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Foreword

The four papers in this volume deal with field construction and testing of bituminous concrete. They should be of particular interest to construction engineers and materials engineers, as well as to those doing research in the field of bituminous mixes.

The Dunn & Gaudette paper covers Ross Count Studies done by the Wisconsin State Highway Commission; this included both laboratory and field studies. The authors conclude that the Ross Count Method offers one of the most practical approaches to establishing minimum mixing times presently available. However, they do feel that it is desirable that the effects and control of the numerous variables be studied before accepting the Ross Count as a proven method.

The paper by Swanson et al., covers a laboratory study of the effect of important variables on the densification of bituminous concrete during rolling. The results indicate that the effects of asphalt viscosity during compaction on stability is noticeable while the relative density void changes are small. An attempt was made to simulate full-scale field rolling conditions insofar as possible in this study. The rubber tire roller and rolling procedure gave a slightly less dense and less stable compacted mix than did the steel roller.

The Walters paper deals with an attempt to determine asphalt content by nuclear method at the job site. This was done adapting currently available commercial equipment.

The report on the Shell Avenue Test Road presents an evaluation of performance over a 3-year period of an experimental asphalt-concrete overlay pavement in California. Because of the existing conditions it was planned that the test pavement should provide information on the resistance to deformation (stability) and fatigue resistance of heavy-duty mixes using conventional asphalt concrete and asphalt concrete with asbestos as a special mineral filler. From an evaluation of the field and laboratory tests together with visual inspection of the performance of the road, conclusions are presented with regard to the ability of the various test pavements to perform under the traffic imposed and within the particular environment.

Contents

Mixing Time Requirements for Bituminous Mixes as Determined by the Ross Count Method

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During the 1963 construction season, tests were performed by the Materials Research Unit of the State Highway Commission of Wisconsin to determine the practicality of using the Ross Count Method of measuring aggregate coating in establishing a minimum wet mixing time for bituminous-concrete mixtures. The resulting effects of reduced mixing times on the mixture properties were measured by Marshall tests.

A preliminary study was conducted in the laboratory before beginning a field study which consisted of Ross Count and Marshall property tests on bituminous-concrete surface mixtures produced from six hot-mix batch plants. Crushed gravel aggregate was in four of the mixtures and crushed limestone in the other two. An 85-100 penetration grade asphalt cement was used in all mixtures.

Five samples were obtained and tested for Ross Count for any given wet mixing time and at least three wet mixing times were used at each plant. Duplicate Marshall specimens were formed in the field for each of three samples obtained for any given wet mixing time.

It was found that the Ross Count Method was a simple and practical procedure to use in the field with the reliability of results dependent on the experience and care of the operator. Ross Count test results show that the current State of Wisconsin specification of 45-sec minimum wet mixing time produced varying degrees of mixing completeness for each plant and mixture. However, an average trend for all mixtures showed nearly 100 percent aggregate coating after 45-sec mixing, and a reduction of mixing time reduced coating with the coating reduction becoming progressively more pronounced as mixing time was decreased.

Statistical evaluation of the test results indicate that the reliability of any one Ross Count decreases as the mixing time decreases. Thus, an increase in number of "counts" would be required for decreased mixing times to maintain a uniform degree of count reliability for all mixing times.

Marshall test results indicate that the mixture properties of all six mixtures were not significantly affected when the wet mixing time was reduced to permit 97 percent aggregate coating. It is concluded that the practical approach offered by the Ross Count Method could conceivably be used to establish and control satisfactorily minimum mixing time requirements. However, because the Ross Count is subject to numerous variables, it is desirable that the effects and control of these variables be studied to determine the full merit of the method as an adequate field control procedure.

•THE USE of an arbitrary mixing time specification for bituminous mixtures has been questioned by producers and highway agency personnel in recent years. The reason

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for concern is that factors other than mixing time may be important; thus, control of mixing should not be based on time alone.

Literature concerning the effect of mixing time was limited to studying the effect of mixing time on asphalt hardening prior to 1959. At the 1959 meeting of the Association of Asphalt Paving Technologists (AAPT) results of two investigations (1, 2) were reported which primarily considered the effect of wet mixing time variation on mixing completeness of bituminous mixtures.

One investigation (1) consisted of studying the efficiency of various pugmill mixers used to produce bituminous paving mixtures. Results of the study indicated random uniformity of aggregate gradation and asphalt distribution (not thorough coating) were completed before the coarse particles were coated. It was determined that the finer aggregate particles were coated before the larger particles; thus, it was proposed that "neither aggregate distribution nor sample to sample variation in A.C. content are controlling factors in time of mixing, but the coating of the coarsest aggregate particles in the mix may be."

During the course of the investigation (1), a method was developed, termed the Ross Count Method, which provided a simple means of numerically measuring the percent of coated particles. Essentially, the method consisted of obtaining a representative sample of bituminous mix as it was discharged from the pugmill mixer and separating the sample into coarse and fine fractions using a sieve of selected size. The coarse particles retained on the sieve were divided further by visually determining if each particle was completely or partially coated. The percent of coated or uncoated particles was computed on the basis of total number of coarse particles in the sample.

General conclusions of the other investigation (2) were that an adequate mixing time can vary with materials and mixers. The following conclusions were given as being applicable to any bituminous-concrete mixing process:

> 1. Random sampling techniques and appropriate control limits can be developed and applied to the mixing of bituminous concrete to serve as an evaluation of mixing efficiency.
> 2. Coating can be evaluated by visual methods. . . .
> 3. The methods of introducing materials and the characteristics of the pugmill are the most significant factors involved in the efficiency of the mixing process.
> 4. Bituminous-concrete mixers should be rated on their individual merits under the conditions that are imposed by a particular job in the field. A blanket mixing time specification cannot logically be applied to all mixers.

These two studies led to an increased interest in determining a satisfactory mixing time based on an actual test measurement rather than an arbitrarily selected period of time. Both studies indicated that coating of aggregate, a measure of mixing completeness, can be evaluated by visual methods. The Ross Count Method is the simplest method proposed to date and has been used recently by several highway agencies to study the usefulness of the method for adjustment of mixing time. Results of these highway agency studies are not presently available, but there are indications that some of the agencies involved are considering adoption of a count method. There are those, however, who argue that film characteristics are much more important than coating of the larger particles in obtaining desired mix properties. Others have suggested that the foremost requirement is to obtain an equilibrium of distribution of asphalt and aggregate.

The discussion of a progress report on the Ross Count Method (3) presented results of studies by the Bureau of Public Roads which showed that uniformity of asphalt and aggregate distribution did occur before complete coating of the larger particles for dense graded mixtures. However, tests made with open graded mixtures indicated 98 percent coating of the larger particles was obtained before uniformity of material distribution was achieved.

From the information presently available on the effects of time on mixing of bituminous-concrete mixtures, and from the above discussion, it is apparent that a thorough mixing time study cannot be made using one asphalt, aggregate and pugmill mixer. Therefore, a mixing time study, utilizing the Ross Count Method, which would encompass various types of asphalts, Wisconsin aggregates and mixers was conducted during the later part of the 1963 construction season.

PURPOSE AND SCOPE

Because previous experience with the Ross Count Method was lacking, a laboratory study was conducted to acquaint personnel with the method and to develop procedures before beginning the field phase of the study program. The ability of one operator to reproduce the "count" of another operator (reproducibility) was investigated during the laboratory phase. The information gained from the laboratory work served as a guide for the sampling and testing procedures used in the field.

One objective of the field study was to obtain information about variations in coating of the aggregate which could be expected at different mixing times for various pugmill mixers and types of asphalt and aggregate. Determination of variations in Marshall properties of the mix due to changes in mixing time was a second objective of the field phase. The two objectives were accomplished by conducting a series of Ross Count tests for various mixing times at six different batch-type bituminous mix plants. It was intended that the data obtained would also be used for a statistical analysis of the variations which occurred for various conditions.

The type of mixtures used for both phases of the study was surface course mix (Gradation No. 3, Section 401.2.5 of the Wisconsin Standard Specifications, 1963). This mix type limitation permitted sufficient data to be obtained for a comparison of the various mix plants involved.

The procedures and discussion of test results are treated separately for the laboratory and field studies of the investigation.

LABORATORY ROSS COUNT STUDY

Material and Equipment

Asphalt and aggregate materials remaining from samples submitted for routine laboratory bituminous mix designs were used in the laboratory phase of the test program. The use of materials previously processed by the mix design laboratory gave the advantage of selecting designs having predetermined aggregate gradation and

TABLE 1
LABORATORY MIXTURE INFORMATION
(Percent Passing Sieve)

Sieve Size	Mixture A[a]		Mixture B[b]		Mixture C[c]		
	Crushed Stone	Sand	Crushed Gravel	Sand	Crushed Stone	Stone Chips	Torpedo Sand
¾ In.	100	-	-	-	-	-	-
½ In.	95	-	99	99	100	-	100
⅜ In.	80	100	87	98	99	100	99
No. 4	54	99	64	97	68	56	91
No. 8	40	94	-	-	45	9	79
No. 10	-	-	49	95	-	-	-
No. 40	-	-	22	89	-	-	-
No. 50	22	7	-	-	15	4	44
No. 80	-	-	8	64	-	-	-
No. 200	10.8	1.9	5.6	42.4	11.2	3.0	10.0

[a]Crushed limestone with blending sand: crushed stone-90%; sand-10%; asphalt content-5.6%.
[b]Crushed igneous and limestone gravel with blending sand: crushed gravel-91%; sand-9%; asphalt content-5.8%.
[c]Crushed limestone with blending sand: crushed stone minus ⅜ in.-30%; stone chips-30%; Torpedo sand-40%; asphalt content-6.1%.

asphalt content data. Three designs were selected from the limited number of retained samples to provide two types of aggregate mixtures: crushed limestone and crushed gravel. The three selected mix combinations are given in Table 1.

A modified Hobart electric mixer was used for mixing. The hot mix was separated into fine and coarse particles on a ¼-in. sieve. The sieve was 17 in. in diameter with a 3¾-in. wooden sidewall.

Test Methods

Aggregate batch sizes were controlled to provide at least 250 coarse particles on the separating sieve. Thus, the batches ranged from 2,000 to 3,500 gm, depending on the aggregate gradation. The asphalt content at the peak of the mix design density curve was selected for each mixture.

Aggregate batches were heated in a gas-heated oven to temperatures ranging from 325 to 390 F. The aggregate was placed in the mixing bowl and the desired amount of asphalt was added to the mixing bowl which was placed on a 1-gm direct-reading scale.

A 2-min wet mixing time has been adopted by the bituminous mix design laboratory to obtain satisfactory mixing of asphalt with a 2,500-gm batch of aggregate using the Hobart mixer. Therefore, 2 min was chosen as a control mixing period for the laboratory study. Changes in mixing time were in ½-min increments and at least three batches were tested for each mixing time.

Preliminary to beginning the laboratory testing program, several trial samples were mixed, separated and counted to check procedures and make any changes deemed necessary. The first batches were separated on a No. 4 sieve resulting in retention of particles believed to be too small for convenient counting. The minimum size particles retained on a ¼-in. sieve, however, were found to be satisfactory for counting. Thus, the ¼-in. sieve was chosen as the separating size for use throughout the study.

Early in the laboratory work it was apparent that the operators processed four or five counts before they were confident of their results. Confusion existed during the initial counting as to what should constitute an uncoated particle. Although most particles were either definitely coated or had definite breaks in surface coating, there were a small number which appeared to have discolored surface areas without an asphalt film. Several of these questionable particles were washed with a degreasing solvent and it became evident that a thin film of asphalt was present. Thus, only those particles which had a definite break in surface coating and/or discoloration were considered to be uncoated.

The initial results indicated that mixing temperature affected the degree of aggregate coating. Therefore, observations of temperatures were recorded for asphalt and aggregate just prior to mixing and for the final mix immediately after mixing. Variations in temperatures of the individual materials undoubtedly had some effect on the coating of particles. Thus, mixes with the same temperature after mixing may have differed in the individual asphalt and aggregate temperatures prior to mixing, resulting in different degrees of coating.

Although the main objective of the laboratory study was to familiarize the operators with procedures of the Ross Count Method, a study of reproducibility of counts was also included to determine the reliability of the method. (Reproducibility refers to the agreement of the count of one operator with the count of another operator for a given sample.) The procedure was as follows:

1. The coarse particles of a mix (those particles retained on the ¼-in. sieve) were split into two samples by quartering.
2. Each split sample was counted by one operator. For example, split sample 1 was counted by operator 1 and split sample 2 was counted by operator 2.
3. Following the original count (separation of coated and uncoated particles), particles of each split sample were recombined and counted by the other operator. For example, split sample 1 was counted by operator 2 and split sample 2 was counted by operator 1.

TABLE 2

LABORATORY ROSS COUNT DATA SUMMARY

Laboratory Mixing Time (min)	Operator	No. of Split Samples Counted[a]	Temperature (F) Asphalt	Aggregate	Mixture	Total No. of Particles	Percent Uncoated Particles	Average Percent Uncoated Particles
			Mixture A					
1½	1	4	285	350	255	324	44.2	
	2	4	285	350	255	336	43.1	43.7
2	1	6	285	375	275	352	33.5	
	2	5	285	370	275	337	32.8	33.2
			Mixture B					
1½	1	7	285	390	310	280	12.8	
	2	7	285	390	310	281	12.1	12.5
2	1	5	280	390	290	279	8.8	
	2	5	280	390	290	283	8.0	8.4
2½	1	4	280	380	280	308	4.4	
	2	4	280	380	280	308	4.0	4.2
3	1	10	280	365	270	325	4.4	
	2	10	280	365	270	325	4.2	4.3
3½	1	8	280	365	260	307	3.4	
	2	8	280	365	260	307	3.5	3.5
			Mixture C					
¾	1	1	-	-	-	622	18.0	
	2	3	-	-	-	573	15.8	
	3	3	-	-	-	572	14.5	15.6
1	1	1	-	-	-	289	9.0	
	2	5	-	-	-	562	10.1	
	3	5	-	-	-	605	9.4	9.7
1½	1	1	-	-	-	216	4.6	
	2	1	-	-	-	555	7.8	
	3	1	-	-	-	349	8.3	6.9

[a]Two split samples equal one mix.

Test Results

Ross Count test data are summarized in Table 2 for the laboratory mixed samples. Only mixture B offered sufficient retained material to provide an adequate number of counts for analysis at various mixing times. Although the results for mixtures A and C are incomplete in themselves, with respect to number of counts for each mixing time, they do serve to substantiate the results for mixture B.

A plot of average Ross Count values at various mixing times for each mixture resulted in the general trend curves shown in Figure 1 which illustrate that the percent of uncoated particles decreased as mixing time increased. It is apparent from Figure 1 that the rate of decrease in percent of uncoated particles will vary for changes in aggregate gradation and asphalt content. The curves of Figure 1 also suggest that the percent of uncoated particles can be expected to vary at any given mixing time due to variations in aggregate type and gradation, and asphalt type and content.

Warden, Ward and Molzan (3) suggest that the relationship of percent uncoated particles and mixing time is represented by an exponential function which plots as a straight line on semi-log paper within the range of 0 to 40 percent uncoated particles. A plot of this type is shown in Figure 2 for mixture B to demonstrate that a straight line is a good indication of the trend of the relationship. The results at the 2½-min mixing time period obviously do not follow the straight-line trend; however, an increased number of counts at this particular mixing time may have given an average value more in line with the majority of the test results.

Table 3 gives the difference between percent uncoated particles or Ross Count values obtained by two operators for a given sample. The average difference for each of the three mixtures was less than one percent, and a combination of the average mixture values resulted in an overall average difference of 0.64 percent. Apparently the operators were assessing the degree of coating on an essentially equal basis, and close agreement of the laboratory counting results provided assurance that field counts would be reliable regardless of which operator made the count.

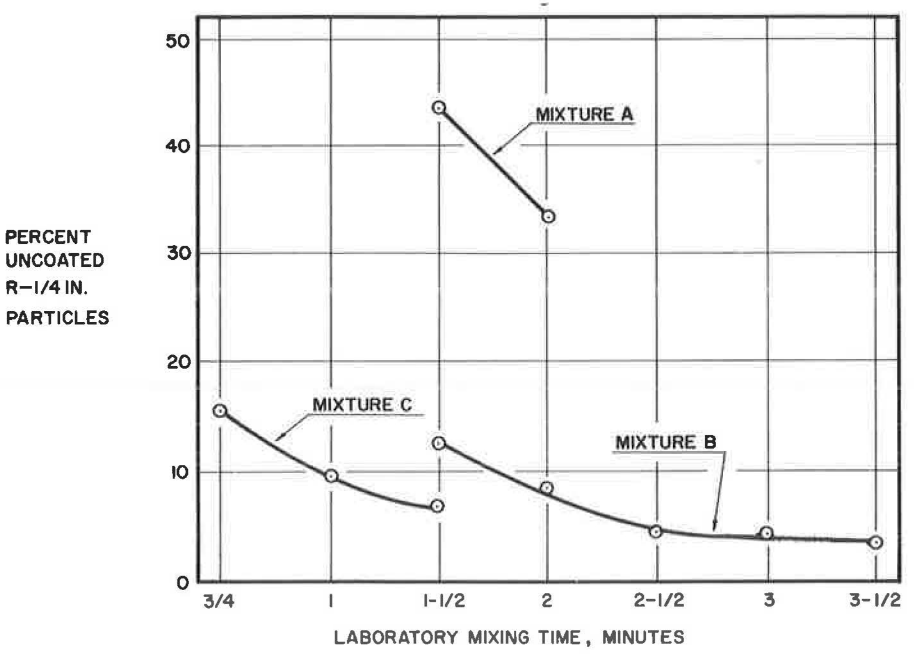


Figure 1. Coating of coarse particles vs laboratory mixing time.

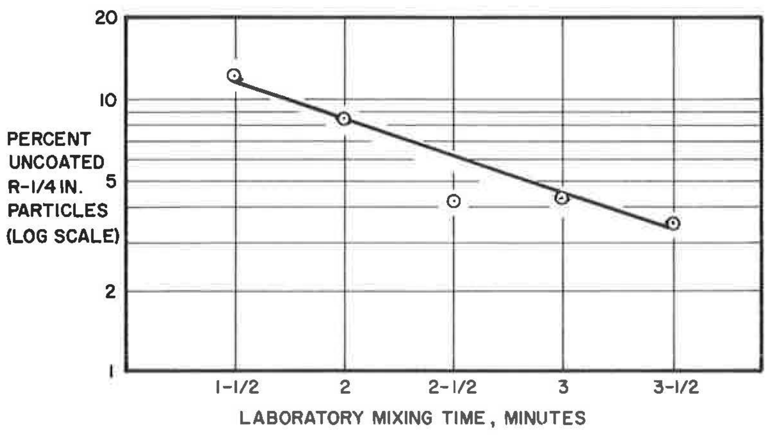


Figure 2. Coating of coarse particles vs laboratory mixing time, mixture B (semi-log plot).

A statistical analysis of the data of Table 3 indicated the differences in count values obtained by two operators for a given sample were significantly different at a 5 percent level of significance, implying there is evidently a systematic difference between the results not due to change alone. It is reasonable to assume the differences in count results are due to the individual bias of operators. It is apparent that operator 1 usually counted more uncoated particles than operator 2, and this observation was reflected in the statistical inference. Bias between operator 2 and 3 was not apparent in the results, as reflected in the statistical acceptance of the hypothesis that no significant difference existed between counts for mixture C. Generally, the statistical

TABLE 3

LABORATORY STATISTICAL DATA SUMMARY

Mixture	Laboratory Mixing Time (min)	No. of Counts	Avg Percent Uncoated R¼-In. Particles			Counting Difference and Std. Dev. (%)
			Operator 1	Operator 2	Operator 3	
A	1½	2	36.9	36.1		0.8[a]
	2	5	33.5	32.8		0.7[a]
						0.83[b]
						0.67[c]
B	1½	6	14.1	13.2		0.9[a]
	2	4	7.3	7.2		0.1[a]
	2½	4	4.4	4.0		0.4[a]
	3	4	2.7	2.4		0.3[a]
	3½	6	3.5	3.6		0.1[a]
						0.51[b]
						0.52[c]
C	1	4		11.0	11.2	0.2[a]
	¾	2		23.3	21.5	1.8[a]
						0.92[b]
						1.12[c]
Overall						0.64[d]
						0.76[e]

[a]Average counting difference.
[b]Average mixture counting difference.
[c]Estimated mixture standard deviation of difference.
[d]Overall mixture counting difference.
[e]Overall estimated standard deviation of difference.

results do not imply that the counting differences are large, but only that the occurring differences are systematic implying that a counting bias existed for operators.

One variable observed to have considerable effect on coating of particles was mixture temperature during the mixing operation. The equipment available at the time the laboratory work was in progress did not permit control of temperatures to the extent desirable or necessary to demonstrate the effect fully. Differences in average count values for mixture B after three minutes mixing serve to illustrate the effect of mixing temperature: The average percentages of uncoated particles for the three mixtures were 4.0, 2.7 and 2.4 for mixing temperatures of 275 F and above, but increases to 5.5 and 6.8 were recorded when the mixing temperatures were decreased to 250 F or below. Variations in viscosity of the asphalt cement due to changes in mixing temperature offer a reasonable explanation for the differences of coating noted for any given mixing time.

FIELD STUDY

The field program was conducted to determine the magnitude of variations in coating of aggregate, as measured by the Ross Count Method, for various mixing times, pugmill mixers, and types of asphalt and aggregate. Tests were performed, also, to determine the effect of mixing time variation on Marshall test properties of the mixes.

Test Variables

The study was limited to six plants producing surface mixtures using 85-100 penetration grade asphalt cements. All bituminous plants were the batch type.

Although many variables were eliminated by the above limitations, the study involved several other factors which may or may not have affected results. The following list of possible variables of the study is suggested:

1. Plant operating mechanism, especially the pugmill type and condition.
2. Mixture variations of asphalt and aggregate materials regarding type, proportioning and quantity.

3. Temperatures of the asphalt in the surge tank, aggregate in the hot bins, and mixing temperature of the combined materials.
4. Operator error resulting from work (sampling, particle counting and Marshall specimen compaction) by six operators.
5. Effect of climate or weather conditions on mix materials, such as the effects of rain on the aggregate stockpile and humidity at the time of mixing and counting.

In addition to the suggested variables, there are inherent and unavoidable variations due to sampling. It is significant, also, that test data accuracy and reliability are only as dependable as the test itself. The Ross Count Method is simple, and to some degree practical, but empirical since it is based on assumptions and involves a "human error" or bias. The errors involved in the Marshall tests are variable depending on conditions such as compaction, number of replicate specimens, and degree of care exercised in handling, curing and actual performance of the various tests.

Materials and Mixtures

Table 4 gives materials and mixture data for the six mixes studied. The data were obtained from laboratory mix design reports for each project. Crushed gravel aggregate was used for four mixtures and crushed limestone for two. Los Angeles wear losses for the two limestone aggregates were high, and aggregate sodium sulfate soundness for mixture 5 was high at 19.6 percent loss after 5 cycles. All mixtures were composed of similar aggregate gradings, so much so that aggregate gradation may not have been a significant variable of the study. The recommended asphalt content range of mixture 6 is high when compared with asphalt content ranges normally recommended.

Plants and Pugmill Mixers

Table 5 gives available data on the plants, plant components, control features, and operating conditions. Pugmill mixer conditions were generally good, as rated visually. All but one plant measured asphalt by weight and all plants were equipped with a time control switch. Pugmill mixing periods for the dry aggregate ranged from 0 to 8 sec and averaged about 5 sec.

TABLE 4

FIELD MATERIALS AND MIXTURE INFORMATION

Item	Mixture					
	1	2	3	4	5	6
Aggregate characteristics						
Type	Gravel	Gravel	Limestone[a]	Gravel	Gravel[b]	Limestone[c]
Fractured particles, %	74	88	98	74	65	99
L. A. wear loss, %	32	21	47	39	28	51[d]
Sodium sulfate soundness loss after 5 cycles, %	10.5	0.9	12.4	11.8	19.6	12.8
Passing sieve size, %:						
3/4 In.	100	100	100	100[e]	100	100
1/2 In.	98	98	95	99	96	99
3/8 In.	83	86	81	88	85	86
No. 4	62	64	60	64	63	59
No. 8	50	51	47	-	52	46
No. 50	19	16	21	-	22	23
No. 200	8.7	6.7	9.3	9.2	9.0	7.5
Asphalt cement						
Pen. grade	85-100	85-100	85-100	85-100	85-100	85-100
Mixture features						
Recommended asphalt content range, %	4.8-6.5	5.4-6.6	5.1-7.1	5.1-6.2	6.0	7.4-8.8
Recommended pugmill mixture temperature range, F	270-300	275-305	265-305	265-295	285	265-305
Maximum compacted laboratory density, pcf	155.4	149.8	152.9	152.6	150.7	146.0

[a]10% blending sand added. [b]15% crushed gravel added. [c]5% blending sand added.
[d]Special Provision Specification permits maximum wear loss of 55%.
[e]Percent passing No. 10 sieve-48, No. 40-23, and No. 80-14.

TABLE 5

PLANT CHARACTERISTICS

Item	Plant 1	2	3	4	5	6
Pugmill mixer manufacturer designation	A	B	C	A	B	B
Visual estimation of pugmill condition	Good	Fair	Good	Good	Good	Fair-Good
Asphalt temperature in surge tank, F	300	305-335	305	290-300	280-285	300-310
Aggregate temperature in hot bins, F	310-320	310	290	315-330	290	315-325
Dryer fuel oil No.	5	2	5	6	5	6
Aggregate control type	Levers	Semi-automatic	Automatic	Hydraulic levers	Automatic	Automatic
Asphalt quantities measured by:	Volume	Weight	Weight	Weight	Weight	Weight
Dry aggregate mixing period, sec	None	5	8	5-8	4	3
Batch capacity, tons	-	-	-	3.0	2.5	3.5
Time for asphalt to enter pugmill, sec	17	-	-	18	14	-
Rated plant capacity, tons/hr	-	140	230	-	-	-
Approximate operating plant output, tons/hr	85	100	140	-	-	-

No attempt was made to determine the individual effects of each plant variable. There is probably much "interaction" or "countereffects," all of which are subject to change for another mix, another day of operation involving changes in weather, and with continual wear or use of the plant.

In addition to pugmill type and condition, inherent plant features which may affect aggregate coating significantly are batch size and time required for the asphalt to enter the pugmill. Controllable features, such as aggregate dry-mixing time and temperatures of materials, are also a part of the overall plant variation effect.

Test Methods

The field study was begun by scheduling preliminary work at a nearby stationary plant in order to familiarize the operators with difficulties to be encountered at a producing plant. As a result of the preliminary work, the operators became acquainted with plant operations and problems that would be encountered, thus becoming prepared for full-scale field operations of sampling and testing.

The convenient site of the stationary plant provided an opportunity to answer several procedure questions always encountered when initiating a project involving unfamiliar operations. It was decided to obtain five counts per mixing time for satisfactory "statistical strength" of the data. It was also decided to mold duplicate Marshall specimens for three samplings at each mixing time in order to obtain a satisfactory cross-section of the mix and mix variability.

Sampling and Sample Preparation.— Mix sampling was considered to be of primary importance if uniform and representative data were to be obtained. Ross Count samples were obtained in a 6-in. diameter by a 6-in. high bucket suspended from a 3-ft handle. Samples were obtained just as the batch was discharged from the pugmill mixer. This sampling method is believed to be very reliable provided the sample is taken at about the midpoint of batch discharge. The method has the disadvantage of requiring the operator to stand on the side of the truck box during loading, which is a somewhat difficult and awkward position until the operator develops a technique.

The Ross Count sample was quickly placed on the ¼-in. sieve size and hand-sieved to allow all passing ¼-in. material to pass through the sieve. It was found that immediate sieving after sampling hastened the operation, but mix characteristics and mixing time were also detected as affecting ease of sieving. The sieve was cleaned readily in a large bucket with fuel oil after each sieving operation. It was necessary to use a wire brush to clean the sieve openings. The sieve was dried completely before admitting another sample.

The R¼-in. particles were spread on paraffin-coated paper (to prevent absorption of asphalt) and allowed to cool. (Throughout this paper the term R¼-in. refers to material retained on the ¼-in. sieve.) The sample was then quartered and two opposite quarters used for the count. This procedure resulted in a total sample of about 700 particles.

Mix material for Marshall specimens was sampled by scooping a pan of mix from a loaded truck. Two specimens were molded from each pan of mix obtained. The mix sample was placed on a gas-heated hot plate to maintain temperature until compaction. The hot plate was also used for heating the compaction molds.

Ross Count Tests.—At least two operators counted any one sample. The Ross Count of approximately 700 coarse particles for each sample consisted of visually observing the coating of each particle. A particle was considered completely uncoated if only a pinpoint area was uncoated. The total number of uncoated particles multiplied by 100 and divided by the total number of coarse particles resulted in the percent uncoated particles for each sample.

Generally, counting was done in the field, but some samples were counted in the laboratory for the last three projects in order to accelerate testing. These latter counts were made as soon as possible after sampling and within a two-week interval in all cases. All samples to be counted in the laboratory were cooled in the field prior to being placed in containers in order to avoid additional coating due to particle contact.

Marshall Tests.—Compaction of Marshall specimens was a manual operation. Compacted specimens were cooled in buckets of cold water prior to being extruded with the Marshall hammer. Each specimen was placed in a small labeled box and transported to the central laboratory for testing. An effort was made to keep specimens supported on a flat surface at all times following compaction, and no specimen was placed on top of another.

Marshall specimens were retained at room temperature in the box containers for 6 to 9 days (generally 7 days) prior to testing. Specimens were tested for density, stability, flow and void content in accordance with normal Marshall test procedures.

Test Results

Ross Count test data are summarized in Table 6. Average percent uncoated particle values at various pugmill wet mixing times are shown in Figure 3. Three mixing times are shown for each mixture. Pugmill mixing times range from 20 to 45 sec but these

TABLE 6

FIELD ROSS COUNT DATA SUMMARY[a]

Mixture	Mixing Asphalt Content (%)	Pugmill Mixing Time (sec)	Pugmill Mix Temp. at Discharge (F)	Ross Count, R¼-In. Particle Size: Coated Particles	Uncoated Particles	Total Particles	Percent Uncoated	Percent Coated
1	5.9	20	285	689	117	806	14.5	85.5
		30	300	780	35	815	4.4	95.6
		45	300	755	6	761	0.8	99.2
2	6.2	20	300	645	95	740	12.8	87.2
		25	295	614	32	646	4.9	95.1
		35	300	678	2	680	0.3	99.7
3	6.1	20	300	607	106	713	14.9	85.1
		25	305	683	33	716	4.6	95.4
		35	300	754	7	761	1.0	99.0
4	5.7	20	310	520	76	596	13.0	87.0
		25	310	639	12	651	1.9	98.1
		35	310	640	0	640	0.0	100.0
5	5.7	25	295	568	111	679	17.2	82.8
		35	285	704	43	747	6.7	93.3
		40	285	722	8	730	1.1	98.9
		45	285	667	13	680	2.0	98.0
6	7.3	30	295	526	185	711	25.2	74.8
		35	290	652	50	702	6.7	93.3
		40	290	569	18	587	3.0	97.0

[a]Each tabulated value is an average of five test results.

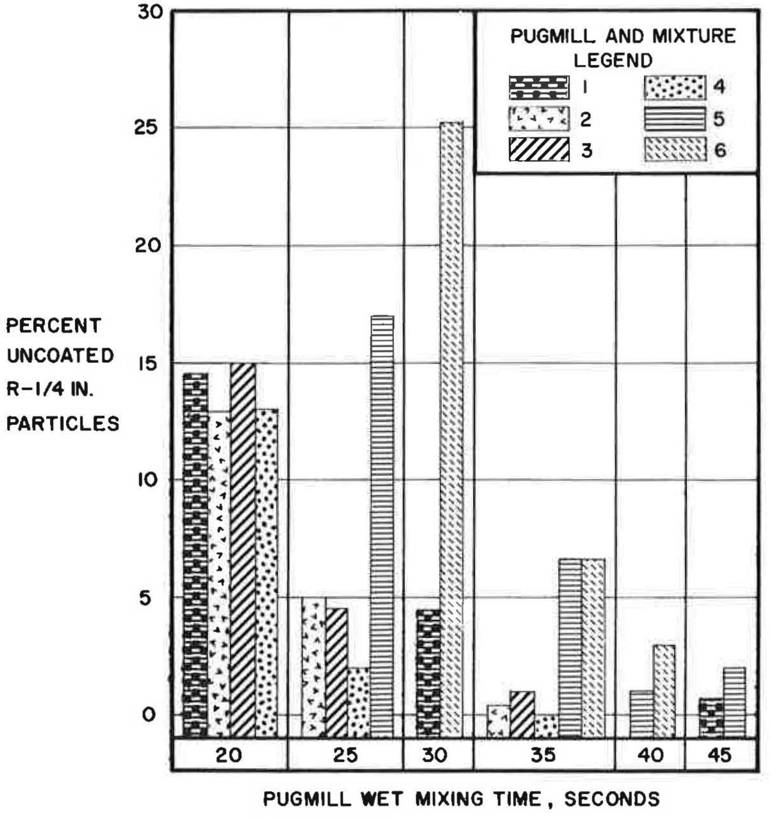


Figure 3. Percent uncoated particles at various pugmill mixing times.

