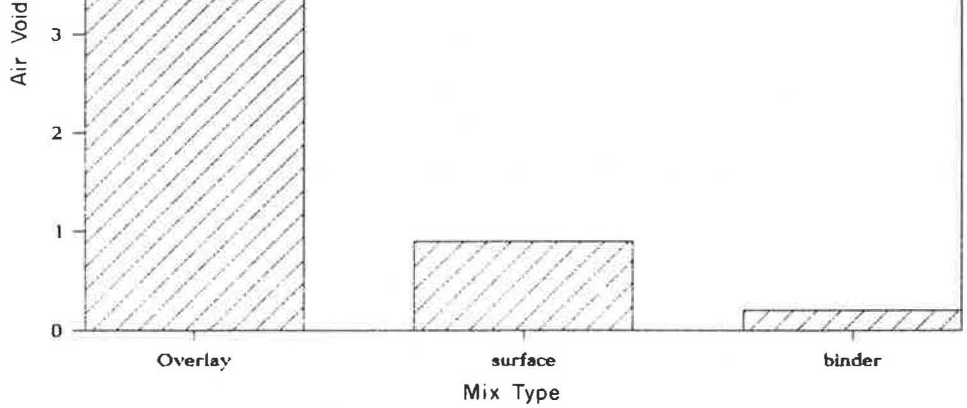

FIGURE 1 Air-void content of westbound I-40 pavement in 1985.

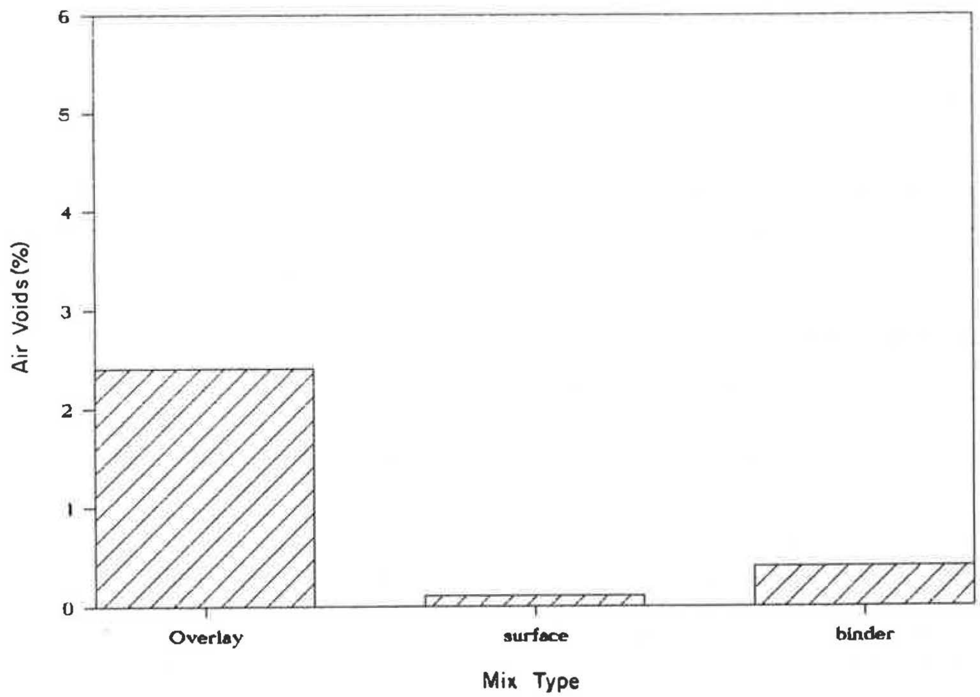

FIGURE 2 Air-void content of westbound I-40 pavement in 1988.