TABLE 7

MARSHALL COMPACTION AND TEST DATA SUMMARY

Mixture	Mixing Asphalt Content (%)	Pugmill Mixing Time (sec)	Compactor Operator	Compaction Temp. (F)	Density (pcf)	Void Content (%)	Stability (lb)	Flow (0.01 in.)
1	5.9	20	1	260	155.2	2.8	1575	10
		30	1	270	155.3	1.3	1520	13
		45	1	270	154.9	1.9	1420	11
2	6.2	20	4	260	149.0	2.0	1010	8
		25	3	265	148.9	1.9	970	9
		35	2	260	150.2	1.6	1300	10
3	6.1	20	3	280	150.9	4.2	1540	5
		25	2	285	151.6	2.7	1490	7
		35	4	275	150.6	2.9	1355	9
4	5.7	20	2	280	151.6	2.7	1375	8
		25	4	290	152.9	1.6	1365	11
		35	2	270	152.4	2.0	1230	12
5	5.7	25	4	270	149.2	3.6	1375	9
		35	2	260	149.8	2.4	1365	10
		40	4, 2, 5	255	149.6	2.5	1335	10
		45	5	260	150.1	2.3	1395	12
6	7.3	30	5, 2	265	149.5	2.5	1615	14
		35	5	270	148.2	2.0	1490	16
		40	2	265	148.3	2.4	1050	16

[a]Specimen test age was normally 7 days but varied from 6 to 9 days. Each tabulated value is an average of six test results.

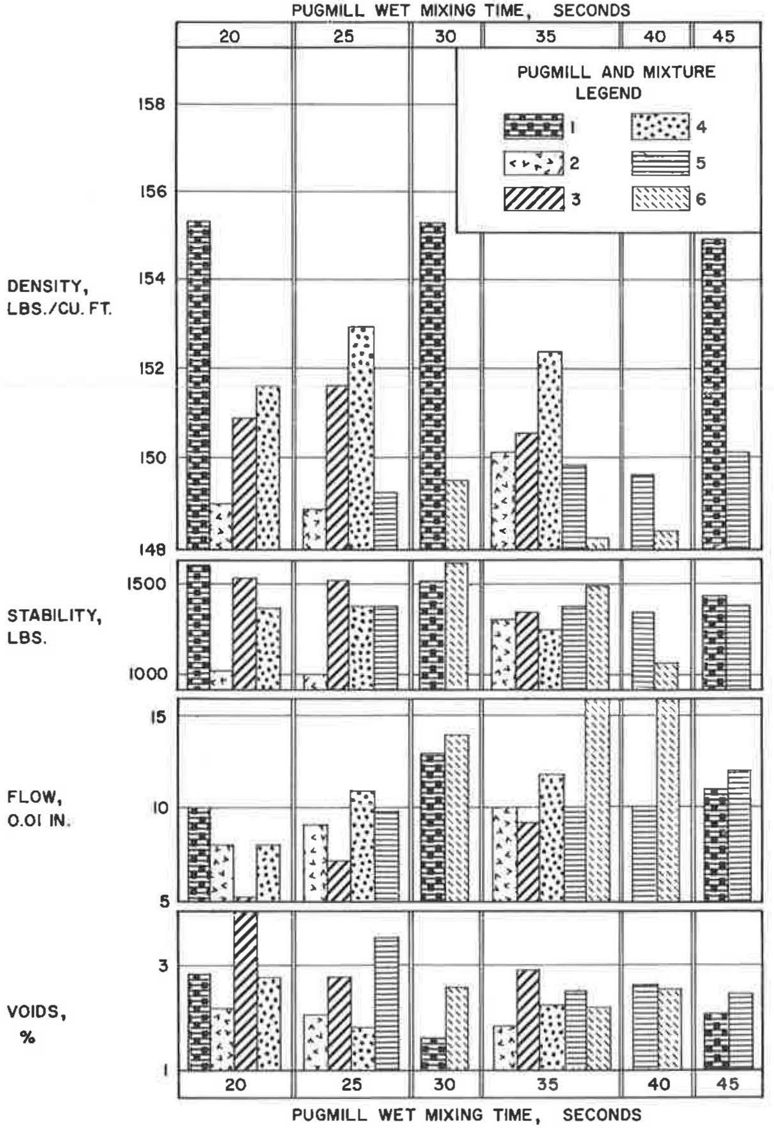


Figure 4. Marshall test properties at various pugmill mixing times.

two extremes were not used for all mixes. In several cases nearly 100 percent coating was observed at 35-sec mixing time. In other cases mixing was so incomplete at 20 sec it would have been impractical to process a count.

Table 7 summarizes the Marshall compaction and test data. Average test results at various pugmill wet mixing times are shown in Figure 4.

Analysis of Test Results

Analysis of test data generally involved working with average test values for each mixture to determine average data trends, and a statistical analysis of data variability. Considering all factors of data variability and desired aggregate coating, minimum wet mixing times were established for each of the six plants and mixes of the study. Also, a comparison of Marshall and Ross Count test results was made for laboratory mixing,

TABLE 8

AVERAGE ROSS COUNT AND MARSHALL TEST VALUE SUMMARY[a]

Mixture	Pugmill Wet Mixing Time (sec)					
	20	25	30	35	40	45
			(a) Ross Count, % uncoated			
1	14.5	9.3[b]	4.4	3.1[b]	1.9[b]	0.8
2	12.8	4.9	2.5[b]	0.3	0.0[c]	0.0[c]
3	14.9	4.6	2.7[b]	1.0	0.0[c]	0.0[c]
4	13.0	1.9	0.9[b]	0.0	0.0[c]	0.0[c]
5	22.3[c]	17.2	11.8[b]	6.7	1.1	2.0
6	62.0[c]	44.0[c]	25.2	6.7	3.0	0.0[c]
Avg	23.3	13.7	7.9	3.0	1.0	0.5
			(b) Density, pcf			
1	155.2	155.3[b]	155.3	155.2[b]	155.0[b]	154.9
2	149.0	148.9	149.5[b]	150.2	150.8[c]	151.5[c]
3	150.9	151.6	151.1[b]	150.6	150.1[c]	149.6[c]
4	151.6	152.9	152.7[b]	152.4	152.1[c]	151.9[c]
5	148.9[c]	149.2	149.5[b]	149.8	149.6	150.1
6	152.1[c]	150.8[c]	149.5	148.2	148.3	148.4[c]
Avg	151.3	151.5	151.3	151.1	151.0	151.1
			(c) Voids, %			
1	2.8	2.0[b]	1.3	1.5[b]	1.7[b]	1.9
2	2.0	1.9	1.7[b]	1.6	1.5[c]	1.3[c]
3	4.2	2.7	2.8[b]	2.9	3.0[c]	3.1[c]
4	2.7	1.6	2.3[b]	2.0	1.7[c]	1.4[c]
5	4.2[c]	3.6	3.0[b]	2.4	2.5	2.3
6	3.5[c]	3.0[c]	2.5	2.0	2.4	2.8[c]
Avg	3.2	2.5	2.3	2.1	2.1	2.1
			(d) Stability, lb			
1	1575	1550[b]	1520	1490[b]	1450[b]	1420
2	1010	970	1130[b]	1300	1460[c]	1620[c]
3	1540	1490	1420[b]	1355	1340[c]	1250[c]
4	1375	1365	1290[b]	1230	1170[c]	1100[c]
5	1360[c]	1375	1370[b]	1365	1335	1395
6	1875[c]	1740[c]	1615	1490	1050	600[c]
Avg	1455	1415	1390	1370	1300	1230
			(e) Flow, 0.01 in.			
1	10	11.5[b]	13	12.4[b]	11.7[b]	11
2	8	9	9.5[b]	10	10.5[c]	11[c]
3	5	7	8[b]	9	10[c]	11[c]
4	8	11	11.5[b]	12	12.5[c]	13[c]
5	8.5[c]	9	9.5[b]	10	10	12
6	10[c]	12[c]	14	16	16	16[c]
Avg	8.3	9.9	10.9	11.6	11.8	12.3

[a]Tabulated values were obtained from average test data except where noted.
[b]Interpolated values were obtained from straight-line connections of data.
[c]Extrapolated values were obtained from straight-line extensions of data.

minimum specification mixing time of 45 sec, and the minimum mixing times established by this study.

Effect of Mixing Time.—Table 8 summarizes average test values for the three mixing periods used for each mixture. Three additional values are included which were obtained by straight-line connections or extensions of average values. The interpolated and extrapolated data are approximate but satisfactory to determine average data patterns shown by Figures 6 and 7. The trends shown can normally be expected with the indicated values being relative and only representative of data obtained.

Figure 6 shows the relationship of Ross Count variation with pugmill wet mixing time. The indicated solid-line curve is representative of 90 individual Ross Counts, supplemented with the interpolated and extrapolated data. The curve shows nearly 100 percent coating is obtained at the present minimum specification mixing time of 45 seconds. Aggregate coating will decrease progressively with reductions in pugmill mixing time.

Figure 7 shows Marshall test data trends resulting from an analysis method as described for Figure 6. Density values were relatively unaffected by mixing time

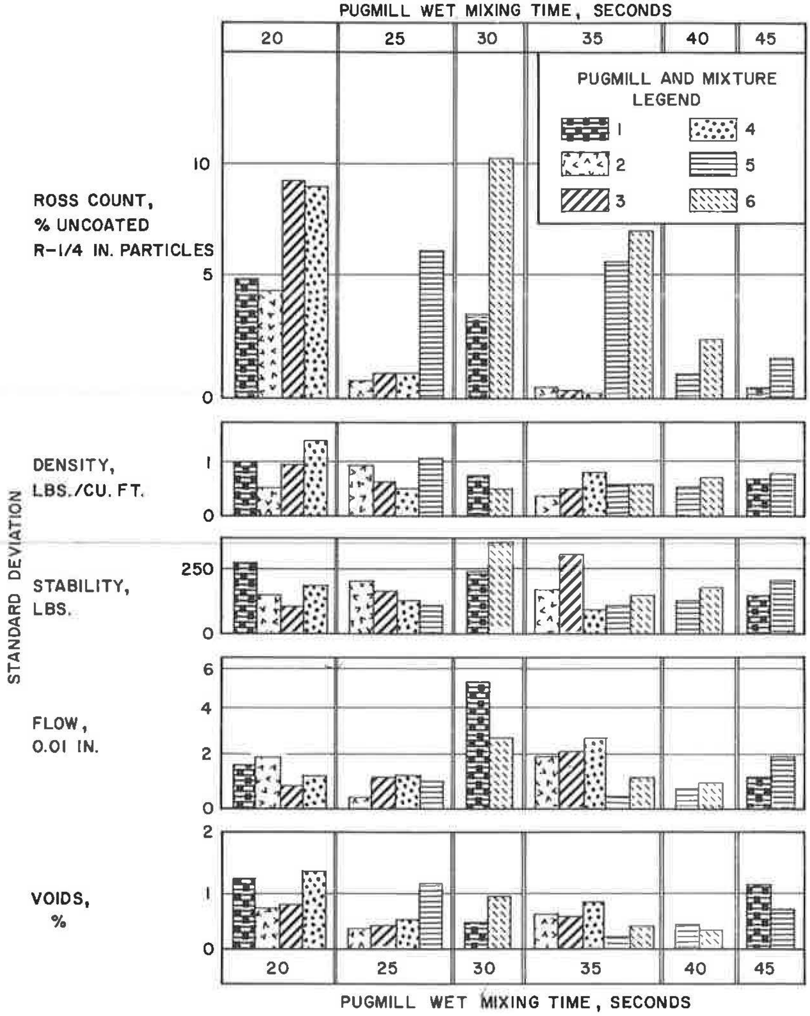


Figure 5. Standard deviation of Ross Count and Marshall test properties at various mixing times.

changes. The decrease in void contents with increased mixing time is explained as a test error. At low mixing times a mix has more uncoated particles which permits higher absorption by the aggregate during the vacuum saturation procedure of the Rice maximum density test. It is possible to correct for aggregate absorption by obtaining a saturated surface-dry weight of mix; however, this procedure is not normally followed in the mix design laboratory.

Marshall stability and flow values decreased and increased, respectively, as mixing time was increased. The variation of flow with mixing time curve in Figure 7 correlates well with the Ross Count vs mixing time curve in Figure 6. For the 20- to 45-sec mixing periods considered, the average flow range was 0.04 in. and stability range 225 lb. The void contents were at or below 2 percent for mixing times above 35 sec. The current requirements of the Wisconsin specifications lists 2 percent as a minimum. Stability became rapidly more critical above 35-sec mixing time and only slightly above the specification minimum of 1,200 lb at the 45-sec mixing period.

Variability of Test Results.—Evidence of considerable variation in test data was apparent in the analysis of results. Inspection of the data suggested that much variation was present in replicate sample test results. It was suspected, also, that changes in mixing time had affected Marshall test properties. A statistical analysis was made to determine the magnitude of test result variability at various mixing times and to determine the degree of significance of the variability.

Table 9 summarizes standard deviation values for all measured properties for each mixture of the study. As in Table 8, straight-line connections and extensions were used to complete the data. Figure 5 shows average values of standard deviation which are directly related to data represented by Figures 3 and 4.

Figure 5 indicates a standard deviation of density of less than 1 pcf in all but one case, which implies that good compaction control was obtained. Normally, the variability of Marshall properties was not affected by mixing time. Any apparent trend of

TABLE 9

STANDARD DEVIATION OF ROSS COUNT AND MARSHALL TEST VALUE SUMMARY[a]

Mixture	Pugmill Wet Mixing Time (sec)					
	20	25	30	35	40	45
			(a) Ross Count, % uncoated			
1	5.13	4.32[b]	3.58	2.51[b]	1.49[b]	0.47
2	4.57	0.75	0.58[b]	0.44	0.28[c]	0.12[c]
3	9.34	1.11	0.70[b]	0.31	0.0[c]	0.0[c]
4	8.99	1.15	0.61[b]	0.10	0.0[c]	0.0[c]
5	6.77[c]	6.42	6.13[b]	5.82	1.01	1.75
6	16.35[c]	13.28[c]	10.24	7.20	2.59	0.0[c]
Avg	8.53	4.51	3.64	2.73	0.90	0.39
$ts/\sqrt{n}$	10.58	5.60	4.52	3.39	1.12	0.48
			(b) Density, pcf			
1	0.93	0.79[b]	0.67	0.65[b]	0.64[b]	0.61
2	0.47	0.86	0.61[b]	0.36	0.12[c]	0.0[c]
3	0.91	0.56	0.50[b]	0.43	0.37[c]	0.31[c]
4	1.35	0.41	0.57[b]	0.73	0.91[c]	1.08[c]
5	1.17[c]	0.99	0.77[b]	0.56	0.47	0.68
6	1.17[c]	0.30[c]	0.47	0.58	0.64	0.70[c]
Avg	0.83	0.65	0.60	0.55	0.53	0.56
$ts/\sqrt{n}$	0.87	0.68	0.63	0.58	0.56	0.59
			(c) Voids, %			
1	1.18	0.81[b]	0.45	0.67[b]	0.87[b]	1.11
2	0.66	0.35	0.45[b]	0.57	0.67[c]	0.78[c]
3	0.75	0.40	0.45[b]	0.50	0.54[c]	0.59[c]
4	1.37	0.46	0.60[b]	0.74	0.90[c]	1.04[c]
5	1.60[c]	1.12	0.65[b]	0.19	0.41	0.66
6	2.05[c]	1.49[c]	0.92	0.37	0.29	0.19[c]
Avg	1.26	0.77	0.59	0.51	0.61	0.73
$ts/\sqrt{n}$	1.32	0.81	0.62	0.53	0.64	0.77
			(d) Stability, lb			
1	278	274[b]	270	233[b]	198[b]	163
2	159	218	203[b]	187	172[c]	157[c]
3	114	182	258[b]	333	408[c]	483[c]
4	209	137	117[b]	99	76[c]	57[c]
5	132[c]	125	120[b]	115	146	239
6	870[c]	635[c]	399	167	199	230[c]
Avg	294	262	228	189	200	222
$ts/\sqrt{n}$	309	275	239	198	210	233
			(e) Flow, 0.01 in.			
1	1.9	3.4[b]	5.4	4.1[b]	2.8[b]	1.5
2	2.1	0.4	1.2[b]	2.1	3.0[c]	3.8[c]
3	0.9	1.2	1.7[b]	2.3	2.9[c]	3.4[c]
4	1.4	1.3	2.2[b]	3.0	3.8[c]	4.7[c]
5	1.4[c]	1.1	0.7[b]	0.4	0.9	2.2
6	6.3[c]	4.6[c]	3.0	1.3	1.1	0.9[c]
Avg	2.3	2.0	2.4	2.2	2.4	2.8
$ts/\sqrt{n}$	2.4	2.0	2.5	2.3	2.5	2.9

[a]Tabulated values were obtained from computed standard deviation values except where noted. Values of n are 5 and 6 for Ross Count and Marshall test data, respectively.
[b]Interpolated values were obtained from straight-line connections of data.
[c]Extrapolated values were obtained from straight-line extensions of data.

variation can be related to the magnitude of the test values since higher test values generally show higher variation.

Generally, regarding Ross Count variability, it is evident from Figure 5 that variability was greatest at low mixing times. Thus, as the number of uncoated particles increased, the expected variability in counting increased.

Test Control Limits.—A further statistical interpretation of standard deviation data is to establish tolerance limits (confidence limits) for the various test properties. Multiplication of the standard deviation by two is one method to obtain tolerance limits (95 percent confidence limits). This procedure could have been followed for each mixture and averaged to obtain tolerance limits for data of all six mixes. However, this method would not be meaningful since it would only be applicable to the six mixes of this particular study, which may or may not be typical. Regardless, the range of tolerance limits would be high due to the wide difference in test properties for the six mixes.

To obtain reasonable tolerance limits, the relationship $\bar{x} \pm ts\sqrt{n}$ was used. The average standard deviation values in Table 9 were used for s and values of n were 5 and 6 for Ross Count and Marshall data, respectively. The resulting limits (deviations from the mean, $\bar{x}$) are shown by the dashed lines of Figures 6 and 7. To interpret correctly the tolerance lines it is necessary to consider the solid lines as representing average data for any given plant and mix. The tolerance lines, then, indicate 95 percent confidence limits for the one plant and mix having the solid-line average values, when sampling, number of replicate samples, compacting, curing and testing procedures are essentially as followed in this study.

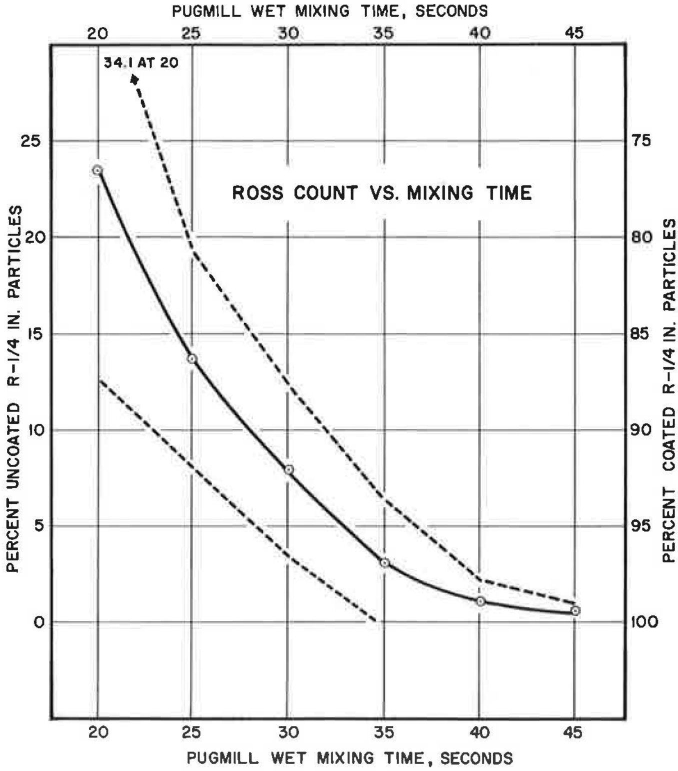
Figure 6 indicates a tolerance range of only 1 percent uncoated particles at the 45-sec mixing time. The range increases rapidly when mixing time is decreased and reaches 21.2 percent at 20 sec. Thus, it is concluded that the reliability of a single Ross Count is decreased as mixing time is decreased. A larger number of counts would be required at low mixing times to establish an accurate Ross Count.

Figure 7 shows the following tolerance limit ranges for a given wet mixing time between 20 and 45 sec using procedures of this study and three sets of duplicate specimens per mixing time: 1.5-pcf density, 500-lb stability, 0.05-in. flow and 1.5 percent voids. It is obvious that any individual Marshall test value should be interpreted liberally. Furthermore, these data warrant increasing the number of replicate specimens in future testing programs from two to an absolute minimum of three replicate specimens per sampling.

Pugmill Mixing Time Recommendations.—Considering all previous discussions, minimum pugmill wet mixing times were established for the six plants and mixes of this study. On the basis of permitting an allowable maximum of 3 percent uncoated coarse particles for a surface mix, absolute minimum mixing times are given in Table 10. Straight-line connections of data were used to determine the mixing time intersection point for 3 percent uncoated particles. Generally, 97 percent coating would be considered adequate for mixing completeness and avoids an unreasonable specification of 100 percent coating which would not allow a tolerance for variation due to chance.

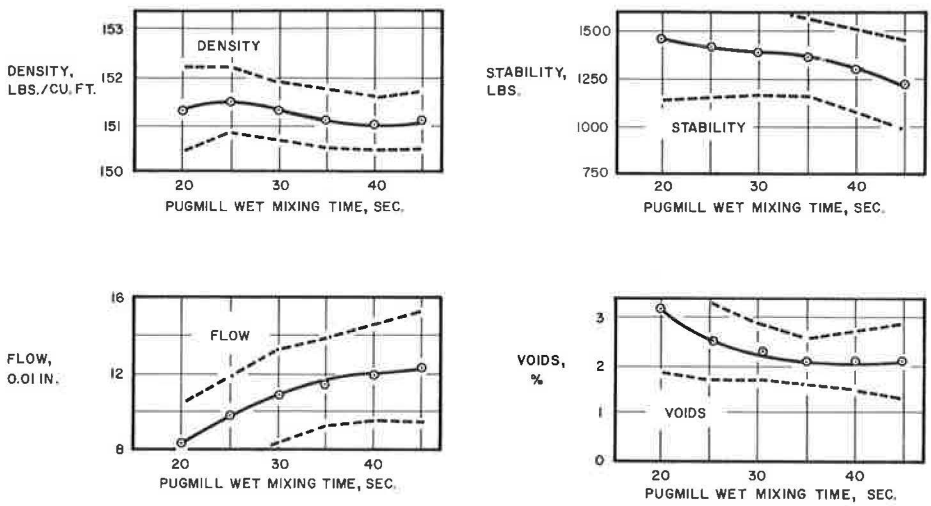
However, a contractor would incur extreme risk by operating at an absolute minimum mixing time. To add a degree of safety, it is recommended that mixing time be set to permit 1.5 percent uncoated particles on the average; which, according to Figure 6, should avoid exceeding the 3 percent uncoated limit. The mixing time increase from absolute to recommended minimum mixing time is very slight—2 to 6 sec for the six mixes of this study.

The recommended mixing times of Table 10 give unmistakable evidence that an arbitrary mixing time of 45 sec was adequate for the six mixes of this study. However, a lower mixing time would suffice in all cases. Obviously, the only way to establish a correct mixing time is to do so for each plant and mix, being aware that many variables are present and that day-to-day changes in mixing time requirements are probable because of changes in materials, weather, plant operation, pugmill condition and sampling. It is possible that certain plants, operating under set conditions to produce a given mix, would require more than 45 sec for adequate mixing as measured by the Ross Count Method.



---- TOLERANCE LIMITS OR 95% CONFIDENCE LIMITS ($\bar{X} \pm TS/\sqrt{N}$) FOR N=5 "COUNTS" PER MIXING TIME FOR ONE PLANT AND MIX.

Figure 6. Variation of particle coating with pugmill mixing time (trend for average data for six plants and mixes).



---- TOLERANCE LIMITS OR 95% CONFIDENCE LIMITS ($\bar{X} \pm TS/\sqrt{N}$) FOR N=6, 3 SETS OF DUPLICATE SPECIMENS, PER MIXING TIME FOR ONE PLANT AND MIX.

Figure 7. Variation of Marshall test properties with pugmill mixing time (trends for average data for six plants and mixes).

TABLE 10

MINIMUM PUGMILL WET MIXING TIMES SHOWN BY ROSS COUNTS

Plant and Mixture No.	Absolute Min. Mixing Time[a] (sec)	Recommended Min. Mixing Time[b] (sec)
1	36	42
2	29	33
3	29	34
4	25	27
5	38	40
6	40	43

[a]Criterion for establishing mixing time was a maximum of 3 percent uncoated $R\frac{1}{4}$-in. particles.
[b]Criterion for establishing mixing time was a mean of 1.5 percent uncoated $R\frac{1}{4}$-in. particles.

Comparison of Mix Properties at Various Mixing Times.—Table 11 summarizes mix properties of the laboratory mixes, field mixes for the current minimum specification of 45-sec mixing, and the absolute and recommended minimum mixing times of Table 10. The comparison provided by Figure 8 shows the results for laboratory mixing are different, generally, from field results at any of the three pugmill mixing times. At the recommended minimum mixing times, void contents are below the current Wisconsin specification minimum of 2 percent. However, low values are also indicated at the 45-sec pugmill mixing time. Low stability values (below the current specification minimum of 1,200 lb) are shown for two projects at the absolute minimum mix time. However, values are within the tolerance limits and are above minimum when the mixing time is increased to the recommended minimum low. The most pronounced differences between laboratory and field mix properties are for mixture 6 which consisted of a high wear loss aggregate and a high asphalt content.

TABLE 11

ROSS COUNT AND MARSHALL TEST VALUES FOR SELECTED MIXING CONDITIONS

Mixture	Marshall Test Properties				Ross Count, % Uncoated $R\frac{1}{4}$-In. Particles
	Density (pcf)	Stability (lb)	Flow (0.01 in.)	Voids (%)	
	(a) Laboratory Mixing[a]				
1	155.0	1750	14	2.3	-
2	149.7	1300	10	2.7	-
3	152.4	1770	11	2.3	-
4	152.6	1570	12	2.5	-
5	147.0	1530	9	3.6	-
6	145.4	1940	9	4.3	-
	(b) 45-Second Minimum Specification Mixing[b]				
1	154.9	1420	11	1.9	0.8
2	151.5*	1620*	11*	1.3*	0.0*
3	149.6*	1250*	11*	3.1*	0.0*
4	151.9*	1100*	13*	1.4*	0.0*
5	150.1	1395	12	2.3	2.0
6	149.1	1720	18	2.1	0.1
	(c) Absolute Minimum Mixing by Ross Count[c]				
1	155.1*	1480*	12*	1.5*	2.9*
2	149.4*	1100*	9*	1.8*	3.0*
3	151.2*	1430*	8*	2.8*	3.0*
4	152.9	1360	11	2.6	1.9
5	149.7*	1350*	10*	2.4*	3.0*
6	148.3	1050	16	2.4	3.0
	(d) Recommended Minimum Mixing by Ross Count[c]				
1	155.0*	1430*	11*	1.7*	1.4*
2	149.9*	1230*	10*	1.7*	1.2*
3	150.7*	1370*	9*	2.9*	1.3*
4	152.8*	1330*	11*	2.5*	1.5*
5	149.6	1330	10	2.5	1.1
6	148.7*	1460*	17*	2.2*	1.2*

[a]Tabulated values are from laboratory mix design reports and represent mixture properties at asphalt content used in field.
[b]Tabulated values followed by an asterisk are extrapolated from straight-line extensions of data.
[c]Tabulated values followed by an asterisk are interpolated from straight-line connections of data.

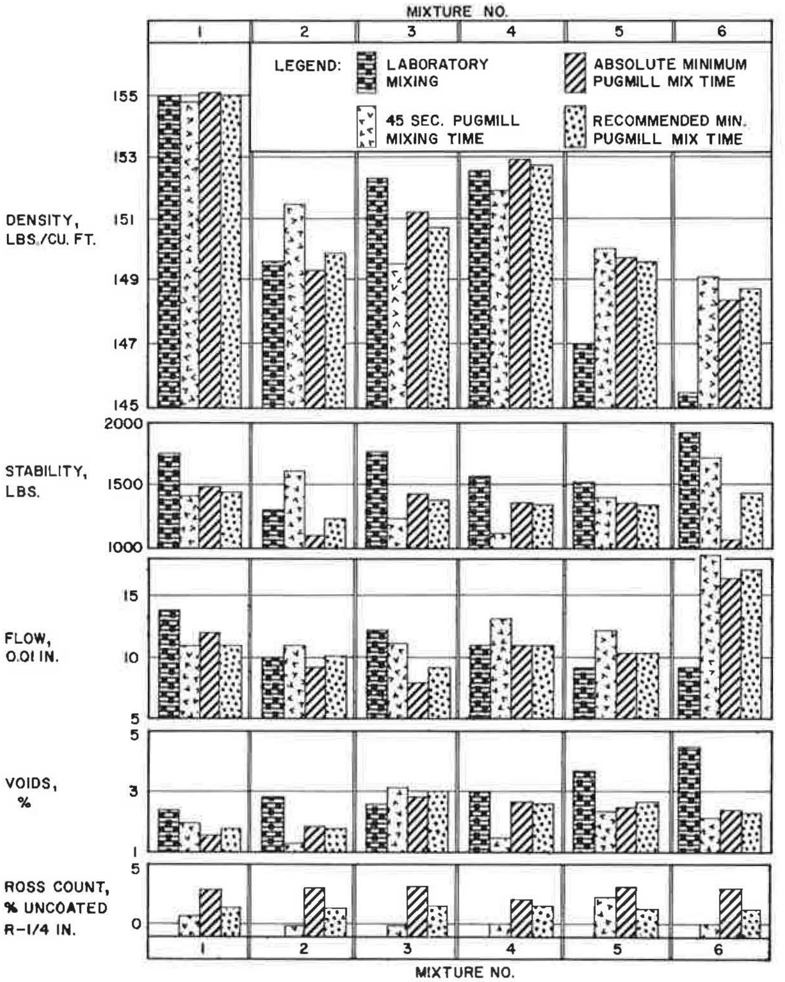


Figure 8. Ross Count and Marshall test values at various minimum mixing times.

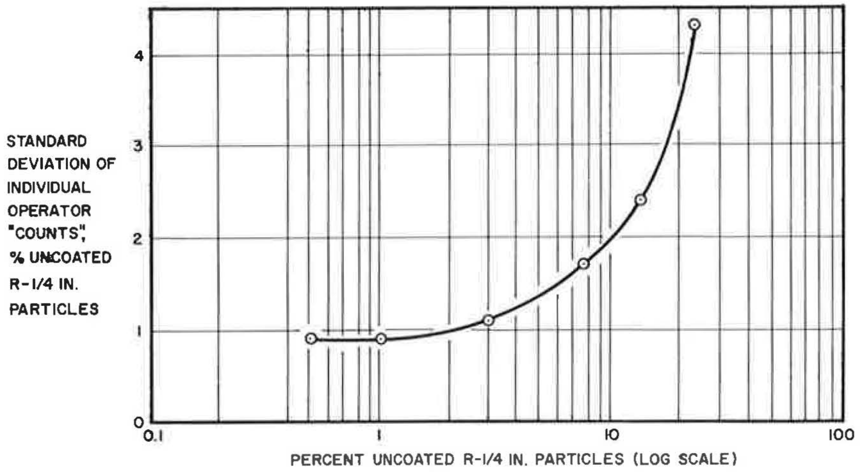


Figure 9. Effect of percent uncoated particles on standard deviation of individual operator "counts" (trend for average data for six plants and mixes).

Figure 8 also shows that the laboratory results generally correlate more closely with field test results for reduced mixing times than with 45-sec mixing time results. This phenomenon suggests that the degree of mixing in the laboratory (in addition to laboratory and field compaction differences) is different from that achieved after 45-sec pugmill mixing. The latter condition may be permitting "over-mixing," or mixing beyond an optimum period, with no apparent additional benefits gained in mix properties. Although sufficient evidence is not presented here to warrant a conclusion, it is worth considering that an optimum mixing time is possible and over-mixing may be nonbeneficial to mix quality.

Statistical Evaluation of Field Ross Count Data.—To avoid excessive delay in counting and to speed field work, two or more operators counted a sample. The statistical analysis of individual operator Ross Counts included paired comparison of counts of operators and standard deviations of counts only due to different operators counting a given split sample.

The data permitted making eight paired comparisons of Ross counts, only two of which were significantly different. It was concluded that operators counted equally and the two exceptions can be explained by one operator lacking experience, and the fact that the major differences are only related to mixture 6.

Standard deviation of counts by different operators was found to vary considerably between mixes. The standard deviation was lowest at high mixing times and increased as mixing time decreased (uncoated particles increased). The trend of standard deviation variation with mixing time on an arithmetic plot is similar to the curve shape of Figure 6. A smooth semi-log plot is obtained (Fig. 9) when the abscissa is changed from mixing time to percent uncoated coarse aggregate. Figure 9 shows an average trend from which actual values for a single project may vary excessively.

Three days were generally required to complete work at any one plant. For the final three projects it was decided to reduce the field work by field-counting only sufficient samples to establish a mixing time range, and transport the remaining samples to the laboratory for counting. Paired comparisons of 35 recounts (the recounts were made about one week after sampling) had previously shown no evidence of a significant difference due to the time at which a given sample was counted. The recount results strongly indicated that comparisons are in closer agreement when experienced counters recount a given sample than when they separate (split) and compare counts for a sample. This result is explained readily by the fact that different particles are being counted in the latter case.

EVALUATION OF ROSS COUNT METHOD

Evidence has been presented that the Ross Count Method could be used satisfactorily to establish and control minimum mixing time requirements. However, it is desirable that further studies be made to determine the effects of certain variables before incorporating the method into a field control program. At this time data are very limited and additional work is required to prove the adequacy of the method.

A Ross Count may be helpful in the case of especially troublesome mixes. In Wisconsin, a mobile bituminous testing laboratory is dispatched to a project when mix quality, as measured by Marshall properties, becomes questionable. If unwarranted mixing is occurring, a reduced mixing time, as established by a Ross Count, may possibly result in some improvement of the Marshall test properties.

Thus, the Ross Count Method could be used on an experimental basis as a part of the mobile laboratory mix control procedure. Should the method show promise as a useful field control tool for mixes, it could eventually become a part of the plant inspector's test duties. Because of the lack of data and insufficient proof of the full merit of the method, it is recommended that the test be used only on an experimental basis until more complete knowledge of the test and test variations is available.

It is advisable that plant inspectors observe coating by visual inspection of mixes. Observation of the mix in a truck is reasonably reliable, generally, for determining if the aggregate is sufficiently coated. A procedure for actual application of the Ross Count Method in establishing a mixing time could consist of observing the degree of aggregate coating and reducing mixing by 5-sec intervals until some uncoated coarse particles are observed. The mixing time should then be increased 5 sec and a Ross Count obtained to verify that satisfactory coating is being achieved.

CONCLUSIONS

The following statements appear warranted on the basis of the laboratory and field study results of this investigation, and considering information from a review of the literature and past experience.

Ross Count Method

1. The Ross Count Method offers a simple and practical means to measure the degree of coating of coarse aggregate in a bituminous mix. Decreases in mixing time are evidenced by corresponding increases in number or percent of uncoated coarse particles.
2. An individual Ross Count is less reliable at low mixing times since the variability of replicate Ross Counts increases as mixing time decreases. Thus, a greater number of counts may be required at low mixing times to establish an accurate Ross Count.
3. Experienced operators are capable of repeat counting of a given sample and, also, showing little variation when counting split samples. The counting variation is a bias, primarily, and increases as the number of uncoated particles increases.
4. The Ross Count at any mixing time is subject to numerous variables, some of which are plant and pugmill conditions, specific mixture characteristics, material changes, amount of asphalt, mixing temperature, plant operation changes, sampling, number of replicate samples counted, counting operators, and climate or humidity.
5. The Ross Count Method offers one of the most practical approaches to establishing minimum mixing times presently available. However, it is desirable that the effects and control of the numerous variables be studied before accepting the Ross Count Method as an adequate standard procedure.

Marshall Test Property Variation with Mixing Time

1. Evidence was presented to show that all six plants of the study could reduce mixing times from the current Wisconsin minimum specified mixing time of 45 sec and produce satisfactory quality mixtures as measured by the Marshall tests.
2. Reduction of pugmill mixing time to a point where 97 percent coating of coarse aggregate is obtained would not significantly affect Marshall test properties. Excessive reduction of mixing was evidenced by "balls" of asphalt in the mix, a sign of incomplete mixing or aggregate coating.
3. It was indicated that Marshall tests on field specimens showed extreme variability, and therefore, test results on an individual specimen should be interpreted liberally. Variation is caused by sampling, compacting, curing and testing procedure variations. Tolerance limits established from this study with six replicate specimens (three sets of duplicate specimens) indicate an acceptable range of 1.5-pcf density, 500-lb stability, 0.05-in. flow and 1.5 percent voids. The data suggest increasing the number of replicate specimens in future studies.
4. Figure 8 gives evidence that laboratory mixing mix properties do not compare as well with mix properties at 45-sec pugmill mixing as at the reduced mixing times of Table 10. It is suggested that "over-mixing" may result in cases when an arbitrary mixing time is used for all mixes. Sufficient evidence is not available at this time to conclude that extended mixing is not beneficial to mix quality; however, it is probable that an optimum mixing time is possible and additional mixing beyond the optimum may not significantly alter mix quality sufficiently to be warranted.

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Effect of Asphalt Viscosity on Compaction of Bituminous Concrete

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A laboratory study was undertaken to determine the important variables that affect the densification of bituminous concrete during rolling. Of principal interest was the influence of asphalt viscosity on the compaction process; also examined were the effects of number of roller passes, type of roller (steel or rubber), hardness of the supporting medium, and environmental temperature. An attempt was made to simulate full-scale field rolling conditions insofar as possible in this study.

The measurements taken on the compacted mix included unit weights, voids, Marshall stabilities and flow. The results indicate that the effects of asphalt viscosity during compaction (between approximate mix temperatures of 300 F and 160 F) on stability is noticeable while the relative density and voids changes are small. The Marshall stability values, however, were found to be several times lower than those expected using a standard Marshall specimen compaction procedure.

The rubber-tire roller and rolling procedure used in this study gave a slightly less dense and less stable compacted mix than did a steel roller.

•COMPACTION of bituminous concrete is a stage of construction which transforms the mix from its very loose state into a more coherent mass, thereby permitting it to carry traffic loads.

Since compaction is a densification process involving the displacement of aggregate particles, the efficiency of the compactive effort will be a function of the internal resistance of the bituminous concrete. This resistance includes aggregate interlock, frictional resistance, and viscous resistance. The interlock and the frictional resistance are primarily functions of the geometry and surface characteristics of the aggregate. The viscous resistance is a function of the viscosity of the binding agent, asphalt.

An increase in density will result in an increase of the strength of the pavement. Previous publications (1, 2, 3) have indicated that the initial compaction during construction will give densities and stabilities that are below those measured in pavements after several years of exposure to traffic.

Theoretical and experimental work by Nijboer (4) divided the major variables in the process of compaction into two general categories: properties of the mix and properties of the roller. The properties of the mix include: (a) angle of internal friction and (b) viscosity of the bituminous mix. The properties of the roller include: (a) the weight of the roller, (b) the length of the roller, (c) the diameter of the roller, (d) the speed of rolling, and (e) the number of coverages. The work described here is an attempt to add to the present knowledge including parameters such as asphalt viscosity, various types of supports, different environmental compaction temperatures, and steel and rubber rollers.

Paper sponsored by Committee on Construction Practices—Flexible Pavement.

PURPOSE AND SCOPE

The general goal of this research project was to attempt to study, in the laboratory, compaction of a given bituminous-concrete mix under full-scale simulated steel and rubber-tire rollers. Specifically, the main purpose was to measure the effects of asphalt viscosity on the compaction process.

Massachusetts Type I surface mix with 6.5 percent of 85-100 Venezuelan asphalt was used for making 12- by 12-in. bituminous-concrete slabs, 2 in. thick (Fig. 1). Altogether about 400 specimens were compacted and the following primary measurements were made: (a) unit weights, (b) void contents, and (c) Marshall stabilities.

Constants and Variables

Steel Roller Compaction.—The roller diameter was 60 in. (in the form of a strip of curved steel plate, see Figs. 2 and 3).

The load was 250 lb per lin in.

Five initial mixing temperatures were used: 325 F, 277 F, 240 F, 217 F, 195 F (to correspond to asphalt viscosities of 100, 300, 900, 2000, and 5000 cps). Environmental placement and compaction temperatures were 80 F and 40 F.

Three hardnesses of supports under the specimen during compaction were K = 100 pci, K = 300 pci, and K = 2000 pci. (K = modulus of support reaction, in pounds per square in. per 1 in. deflection or in pounds per cubic inch.)

There were 1, 3, 6, and 18 coverages, and the time lapse between coverages was 2 min.

Figure 1. 12- by 12- by 2-inch specimen.

Figure 2. Compaction machine.

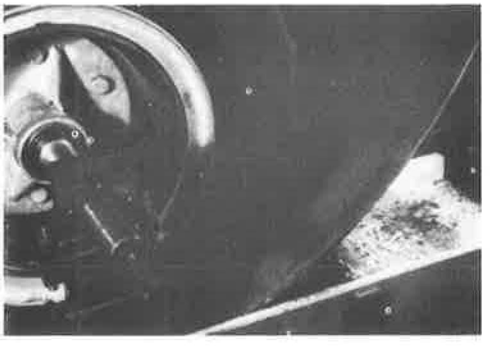

Figure 3. Close-up of steel roller.

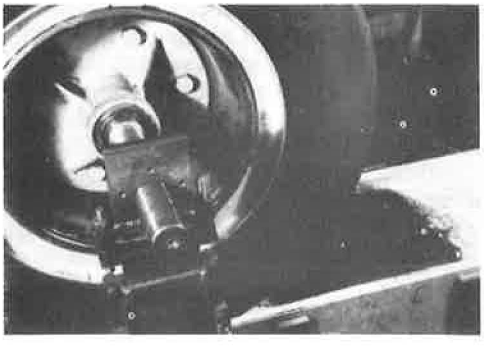

Figure 4. Close-up of rubber roller.

Rubber-Tired Roller Compaction.—The roller tire was 7.50 × 16 (Fig. 4) with a load of 3500 lb, and a 100-psi tire pressure. Initial mixing temperatures, and corresponding viscosities were the same as for steel roller compaction.

Environmental placement and compaction temperature was 80 F, with two hardnesses of supports: K = 100 pci and K = 2000 pci.

The same coverages and time lapse between coverages were used.

Miscellaneous.— Several additional trial tests were run to check the sensitivity of the results at extreme compaction temperatures, high base support values (concrete slab support), changes in roller loads and diameter, etc.

REASONS BEHIND VARIABLES

Before the actual test specimens were made, a number of preliminary experiments were conducted.

Dimensions of the Specimens

A simulation of road conditions was attempted. The thickness of 2 in. was chosen in order to use the Marshall procedure for strength measurements (in practice the Massachusetts Type I top course is usually compacted in layers thinner than 2 in.).

In order to reduce the size of the mix batches, trial experiments were conducted compacting the mix in various sized slabs, taking cores and comparing their density-void-stability values. The smallest specimen still simulating a "continuous" bituminous concrete mat was found to be in this case around 12 by 12 in. Thus all compacted specimens were made this size. They weighed about 22 lb each.

Choosing the Base Supports

Bituminous concrete may be placed on supports having different hardness or stiffness. To investigate the support values that would indicate differences in the compacted product, a series of preliminary compaction tests was conducted and it was found that supports with K values above 2000 gave similar results; there was a slight change in density-stability values at K values lower than 2000. This led to the choice of three supports: K = 100, K = 300, and K = 2000, which were simulated by 1-in. thick pads of foam rubber, urethane elastomer, and hard rubber, respectively. The K values, in pounds per cubic inch, were determined by compressing a 12.5 sq in. area of these pads, which were sandwiched between two steel plates, and measuring the load-deformation characteristics.

Rolling Frequency Procedure

Steel and rubber tire rollers with what were considered reasonable dimensions and unit pressures were chosen. A speed of 2 mph was assumed acceptable. In order to simulate the time interval between roller coverages a few field observations were made and it was decided that a 2-min interval between each coverage of the roller would closely approximate average field conditions. As will be shown later this time interval is not very critical; compaction results using the 2-min data can be calculated for other time intervals if necessary.

Other Variables

The materials, mix proportions, mix temperatures during mixing, placing, and compaction were selected according to observations and judgment, simulating conditions in Massachusetts. Added were some extreme conditions, such as mixing and compaction at mix temperatures below 200 F.

The compaction with the steel roller was conducted at two environmental temperatures, 40 F and 80 F, to observe cooling rates of the mix and their effect on compaction.

Figure 5. Mixing machine.

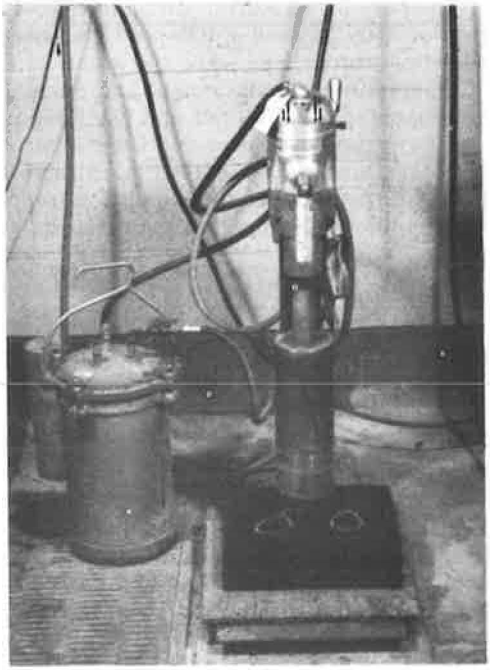

Figure 6. Core drill.

SPECIMEN PREPARATION

The aggregates (22 lb) were heated in an oven overnight to the desired temperature before the asphalt was added. Prior to mixing, the asphalt was heated to the same temperature as the aggregates. Both components were combined and mixed in a covered and Styrofoam-insulated mixing bowl (Fig. 5) for one min. The hot mix was then placed in the compaction box. The following sequence was carried out:

Time Sequence (min)	Action
0- 1	Mixing
1- 8	Transporting, placing and knock-down coverage
at 8	First coverage with roller
8-10	Wait
at 10	Second coverage
10-12	Wait (repeating coverages to completion)

In the case of the steel wheel roller, a knock-down coverage (of 50 lb/in.) was used before the standard 250 lb/in. compaction load was applied. In the case of the rubber roller, a knock-down coverage (50 lb/in.) using a steel roller was applied. Then the standard compactive load of 3500 lb and 100-psi tire pressure was used.

The procedure for the compaction with the rubber roller had to be modified, because the tire traversed only half the width of the specimen with each coverage. One full coverage in 2 min for all points of the specimen was achieved by applying one coverage per min on alternate halves of the specimen.

In all cases, continuous readings of the temperatures of each specimen were taken. After the specimens had reached room temperature, they were removed from the compaction box. The specimens were allowed to stay overnight after which they were placed at 40 F for three hours before the coring operations were performed (Fig. 6). Two cores, 4 in. in diameter and 2 in. thick, were cut out of each compacted specimen. After drying and obtaining specific gravities (and by inference void contents), Marshall stability-flow measurements on both cores were taken (ASTM D-1559-60T).

TEST RESULTS—VISCOSITY MEASUREMENTS

Viscosity of Asphalt

Since the major variable in the entire program was the asphalt viscosity, measurements were made on the fresh and the extracted asphalt.

Fresh Asphalt.—The absolute visocity as a function of temperature was determined experimentally using a Brookfield "Synchro-Lectric" HAT model (range 0-16,000,000 cps) Viscometer. The asphalt to be tested was placed in a 600-ml beaker and suspended in a constant-temperature oil bath. When the desired temperature was reached uniformly in the beaker, the viscometer with spindle No. 1 in place was lowered into the asphalt. Shear stress readings were taken covering a range of 0.5-100 rpm. Stress at each shear rate was plotted on a graph of log (shear stress) vs log (shear rate) and a straight line was drawn through the points. Viscosity at each temperature was calculated at the intersection of these lines with a line of constant energy (RPM x shear stress = constant). The asphalt behaved in a Newtonian fashion with only slight tendencies to be thixotropic (in the range investigated, viscosity decreased with increasing shear rate by only 7%). The results could be plotted as a straight line on a log-log (viscosity) vs log (absolute temperature F) basis (Fig. 7). The final results, after curve fittings by the least squares method, were:

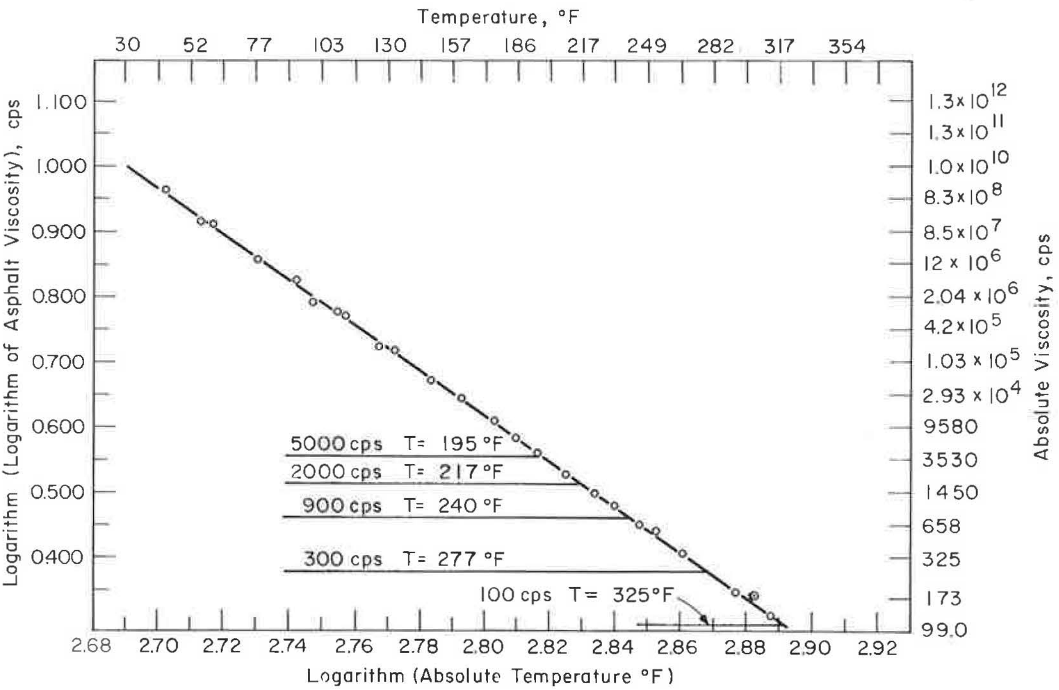

Figure 7. Initial asphalt viscosities and temperatures used in compaction experiments.

$$\log(\log \eta) = 10.1497 - 3.4030 \log(T_{abs}) \pm 0.0042$$

where

η = viscosity, cps; and
T_{abs} = 560 + °F.

This equation holds for temperatures ranging from 40 F to 377 F.

The above results were checked using the Saybolt-Furol viscosity method. Five points were obtained between 210 F and 168 F. Using the approximate relation that absolute viscosity (cps) equals 2 times the Saybolt-Furol time (in sec), the results checked very well with the Brookfield measurements. Additional points to 40 F were obtained by the Shell sliding plate viscometer.

Extracted Asphalt.—Since the viscosity of the asphalt was the single most important variable of the research project, it was clear that the variations in the viscosity of the asphalt due to hardening during mixing and compaction had to be checked. After compaction the asphalt was extracted and distilled using the modified Abson procedure ASTM D-762-49. The viscosity of the extracted asphalt was then determined between 40 and 325 F. The Saybolt-Furol viscometer was used from 325 to 160 F, and to complete the cycle, the Shell sliding plate microviscometer was employed from 160 to 40 F.

The results obtained are shown in Figure 8: one curve gives the viscosity-temperature relationship for the asphalt before heating-mixing and the other two curves, viscosities for the asphalt extracted from mixes which were heated and prepared at 325 F and 195 F.

Cooling of the Mix

The procedure called for heating both the mix and the aggregates to the desired temperature, mixing and compaction. During these various operations the temperature

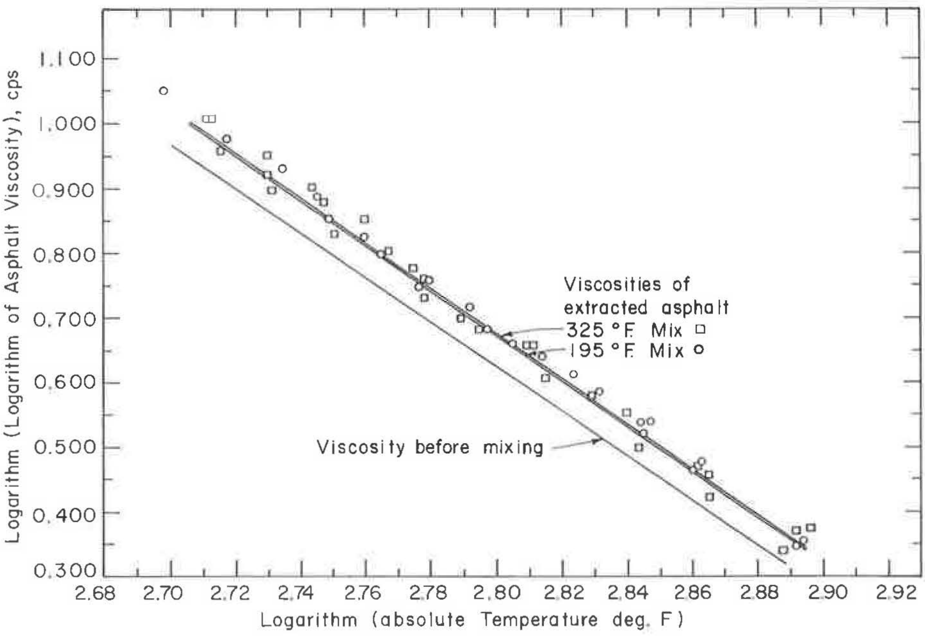

Figure 8. Logarithm of the logarithm of the asphalt viscosity vs the logarithm of the temperature in deg F absolute.

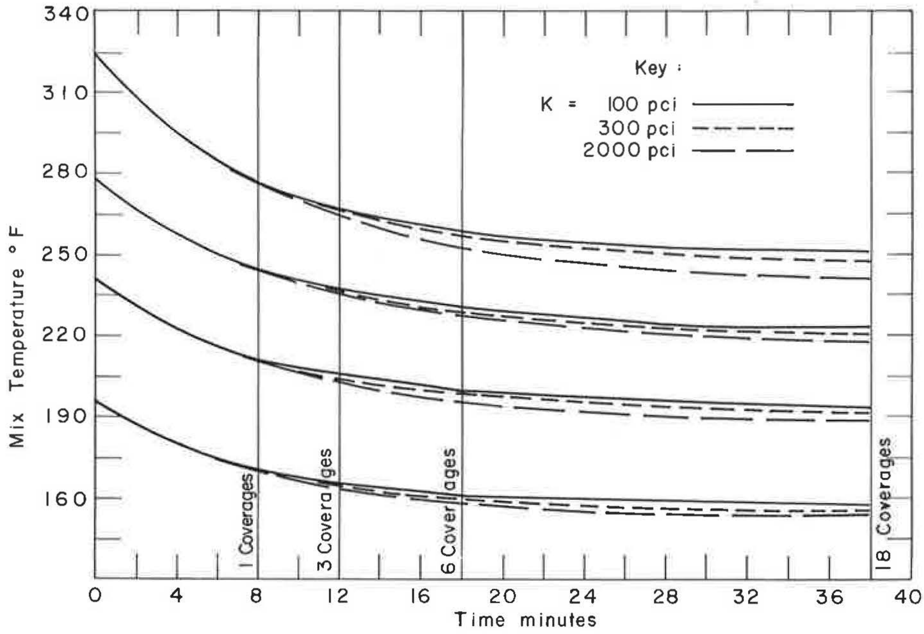

Figure 9. Temperature decay with time at 80 F environmental temperature, steel and rubber rollers, various initial mix temperatures.

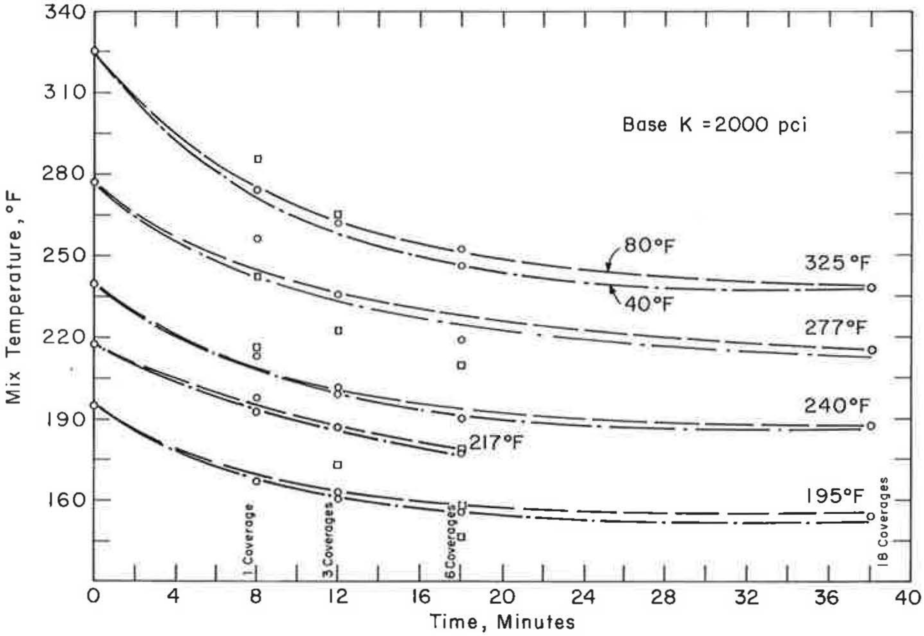

Figure 10. Temperature decay vs time, for steel-rolled specimen.

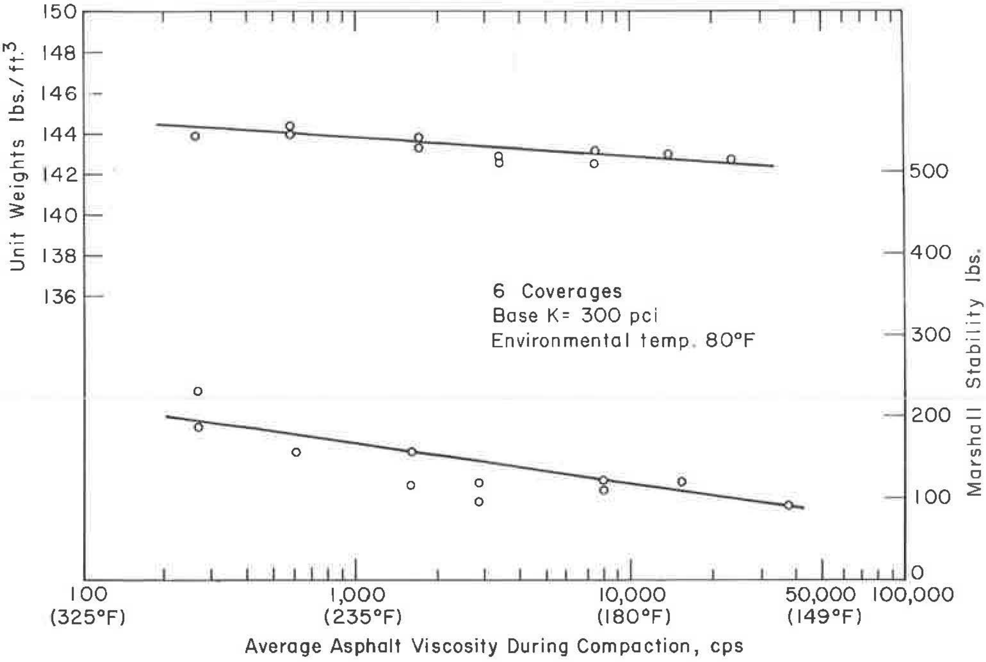

Figure 11. Unit weights and Marshall stability vs average asphalt viscosity, steel roller.

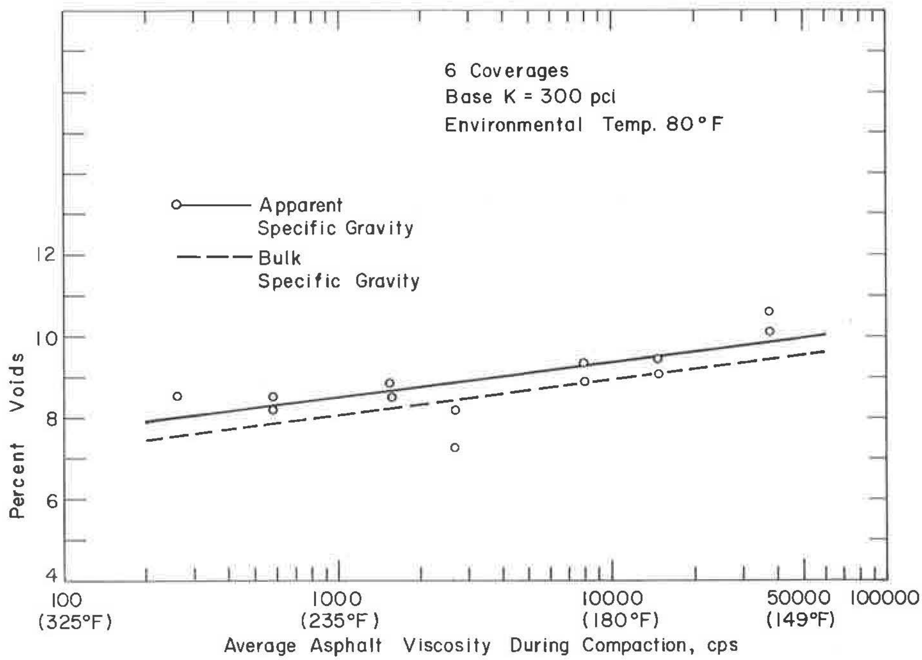

Figure 12. Percent voids vs average asphalt viscosity, steel roller.

of the mix was decreasing and the viscosity of the asphalt was increasing. The temperature-time curves for the various initial mix conditions and types of supports are given in Figure 9.

The cooling rate of the mix was slightly different for various types of base supports used (Fig. 9). This was due to the better conductivity of solid rubber, for example, (K = 2000) as compared to foam rubber (K = 100).

The cooling during the compaction depends also on the environmental compaction temperature (40 or 80 F) as indicated in Figure 10.

Because the viscosity (the temperature) of the mix was changing during the compaction, a certain "average compaction viscosity" had to be used for presentation of data. This was done by taking the temperature of the mix at the time of the first coverage and the temperature at the last coverage, and obtaining the average of these two; the corresponding viscosity was then calculated and designated as "average asphalt viscosity during compaction, cps."

This average compaction viscosity will not be exactly the viscosity of the fresh asphalt because some hardening during the mixing and compaction has taken place. The amount of hardening or increase in viscosity would be reflected by a curve somewhere between the viscosity of the fresh asphalt and the viscosity of the extracted asphalt shown in Figure 8. This amount of hardening was of little importance in the results (the unit weights would change by about 0.2 percent), and therefore, for convenience, the viscosity of the fresh asphalt was always used in the various figures presented in this paper.