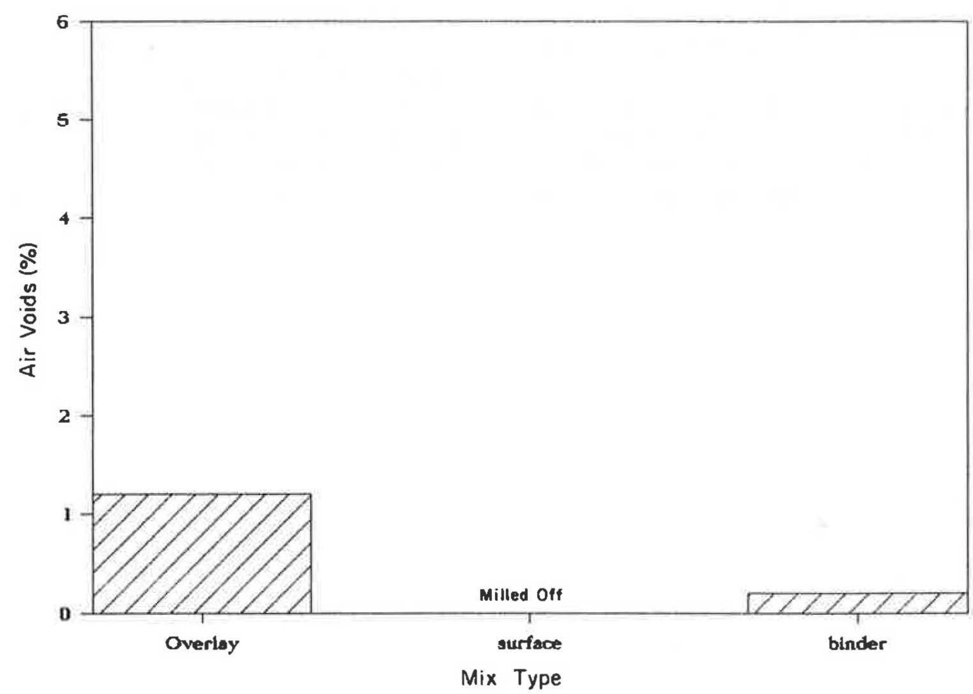

FIGURE 3 Air-void content of eastbound I-40 pavement in 1988.

overlaying with a minimum depth surface course may be a short-term solution, if the pavement layers beneath the overlay are in a plastic state. Results from the 1985 and 1988 core tests show that the 75-blow design procedure and field requirements are definitely an improvement over the older procedures, but they may be insufficient to resist deformation caused by the increasing weight and volume of heavy interstate highway traffic.

The 75-blow design procedure has not adequately defined the ultimate voids in the mixture. In this case, for example, the 75-blow mix design used in the westbound lanes was compacted to 6 percent air voids during construction. The design air-void content was 4.1 percent, and cores taken in 1985 showed that 6 yr of interstate traffic had consolidated the mix to a void content of 4.5 percent. The design air void content closely predicted actual air void content, but, unfortunately,

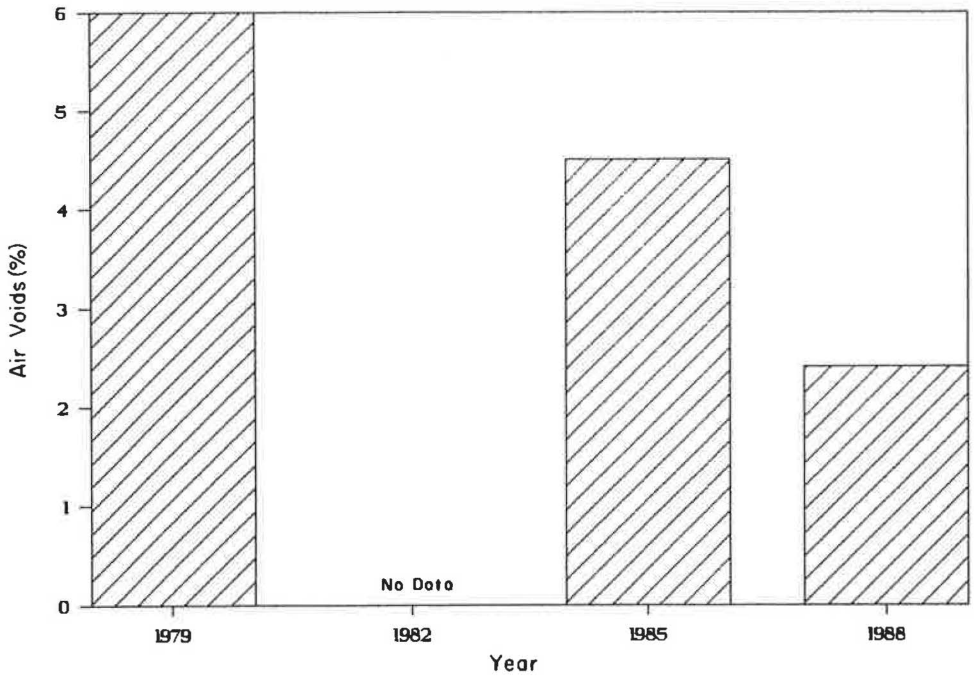

FIGURE 4 Air-void content of westbound I-40 overlay.