Typical Examples

To maintain uniformity in the figures, the average asphalt viscosity during compaction is usually plotted against the other three measurements: density, voids and stability of the mixes. This is illustrated in Figures 11 and 12 for steel roller, 6 coverages, K = 300 and environmental compaction temperature of 80 F. One of the significant findings is that the physical properties of this mix altered proportionally to the logarithm of the average asphalt viscosity during compaction.

TEST RESULTS—COMPARISON OF STEEL AND RUBBER ROLLERS

In the following, the effect of different variables on the density-voids-stability characteristics for each type of roller is discussed.

Effect of Environmental Compaction Temperature

In the series of experiments with the steel roller, two environmental compaction temperatures were used: 40 F and 80 F. The room was brought to the designated temperature and then the mix was placed and compacted.

Figure 13 gives an example of what happens to the unit weight and the stability of the mix when compacted during cold and warm environmental temperatures. The main difference is that the average asphalt viscosity during the compaction is lower in the case of the 40 F temperature compared with the 80 F, other variables being equal. This simply results in slightly lower densities and stabilities, and both curves should follow the same line.

Some deviations from this pattern are expected due to the fact that simple average temperatures (viscosities) are used from the average asphalt viscosity during compaction.

This observation is of practical importance because it indicates that bituminous concrete can be placed and rolled in cold weather at temperatures below 40 F, provided that the needed number of coverages is applied within a short time. Heating the mix to unusually high temperatures is not as promising as repetitious rolling at normal mix laying temperatures (say 325 to 250 F).

The effect of environmental temperature on rubber-wheel rolling was not investigated on the assumption that the trends would be similar to the case previously discussed.

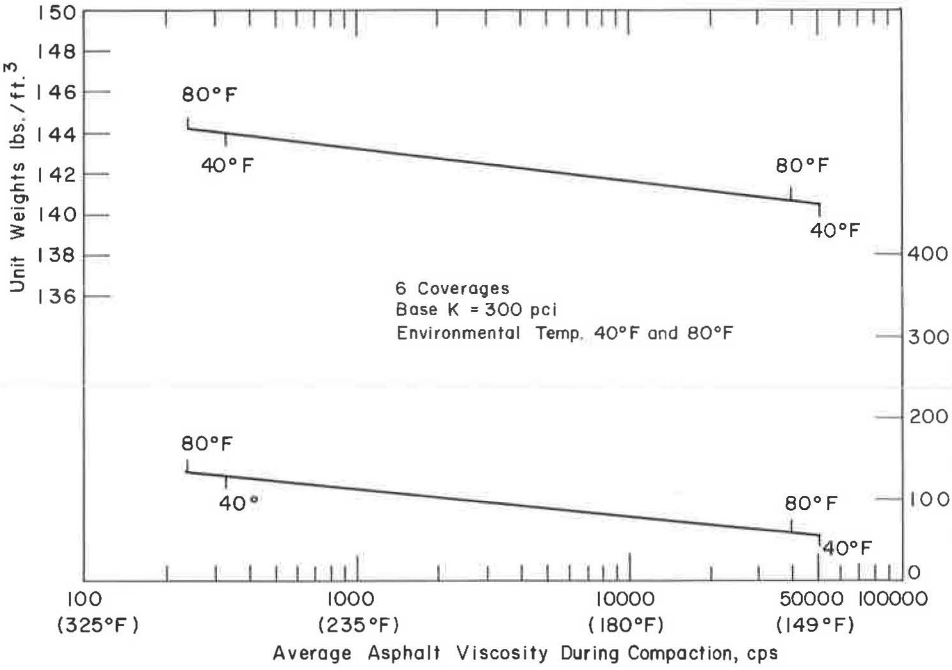

Figure 13. Unit weights vs average asphalt viscosity, steel roller.

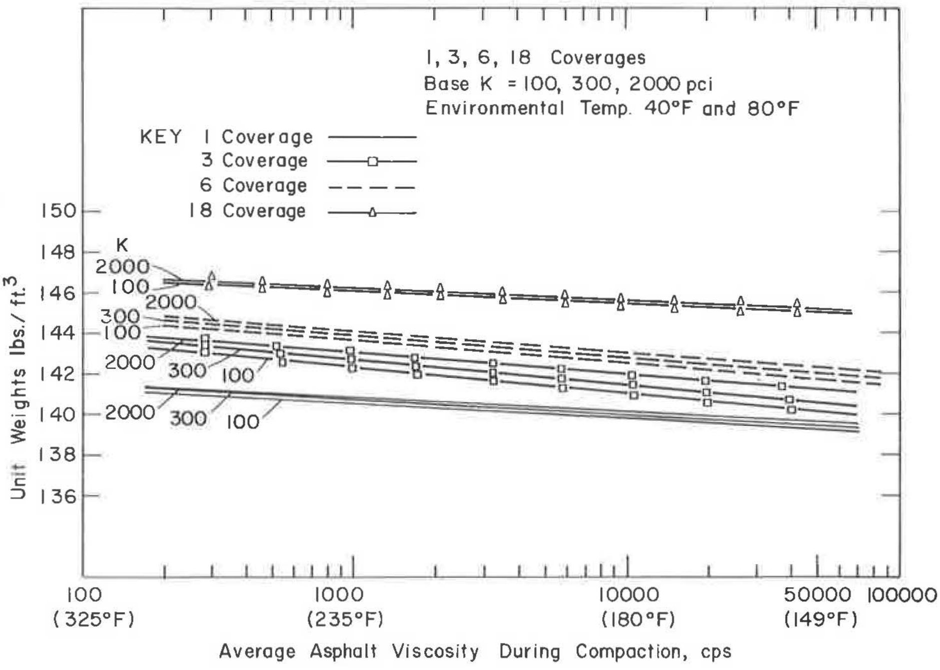

Figure 14. Unit weights vs asphalt viscosity, steel roller.

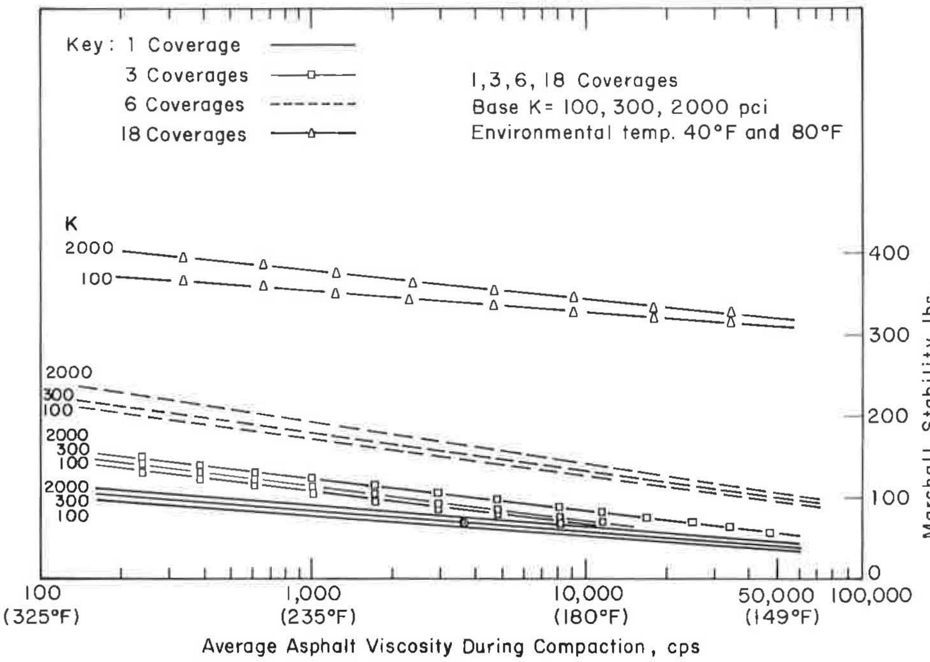

Figure 15. Marshall stability vs average asphalt viscosity, steel roller.

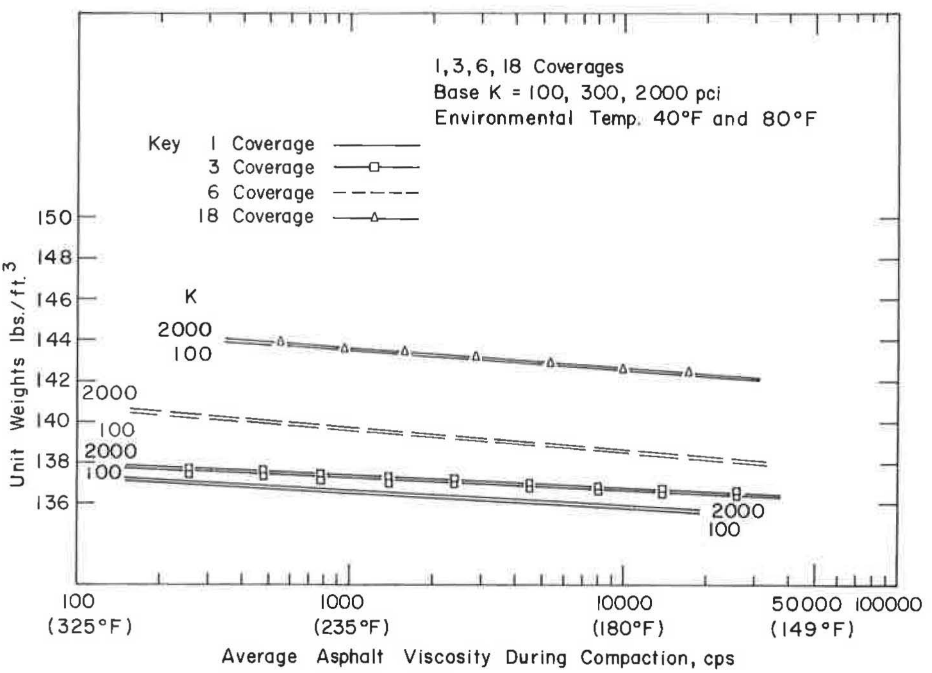

Figure 16. Unit weights vs average asphalt viscosity, rubber roller.

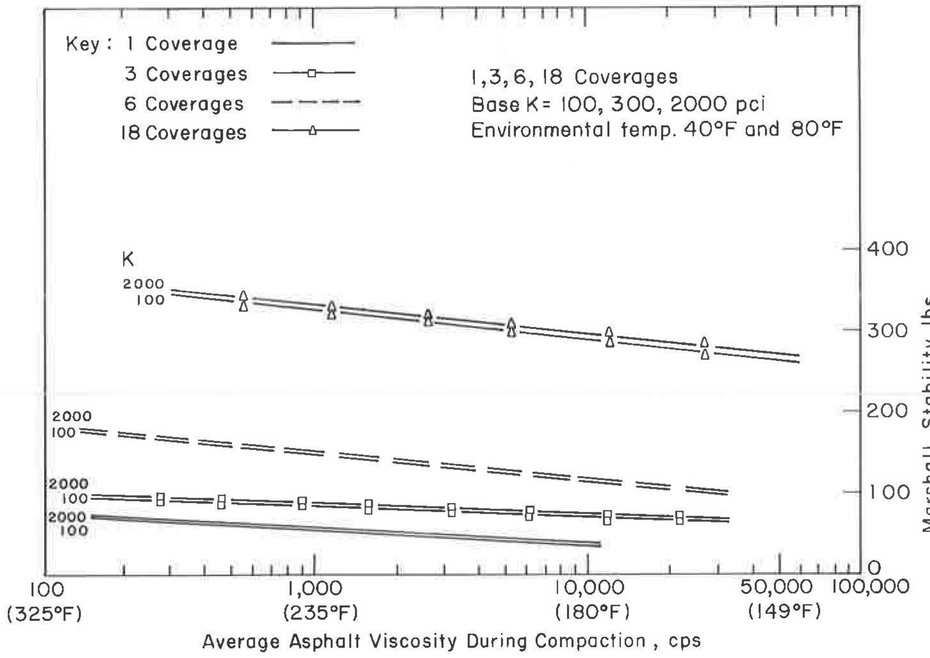

Figure 17. Marshall stability vs average asphalt viscosity, rubber roller.

Effect of Base Support Stiffness

Three types of base supports were used with reactions K equal to 100, 300 (in steel roller series only), and 2000 pci. Although the K = 100 (and maybe K = 300) base is seldom encountered in practice, it was chosen as an extreme case.

Figures 14 and 15 summarize the comparisons for the steel roller and Figures 16 and 17 for the rubber tire. There are only slight differences in the densities and stabilities, the harder base giving the higher values. Compaction checks using a concrete base (both smooth and rough) with K = 4,000,000 gave similar results to those of the K = 2000 base (solid rubber). In other words, the effect of base course stiffness or smoothness on density-stability of a 2-in. thick layer under compaction conditions described in this report was found to be small from a practical point of view.

Master Curves

From the previous findings it became apparent that the effects of the environmental compaction temperature and the base support stiffness were small. This permitted a compounding of the various curves to obtain about 80 points for plotting unit weight, stability, and void curves for the steel roller compaction as shown in Figures 18 to 25. Although a slight error is introduced due to compounding the points from various base supports, more reliable trend curves are obtained.

In Figure 20, the density-stability values are plotted against the average asphalt viscosity during compaction for 6 coverages of a steel roller. The curves are based on about 80 points each and the following trends can be observed:

1. For the unit weights, if a variation on both sides of the average curve ± 1.5 percent is accepted, 95 percent of all points will lie within this range (Fig. 20).

2. For the Marshall stabilities, assuming a range of ± 25 percent, about 80 percent of the points are inside the limits. This indicates that the variability in the Marshall stability values is considerable.

3. For the voids (based on apparent specific gravity) about 95 percent of the points lie within ± 15 percent of the average curve.

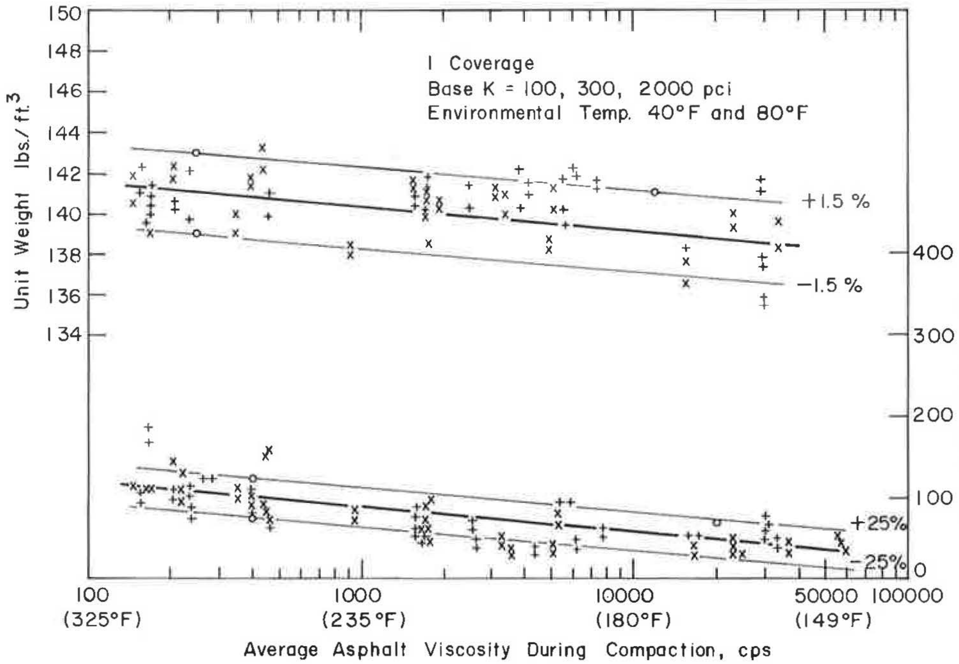

Figure 18. Unit weights and Marshall stability vs average asphalt viscosity, steel roller.

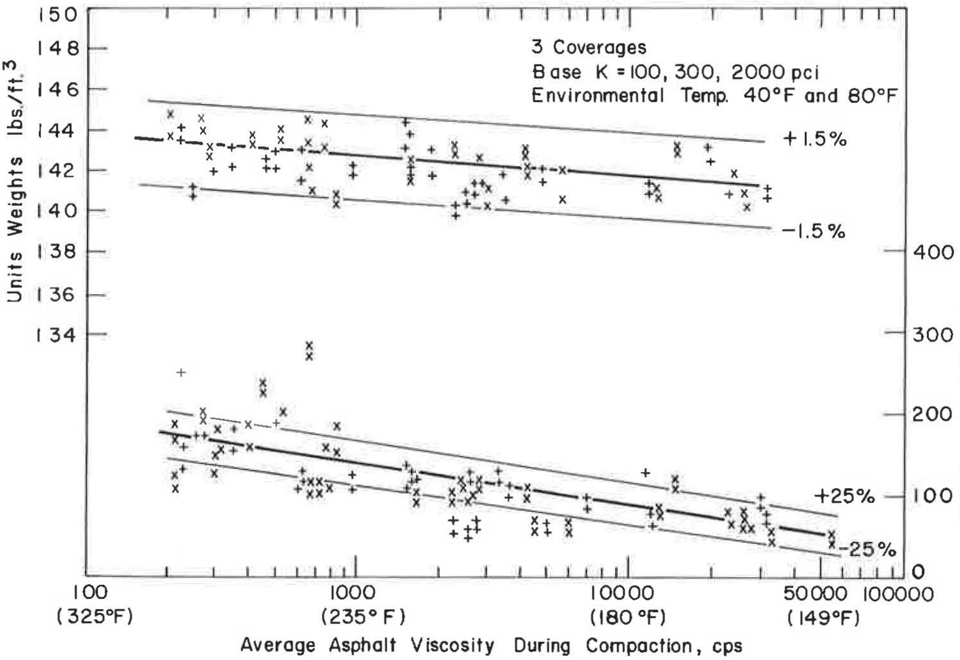

Figure 19. Unit weights and Marshall stability vs average asphalt viscosity, steel roller.

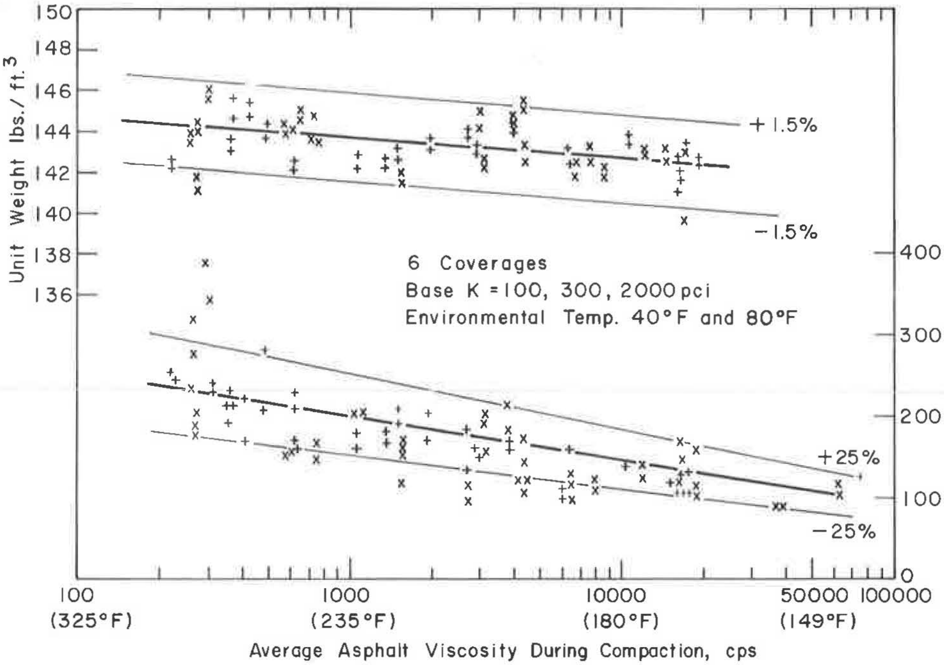

Figure 20. Unit weights and Marshall stability vs average asphalt viscosity, steel roller.

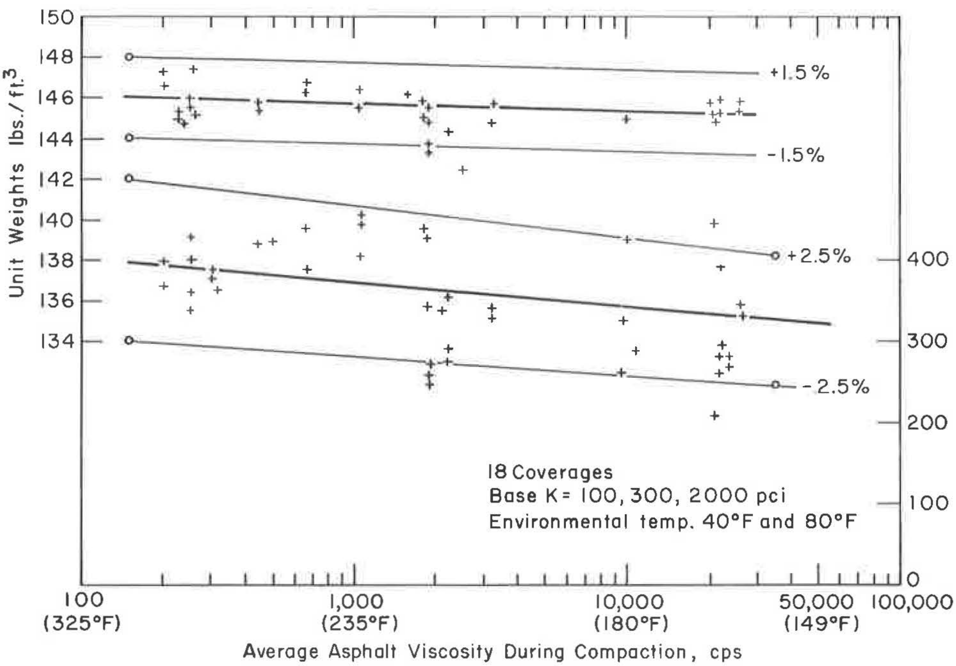

Figure 21. Unit weights and Marshall stability vs average asphalt viscosity, steel roller.

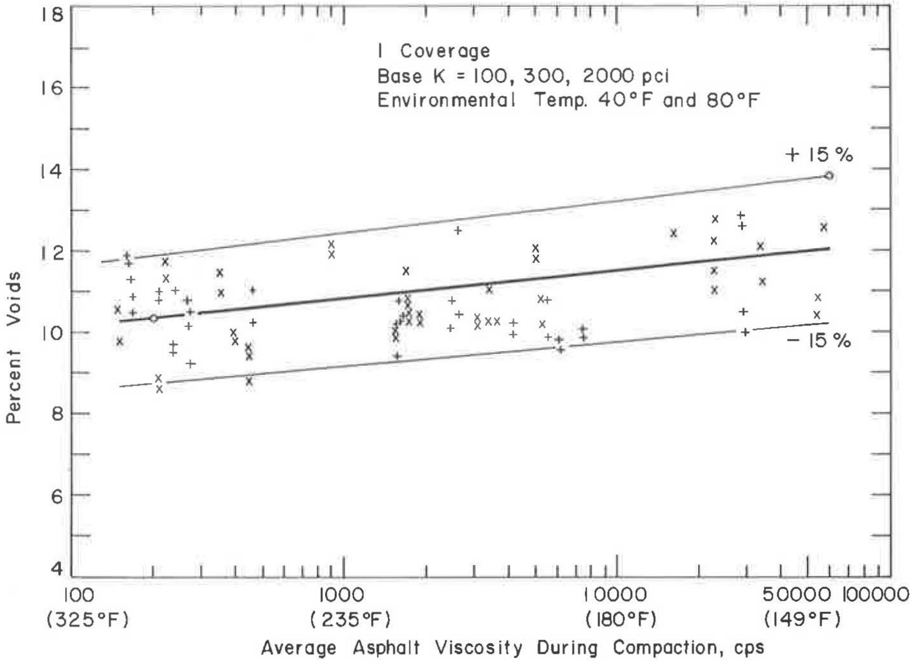

Figure 22. Percent voids vs average asphalt viscosity, steel roller.

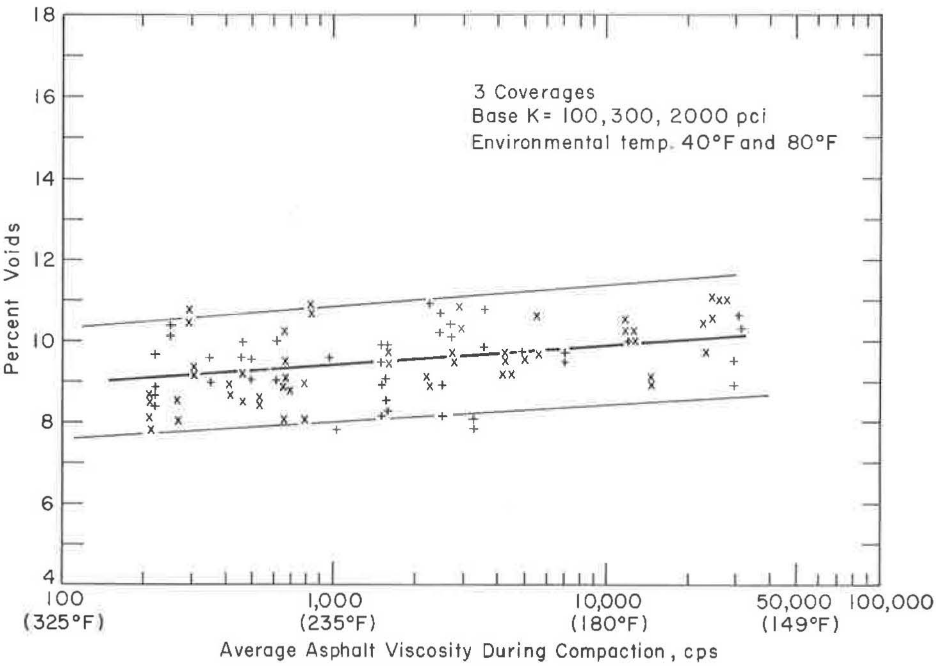

Figure 23. Percent voids vs average asphalt viscosity, steel roller.

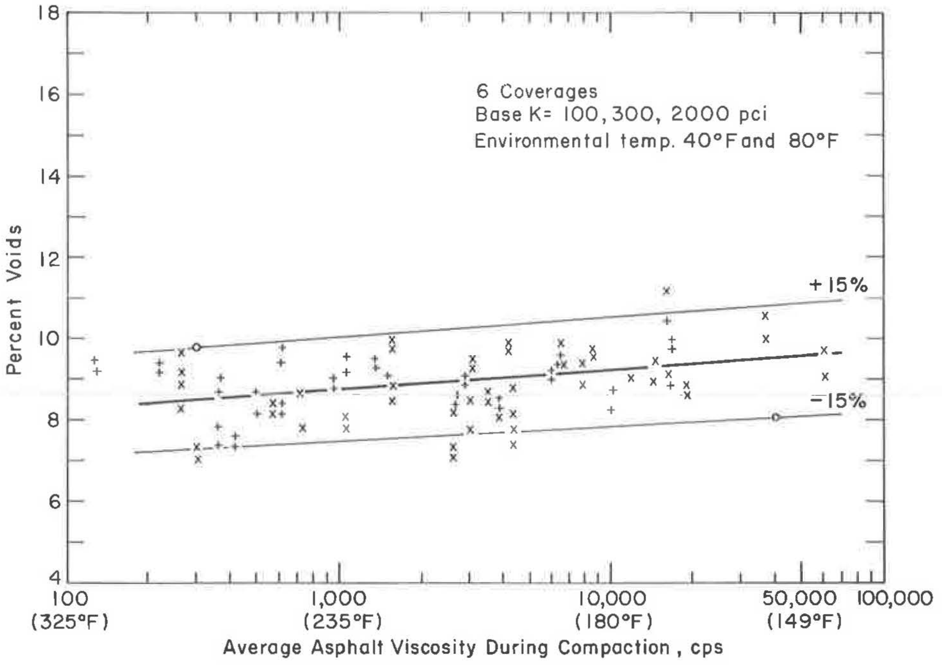

Figure 24. Percent voids vs average asphalt viscosity, steel roller.

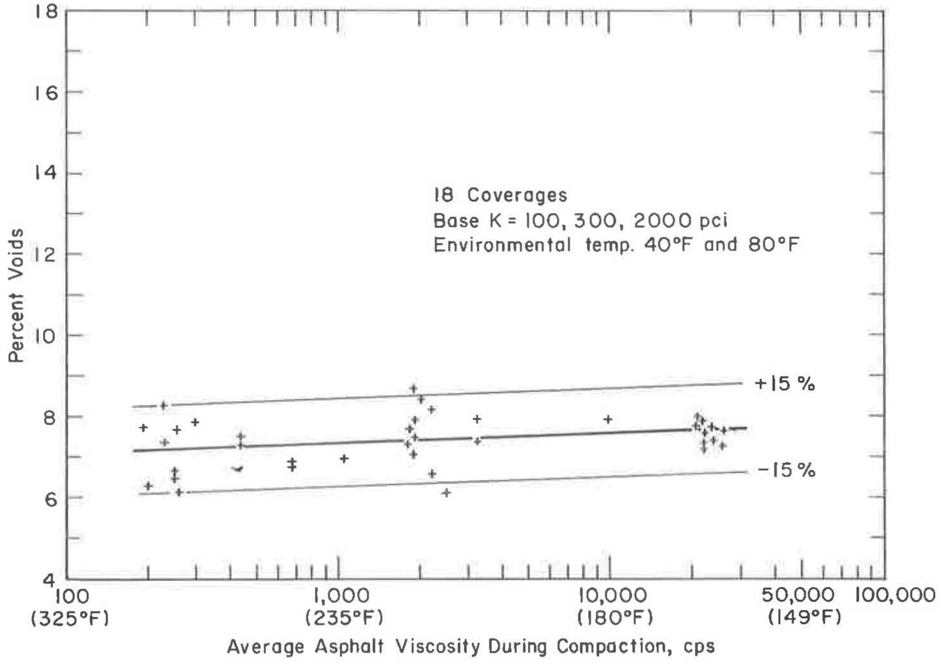

Figure 25. Percent voids vs average asphalt viscosity, steel roller.

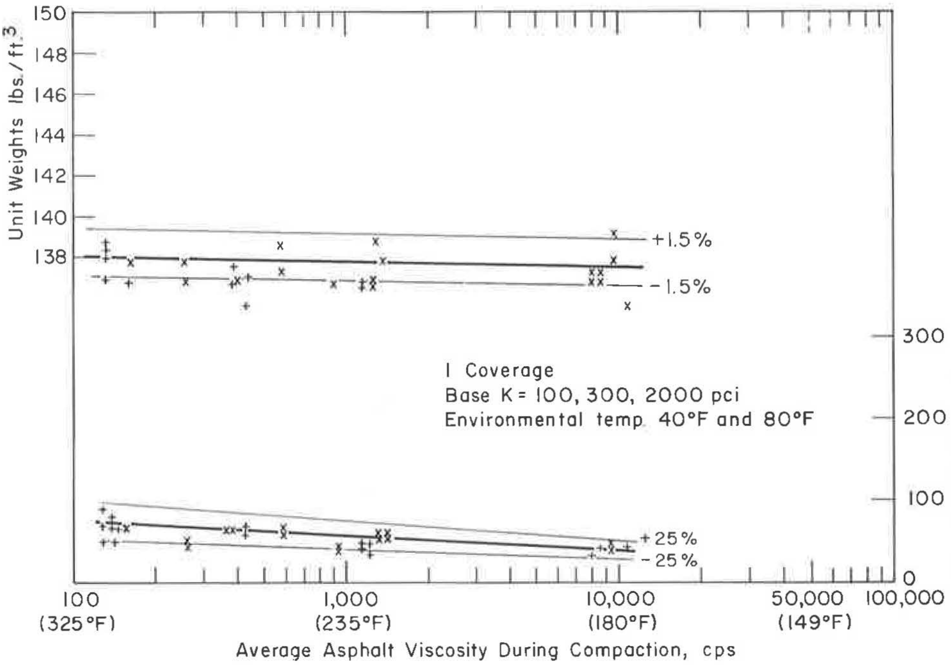

Figure 26. Unit weights and Marshall stability vs average asphalt viscosity, rubber roller.

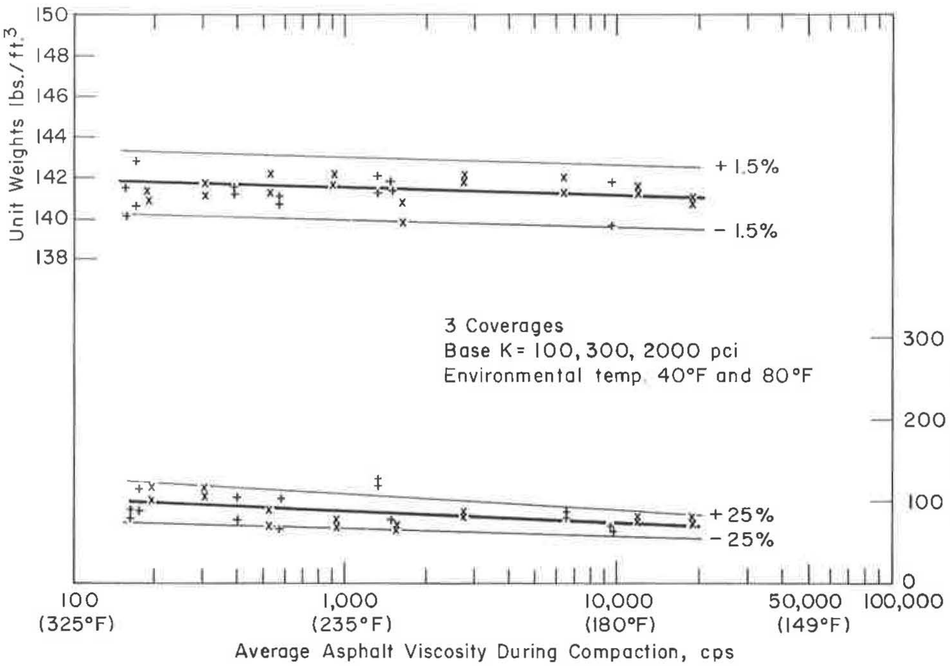

Figure 27. Unit weights and Marshall stability vs average asphalt viscosity, rubber roller.

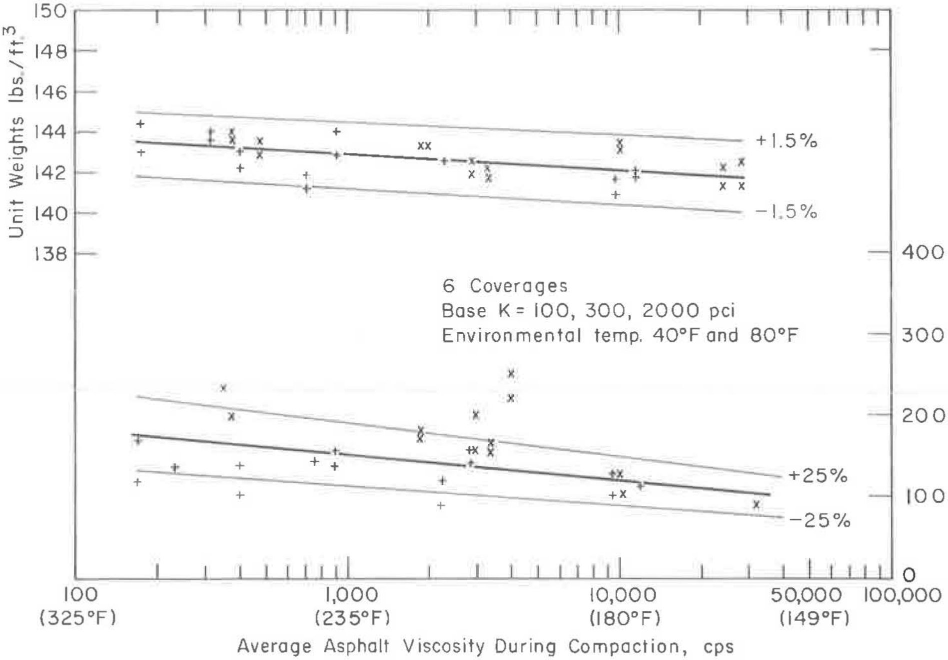

Figure 28. Unit weights and Marshall stability vs average asphalt viscosity, rubber roller.

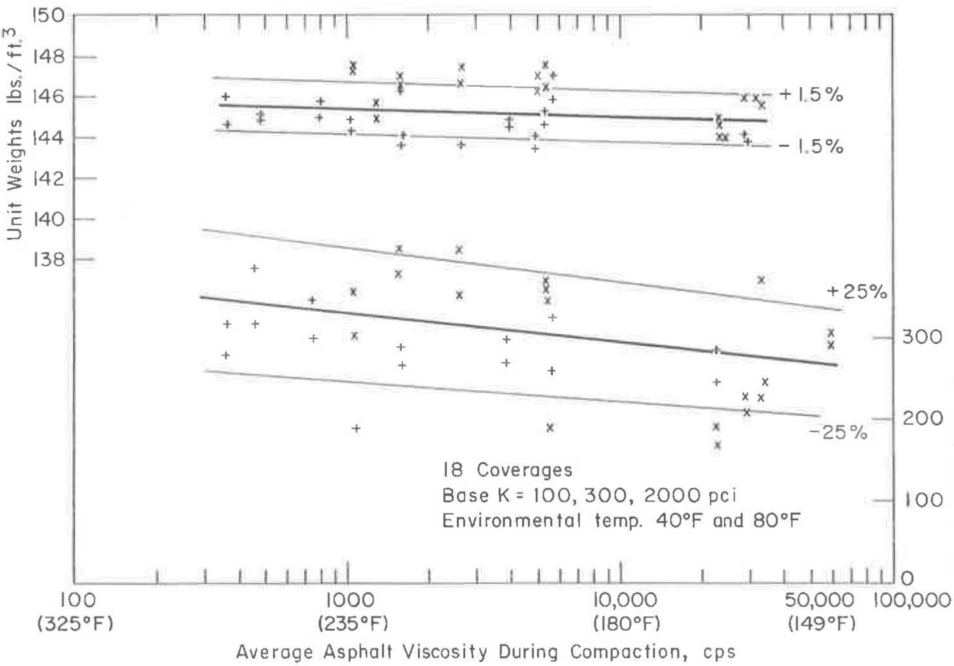

Figure 29. Unit weights and Marshall stability vs average asphalt viscosity, rubber roller.

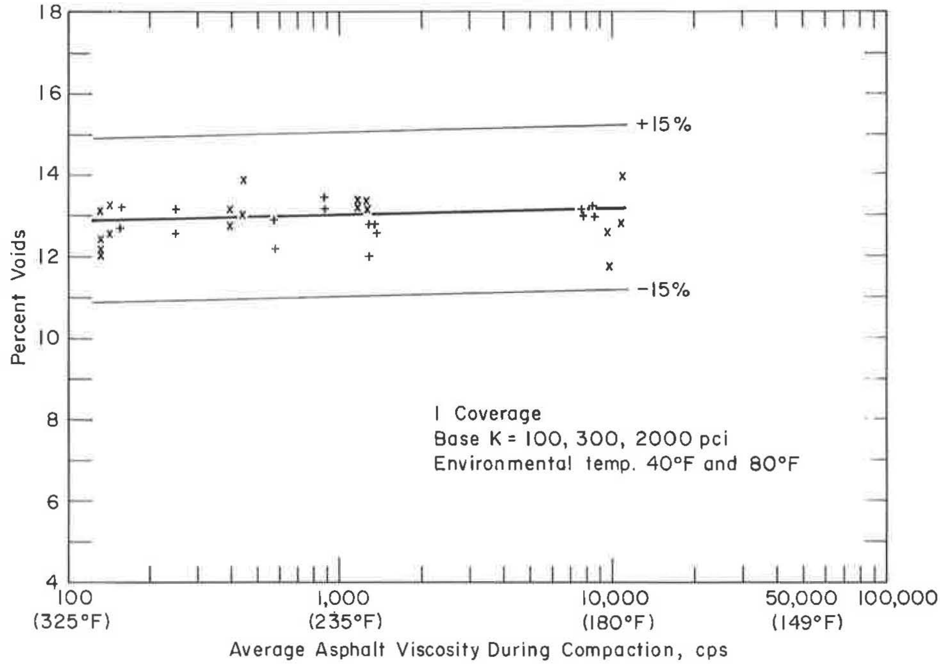

Figure 30. Percent voids vs average asphalt viscosity, rubber roller.

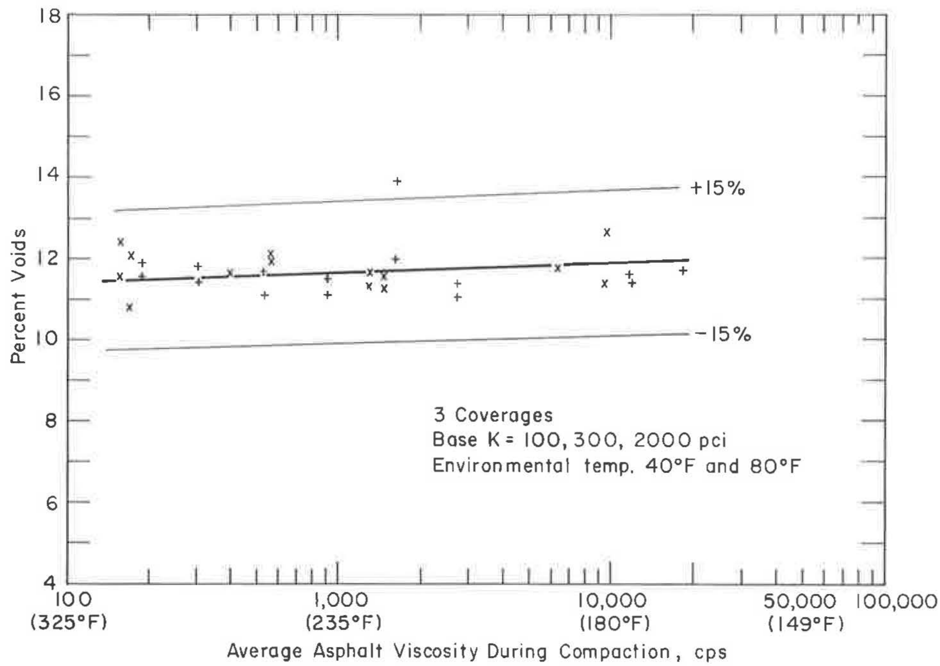

Figure 31. Percent voids vs average asphalt viscosity, rubber roller.

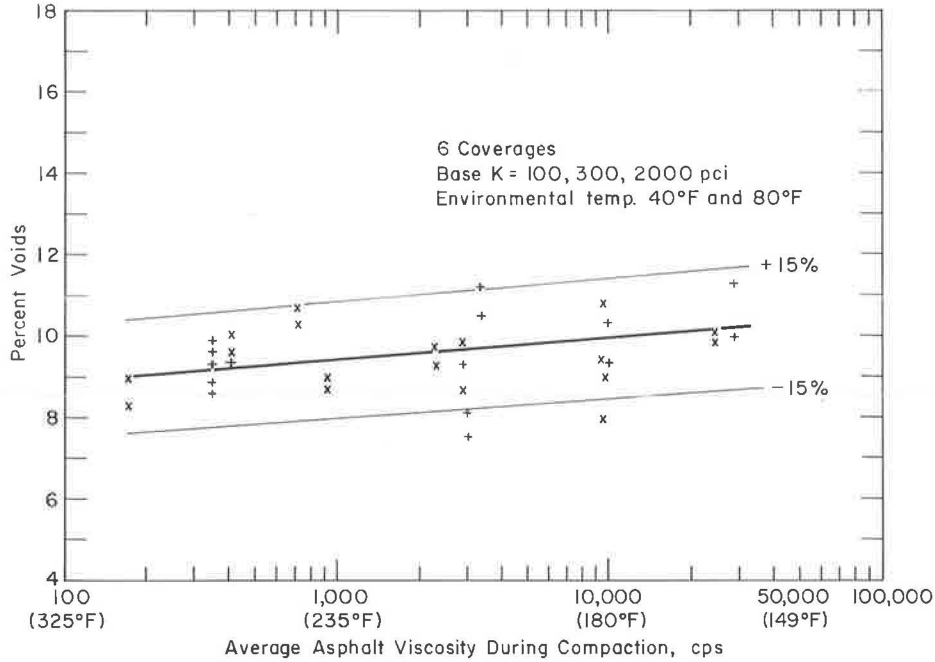

Figure 32. Percent voids vs average asphalt viscosity, rubber roller.

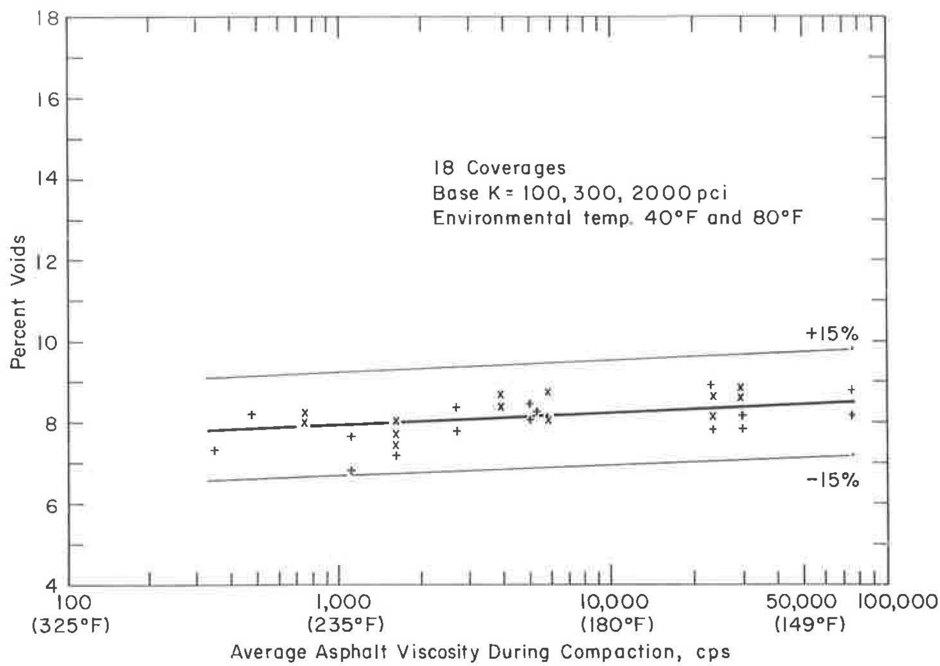

Figure 33. Percent voids vs average asphalt viscosity, rubber roller.

Similar trends are observed for the other coverages and the rubber roller. The number of specimens was less in the rubber roller series (Figs. 26 through 33).

Steel Versus Rubber Rolling

Figures 34 to 41 compare the effectiveness of a steel wheel roller (60-in. diameter, 250 lb/lin in.) and a rubber-tired roller (3500-lb load, 100-psi pressure). The main conclusion is that the specific rubber tire roller used was slightly less effective in compaction than the steel roller. Although from the surface it appeared that the rubber roller gave a more densely compacted mix, this apparently was not true inside the specimen. However, the differences between the two are not great, especially at larger number of coverages.

Several compaction experiments were conducted applying 9 coverages with steel roller and 9 with rubber. The resulting densities and stabilities were about the same as with the steel roller after 18 coverages.

Effect of Coverages

So far, all presentations have been made on the basis of viscosity vs density-stability for a given number of compaction coverages. Figures 42, 43 and 44 are plotted to compare the effect of coverages on the properties of the compacted mix.

It is emphasized that these curves are tied in and influenced by the particular 2-min time interval between each coverage. If, say, ½-min intervals between the coverages had been used, slightly different curves would be obtained.

Unit Weights.—Figure 42 shows the effects of number of coverages on unit weights approximating a semilogarithmic function. This is apparently due to the decreasing efficiency in densification of each successive coverage. At the beginning of the compaction, the bituminous mix is in a loose state; as more roller coverages are applied, the particles are pushed closer together, establishing more contact and the densification rate decreases rapidly.

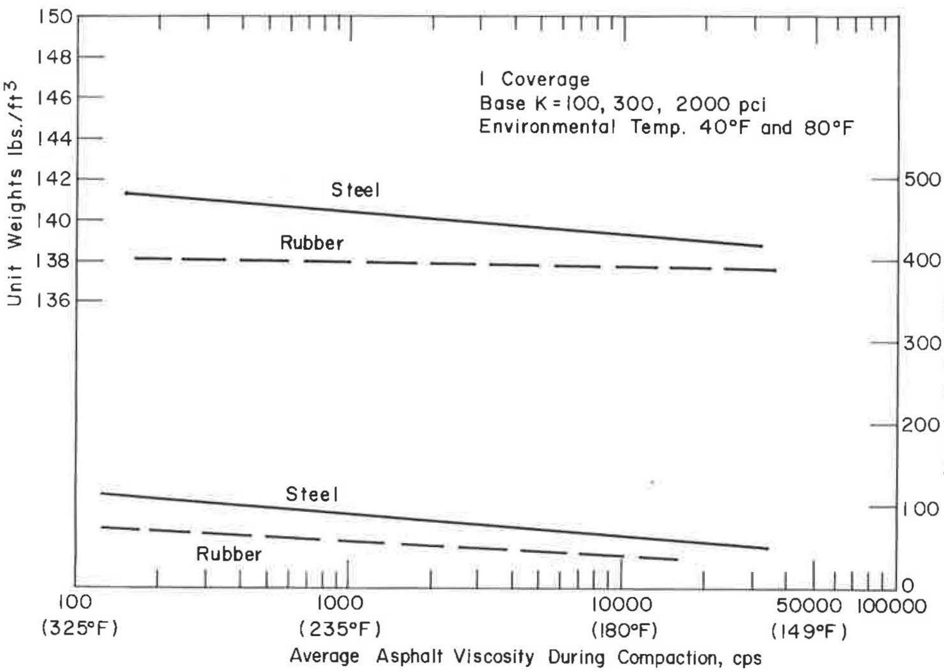

Figure 34. Unit weights and Marshall stability vs average asphalt viscosity, steel and rubber rollers.

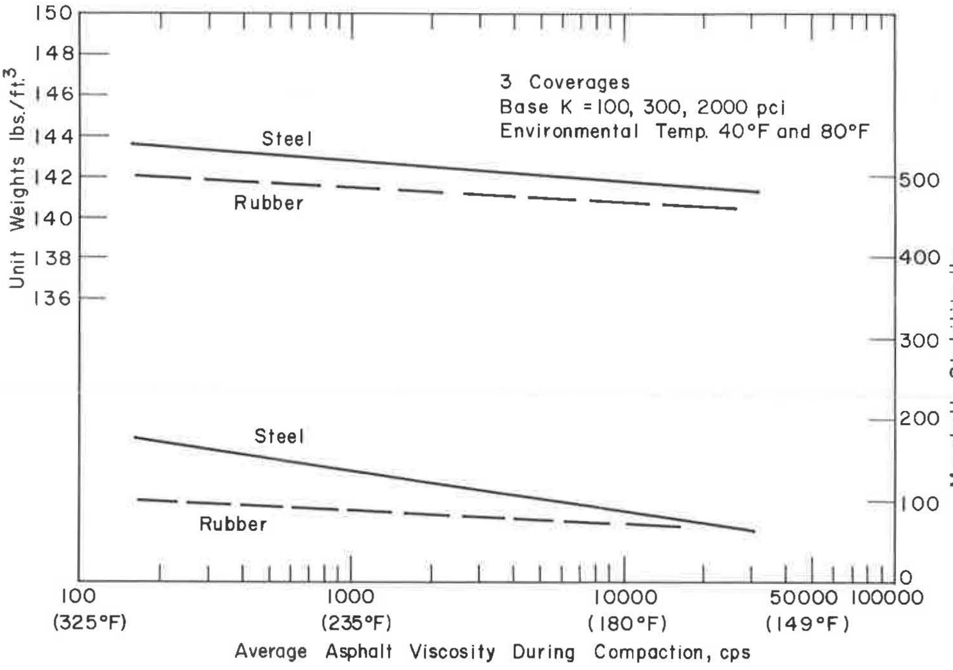

Figure 35. Unit weights and Marshall stability vs average asphalt viscosity, steel and rubber rollers.

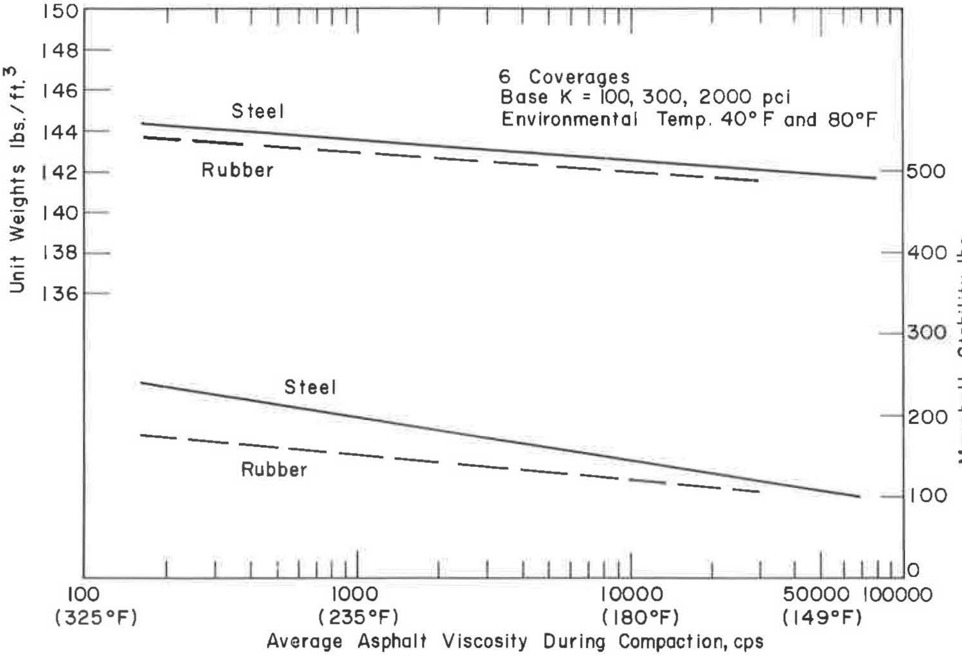

Figure 36. Unit weights and Marshall stability vs average asphalt viscosity, steel and rubber rollers.

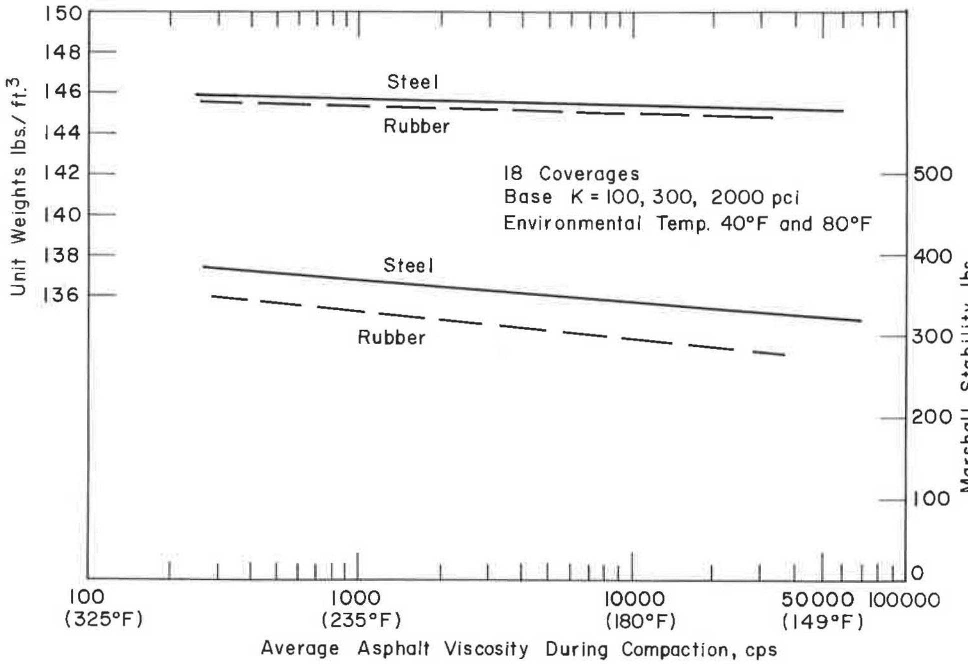

Figure 37. Unit weights and Marshall stability vs average asphalt viscosity, steel and rubber rollers.

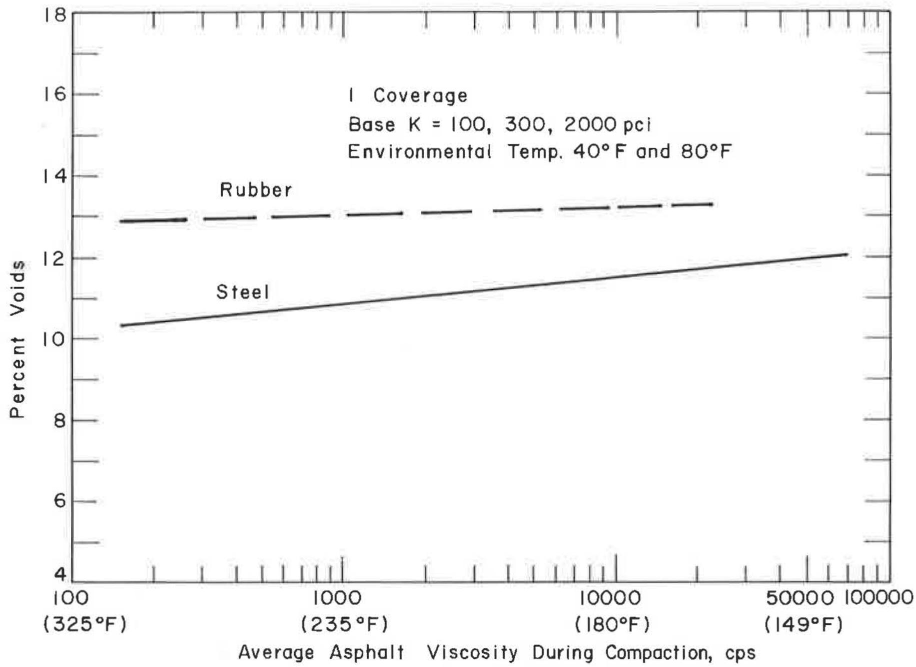

Figure 38. Percent voids vs average asphalt viscosity, steel and rubber rollers.

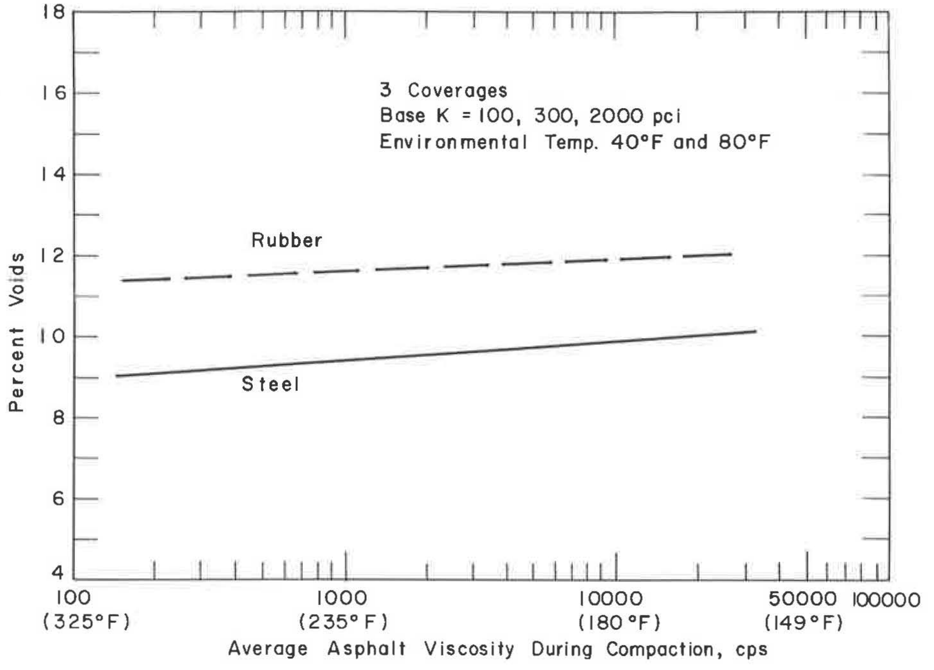

Figure 39. Percent voids vs average asphalt viscosity, steel and rubber rollers.

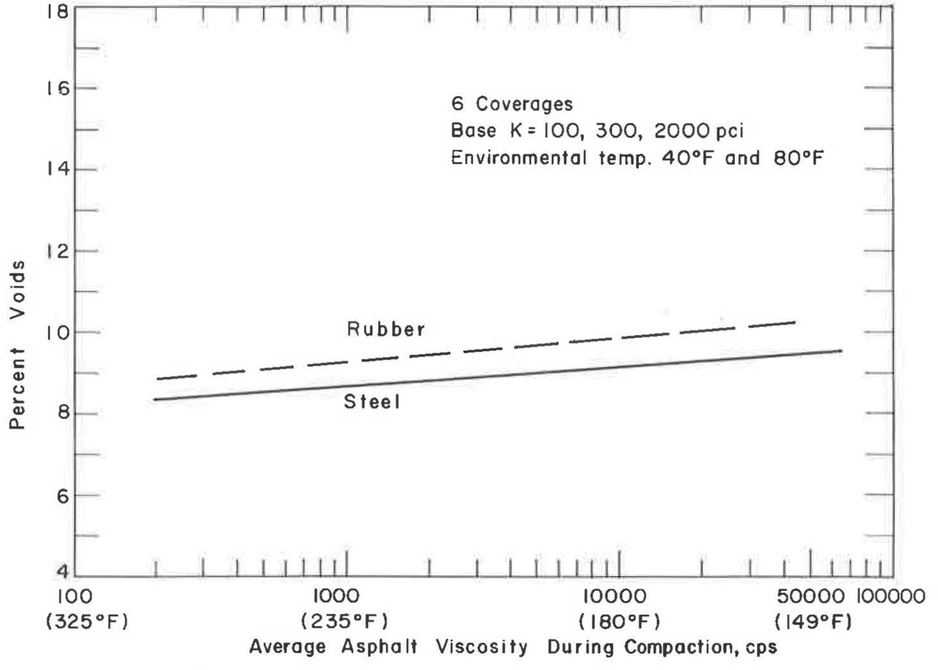

Figure 40. Percent voids vs average asphalt viscosity, steel and rubber rollers.

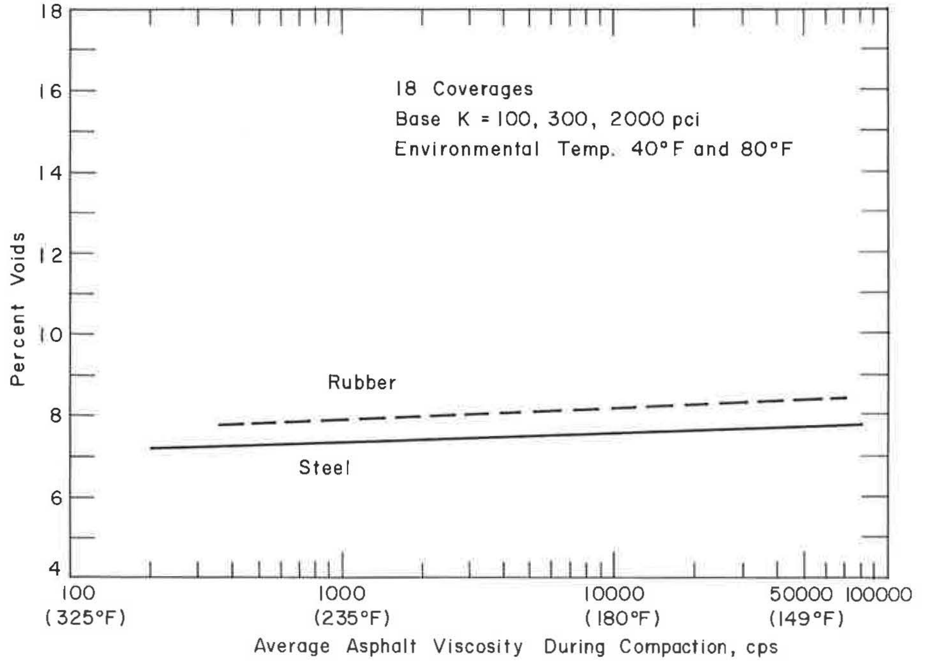

Figure 41. Percent voids vs average asphalt viscosity, steel and rubber rollers.