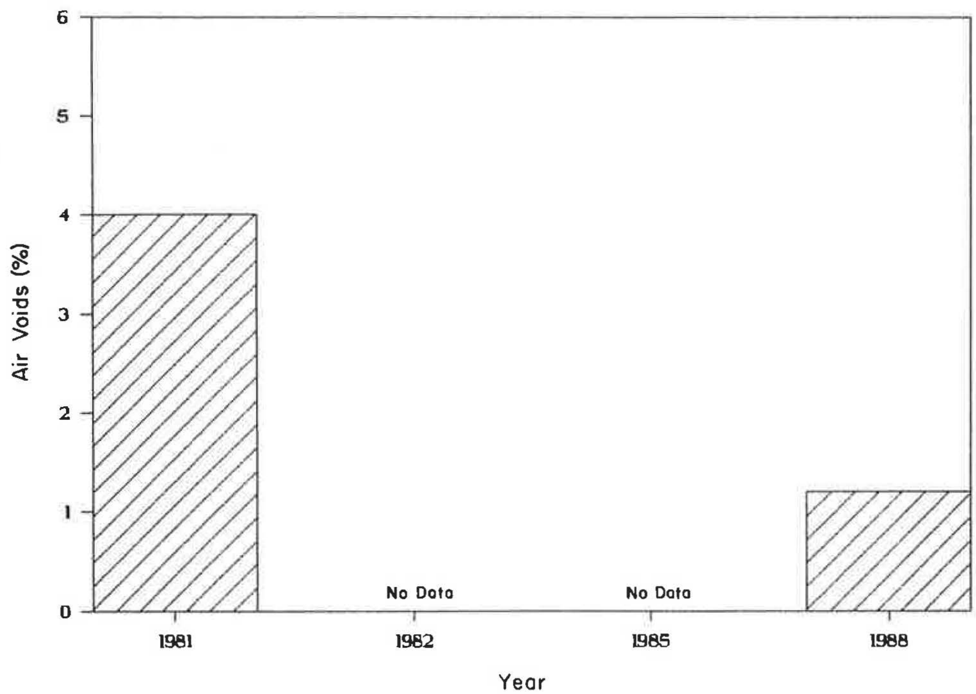

FIGURE 5 Air-void content of eastbound I-40 overlay.

consolidation from traffic did not stop there. By 1988, after 9 yr, the mix had further consolidated to 2.4 percent void content and the mix may be approaching a plastic condition (see Figure 4). The eastbound overlay is in worse condition. It was designed at a void content of 3.4 percent, and the mix was compacted to 4 percent air voids during construction. In 1988, after 7 yr, the average air void level had dropped to 1.2 percent. This mix is definitely in a plastic condition (see Figure 5).

Evaluation of the I-40 overlays shows that improvements in mix design and quality control have helped; however, current procedures for the aggregates available in Eastern Arkansas are still inadequate for interstate highway traffic. A 6-yr life for an overlay on the interstate highway is better than 6 mo, but it is still not the life that had been counted on.

In 1976 and 1977, about the same time that the Interstate 40 overlay was being constructed, two similar projects were under way on Interstate 30 near Little Rock. A typical section from the projects on I-30 had essentially the same composition as the I-40 overlay, i.e., 300 lb/yd^2 of bituminous crushed-stone base course, 400 lb/yd^2 of ACHM binder course, and 165 lb/yd^2 of ACHM surface course. Both Interstate projects used the same asphalt mix designs, although the I-30 project in Pulaski County was started 1 yr earlier. The surface mix for these jobs was composed of syenite coarse aggregate with 15 percent natural sand and 5 percent limestone dust. The mix design was a 50-blow modified Marshall. Optimum asphalt content was 5.6 percent and the design air void percentage was 3.0.

The two I-30 overlays, with a traffic history similar to that of I-40, have held up very well after 11 yr. Rutting in the wheelpaths is less than ¼ in. Apparently the I-30 50-blow mixes have outperformed the I-40 mixes that were designed with new mix requirements and a 75-blow Marshall mix design.

CONCLUSION

Some design and construction procedures still need to be revised to produce an asphalt hot-mix overlay that will resist today's heavy loads and volume on an interstate highway. Arkansas is reviewing fine aggregate shapes and gradations with the intent of limiting quantities and types of natural sands. A comparison of compactive effort of laboratory specimens with actual compactive effort on the roadway is needed. It is believed that the degree of crushing on some coarse aggregates is insufficient to provide adequate angular shape. Generally, Arkansas is in the process of tightening specification tolerances for aggregate gradations, AC contents, and air voids of the job-produced mix, as well as for rolling and compactive efforts on the roadway.

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Asphalt Concrete Mixtures as Related to Pavement Rutting: Case Studies

M. J. HENSLEY AND RITA B. LEAHY

The objective of this paper is to document case studies of pavements that have rutted prematurely. The investigation of the nature and cause of the rutting includes study data from cores, beams across the lane widths, mix designs, nondestructive testing, and traffic data. Three projects are presented in this documentation, two of which rutted prematurely and one of which is identified as rutting-resistant. The rutting in the first project was caused by overcompaction in construction, which was the result of mix design, quality control, and inadequate construction standards. Premature and long-term rutting were both shown in the second project. The rutting was found in all layers of the asphalt, and rutting continued after each corrective action. This problem resulted from a combination of mix and structural designs. The third project used a 4-in.-thick stone-filled binder and surface over a 6-in.-thick, regular dense-graded asphalt base. The stone-filled mixtures are generally higher in initial air-void content and can be compacted to a higher percent of laboratory density in construction. They do not seem to be sensitive to asphalt content and are in that respect not subject to additional compaction under traffic. The void-in-mineral-aggregate curve is usually very flat and shows little change with the asphalt content.

The data presented in this paper are based on projects in three states: Interstate 55 in East Arkansas; the Turner Turnpike near Oklahoma City, Oklahoma; and U.S. 54 in Kingman, Kansas, approximately 35 mi west of Wichita. The Arkansas and Oklahoma projects involved case studies of pavement rutting utilizing data from laboratory testing of field cores or trench sections and nondestructive deflection measurements. The Kansas project involved the use of a stone-filled mix design for both binder and surface courses.