Figure 42 has been plotted for two average asphalt viscosities during the compaction—200 cps and 2000 cps, corresponding to 283 F and 217 F, respectively.

Marshall Stabilities.—Figure 43 shows the increase in stability with coverages. If plotted on a linear scale, an asymptotic curve is obtained showing less and less effect on strength with each coverage. The most significant observation is the low Marshall stability values even at 36 coverages (few specimens were compacted with 36 coverages). Cores taken from a freshly compacted similar mix from a pavement in Massachusetts gave values between 350 and 550, and pavements a few months old gave values in the range of 550 to 1150, indicating that the laboratory compaction values are probably reasonable. The laboratory mix, compacted by the Marshall method (50 blows) gave stability values of 1500 lb and more. Thus the stabilities are increased with the use of the pavement by traffic, possibly reaching the 50-blow Marshall values after several years of service. This presents a problem to the designer: should he work with a 350-lb stability or a 1500-lb stability in the design?

Void Content.—Figure 44 shows void content vs number of roller coverages. This again is a semilogarithmic function and the explanation for this relationship is similar to that presented in the discussion of density. These voids are based on apparent specific gravity of the aggregate. If bulk specific gravity is used, the void content would be about ½ percent lower (see Fig. 12); if effective specific gravity is used, the void contents would be between the apparent and the bulk values.

Guide for Field Compaction

The compaction experiments were set up in an attempt to simulate certain field compactions as closely as possible. Although the values obtained apply to one particular standard mix, it is possible that the general principles apply to other practical mixes in use.

Present Massachusetts specifications call for field compaction 95 percent or better of Marshall 50-blow density. Figure 45 shows a comparison between this minimum required density and the densities obtained with a steel roller in the laboratory. If

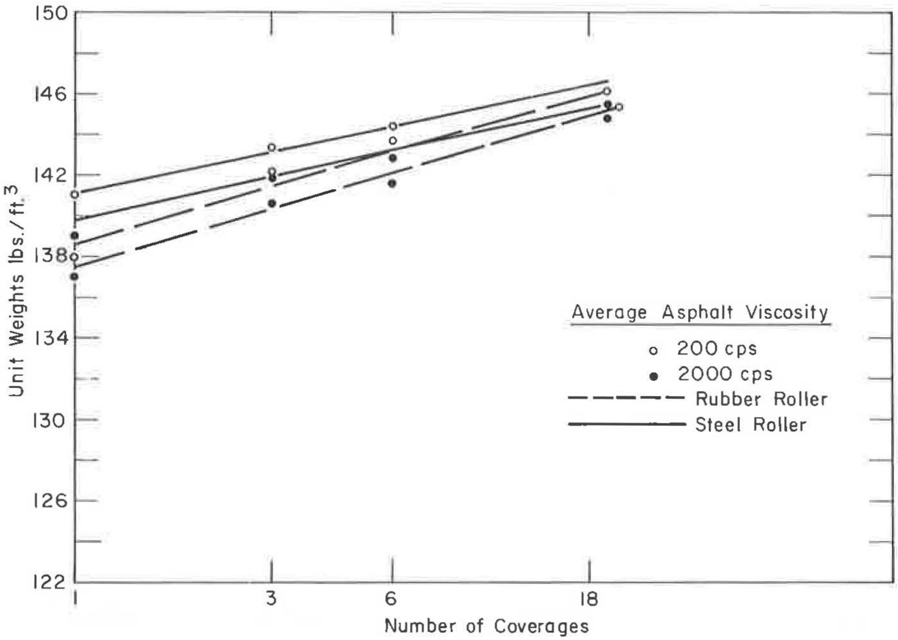

Figure 42. Unit weights vs number of coverages, steel and rubber rollers.

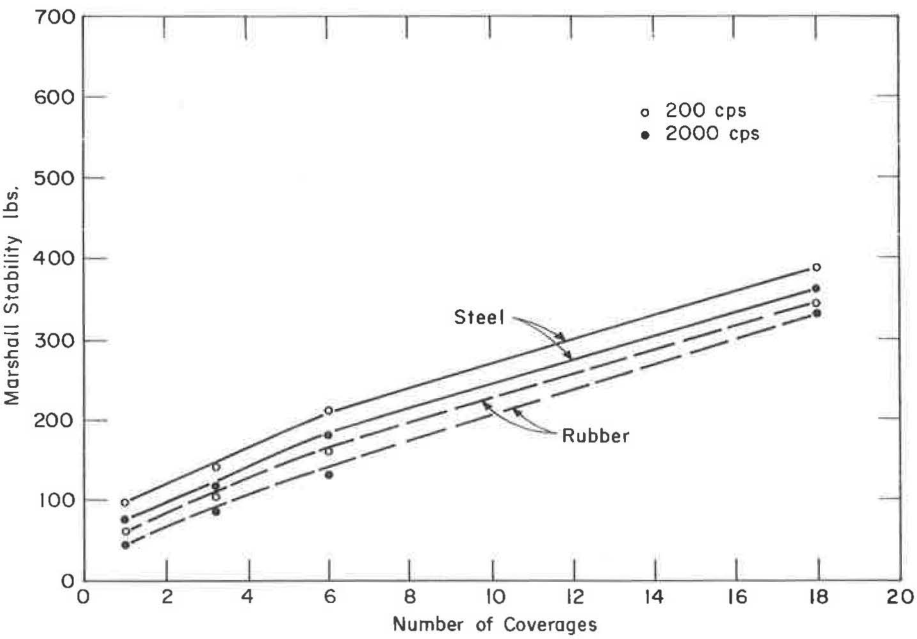

Figure 43. Marshall stability vs number of coverages, steel roller.

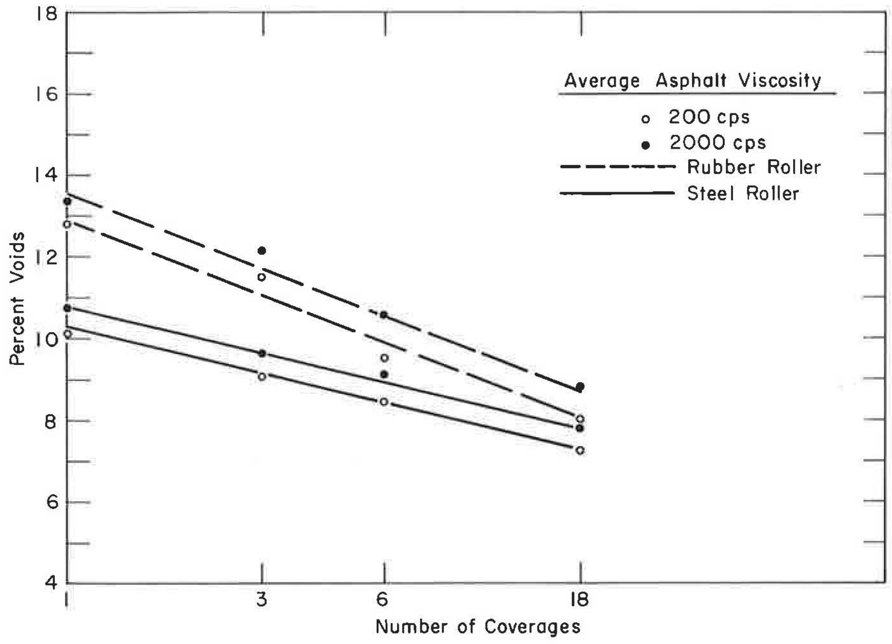

Figure 44. Percent voids vs number of coverages, steel and rubber rollers.

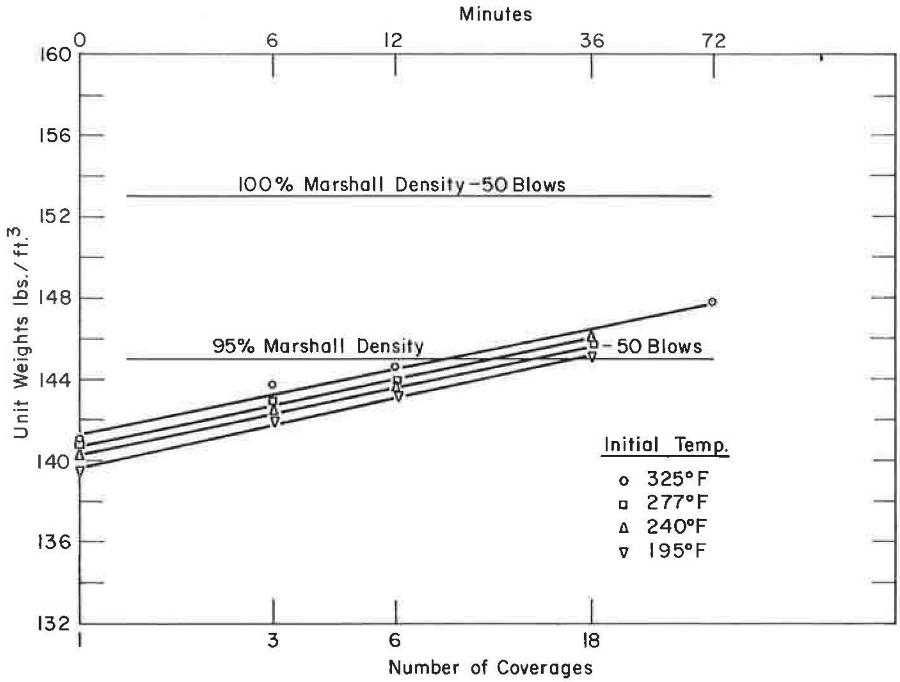

Figure 45. Unit weights vs number of coverages, steel roller, showing specified densities.

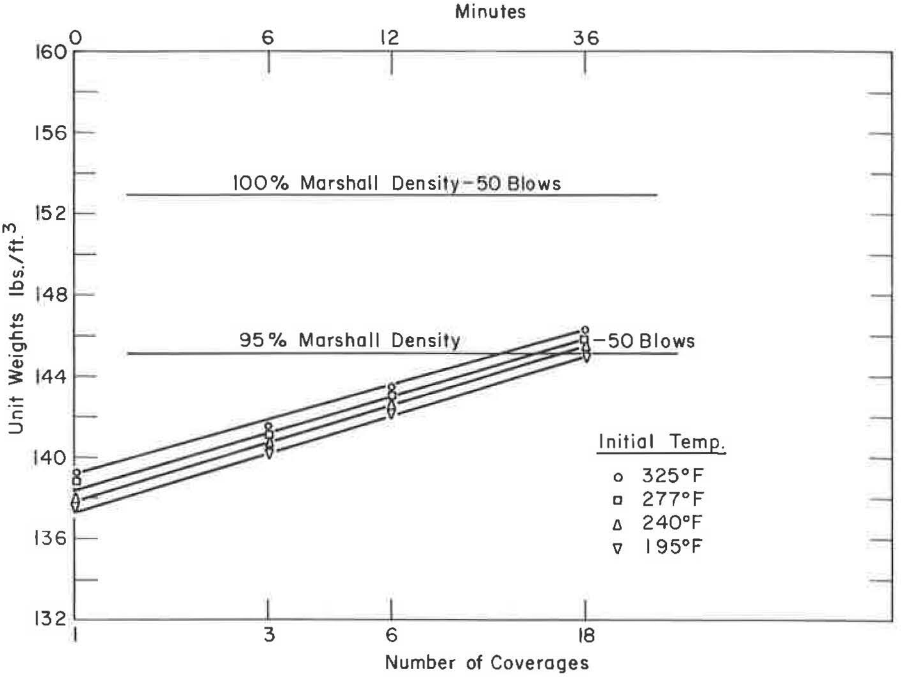

Figure 46. Unit weights vs number of coverages, rubber roller, showing specified densities.

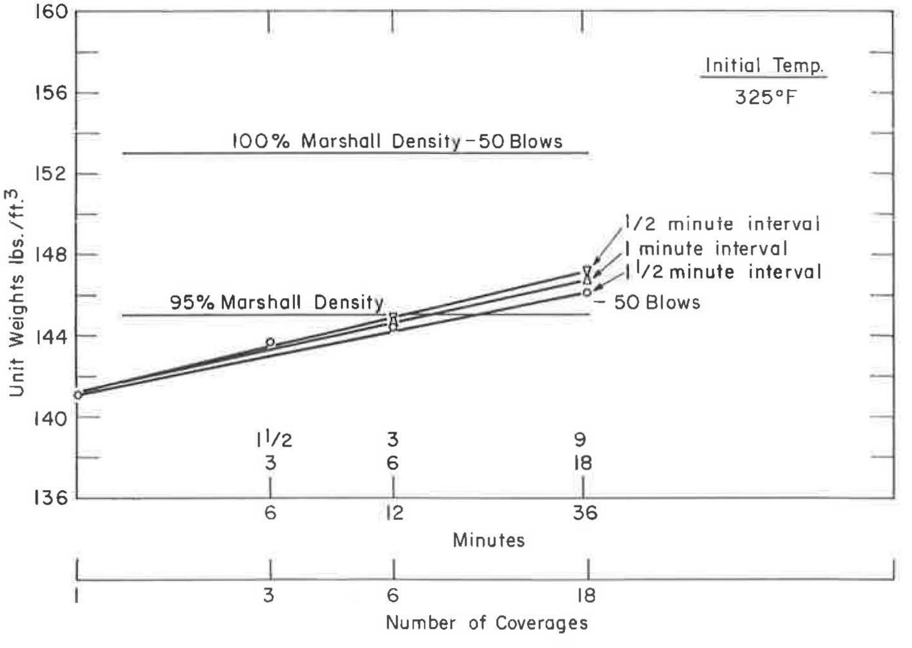

Figure 47. Unit weights vs number of coverages, steel roller, showing influence of frequency of coverages.

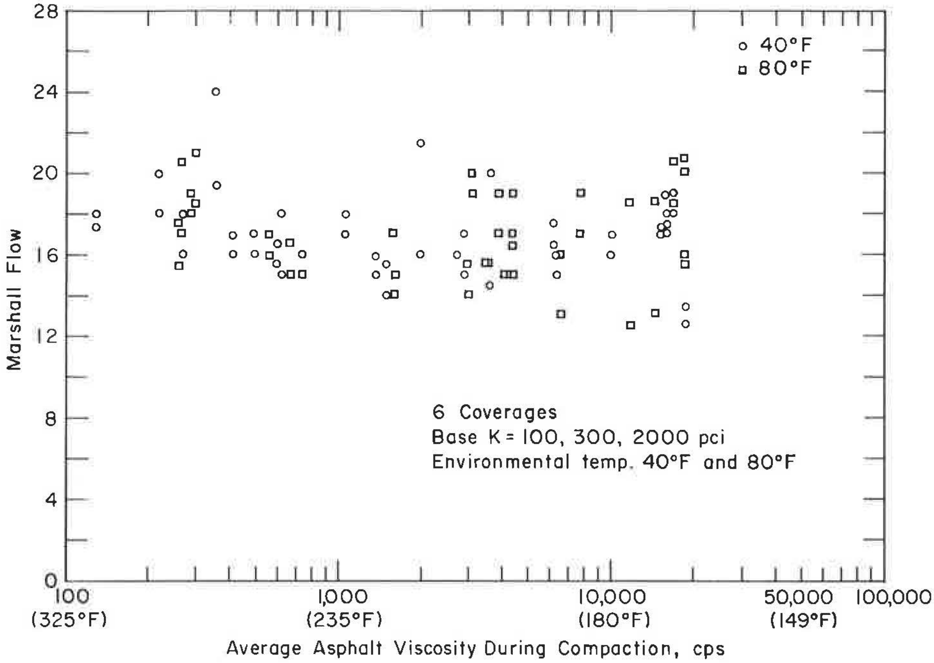

Figure 48. Marshall flow vs average asphalt viscosity during compaction, steel roller.

this laboratory compaction is assumed to be realistic, about 8 coverages are needed to get this density on the road provided that a 325 F mixing temperature is used and other conditions are similar to those used in the laboratory. If the initial mixing temperature is 195 F, about 17 coverages are needed. With a rubber roller, 11 and 20 coverages are needed for the same results (Fig. 46).

The time lapse between mixing and placing of the mix may be greater in the case of field work. However, the drop in temperature of the mix during the transport of a large bulk may very well compare with the heat losses of the 22-lb laboratory batch. If this is not true, temperature of the mix can be measured on the road and adjustments made to compare the results with Figures 45 or 46.

Changing Time Interval Between Coverages

All the curves presented so far are based on 2-min intervals between coverages. If this time is shortened, say, to 1 min or $\frac{1}{2}$ min or less, the number of coverages needed to obtain a given density is reduced.

The effect of the variation of the rolling interval is shown in Figure 47 for the steel roller.

From the cooling curves (Figure 9) the temperatures were obtained for 1-min and $\frac{1}{2}$-min intervals. From the compounded (master) curves of unit weights, Marshall stabilities and voids, the new values of the properties of the mix were obtained using the higher average asphalt temperatures during mixing. There is an increase in the unit weights depending upon the number of coverages (about 1 lb for 18 coverages). The Marshall stabilities would also be slightly increased. It is apparent that an increase would occur with an increased number of roller coverages. The difference (in Fig. 47) between 1 and 18 coverages means an increase of about 6 lb in unit weight ($\frac{1}{2}$-min coverage interval).

The results of rubber wheel compaction and those at other than 325 F initial temperatures show similar trends.

Other Physical Measurements

In addition to unit weights, voids, and Marshall stabilities, measurements of Marshall flow were taken. The Marshall flow values were somewhat erratic. This was probably due to the relatively low Marshall stability values encountered all through the study. Figure 48 gives an example for flow values.

CONCLUSIONS

The results of this study are based on laboratory compaction and testing of one specific dense graded bituminous-concrete mix. Attempts were made to simulate compaction on the road. The following conclusions appear warranted:

1. The effect of asphalt viscosity in the mix during compaction on density-voids was found to be relatively small. For instance, Figure 20 shows that the same mix compacted at temperatures at which the asphalt viscosity ranged between 200 and 20,000 cps had average unit weights of 144.4 and 142.3 lb, respectively, or a difference of 1.5 percent.
2. The effect of asphalt viscosity in the mix during compaction on Marshall stability was found to be noticeable. For instance, Figure 20 shows that the same mix compacted at temperatures equal to asphalt viscosity of 200 and 20,000 cps had average stabilities of 235 and 130, or a difference of 80 percent.
3. A semilogarithmic relationship appears to exist between the density-voids and Marshall stability of the compacted mix and the average asphalt viscosity during compaction (for a given number of roller coverages).
4. A semilogarithmic relationship appears to exist between the density and voids and the number of coverages.
5. An asymptotic relationship appears to exist between the Marshall stability and the number of coverages.
6. The Marshall stabilities obtained during the rolling compaction were much lower than those obtained by a 50-blow standard Marshall compaction.
7. The effect of the stiffness of base support on the compaction process of a 2-in. bituminous concrete mat was found to be small, although the harder bases gave slightly higher densities and stabilities.
8. In this study, the steel roller was more effective per coverage than the rubber roller, giving about 2 to 3 percent higher densities.

RECOMMENDATIONS

The results described in this report are primarily applicable to a given type of "standard" bituminous mix. Therefore, the following investigations would be of interest:

1. The high temperature rheological properties of the mix tested in this project should be determined. This may lead to simple ways of predicting the behavior of any mix under a roller.
2. Whether other gradations and asphalt contents give similar results should be investigated.
3. The low Marshall stabilities of a compacted mix are not desirable. Increasing the number of coverages before the mix cools appears to be the most effective way to get higher density and stability. Research on a theoretical and practical level should be attempted to find methods by which better initial compaction can be attained.

ACKNOWLEDGMENT

The study described in this report was supported by the Commonwealth of Massachusetts and the United States Bureau of Public Roads.

The authors express thanks to the Staff of the Materials Research Laboratory for their help and suggestions, with special thanks to Robert K. Ashworth, Jr., who was responsible for the preparation of mixes and compaction control.

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Nuclear Asphalt Content Determination at the Job Site

H. W. WALTERS, Assistant Highway Engineer, Colorado Department of Highways

Quick asphalt content determinations immediately after the asphalt surface course has been placed are necessary for quality control of the amount of asphalt cement in the hot mix. Modern hot plants capable of producing 200 tons or more per hour turn out significant quantities of asphalt concrete between the time a sample to be tested for asphalt content is obtained and the test results are reported.

Correlation with the reflux extraction tests and plant checks is reported in this paper, as well as the procedure involved when using the neutron probe. Asphalts used were 85/100 and 120/150 penetration grade from three different suppliers. Aggregates used came from six different sources. Projects on which this new method was tried ranged in location over a 200-mile area of southeastern Colorado.

Although this new method has not been adopted as standard procedure in Colorado, it is being used as a quick field check on asphalt content in CDH District Two with good results.

•IN AN EFFORT to achieve better control testing of asphalt, District Two of the Colorado Department of Highways has conducted a research program using the Troxler Nuclear Moisture and Density equipment to measure specific gravity and asphalt content. This paper covers the research to date on asphalt content determinations.

The principle of the nuclear test of asphalt content is the same as the nuclear test of moisture in soils. The hydrogen content in the asphalt, like the hydrogen content of water, is the main cause of thermalization of the neutron.

EQUIPMENT

The only additional piece of equipment used with the Troxler surface moisture gage, Model 104-115, is an experimental test chamber. This unit is designed to contain that portion of the neutron field of the surface moisture gage that is not used in the measurement of the sample.

The sample is contained in a 1-gal container, $6\frac{5}{8}$ in. in diameter and $7\frac{1}{2}$ in. in height. The weight of the sample is 15 lb for grading "C" material having 30 to 60 percent passing the No. 4 sieve, and 13 lb for grading "D" material having 15 to 80 percent passing the No. 10 sieve.

TESTING

The sample of asphalt is taken from the roadway directly behind the asphalt paving machine. It is then placed in a shallow mixing pan and the temperature is checked. The temperature of the material should be between 200 F and 230 F when put into the 1-gal container. At this temperature the material may be scooped into the container without any compactive effort, and leveled off to the top of the container at the desired weight. A transite lid, $\frac{1}{4}$ in. thick by $6\frac{3}{4}$ in. in diameter, with the outer edge recessed $1\frac{1}{16}$ in. so that the lid can be centered into the container, is fitted onto the container,

Paper sponsored by Committee on Construction Practices—Flexible Pavement.

Figure 1. Aggregate materials for master standards.

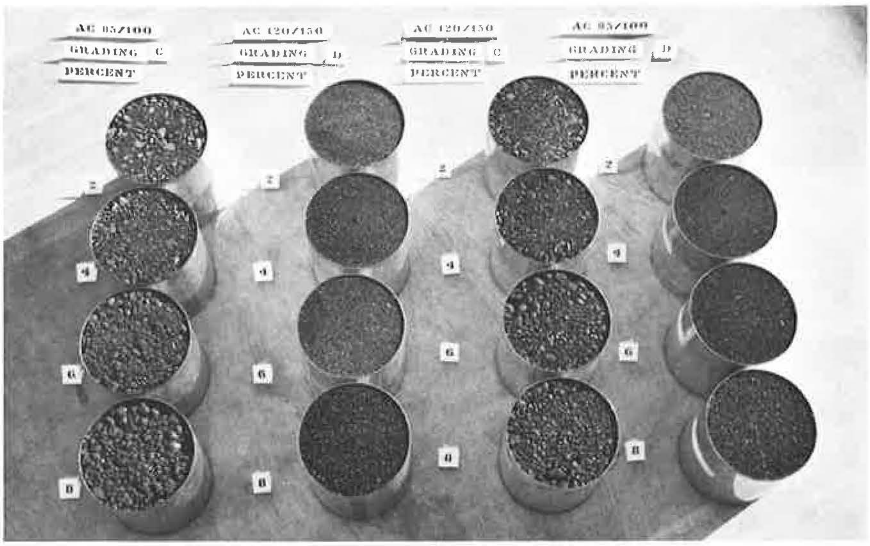

Figure 2. Standards for the four master curves.

then removed. This finishes the leveling of the sample. The container is then loaded into the testing chamber using a special bail. After the container is in place, the transite lid is carefully replaced on the container and the Troxler surface moisture gage positioned on top of the assembly.

A spring-loaded base in the bottom of the test chamber holds the sample container slightly above the surface of the testing chamber. The weight of the moisture gage presses the container flush with the surface of the test chamber. This arrangement insures a good contact between the bottom surface of the moisture gage and the top surface of the container assembly.

Five 1-min readings are taken and averaged; the average count divided by the normal standard count is then used for moisture determinations. This is obtained on the moisture reference standard, which is part of the standard nuclear equipment. The result is expressed as "percent of standard."

The percent of asphalt content is found from a calibrated curve opposite the percent of standard. This completes the nuclear test of asphalt content.

Figure 3. Asphalt sample from the roadway.

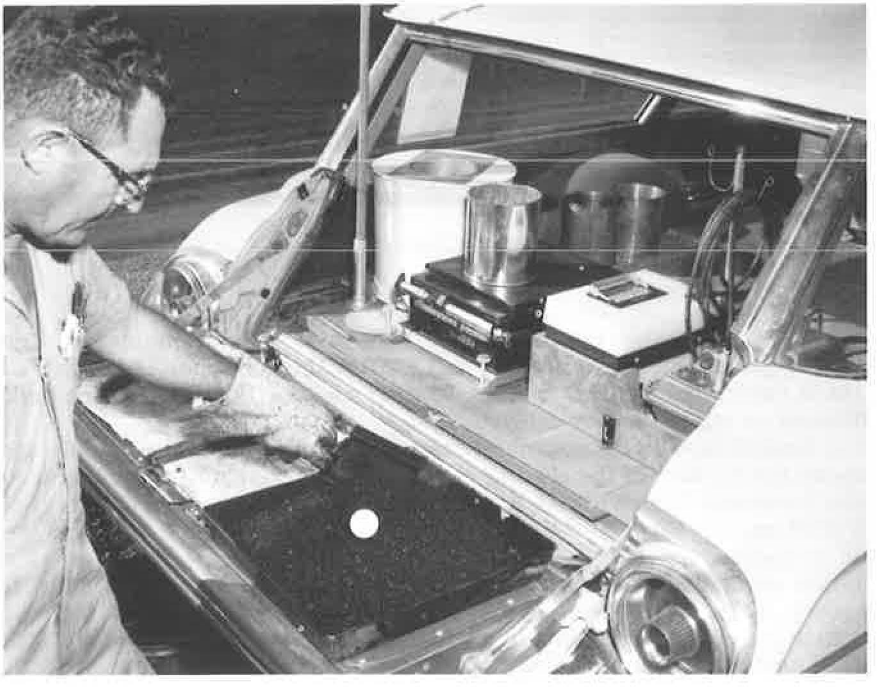

Figure 4. Checking temperature of asphalt in mixing pan.

Figure 5. Putting asphalt sample into gallon container.

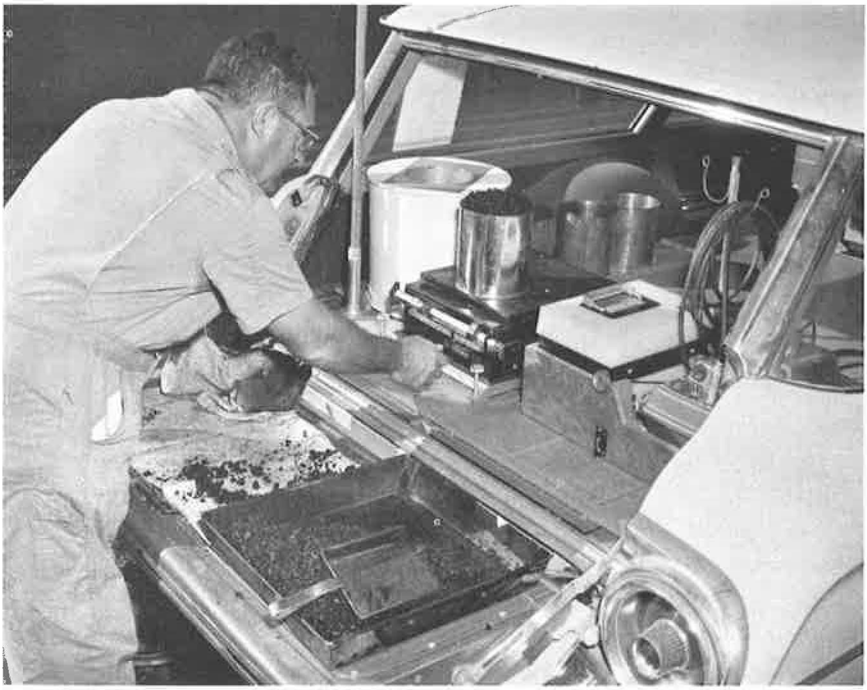

Figure 6. Weighing asphalt sample.

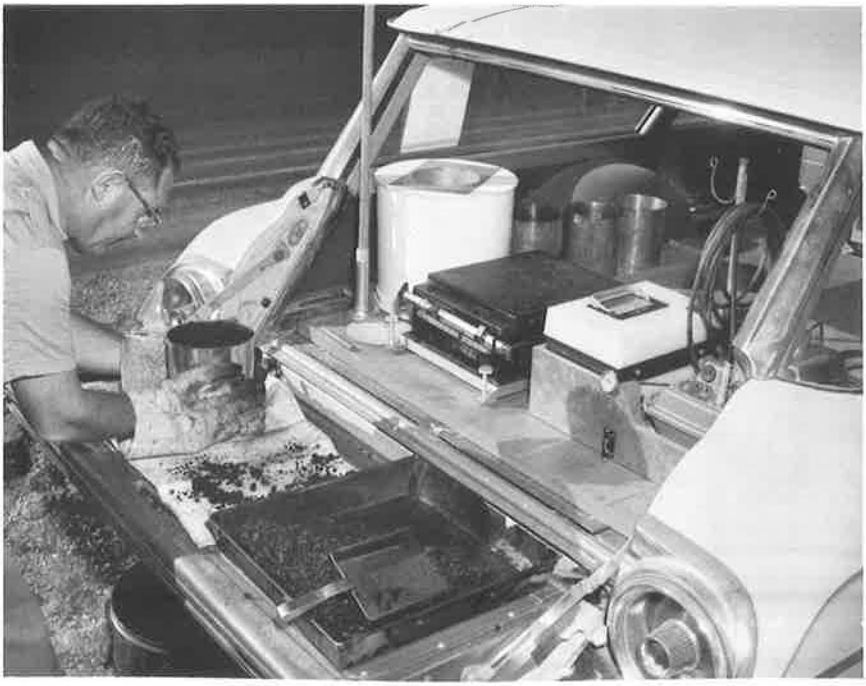

Figure 7. Settling material with rotating motion.

Figure 8. Leveling the material.

Figure 9. Fitting transite lid to container.

Figure 10. Lowering container into chamber.

Figure 11. Replacing transite lid on container.

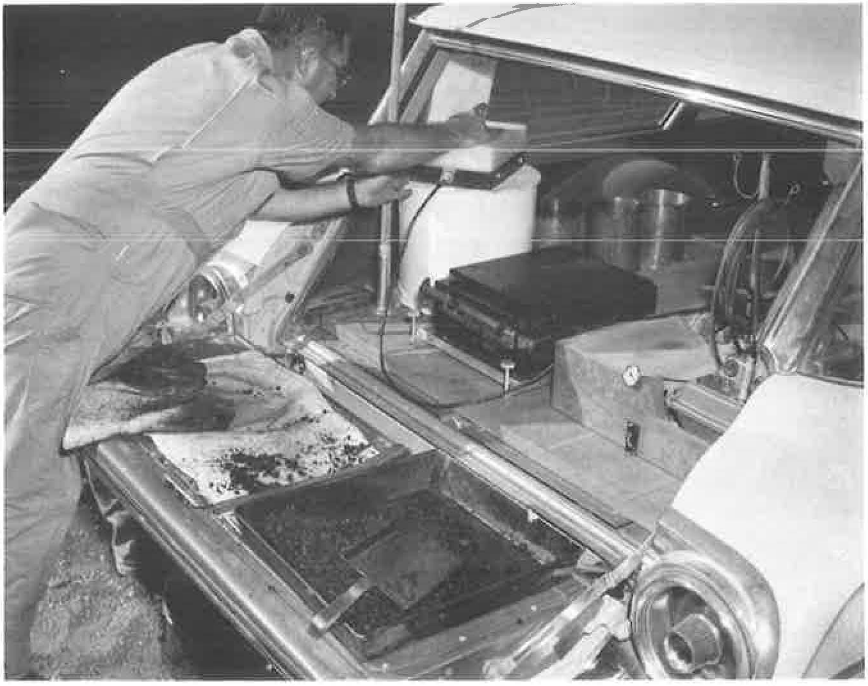

Figure 12. Placing Troxler neutron probe on chamber.

Figure 13. Taking five minute reading.

CALIBRATION

The curve used to determine the asphalt content is calibrated from standards of 2.0, 4.0, 6.0 and 8.0 percent. These standards are made by weighing the dry aggregate material and the asphalt in calculated proportions to form the percent required, and then mixing thoroughly by hand. The temperature of the aggregate and asphalt is held at approximately 280 F to 300 F during the weighing and mixing.

FIELD STUDY

The field comparison study consisted of taking nuclear tests of asphalt content and comparing the results with reflux methods and with the plant calculations.

Because of the time involved in the reflux test, it was difficult to obtain many comparisons per day. However, by taking additional nuclear tests throughout the day and averaging these tests, the comparison between the plant calculations for the day and average nuclear values for the day became more realistic. The equipment used for nuclear testing of asphalt is completely portable and the entire test may be performed wherever the sample is obtained. It is not necessary to return to the field test lab.

The first attempt to measure asphalt content began at Breed-Monument project I-25-2(48) 150. There were two pits used on this project, one for Item 32 and the other for Item 34. Examples of the average gradation of each pit are shown.

Higginson Pit (Granite)				Pioneer Pit (Limestone)			
Screen	% Passing	Screen	% Passing	Screen	% Passing	Screen	% Passing
3/4 In.	100	No. 40	15	3/4 In.	100	No. 30	25
No. 4	55	No. 200	6.9	1/2 In.	100	No. 80	12
No. 10	35			No. 4	54	No. 200	7.4
				No. 8	40		

Asphalt 85/100 AC Hydrated Lime 0.5% and 1.0%.

TABLE 1

COMPARISON OF ASPHALT CONTENT

Date 1963	Reflux	Nuclear	Diff.	Plant	Nuclear	Diff.
5/23	6.71	6.97	+.26	6.40	6.97	+.57
5/24	6.57	6.17	-.40	6.40	6.17	-.23
5/25	6.29	6.41	+.12	6.27	6.41	-.14
5/27	6.29	6.25	-.04	6.31	6.25	-.06
5/28	5.97	5.93	-.04	6.21	5.93	-.28
5/28	Note: Sample tested by Nuclear method sent to Denver Materials Laboratory. Their report was 5.89%, Nuclear value, 5.93%, +.04% Difference.					
6/5	6.33	6.18	-.15	6.19	6.18	-.01
6/6	6.41	6.03	-.38	6.04	6.03	-.01
6/7	6.35	6.22	-.13	6.04	6.22	+.18
6/10	5.94	6.01	+.07	5.72	6.01	-.29
6/11	6.55	6.27	-.28	6.08	6.27	+.19
6/12	6.44	6.22	-.22	6.27	6.22	-.05
6/13	6.33	6.25	-.08	6.18	6.25	+.07
6/14	6.02	5.97	-.05	6.15	5.97	-.18
6/14	Note: B.P.R. test sent to Denver Materials Laboratory. Their report was 5.75%, Nuclear value, 5.62%, -.13% Difference.					
6/15	6.62	6.25	-.37	6.20	6.25	+.05
6/17	6.67	6.22	-.45	6.20	6.22	+.02
6/18	6.15	6.16	+.01	6.24	6.16	-.08
6/20	6.55	5.97	-.58	6.03	5.97	-.06
6/21	6.52	6.14	-.38	6.08	6.14	+.06
6/22	6.34	6.17	-.17	6.07	6.17	+.10
6/24	5.99	5.87	-.12	5.83	5.87	+.04
6/25	5.96	6.05	+.09	5.91	6.05	+.14
6/26	6.11	5.89	-.22	5.87	5.89	+.02
6/27	6.07	5.86	-.21	5.83	5.86	+.03
6/28	5.92	5.87	-.05	5.80	5.87	+.07
6/29	5.87	5.80	-.07	5.78	5.80	+.02
7/1	5.83	5.83	.00	5.70	5.83	+.13
7/2	5.83	5.82	-.01	5.70	5.82	+.12
7/3	5.85	5.73	-.12	5.69	5.73	+.04
7/8	5.78	5.84	+.06	5.88	5.84	-.04
7/9	5.86	5.83	-.03	5.84	5.83	-.01
7/10	5.78	5.87	+.09	-	5.87	-
7/11	5.85	5.76	-.09	5.70	5.76	+.06
7/13	5.64	5.78	+.14	5.66	5.78	+.12
7/15	5.61	5.78	+.17	5.67	5.78	+.11
7/16	5.54	5.47	-.07	5.60	5.47	-.13
7/17	5.74	5.86	+.12	5.65	5.86	+.21
7/18	5.61	5.70	+.09	5.65	5.70	+.05

Table 1 shows the comparison between the reflux, nuclear and plant averages per day.

Of the 176 tests taken, the average deviation between the reflux and nuclear was -0.19% + 0.11%. The average deviation between the plant and nuclear was +0.11% -0.11%.

Comparisons became more favorable as testing progressed, because of a better understanding of how the sample should be prepared for testing.

The next study consisted of testing on different projects using a curve calibrated on one asphalt material to determine what amount of recalibrating would be necessary for each project.

The following results are on Project F 034-1(5) Harrison Road to U.S. Highway 24. The aggregate gradation is shown.

Strauss Pit (Granite)	
Screen	% Passing
¾ In.	100
No. 4	58
No. 10	33
No. 40	14
No. 200	7.8

Asphalt 85/000 AC Grading C Hydrated Lime 1.0%.

Comparison between the reflux and nuclear determinations on Project F 034-1(5) Harrison Road to U. S. Highway 24:

Date 1963	Reflux	Nuclear	Diff.
11/18	6.16	6.02	-.14
11/19	6.36	6.20	-.16
11/20	5.98	6.20	+.22

Project F 031-1(6) Penrose (Asphalt 120/150 AC Grading C Hydrated Lime 1.0%):

Date	Reflux	Nuclear	Diff.
4/2/64	5.85	5.99	+.14

It was necessary to calibrate a different curve for asphalt 120/150 penetration.

Project S 0002(25) Springfield to Vilas (Asphalt 120/150 Grading D Hydrated Lime 1.0%). In this case a curve was calibrated using Grading "D" material before testing began. Calibration standards were made using aggregate from the Freeman Pit No. 1 near Calhan.

The gradations below show the comparison between the Freeman Pit No. 1 and the Rutherford Pit used on the Springfield to Vilas project.

Curve		Project	
Freeman Pit No. 1 (Sand & Gravel)		Rutherford Pit (Sand & Rock)	
Screen	% Passing	Screen	% Passing
¾ In.	100	¾ In.	100
½ In.	100	½ In.	-
No. 4	93	No. 4	82
No. 10	73	No. 10	64
No. 40	21	No. 40	21
No. 200	9.4	No. 200	10

Comparison between the reflux and nuclear determinations on Project S0002(25) Springfield to Vilas:

Date 1964	Reflux	Nuclear	Diff.	Plant	Nuclear	Diff.
4/6	7.26	7.00	-.26	7.10	7.00	-.10
4/7	7.01	7.16	+.15	-	7.16	-
Note:	Changed plant at noon					
4/7	6.79	6.60	-.19	-	6.60	-
4/8	6.86	6.80	-.06	6.80	6.80	.00
4/9	-	6.80	-	6.77	6.80	+.03

Project C-22-0059-07 Campo (Asphalt 120/150 AC Grading C Modified no lime):

Curve		Project Pit (Limestone)	
Strauss Pit (Granite)			
Screen	% Passing	Screen	% Passing
¾ In.	100	¾ In.	100
No. 4	58	No. 4	55
No. 10	33	No. 10	43
No. 40	14	No. 40	20
No. 200	7.8	No. 200	6.9

Comparisons below are between reflux and nuclear only.

Date 1964	Reflux	Nuclear	Diff.
4/15	5.85	6.01	+.16
4/16	5.92	5.99	+.07

Project S0016(27) Olney Springs to Ordway (Asphalt 85/100 AC Grading C Modified 1.0% Hydrated Lime):

Curve		Project Pit (Granite)		Note Specs
Higginson Pit (Granite)				
Screen	% Passing	Screen	% Passing	
¾ In.	100	¾ In.	100	-
½ In.	-	½ In.	100	100
No. 4	55	No. 4	80	65 to 85
No. 10	35	No. 10	66	60 to 75
No. 40	15	No. 40	29	-
No. 200	6.9	No. 200	7.4	7 to 12

The following list includes the average values per day between the reflux and nuclear methods on Project S0016(27) Olney Springs to Ordway.

Date 1964	Reflux	Nuclear	Diff.
5/4	6.40	6.68	+.28
5/5	6.59	6.59	.00
5/6	6.94	6.94	.00
5/11	6.66	6.72	+.06
5/12	6.18	6.23	+.05

The average deviation of 53 tests using a calibrated curve from one aggregate source to measure an asphalt mix using another aggregate source was +0.14% -0.16%.

The research study to date has revealed that if the penetration range of the asphalt and the type of gradation (Grading C or D) remain the same, there is very little adjusting of the calibrated curve due to different suppliers of the asphalt or type of rock. However, if the penetration or type of grading differs, a new curve must be calibrated.

There are four master curves calibrated:

Curve No.	Asphalt	Aggregate
1	120/150 penetration	Grading C
2	85/100 penetration	Grading C
3	120/150 penetration	Grading D
4	85/100 penetration	Grading D

The slope of the asphalt content curve is less than the moisture curve because only part of the normal volume of the field is used. Research is being conducted at this time to determine a method of improving this.

SCREEN ANALYSIS

Gradation tests are sampled from the hot bins at the plant. The size of the sample measured in the nuclear test eliminates, to a greater degree, the variation in gradation from sample to sample.

MOISTURE

The determination of moisture in the bituminous mixture is obtained by AASHO method T 110, which employs a metal still, annealed glass trap and water-cooled reflux glass tube-type condenser. The solvent used is Xylol.

A moisture test was made twice a day, provided weather conditions remained constant.

CONCLUSIONS

The nuclear method of measuring asphalt content requires approximately twenty minutes for the complete test. This includes taking the sample from the roadway, handling of the material in preparation for nuclear measurement, the nuclear measurement, and the calculations.

Of the 229 nuclear tests represented in this report, 229 comparisons have been between the reflux method and the nuclear method, and 41 comparisons have been between the plant calculated values and the nuclear method, making a total of 270 comparison tests. Of these, 15 tests have exceeded 0.3 percent difference and the cause is believed to have been determined.

While the values of asphalt percents in a mix are small as compared to moisture tests in soil materials, attention should be directed to the fact that with soils, the material being measured may vary in composition from test to test on a single project. In the measurement of asphalt content, the material being measured is select processed material and if the nuclear method is to be used on the project, it may be easily calibrated for this material.

The sample to be measured is 13 to 15 lb in weight, depending on the gradation of the material. This volume may be handled with reasonable accuracy when preparing a sample for testing.

ACKNOWLEDGMENT

Appreciation is expressed to the field personnel of District II and to the reproduction staff for their assistance in preparing this paper.

Discussion

TAJAMAL HUSSAIN QURESHI, Superintending Engineer, Government of West Pakistan, C & W Department (47-F, Modeltown, Lahore)—Mr. Walters is to be thanked for presenting his paper. The rapidity and reasonable degree of accuracy with which the asphalt content measurements can be taken by this technique deserve serious consideration of its adoption as a standard method. In support of this technique, extracts from recent research at North Carolina State College at Raleigh are presented.

Hydrogen is present in the hydrocarbons which constitute asphalt. The aggregate in the asphaltic concrete can have hydrogen either in the moisture accompanying the aggregate or in the adsorbed layers of the mineral fraction. The aggregate used for asphalt concrete is generally free from such fine fractions which may have adsorbed water to a considerable extent. If some such fraction is present, however, the adsorbed water along with free moisture is almost entirely lost when the aggregate is heated in the dryer. Therefore, the presence of hydrogen in asphaltic concrete is almost entirely due to the hydrocarbons.

The fact that the hydrogen content of the asphalt concrete is found in the hydrocarbons of the asphalt and that hydrogen is the most effective neutron moderator serves as the basis for asphalt content determination by the neutron moderation technique.

Experimental Procedure

The neutron moderation technique is based on the hydrogen content of the asphalt portion of the asphaltic concrete. As such, the count rate depends on the quantity of asphalt present in a sample. It was concluded, therefore, that keeping the percentage of asphalt the same in a sample and varying its dimensions or density would affect the count rate. Accordingly, the initial studies were directed to the determination of the smallest practicable size of the sample beyond which the count rate would not be affected.

In the experiments the following apparatus was used:

1. Source: 3 millicurie R_a - B_e.
2. Reflector: Polyethylene reflector (originally developed for the soil moisture meter).
3. Detector: Neutron Moisture Probe Model 104 Serial 016629 (Troxler).
4. Scaler: Model 200B Serial 203 (Troxler).

Effect of Lateral Dimensions.—To study the effect of lateral dimensions, an asphalt-concrete sample was prepared by mixing 7 percent asphalt with well-graded aggregate in the laboratory and rolling it to 10- by 2- by 2-in. size. Count rate was taken by placing it on the polyethlene reflector. The size of this same sample was then reduced to 8 by 8 by 2 in. and then to 5½ by 5½ by 2 in. For each of these sizes the count rate

TABLE 2

EFFECT OF LATERAL DIMENSIONS ON COUNT RATE
(Asphalt percentage by weight, 7%)

Test No.[a]	Dimensions of Specimen (in.)	Counts	Count Rate per Min	Avg. Count Rate
1	10 × 10 × 2	41813	10453	
	overturned	41243	10310	10,381
2	8 × 8 × 2	40034	10008	
	overturned	40388	10097	10,052
3	$5\frac{1}{2} \times 5\frac{1}{2} \times 2$	32573	8143	
	overturned	31240	7810	7,976

[a]Time = 4 min.

TABLE 3

EFFECT OF THICKNESS ON COUNT RATE

Test No.[a]	Specimen Thickness (in.)	Counts	Count Rate per Min
1	2.90	42452	10,613
2	5.90	44673	11,168
3	8.15	45168	11,292
4	10.40	45713	11,428
5	11.80	46232	11,558

[a]Time = 4 min.

was taken by first placing it on one face and then on the other to guard against any variation due to nonuniformity within the sample. The results are given in Table 2. A $9\frac{3}{4}$- by $7\frac{3}{4}$- by 3-in. size was adopted since a mold was available for further experiments.

Effect of Thickness.—Similarly, count rates were taken by successively increasing the thickness of the samples on the reflector. It was found that the count rate increased for every added thickness (Table 3). A 3-in. thickness of the samples was adopted for further experiments.

Relationship Between Asphalt Content and Count Rate

Varying densities also affect the count rate. Accordingly, it was considered necessary to use samples of the same dimensions throughout the studies and, to obtain uniform density for every sample, the same quantity of asphalt concrete was compacted to the same size in every case.

A mold to give a size of $9\frac{3}{4}$ by $7\frac{3}{4}$ by 3 in. was used throughout, and the weight of asphaltic concrete contained in the sample was 7.7 kg in every case. Due to the difficulty of obtaining exact measurements and weights, a tolerance of ± 1 percent was accepted as a maximum. All samples used in the experiments were within this variation.

Having determined a procedure for preparation of samples of uniform dimensions and uniform densities, samples of asphaltic concrete were prepared containing 2 to 7 percent asphalt. The count rates are given in Table 4. Count rate vs asphalt percent-

TABLE 4

RELATIONSHIP OF ASPHALT PERCENTAGE TO COUNT RATE

Test No.[a]	Asphalt (%)	Counts	Count Rate
1[b]	-	120,133	30,033
2	2	51,547	12,887
3	3	53,359	13,339
4	4	55,897	13,974
5	5	58,166	14,541
6	6	60,040	15,010
7	7	61,753	15,438
8	7	62,114	15,528
9	6	60,605	15,151
10	5	58,860	14,715
11	4	56,231	14,058
12	3	53,716	13,429
13	2	51,666	12,916
14[b]	-	120,633	30,158

[a]Time = 4 min.
[b]Polyethylene standard block.

Note: Tests Nos. 8 through 14 are repetitions of 1 through 7 in reverse order.

age was plotted in Figure 14, and a linear relationship was established. It was, however, noticed that the count rate drifted considerably during the experiments, as seen from the count rates recorded at different instants for the polyethylene block, as well as those recorded for the samples by repeating the counting. To eliminate errors due to this drifting, the next set of readings was obtained for each sample alternately with the standard polyethylene block, so that a count rate was available for the standard block before and after the count rate for each sample, and a count rate ratio could be obtained. To minimize errors due to any nonuniformity within the sample the count rate was taken by placing each sample first on one face and then reversing it. The record of count rates and count rate ratios is given in Table 5.

Figure 15 was plotted between count rate ratio and asphalt percentage and

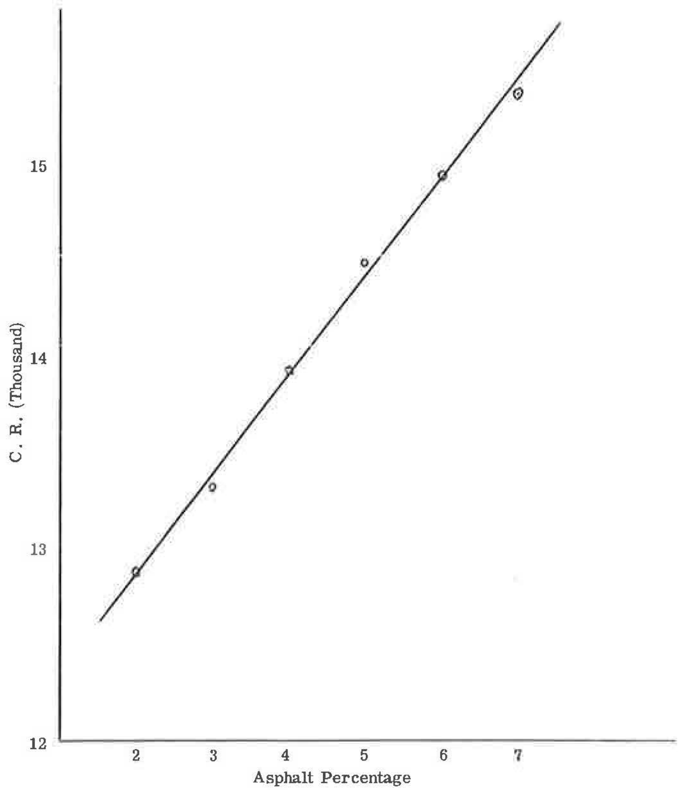

Figure 14. Relationship of asphalt percentage to count rate.

TABLE 5

RELATIONSHIP OF ASPHALT PERCENTAGE TO COUNT RATIO OF SPECIMEN AND POLYETHYLENE STANDARD BLOCK

Test No.[a]	Item	Counts	Count Rate	Count Rate Avg.	Count Rate Avg. of Poly. Std.	Count Rate Ratio
1	Poly. std.	90,017	30,006			
2	2% Asphalt sample	37,859	12,620	12,636	30,058	0.42038
3	Reversed	37,956	12,652			
4	Poly. std.	90,331	30,110			
5	3% Specimen	39,314	13,105	13,241	30,109	0.43976
6	Reversed	40,132	13,377			
7	Poly. std.	90,323	30,108			
8	4% Specimen	40,990	13,663	13,750	30,103	0.45676
9	Reversed	41,514	13,838			
10	Poly. std.	90,293	30,098			
11	5% Specimen	42,340	14,113	14,275	30,096	0.47431
12	Reversed	43,315	14,438			
13	Poly. std.	90,285	30,095			
14	6% Specimen	44,290	14,763	14,708	30,126	0.48821
15	Reversed	43,957	14,652			
16	Poly. std.	90,473	30,157			
17	7% Specimen	44,851	14,950	15,099	30,151	0.50077
18	Reversed	45,747	15,249			
19	Poly. std.	90,435	30,145			

[a]Time = 3 min.

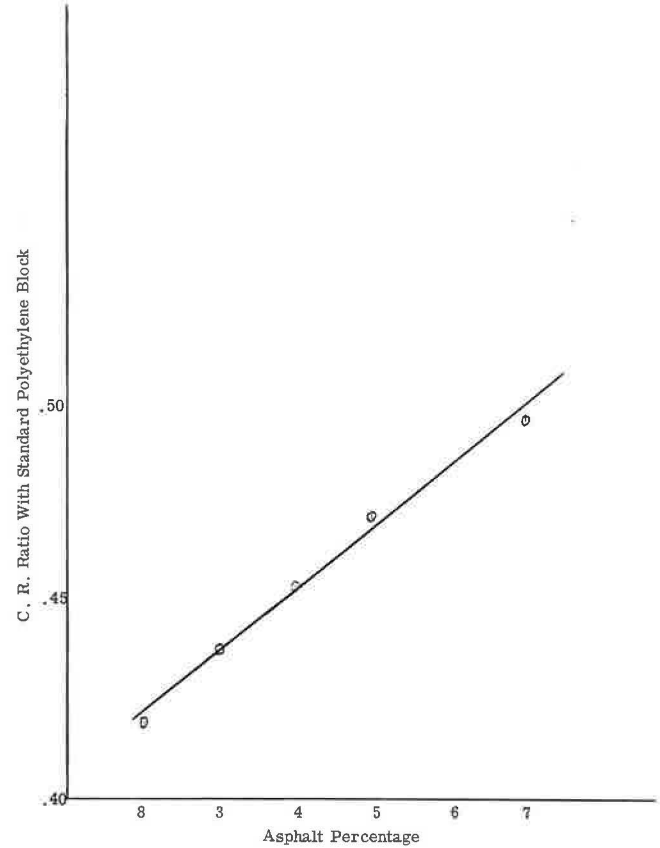

Figure 15. Relationship of asphalt percentage to count rate ratio.

indicates a linear relation with a straight line passing very close to each point given by the observations.

The linear relations established by the experiments indicate that the maximum deviation from the fitted curve is less than ± $\frac{1}{4}$ percent asphalt.

H. W. WALTERS, Closure—I am very pleased with the discussion by Tajamal Hussain Qureshi on Nuclear Asphalt Content Determination.

Considering the fact that there was no communication between us during the time this research work was being done, I am impressed with the similarity of the results.

Three-Year Evaluation of Shell Avenue Test Road

SHELL AVENUE TEST ROAD COMMITTEE,[1] W. A. Garrison, Chairman

The purpose of this report is to present an evaluation of performance over a 3-yr period of an experimental asphalt-concrete overlay pavement constructed on Shell Avenue in Contra Costa County, California. The overlay pavement was constructed on an existing pavement exhibiting fairly large deflections under a 15,000-lb axle load, and subjected to a large proportion of truck traffic in terms of the average daily traffic applied to the highway. Because of the existing conditions, it was planned that the test pavement should provide information on the resistance to deformation (stability) and fatigue resistance of heavy-duty mixes using conventional asphalt concrete and asphalt concrete with asbestos as a special mineral filler.

The test pavement is approximately 3,200 ft long and is divided into 4 sections, 2 with mixtures containing asbestos and 2 control sections without asbestos.

Instrumentation was installed in the pavement at the time of construction to measure dynamic deflections, bending strain, and temperature.

The report is concerned with an evaluation of periodic measurements of deflection, strain, and temperature; laboratory evaluation of cores, including density, stability as measured by the Hveem stabilometer, and viscosity at different levels in the overlay as measured by the sliding plate microviscometer; skid resistance and road roughness measurements.

From an evaluation of the field and laboratory tests, together with visual inspection of the performance of the road, conclusions are presented with regard to the ability of the various test pavements to perform under the traffic imposed and within the particular environment.

•THE PURPOSE of this report is to present an evaluation of performance over a 3-yr period of an experimental asphalt concrete overlay pavement.

A previous report (1) discussed the background of circumstances which led to the decision to undertake this full-scale field investigation of ways and means of producing heavy-duty, high-quality surfacings and to explain the various levels of performance by means of physical measurements. Following an extensive period of study and planning, it was decided to limit the investigation to an evaluation of the potential benefits of using asbestos as a filler in asphalt concrete. The field test site selected for this investigation, as well as procedures used in construction, are described in some detail in that report.

The overlay pavement was constructed on an existing pavement exhibiting fairly large deflections under a 15,000-lb axle load and subjected to a large proportion of truck traffic in terms of the average daily traffic applied to the highway. Because of the existing conditions, it was planned that the test pavement should provide information on the resistance to deformation (stability) and fatigue resistance of heavy-duty mixes using conventional asphalt concrete and asphalt concrete with asbestos as a special mineral

[1] Committee membership and functions are presented in Appendix A; this committee was organized to administer the Shell Avenue Test Road and is not a part of the Highway Research Board's committee structure.

Paper sponsored by Bituminous Division of Department of Materials and Construction.

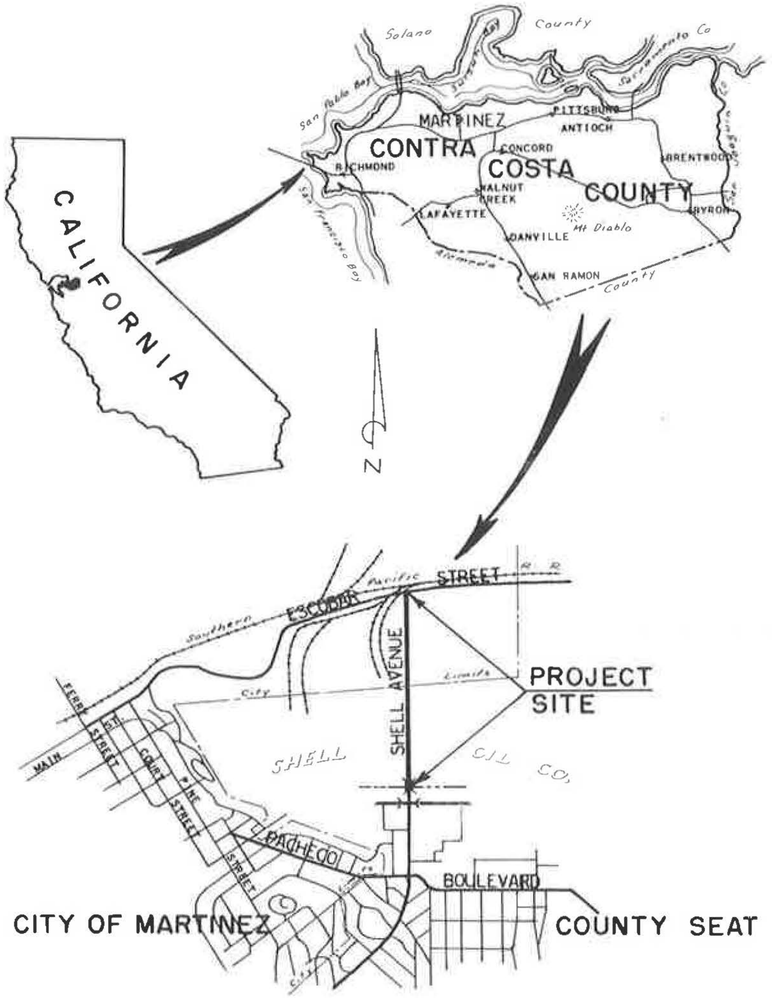

Figure 1. Site location—Shell Avenue Test Road.

Figure 2. General view of test road, before resurfacing.

Figure 3. General view of test road, after resurfacing.

filler. The experiment was designed to evaluate the relative performance of asbestos filler with 2 grades of asphalt cement when compared with comparable materials without filler.

After a year of study, during which preliminary laboratory tests and field evaluations were conducted, Shell Avenue in Martinez, California, was selected as the test road site. Specifically, an existing pavement section (Fig. 1) was selected as the test site; construction involved the placement of a nominal 3-in. resurfacing course of asphalt concrete. Shell Avenue is an industrial road carrying large numbers of heavy tank trucks and other commercial traffic. The right-of-way bisects the Shell Oil Company industrial properties. The length of the project is approximately 3,200 ft, right-of-way is 40 ft wide, and the paved roadway section is 24 ft wide. Figures 2 and 3 are overall views of the road before and after resurfacing, respectively.

SECTION LAYOUT

Figure 4 is a schematic layout of the test project showing the location of the various test sections and describing the overlay composition. Section 4-W is composed of 40-50 penetration asphalt with asbestos filler. This section is 1,280 ft long as compared to 640 ft for the other sections. This additional length was needed in order to include an area of high deflections found in the northern limits of the project as represented by the northernmost 640-ft section. For purposes of this report, the 4-W section was divided into two subsections of approximately equal length and designated 4-W-1 (high deflections) and 4-W-2 (normal deflections).

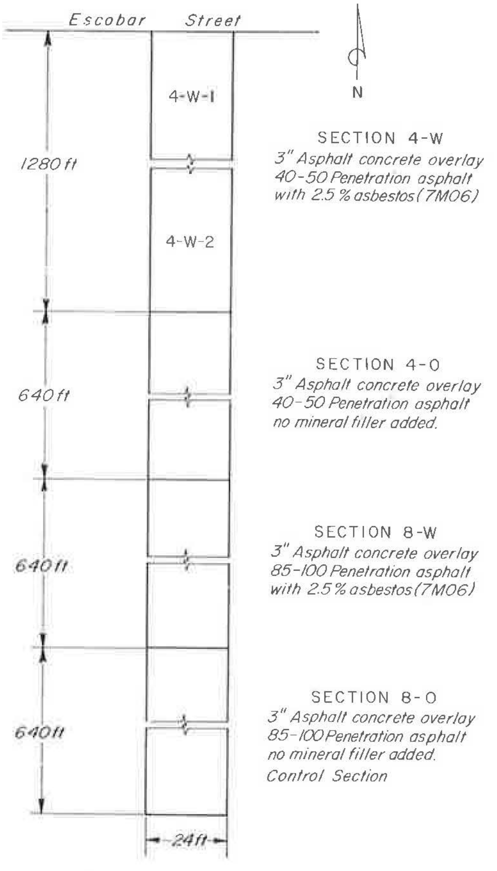

Figure 4. Schematic layout of test sections.

One of the features of this investigation which helps simplify the analysis is the manner in which traffic must operate within the limits of the project. There are no side entrances; hence, traffic must proceed through the entire length of the project, providing a continuous traffic condition for each lane. Southbound trucks were predominantly loaded, whereas northbound trucks were unloaded, making it possible to evaluate performance by lanes at 2 traffic levels. To the extent that other factors, i.e., deflection and preconstruction conditions, are similar, the difference in performance between lanes can reasonably be associated with the difference in traffic.

The 3,200-ft test pavement was divided into 4 sections in which the nominal 3-in. asphalt-concrete overlay had the following mix proportions:

1. Dense-graded aggregate, 40-50 penetration asphalt cement, 2.5 percent asbestos fiber, asphalt content 6.7 percent. Designated Section 4-W.
2. Dense-graded aggregate, 40-50 penetration asphalt cement, asphalt content 5.8 percent. Designated Section 4-0.
3. Dense-graded aggregate, 85-100 penetration asphalt cement, 2.5 percent asbestos fiber, asphalt content 6.4 percent. Designated Section 8-W.

4. Dense-graded aggregate, 85-100 penetration asphalt cement, asphalt content 5.5 percent by dry weight of aggregate. Designated Section 8-0.

PRELIMINARY INVESTIGATIONS

Before construction of the test section, certain preliminary investigations were required, including overlay thickness design, design of the asphalt-concrete mixtures, and a survey of the condition of the existing pavement.