Historically, the term *permanent deformation* was used to describe any distortion of the pavement surface, including shoving and pushing due to mix instability (*1*). Today, however, this term is used for longitudinal depressions, or ruts, that form in the wheelpaths because of "consolidation and/or lateral movement in one or more of the component pavement layers due to repeated, transient load applications" (*2*). Because rutting appears only as a change in the transverse surface profile, it was often erroneously blamed on surface instability. Investigations at the AASHO Road Test, however, revealed that permanent deformation occurred in all layers of the pavement system.

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Generally, for properly designed and constructed asphalt pavements, the rut depth on the inside wheelpath (IWP) will be greater than the rut depth on the outside wheelpath (OWP) because the pavement makes the greatest response to its load at the IWP. Greater rut depth in the OWP would indicate instability in any or all of the component layers of the pavement system. On the other hand, nondestructive deflection measurements are usually greater in the OWP. If the wheelpath deflections are equal, the rut depths, similarly, should be equal.

I-55 IN EAST ARKANSAS

Construction Background

In 1976, a 9.8-mi section of the original 10-in.-thick portland cement concrete pavement was undersealed and overlaid. The 10-in. overlay design consisted of the following layers: approximately 3½ in. of crack relief; 3½ in. of binder (or more, as required) for leveling; 2½ in. of surface; and an open-graded friction course.

Based on a 50-blow Marshall design, the mix properties were as shown in Table 1.

During construction of the overlay, traffic was routed onto the parallel frontage roads, which had been previously overlaid with 3½ in. of the same mix that was to be used on the main lanes. Although the frontage roads rutted shortly thereafter, no adjustments were made to the mix design for the main lanes. As construction was completed and safety precautions were met, traffic was rerouted to the main lanes in early September. Although the entire 9.8-mi overlay had not yet been completed, rutting was observed within 30 days on those sections opened to traffic. An observation point to monitor the rutting was established in the vicinity of a truck weigh station, where the rut depth in early October measured approximately ½ in. From October to early December, the rut depth increased an additional 1/16 in., resulting in a maximum rut of 9/16 in.

Investigation

In December 1976, the Asphalt Institute was asked to help determine the cause of the rutting. Five 6-in. cores were cut through the full depth of the overlay from the outside lane, one each in the IWP and OWP, and three outside the wheelpaths. In addition to the cores, a beam section spanning the

TABLE 1 MIXTURE PROPERTIES OF I-55

PROPERTY	SURFACE	BINDER
Unit Wt (lb/ft^3)	147.8	148.4
Air Voids (%)	4.2	4.0
Stability (lb)	1425	1000
Flow (0.01 in)	12	11
VMA[1]	16.2	13.5
G_{mm}	2.469	2.477
AC (% by wt of mix)	5.3	4.3

(1) VMA back calculated using G_{mm} and G_{se}

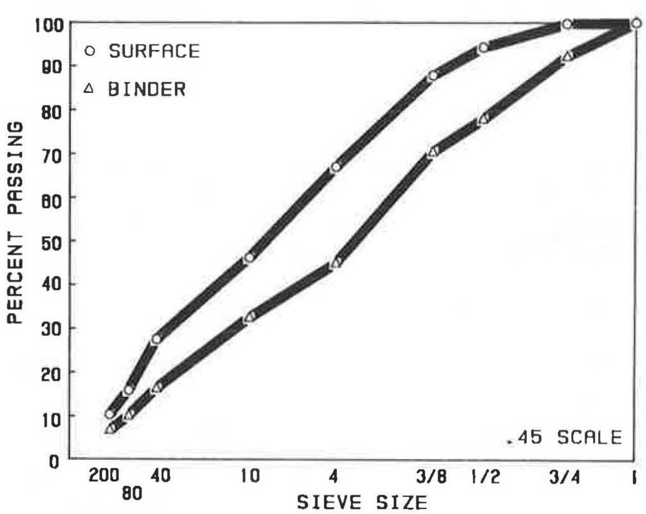

FIGURE 1 Gradation of extracted aggregate (I-55).

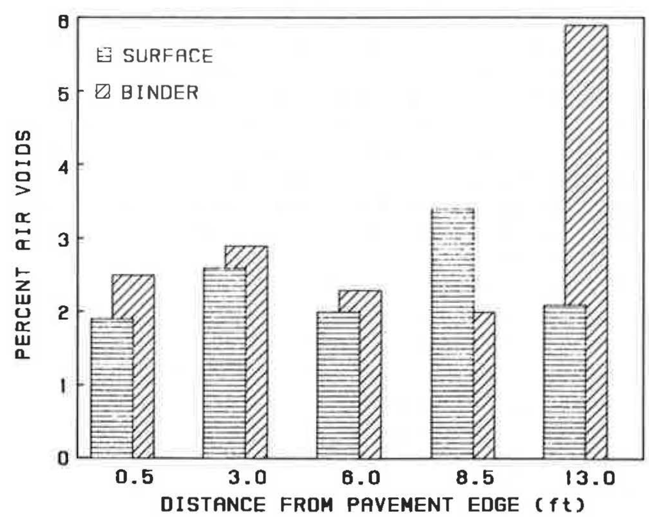

FIGURE 2 Variation in air voids (I-55).

wheelpaths was cut from the traffic lane. The crack-relief layer, consisting of 3-in. top-size aggregate, could not be removed intact, yielding a beam thickness of approximately 5.5 in. From the beam section, eight additional cores were cut from both distressed and nondistressed areas. The cores were trimmed to Marshall size specimens (4-in. diameter and 2.5-in. height) where feasible and used in Marshall and Hveem testing as well as for extraction and recovery tests.

Shown in Figures 1 and 2 and Tables 2 to 4 are average core properties and test results.

Traffic

Traffic counts indicated consistently heavy use by trucks: approximately 189 equivalent single-axle loads with 21 percent truck traffic, as projected in the overlay design.

Nondestructive Testing

Although deflection measurements were made with both the Dynaflect and the Falling Weight Deflectometer devices, only the Dynaflect data are presented here. The data set included deflection measurements at 32 locations in each lane. Average deflections for both north- and southbound lanes were essentially the same: the northbound lane had a mean of 0.46 mil for the inside lane and 0.45 mil for the outside lane, whereas both of the southbound lanes had a mean deflection of 0.47 mil. The coefficients of variation were less than 10 percent. Using computer software that accompanied the Dynaflect device, the average moduli for the dense- and open-graded layers were calculated to be 455,500 and 112,500 psi, respectively.

Because the core properties, test results, and measured deflections for both lanes were nearly identical, it seems rea-

TABLE 2 CORE PROPERTIES OF I-55

PROPERTY	SURFACE	BINDER
Unit wt (lb/ft^3)	150.1	151.6
Air Voids (%)	2.6	2.4
VMA (%)	14.4	12.7

TABLE 3 GRADATION OF EXTRACTED AGGREGATE FROM I-55

	PERCENT PASSING	
SIEVE SIZE	SURFACE	BINDER
1 in	100.0	100.0
3/4	99.8	92.3
1/2	94.3	77.9
3/8	87.9	70.5
No 4	67.1	45.0
10	46.3	32.6
40	7.4	16.3
80	15.8	9.9
200	10.2	6.7
Filler/AC	1.9	1.5

TABLE 4 PROPERTIES OF RECOVERED ASPHALT FROM I-55

PROPERTY	SURFACE	BINDER
Asphalt Content	5.4	4.8
(% by wt of mix)		
Viscosity		
@ 140 ^{0}F (poise)	2181	3370
@ 275 ^{0}F (cSt)	499	570
Penetration		
(77 ^{0}F, 5 s, 100 g)	79	54

sonable to conclude that the underlying layers were relatively stable and that the primary source of the structural weakness was associated with the surface and binder courses. Condition surveys indicated a plastic and horizontal movement of the upper pavement layers toward the outside edge. This was further shown by snowplows clipping the friction course at the outer pavement edge and on each side of the wheelpaths.

Summary of Study

Data from the mix designs and condition surveys point to the surface course as the primary source of rutting, with some contribution from the binder course. The design asphalt content of the surface course, 5.3 percent, was above (to the right of) the minimum void in mineral aggregate (VMA), suggesting that the mix would be susceptible to plastic flow. Furthermore, densities of the field cores from both surface and binder courses were greater than 102 percent of the laboratory-compacted density, indicating that the mix had been overcompacted. Average void contents for the surface and binder cores were 2.6 percent and 2.4 percent, respectively—well below the 4.0 percent specified in the mix design. The void content was slightly higher in some rutted areas, indicating that traffic had actually decompacted the mat. The VMA values from the field cores appeared to be adequate (14.4 percent for the surface and 12.7 percent for the binder), but both were lower than the values calculated at the design asphalt content. Although the gradation of the extracted aggregate did not reveal any particular problem, the filler-to-asphalt ratios for the surface and binder of 1.9 and 1.4, respectively, both exceeded the generally accepted upper limit of 1.2 recommended by the FHWA in Technical Advisory 5040.24, *Asphalt Concrete Mix Design and Field Control.*

From the beam cores, the average Hveem stabilometer values were 12 for the surface and 27 for the binder. Although the average Marshall stability values of 1,335 lb (surface) and 1,550 lb (binder) were acceptable based on existing criteria, the flow values of 15 (surface) and 20 (binder) were high.

To summarize, the apparent causes of the pavement rutting were selection of the design asphalt content above the minimum VMA and high filler-to-asphalt ratios for both the surface and binder courses, which led to overcompaction of the mat.

Corrective Action

Before this project was accepted in October 1978, the Arkansas Highway and Transportation Department required the contractor to place an additional open-graded friction course on the rutted area. This removed the appearance of all rutting at the time. In December 1982, the rut depth was measured at ½-in. on the IWP. In 1985, approximately 1.5 in. was milled off the surface and overlaid with a mixture based on a 75-blow Marshall, as outlined in the Asphalt Institute's MS-2, *Mix Design Methods for Asphalt Concrete.* As of this writing, the pavement is performing satisfactorily without any measurable rutting.

MILLED & 2 in OL (1981)
3 in OL (1973)
2 in OL (1962)
2 in WEARING COURSE
3 in TYPE 'A' AC
7 in AGG BASE
12 in SELECT SUBGRADE
SUBGRADE

10.6 in

FIGURE 3 Turnpike pavement structure.

TURNER TURNPIKE IN OKLAHOMA

Construction History (*3*)

Opened to traffic in 1953, the 90-mi Turner Turnpike connects Oklahoma City and Tulsa. The original structure as shown in Figure 3 consisted of approximately 12 in. of select subgrade, 7 in. of stabilized aggregate base, and 5 in. of asphalt concrete.

According to maintenance records, surface irregularities were observed as early as 1959. Condition surveys noted fatigue and shrinkage cracking and localized structural failures. A 23-mi section was overlaid with a nominal thickness of 2 in. in 1962 and 3 in. in 1973. In 1981, all four lanes were milled to various depths and overlaid with approximately 2 in. of asphalt concrete (55 percent virgin and 45 percent reclaimed material). Shortly after completion, rutting was observed in the outside lane of the westbound lanes (WBL). The project consultant concluded that this was due to Texas Gyratory Hveem stabilities below the recommended minimum of 40. As a result, a 12-mi section of the WBL was milled and replaced prior to final acceptance of the project. This section, however, rutted shortly thereafter. Because the rutting appeared to be confined primarily to the WBL, trench sections were cut across the lane at Mileposts 1.6 and 11.5. Visual observation of the component layers provided conflicting data. At Milepost 1.6 the rutting appeared to be confined to the top layer, whereas at Milepost 11.5, rutting had occurred in all layers.

A brief review of the road's traffic history might prove useful before turning to the investigation of the causes of the rutting and remedial measures taken. In only 7 yr of operation, the turnpike had carried the traffic expected for the first 12 yr. The total traffic volume in 1980 was more than twice that originally predicted, with truck traffic nearly three times the original prediction.

Field Investigation

Based on the conflicting data from the trench sections, a comprehensive program of materials sampling and testing of the 23-mi west end section of the turnpike was undertaken. Using Dynaflect, Austin Research Engineers measured pavement deflections every 0.2 mi in the wheelpath (OWP on the outside lane and IWP on the inside lane) for a total of 500 data points. Standard Testing and Engineering Company of Oklahoma City extracted 65 full-depth cores at 16 different locations (see Figure 4 for core location relative to wheel-

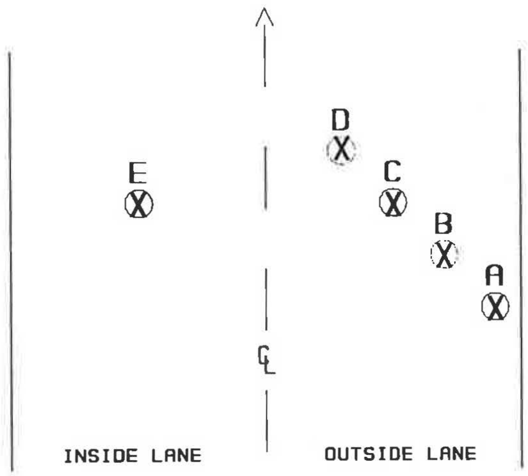

FIGURE 4 Core locations (Turner Turnpike).

path). Laboratory evaluation of the cores included such tests as density/voids analysis, extraction and gradation of the aggregate for the asphalt-bound layers, California bearing ratio, Atterberg limits, and gradation for the base and subgrade.

The results of the density/voids analysis and extraction are shown in Tables 5 and 6. These same results are shown graphically in Figures 5 and 6. Based on the consolidation of the mixture, as shown in Figures 5 and 6, it was concluded that rutting was evident in all the layers.

Layers 3 and 4, which represent the initial construction, indicate excessive deformation due to both consolidation and plastic movement. The 1962 overlay, represented by Layer 2, did not rut in the WBL but it did crack severely. The next corrective action (the 1981 overlay) showed evidence of rutting in both lanes. The most severe rutting was observed in the OWP of the westbound lane. Accordingly, 12 mi were milled and replaced with essentially the same mix utilizing virgin materials. The rest of the project began to rut shortly thereafter. A summary of rut-depth measurements following the 1981 construction is shown in Table 7.

Nondestructive Testing

Based on the Dynaflect data, the 23-mi section was divided into four statistically different sections, two each in the eastbound and westbound lanes. Table 8 shows that the subbase has a lower modulus than that of the subgrade. The asphalt concrete modulus is also low when compared with typical values for asphalt concrete moduli.

Summary of Study

As noted previously, permanent deformation was observed in all of the asphalt-bound layers. The data suggested that several factors led to this rutting. The initial failure could be attributed to structural design, as shown by the load-associated cracking and rutting in the wheelpaths. This structural failure was the result of a weak subbase that had a modulus lower than that of the subgrade. The rutting, which appeared to be greater in the OWP, provided additional confirmation of the instability of the underlying layers. Also, the pavement structure was inadequate for the actual load and volume of truck traffic. Unfortunately, rehabilitation efforts compounded the problem since the overlay mixes had low air voids and high filler-to-asphalt ratios. These types of mixes are typically susceptible to excessive permanent deformation.

In 1983, the surface was milled and replaced with approximately 2.5 in. of a dense-graded, high-stability mix (40+ Hveem) compacted to a minimum of 4 percent air voids. An open-graded friction course was placed as the final wearing course. According to turnpike officials, this corrective action has eliminated the deformation associated with the plastic mix, although the presence of the unstable subbase is still obvious, as evidenced by continued rutting in the OWP.

U.S. 54 IN KANSAS

Design and Construction

In an effort to prevent rutting and cracking, this project was redesigned by the contractor as part of the Kansas Department of Transportation (KDOT) Value Engineering program. The 19-mi project west of Kingman involved not only overlaying the existing pavement but also placing full-depth asphalt on the relocated sections. Although paving was not completed until November 1988, traffic was allowed on the completed sections of the binder (BM-7) and base (BM-2) in early 1987.

A 500-ft portion of a relocated section was identified to monitor the pavement performance. In cooperation with KDOT and the paving contractor, aggregate, asphalt cement, mix, and cores from this section were obtained by the Asphalt Institute for laboratory evaluation of the engineering properties of the mix. The laboratory-measured properties are being compared with those determined by nondestructive deflection measurements and condition surveys.

The full-depth asphalt section consists of 6 in. of dense-graded asphalt base (BM-2), 3 in. of stone-filled binder (BM-7), and 1⅛ in. of stone-filled surface (BM-1). Stone-filled mixes typically have aggregate gradations well below the maximum density line, as shown in Figure 7 (see also Table 9). This is accomplished by increasing the coarse aggregate fraction by as much as 15–25 percent.

The mix properties based on a 50-blow Marshall design are shown in Table 10.

Nondestructive Test

To compare engineering properties of this pavement system with those of the Arkansas and Oklahoma projects previously discussed, Dynaflect deflection data were used to back-calculate the elastic modulus of each layer. To obtain a representative measure of the modulus, deflections were recorded at 20 different locations in both wheelpaths and between the wheelpaths. The moduli were back-calculated to be 500,000, 350,000, and 200,000 psi for the surface, binder, and base, respectively. These values compare favorably with those of Interstate 55 and, considering the reduced volume of traffic projected for this primary road, it is unlikely that there will be significant permanent deformation.

TABLE 5 AVERAGE PERCENT VOIDS AND VMA, TURNER TURNPIKE

	Core Location[1]				
	A	B	C	D	E
Eastbound Lanes					
Layer 1					
Voids (%)	1.3	.9	1.9	.9	2.0
VMA (%)	13.2	13.1	13.7	13.0	13.8
Layer 2					
Voids (%)	3.4	1.7	3.7	1.3	1.0
VMA (%)	16.8	15.2	16.9	13.7	14.3
Layer 3					
Voids (%)	4.2	2.1	3.9	2.2	3.8
VMA (%)	15.1	13.4	15.3	13.2	15.3
Layer 4					
Voids (%)	5.4	2.6	3.7	6.6	6.6
VMA (%)	16.4	13.9	14.7	14.5	17.8
Westbound Lanes					
Layer 1					
Voids (%)	2.3	1.6	2.6	2.1	3.0
VMA (%)	13.8	13.6	14.1	13.8	14.8
Layer 2					
Voids (%)	2.8	3.1	3.2	3.0	2.7
VMA (%)	15.8	15.7	16.2	15.8	15.6
Layer 3					
Voids (%)	3.0	1.1	2.4	1.8	4.5
VMA (%)	15.6	13.8	14.8	14.2	16.6
Layer 4					
Voids (%)	6.6	3.8	5.4	3.6	6.9
VMA (%)	18.2	16.0	17.2	16.7	18.5

(1) Locations relative to the wheelpaths are shown in Figure 4.

TABLE 6 AVERAGE GRADATION OF EXTRACTED AGGREGATE FROM TURNER TURNPIKE

	LAYER			
SIEVE SIZE	FIRST	SECOND	THIRD	FOURTH
1 1/2 in				100.0
1				99.7
3/4	100.0	100.0	100.0	98.4
1/2	92.6	99.1	98.1	92.4
3/8	85.4	91.7	89.2	86.1
No 4	65.3	64.0	67.3	66.9
10	48.5	45.9	47.2	48.6
40	26.9	29.5	30.3	31.1
80	16.0	18.4	17.6	18.5
200	8.7	7.5	5.6	6.1
AC (%)	4.7	5.5	5.3	5.0
Filler/AC	1.9	1.4	1.1	1.2

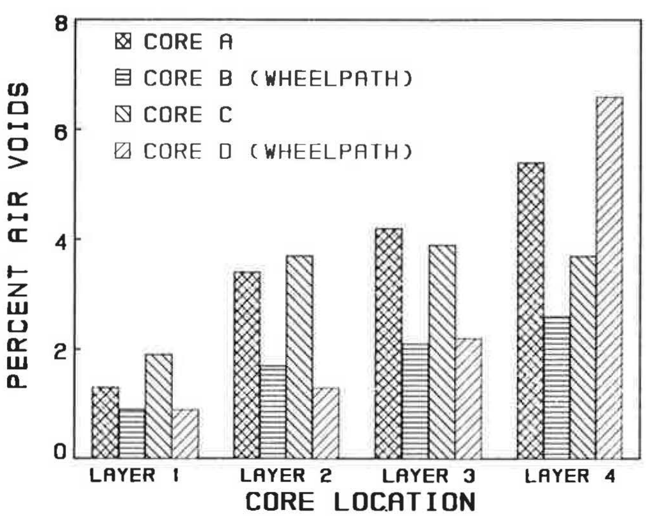

FIGURE 5 Eastbound lane voids (Turner Turnpike).

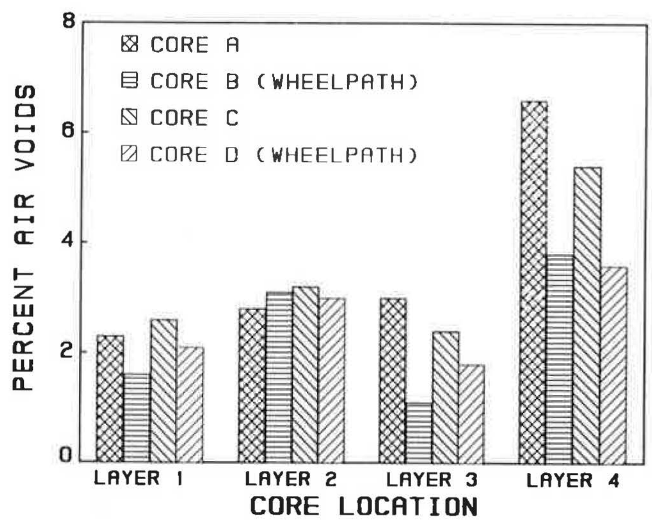

FIGURE 6 Westbound lane voids (Turner Turnpike).

TABLE 7 RUT-DEPTH OF OUTER WHEELPATH ON TURNER TURNPIKE

	Eastbound Lane	Westbound Lane
Mean (in)	0.51	0.36
Std Dev	0.16	0.27
CV (%)	31.4	75.0

TABLE 8 ELASTIC MODULI BASED ON DYNAFLECT DATA FOR TURNER TURNPIKE

		ELASTIC MODULUS (psi)			
		AC	Base	Subbase	Subgrade
MILE POST					
EBL	2.00 - 18.65	170,000	35,000	18,000	25,500
	18.65 - 26.00	150,000	55,000	18,000	18,800
AVERAGE MODULUS		160,000	45,000	18,000	21,500
WBL	2.00 - 19.35	130,000	35,000	15,000	22,000
	19.35 - 26.00	100,000	45,000	15,000	18,900
AVERAGE MODULUS		115,000	40,000	15,000	20,450

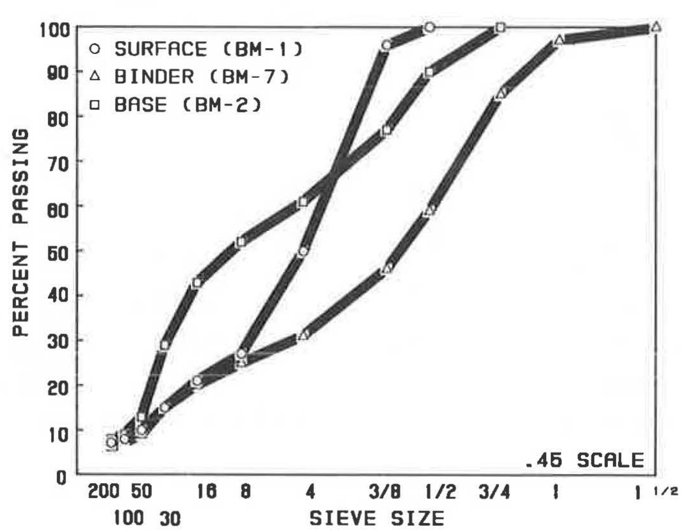

FIGURE 7 Aggregate gradations (Kansas U.S. 54).

TABLE 9 JOB MIX FORMULA FOR KANSAS U.S. 54

	Percent Passing		
SIEVE SIZE	SURFACE	BINDER	BASE
1 in		97	100
3/4		85	100
1/2	100	59	90
3/8	96	46	77
No 4	50	31	61
8	27	25	52
16	21	20	43
30	15	15	29
50	10	9	13
100	8	7	9
200	7	6	8
AC (%)	5.5	4.5	5.3
Filler/AC	1.3	1.3	1.5

TABLE 10 MIX PROPERTIES FOR KANSAS U.S. 54

PROPERTY	SURFACE	BINDER	BASE
Unit wt (lb/ft^3)	144.0	143.6	146.6
Air Voids (%)	5.4	6.9	3.2
VMA[1] (%)	14.0	13.8	13.0
VFWA[2] (%)	61.4	50.5	75.3
G_{mm}	2.436	2.464	2.430
Stability (lb)	2140	2100	1473
Flow (.01 in)	12	14	10

Summary of Study

Although portions of the project were opened to traffic for almost a full year before the surface course was placed, there was no evidence of measurable rutting. In the absence of intermediate fines and with the addition of coarse material, it is likely that the stone-filled mixes will be less sensitive to asphalt content and less susceptible to additional consolidation. Additional condition surveys and deflection measurements are planned to further document the effectiveness of the stone-filled mixes in minimizing permanent deformation.

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