Thickness Design

In order to determine the desirable thickness of asphalt-concrete overlay, the California Division of Highways' method of design was utilized. Soil borings were made along the test route for evaluation of the existing subsurface materials. Hveem R-value tests were performed on samples of the underlying soil and base aggregates; a cohesiometer value was determined for the surface courses of asphalt concrete; and a future traffic index was estimated from available traffic counts. These data indicated that a 3-in. overlay would provide adequate cover in the better areas, but could be expected to be inadequate in the poorer areas. Since a certain amount of early distress in the overlaid

TABLE 1

CHARACTERISTICS OF AGGREGATE

Characteristic	Value
Specific gravity	
Coarse aggregate (¾-in. × No. 8)	
ASTM apparent	2.89
ASTM bulk	2.84
Fine aggregate	
ASTM apparent	2.82
ASTM bulk	2.64
LA abrasion, 500 rev.	18
Sand equivalent	41

TABLE 2

CHARACTERISTICS OF ASPHALT CEMENTS

Test	Original Samples		Contractor's Storage at Time of Construction		
	85-100	40-50	85-100	40-50	40-50[1]
Pen. at 77 F	82	40	93	36	43
Ductility, cm, at 77 F	150+	150+			
Soft. Pt., R & B, F	115	128			
Viscosity					
At 77 F, poises	1.1×10^6	-			
At 140 F, poises	1393	4706			
At 180 F, poises	102.5	-			
At 275 F, stokes	2.58	4.92			
Viscosity at 275 F, SSF			122	264	
Flash point, COC, F	-	550			
Xylene equivalent			30-35	26-30	
Flash point, PMCT, F			465	-	475
Penetration ratio			30	38	
Solubility in CCl_4, %			99.9	99.9	
Thin film oven test					
Loss on heating, %			0.41	0.46	
Ret. of pen., %			60	67	
Duct. of residue			111+	111+	
Specific gravity, 77/77 F	1.015	1.020			

[1]From supplier's storage at time of shipment.

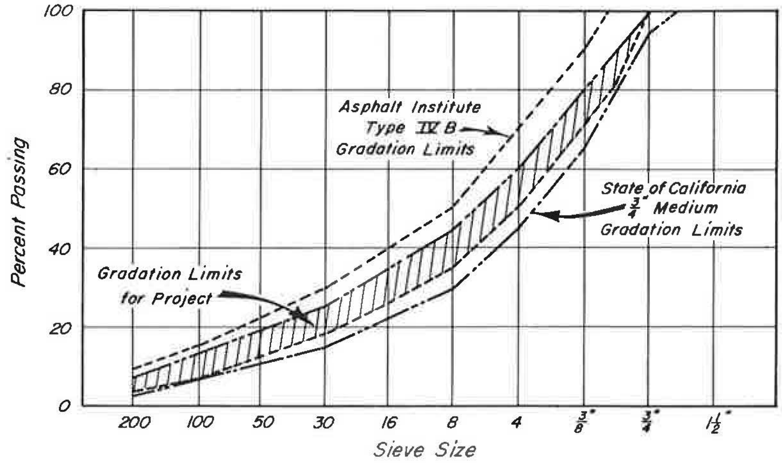

Figure 5. Aggregate gradation limits.

TABLE 3

MIX PROPERTIES AT DESIGN ASPHALT CONTENTS

Mix No.	Asphalt Content (% of Dry Wt of Agg.)	Relative Stability (S)	Cohesiometer Value (C)	Unit Weight (pcf)	% Voids Total Mix
8-0	5.5	45	340	154.5	5.2
8-W	6.4	43	400	151.8	5.4
4-0	5.8	49	600	154.1	5.2
4-W	6.7	48	520	151.8	5.2

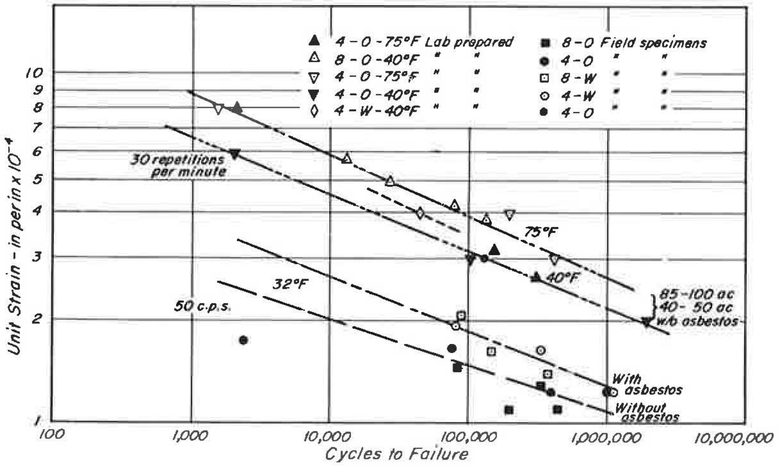

Figure 6. Fatigue test results on laboratory and field-compacted specimens.

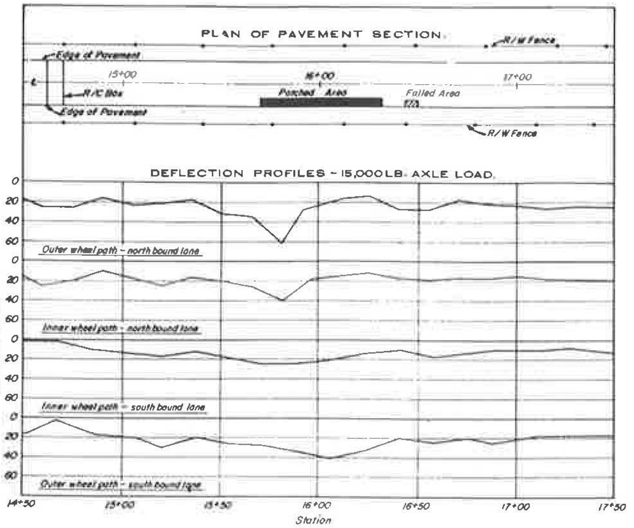

Figure 7. Typical deflection profile—low deflection area section 4-0.

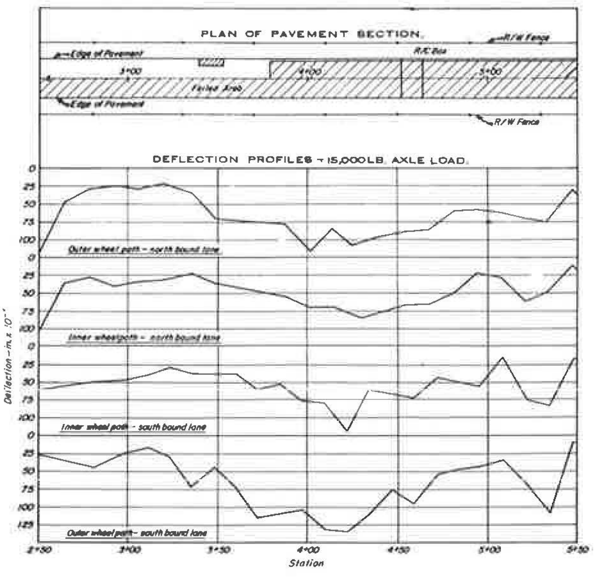

Figure 8. Typical deflection profile—high deflection area section 4-W-1.

pavement would contribute to the success of the experiment, it was decided to overlay the existing pavement with 3 in. of asphalt concrete, placed in 2 equal lifts.

Mix Design

Four different mixtures were utilized in the project. The aggregate for all of the mixtures was a crushed basaltic type material obtained from a local quarry; it has an excellent service record in pavements in the area. Typical test properties of this material are given in Table 1.

Figure 9. Typical pavement failure.

Figure 10. Typical pavement failure.

Figure 11. Typical pavement failure.

Figure 12. Typical pavement failure.

TABLE 4

SUMMARY OF ALL CRACKING IN EXISTING PAVEMENT PRIOR TO RESURFACING

Section	Area (sq ft)	Percent
4-W-1	8842	80.3
4-W-2	6398	48.2
4-0	345	4.5
8-W	533	5.0
8-0	1435	13.3

The asphalts used are from California crudes as produced and supplied by the Shell Oil Company. Results of standard tests on both the 85-100 and 40-50 materials are given in Table 2.

In order to comply with both The Asphalt Institute specifications Type IV-b and the California Division of Highways ¾-in. maximum medium gradations, the overlapping portion of these gradation bands was utilized as the limits for the asphalt concrete used for the project. The specification limits are shown in Figure 5; this gradation specification is more restrictive than either of the specifications named.

The design asphalt content for each of the mixes was selected on the basis of tests conducted by The Asphalt Institute laboratory at College Park, Md. Recommended asphalt contents, and also mix properties for each design, are given in Table 3. Values were selected on the basis of results of both the Hveem and Marshall stability tests.

While not a part of the actual mixture design, constant-strain amplitude fatigue tests were conducted on laboratory-prepared beam specimens of the mixes without asbestos at the design asphalt contents and at approximately the densities obtained during field compaction (Fig. 6). Results of tests on one series of specimens containing the 40-50 penetration asphalt and asbestos, also conducted at 40 F, are also shown in Figure 6.

These tests were performed with apparatus described elsewhere (2) at a frequency of loading of 30 applications per min and a duration of loading of 0.1 sec. For purposes of comparison, test results for a series of slabs sawed from the 4-0 section of pavement immediately after construction are also presented. Essentially the same fatigue life at 300×10^{-6} in./in. strain is obtained for both the laboratory-prepared and field-compacted specimens.

An additional series of constant-stress amplitude fatigue tests was performed on field specimens at 32 F and a frequency-of-stress application of 50 cps. These data are presented in Figure 6. Essentially the same trends were obtained in these tests on the field specimens as were obtained on the laboratory-prepared specimens.

Initial Deflection and Crack Survey

A condition survey was made of the existing pavement by measuring deflections throughout the length of the project and by conducting a crack survey. Deflections were measured in both the inner and outer wheelpaths of the northbound and southbound lanes using the traveling deflectometer developed by the California Division of Highways. Except for the first approximately 600-ft length of the project (the northern section), the initial deflections were relatively uniform. A representative deflection for a section of this latter (major) portion is shown in Figure 7. For comparison, deflections from the northern 600 ft are shown in Figure 8. Deflections as high as 0.135 in. were obtained in this area. Areas which were patched or considered to be failed as a result of the crack survey are also shown (Figs. 7 and 8). Figures 9 through 12 show typical examples of the failed areas.

The initial crack survey was conducted by outlining the cracked areas and converting them to square feet of cracking. Table 4 summarizes all cracking present in the existing pavement before resurfacing.

INSTRUMENTATION

To assist in the interpretation of the performance data obtained from the test road, instrumentation was installed in the pavement at the time of construction to measure dynamic deflections, bending strains, and pavement temperature. This instrumentation consisted of linear variable differential transformers, variable-resistance bonded wire strain gages, and thermocouples.

A typical linear variable differential transformer (LVDT) installation is shown in Figure 13. Four such installations were constructed in each test section. Although fixed in position (a possible disadvantage), these gages have the advantage, when compared to the Benkelman beam, of being able to determine the complete deflection profile for any tire configuration, and for deflections of the pavement under rapidly-moving wheel loads.

The bonded wire strain gages were installed to provide data on the bending strains induced in the resurfacing by moving wheel loads. These gages were installed on top of the existing surface prior to resurfacing near each of the LVDT installations. Two gages were placed at a specific location, one oriented parallel to and the other normal (or transverse) to the direction of traffic. After the resurfacing had been placed, two gages were also installed on the pavement surface in approximately the same locations as the gages bonded to the existing pavement. By placing both sets of gages near the LVDT installations, the radius of curvature of the deflected surface as determined from the LVDT could be related to the measured bending strains. Typical recordings of deflection and strain are shown in Figure 14 for a 15,000-lb axle load moving at creep speed at gage point 18.

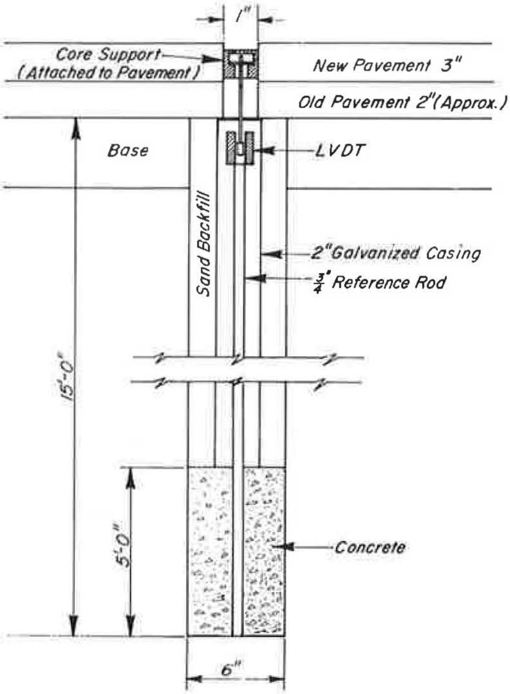

Figure 13. Typical linear variable differential transformer (LVDT) installation.

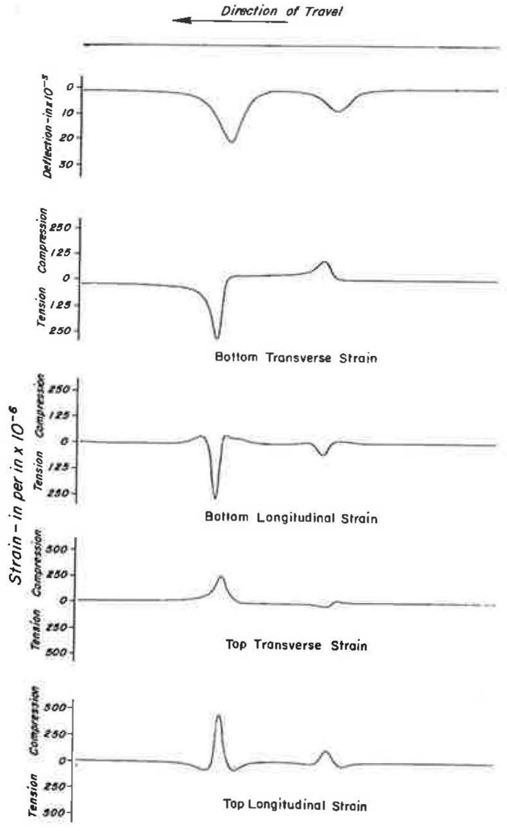

Figure 14. Oscillogram illustrating strain and deflection variation with passage of a 15,000-lb single-axle load at gage point 18.

As noted, one of the objectives of the test road project was to study the resistance of heavy-duty mixtures to fatigue cracking. Current evidence would indicate that the magnitude of the tensile strain repeatedly applied appears to be a satisfactory criterion for ascertaining the development of fatigue cracking. The strain gages thus provide a direct measure of strains occurring in the pavement. It was hoped that these measured values could be related to laboratory-determined values such as those in Figure 6.

Since temperature has an effect on the flexural stiffness of asphalt concrete, thermocouples were installed in each section to measure the temperature near the surface, at middepth, and at the bottom of the overlay. Temperature measurements were made each time the deflection and strain measurements were obtained.

CONSTRUCTION MEASUREMENTS

In order to have a more complete record of initial properties of the asphalt concrete as placed, and of construction procedures and conditions, a number of special tests were made.

Table 5 gives the results of water permeability tests made on the completed resurfacing approximately 20 hr after construction. Equipment and techniques used are as developed by the California Division of Highways and described in detail in Test Method No. Calif. 341-A. A tentative limit of 150 ml/min has been suggested (3) as a maximum acceptable permeability, with consideration being given to use of a seal coat on pavements with measured permeability greater than this limit.

Results of air permeability tests are given in Table 6. Equipment and techniques used for conducting this test are those developed by the California Research Corporation (4). No limiting values of air permeability have been suggested for general use.

TABLE 5

WATER PERMEABILITY DATA 20 HOURS AFTER CONSTRUCTION

Test Section	Water Permeability (ml per min)							
	Leveling Course				Surface Course			
	OWP[1]	BWP[1]	IWP[1]	Avg	OWP	BWP	IWP	Avg
4-W	20	25	105	64	15	14	25	16
4-0	160	180	255	206	45	25	25	33
8-W	38	33	42	38	34	28	25	29
8-0	38	35	55	40	40	28	48	38

[1] OWP—Outer Wheelpath; BWP—Between Wheelpaths; IWP—Inner Wheelpath.

TABLE 6

AIR PERMEABILITY DATA IMMEDIATELY AFTER CONSTRUCTION

Test Section	Air Permeability (ml per min at 0.25-in. pressure)			
	Northbound		Southbound	
	Average	Range	Average	Range
4-W	21	1-118	104	55-150
4-0	162	13-428	499	188-811
8-W	19	3-79	187	112-289
8-0	101	40-189	137	75-191

TABLE 7

REPRESENTATIVE MIX TEMPERATURES DURING CONSTRUCTION

Test Section	Temperature (F)		
	Placing	Initial Rolling	Pneumatic Rolling
4-W	231	206	162
4-0	266	196	180
8-W	221	213	189
8-0	233	221	179

TABLE 8

SUMMARY OF TEST RESULTS ON CORES TAKEN IMMEDIATELY AFTER CONSTRUCTION

Test Section		Unit Weight (pcf)		% Air Voids (avg)	Individual Cores					
					Hveem Tests			Marshall Tests		
		Avg	Range		S	C	Unit Wt (pcf)	Stab. (lb)	Flow (0.01 in.)	Unit Wt (pcf)
4-W	N	152.7	152.5-153.3	1.6	23	405	152.5	2622	43	152.5
	S	148.6	147.8-149.3	4.2	18	282	149.3	1922	54	148.1
4-0	N	152.4	150.8-153.4	3.0	26	247	153.4	1584	30	153.1
	S	149.9	149.2-151.1	6.0	18	137	149.1	1176	26	150.1
8-W	N	152.3	151.8-153.0	2.2	17	409	153.0	1488	28	151.9
	S	148.6	146.0-149.4	4.9	15	246	147.9	895	28	149.4
8-0	N	155.5	154.4-156.4	1.4	20	214	155.8	1950	29	156.4
	S	153.7	153.2-154.4	2.6	23	212	153.6	1406	22	154.4

Thermocouples were installed in the mix at the time of construction to develop information on mix temperatures during placing and rolling. Table 7 gives representative temperatures at approximately ¾ in. below the surface as measured by these thermocouples during construction operations.

Immediately following completion of the resurfacing, cores were cut at selected locations in each test section. Test results are summarized in Table 8. Extraction tests were conducted on samples of the asphalt concrete obtained at the hot-mix plant during production. Aggregate gradation and asphalt content as determined from these samples are given in Table 9 and compared with specified gradation limits and design asphalt content.

TABLE 9

RESULTS OF EXTRACTION TESTS ON PLANT SAMPLES

Sieve Sizes	Percent Passing				
	Specification Limits	Section 4-W	Section 4-0	Section 8-W	Section 8-0
¾ In.	100	100	100	100	100
½ In.	-	90	85	88	88
⅜ In.	70-80	76	64	68	66
No. 4	50-60	64	54	56	54
No. 8	35-45	42	35	41	35
No. 16	-	30	26	32	25
No. 30	18-25	23	22	27	20
No. 50	-	16	18	20	16
No. 100	-	10	10	11	10
No. 200	4-7	5.8	6.8	6.3	6.8
Asphalt Content[1]		7.0	5.3	6.0	4.9
Design Asphalt Content[1]		6.7	5.8	6.4	5.5

[1] Percent of dry weight of aggregate.

The average measured thickness of asphalt-concrete resurfacing, calculated from measurements made on 111 cores taken during the entire period of study, was 3.04 in.

TEST PROGRAM

To measure properly changes occurring in the various test sections and to attempt to relate these changes to traffic and environment during the test period, a comprehensive series of field and laboratory measurements was planned. The test program included, where appropriate, the following measurements before and after pavement construction:

1. Traffic and load surveys;
2. Deflection measurements with the traveling deflectometer of the California Division of Highways;
3. LVDT deflection and strain measurements at permanent gauge installations;
4. Tests on cores for determination of changes in mix and asphalt properties with time;
5. Precise levels;
6. Road surface measurements including skid resistance and road roughness;
7. Condition (cracking) surveys by visual observation.

Traffic Counts and Index

The 1964 California Division of Highways procedures were followed in evaluating traffic characteristics on the project during the initial 3-yr period following construction. To convert traffic into a traffic index, it was necessary to have some knowledge of truck traffic, axle configuration, and load.

Visual truck traffic counts, including axle configuration, were made at 4 different times through July 1964. These counts were made during the weeks of Aug. 25-Sept. 1, 1961; Aug. 27-31, 1962; Sept. 24-28, 1962; and March 4-7, 1963. The computations showed that traffic was essentially constant for the first 3 traffic counts. The final count was made after the opening of the Benicia-Martinez Bridge which permanently rerouted certain truck traffic from the test road to a new highway. This last count was substantially lower than on previous dates; however, it is believed to be representative of present and future traffic.

Typical gross load information (loaded and unloaded) was recorded as part of the traffic count, and was used to determine an equivalent wheel load factor (loaded and unloaded) for 2-, 3-, 4-, and 5-axle trucks.

To determine equivalent wheel load from gross load data, it was necessary to distribute the load to the various axles. A typical axle configuration for each type of truck was assumed, based on what is believed to be the most common truck design operating in the area. The assumed load distribution between axles is based, to a large extent, on information on truck weights from the AASHO Road Test. Both of these assumptions could be subject to some variation. It is believed, however, that the assumptions made will result in a reasonable calculation of average equivalent wheel load factors.

With the loads for each wheel determined, it was possible to convert to an EWL factor using the 1963 equations of the California Division of Highways. On the basis of these load factors, the traffic was converted to equivalent 5000-lb wheel loads for northbound and for southbound traffic. The traffic index was then computed from the following formula:

$$TI = 6.7\left(\frac{EWL}{10^6}\right)^{0.119}$$

The estimated southbound traffic index is 6.9 and the northbound traffic index 6.3 for the 3-yr period.

Cracking Surveys

Pavement cracking surveys were made at 4 intervals from the inception of the project to the end of the study period. The initial survey made in March 1961 (before the overlay) represents the preconstruction condition and was discussed above. Subsequent crack surveys were made in March 1962, August 1962, March 1963, and June 1964.

To show the progressive manner in which cracking developed, cumulative percent of pavement cracked, by lane and wheelpath, is given in Table 10. This tabulation is simply the length in which cracking was present, expressed as percent of total length.

For purposes of analysis, the results of the preconstruction and June 1964 cracking surveys only were used, and a refinement in the measurement of cracking was made. Table 11 gives these results. Since cracking is expressed here as percent of total area, rather than length, values shown here cannot be compared with those in Table 10.

It is important to point out that certain limitations in construction did not allow precise control of placing of the various types of materials within established boundaries; therefore, 100-ft transitions between sections were eliminated from the analysis.

TABLE 10

PROGRESSION OF CRACKING AS SHOWN BY RESULTS OF THREE POST-CONSTRUCTION SURVEYS

Section	Date	Cumulative Percent of Pavement Length Cracked				
		Southbound Lane		Northbound Lane		Total for Section
		IWP	OWP	IWP	OWP	
4-W-1	Mar 1962	4	0	0	7	3
	Oct 1962	18	45	27	42	33
	Mar 1963	56	45	31	45	44
4-W-2	Mar 1962	47	0	0	0	12
	Oct 1962	47	2	0	0	12
	Mar 1963	49	3	26	3	20
4-0						(No cracking)
8-W	Mar 1962	11	0	0	0	3
	Oct 1962	13	0	1	0	3
	Mar 1963	19	1	2	7	7
8-0						(No cracking)

TABLE 11

COMPARISON OF CRACKING MEASURED IN JUNE 1964
WITH PRECONSTRUCTION CRACKING

Section	Date	Percent of Pavement Area Cracked				
		Southbound Lane		Northbound Lane		Total for Section
		IWP	OWP	IWP	OWP	
4-W-1	Preconstr.	98	98	68	57	80
	June 1964	20	16	15	18	17
4-W-2	Preconstr.	50	55	37	52	48
	June 1964	31	11	9	10	15
4-0	Preconstr.	0	18	0	0	5
	June 1964	0	0	0	0	0
8-W	Preconstr.	7	13	0	0	5
	June 1964	12	9	2	2	7
8-0	Preconstr.	30	18	6	0	13
	June 1964	0	0	0	0	0

The cracking survey made in June 1964 resulted in the identification of two important crack patterns. The predominant type of crack is longitudinal, occurring generally within the limits of the wheelpath. Some other cracking, often referred to as "chicken-wire" or "pattern" cracking, was also observed. Both longitudinal and chicken-wire cracking were plotted regardless of the degree or amount of progression (i.e., "hair-line" cracking was included), and the area reported was based on the presence or absence of cracking, without regard to degree or severity. In some instances, where cracking is just barely discernible, this criterion of performance could be considered as a very severe judgment of distress. However, it is likely that these hairline cracks are signs of impending distress, given enough time and traffic.

Deflection Surveys with Traveling Deflectometer

Pavement deflection surveys were made with the California Division of Highways traveling deflectometer (5). This equipment measures surface deflections under a 15,000-lb single-axle load and provides an analog trace of the deflected basin along the longitudinal axis.

Deflection measurements were made with this equipment 5 times up to June 1964. These deflection runs were made on: March 27, 1961 (preconstruction); Sept. 20, 1961; March 8, 1962; Aug. 31, 1962; and March 18, 1963. Since the parameters of the deflection test are considered of value primarily as predictors of future performance, it was desirable to select data from earlier tests (i.e., either the Sept. 1961 or March 1962 series of measurements) as a basis for the analysis. The March 1962 run was selected for 2 reasons:

1. The Sept. 1961 run was taken too soon (approximately one week) after construction to allow the mix to assume a condition representative of its long-term characteristics.
2. There was less scatter in the data of March 1962, indicating somewhat more reliable information.

Average deflections for Sept. 1961 and March 1962 for the southbound lane are shown separately by section and wheelpath in Figure 15. Similar data for the northbound lane are shown in Figure 16. Although the Sept. 1961 deflection measurements are not used in the analysis, they are included in these figures to illustrate the considerable reduction in deflections from immediately following construction to only a few months later.

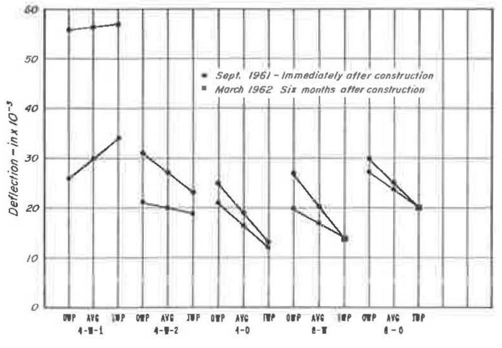

Figure 15. Average deflections by section and wheelpath for the southbound lane.

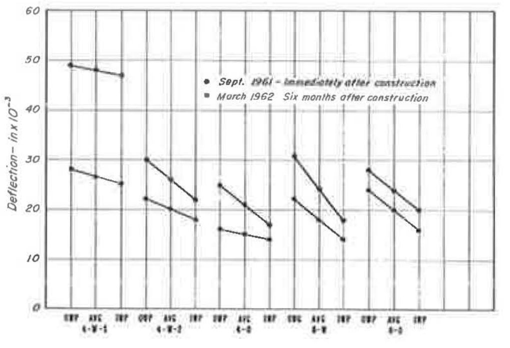

Figure 16. Average deflections by section and wheelpath for the northbound lane.

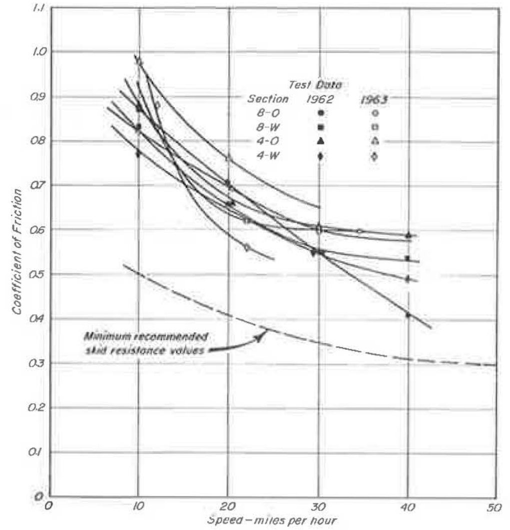

Figure 17. Summary of 1962 and 1963 skid resistance measurements—wet pavement tests.

Surface Measurements

Skid resistance tests were performed on each of the test sections in March 1962 and Dec. 1963. Tests were conducted with the University of California skid resistance equipment using a 1958 Standard Test Tire (6). Values obtained for coefficient of friction for all sections (Fig. 17) are in the range considered to give skid resistance comparable to well-constructed State Highways in California.

Road roughness tests were performed in Nov. 1961, using University of California equipment (7). The roughness index at 20 mph was 106 in./mi for the southbound lane and 102 in./mi for the northbound lane. Good riding quality is indicated since values less than 125 in./mi are considered satisfactory for this type of facility.

Precise level surveys were made on the surface immediately following placing of the resurfacing and periodically thereafter. Comparison of the results of these surveys indicates no measurable rutting or distortion. Comparison of density measurements made on cores taken immediately after construction with those taken in 1962 and 1963 indicates no significant trend toward densification under traffic. This tends to verify conclusions made from the precise level surveys.

Deflection (LVDT) and Strain Data

Table 12 summarizes deflection, strain and temperature data obtained from the deflection and strain gage installations in the various sections during the period Oct. 1961 to April 1963. Since the measurements covered periods of time of as much as an hour at a particular gage point, the temperature range during this interval is given in many instances.

At a particular point, measurements were taken both of strain and deflection for various positions of the wheel load with respect to the gage installation and with the truck (single rear-axle, dual tires, 15,000-lb axle load) traveling at creep velocity (1 to 3 mph). These measurements at the various positions permit development of

TABLE 12

SUMMARY OF DEFLECTION AND STRAIN MEASUREMENTS AT FIXED GAGE INSTALLATIONS RESULTING FROM 15,000 LB AXLE LOAD ON DUAL TIRES TRAVELING AT CREEP SPEED (1—3 MPH)

Section	Gage Point	Date	Air Temp. (F)	Pavement Temperature (F)			Maximum Deflection (in.)	Maximum Observed Tensile Strain (in. per in. × 10^{-6})			
								Bottom		Top	
				Bottom	Middle	Top		Transverse	Longitudinal	Transverse	Longitudinal
4-W-1	1	17 Apr 62	-	95-102	106-114	116-121	0.040	35	*	150	100
	4	17 Apr 62	-	102-104	112	120-116	0.056	60	*	120	75
	4	28 Aug 62	-	101-111	124-140	127-134	0.056	40	*	230	100
4-W-2	5	26 Oct 61	-	74-76	76-78	78-84	0.025	25	15	25	25
	5	27 Dec 61	44	47-50	48-50	49-51	0.0185	40	30	50	20
	5	27 Dec 61	44	48	49	50	0.018	65	60	15	5
	5	17 Apr 62	73-81	83-93	90-103	102-119	*	40	10	90	70
	5	28 Aug 62	-	105-107	115-116	129-124	0.036	100	95	145	235
	6	1 Nov 61	-	63	62	62	0.018	30	80	75	65
	6	27 Dec 61	45	48	48	50	0.018	90	75	10	20
	6	18 Apr 62	-	74-78	76-82	78-86	0.014	400	*	*	65
	6	29 Aug 62	-	76-100	76-108	78-119	0.0205-0.023	475	*	130	250
	7	17 Apr 62	-	99	109-102	118-107	0.060	80	110	125	70
	7	12 Apr 63	-	78	84	90-92	0.0275	70	105	40	40
4-0	9	26 Oct 61	-	76	78	78-77	0.028	25	75	75	50
	9	22 Nov 61	-	56	55	55	0.0165	20	25	25	15
	9	26 Dec 61	56	60	64	66	0.022	20	20	15	25
	9	27 Dec 61	43-40	50	48	46	0.016	25	25	25	35
	9	18 Apr 62	-	80	83	86	0.033	15	80	85	45
	9	30 Aug 62	-	77-79	76-80	77-84	0.025	70	75	70	50
	9	12 Apr 63	-	85-80	90-89	90-88	0.021	35	115	40	60
	10	18 Apr 62	-	85-91	90-98	97-105	0.060	35	400	95	90
	10	12 Apr 63	72-70	-	82-88	92-86	0.032	225	450	35	50
8-W	14	25 Oct 61	-	67-78	70-84	75-86	0.041	20	15	175	45
	14	22 Nov 61	-	56	55	55	0.0235	5	30	40	15
	14	27 Dec 61	46	54	55	55	0.025	10	20	50	40
	14	18 Apr 62	-	96	103	103	0.041	90	145	70	150
	14	31 Aug 62	-	80-84	82-100	88-104	0.027	25	50	140	150
	14	13 Apr 63	70-73	70	80-84	84-85	0.019	*	45	65	65
	15	26 Oct 61	-	68-72	74-80	78-82	0.0435	0	*	90	40
	15	26 Dec 61	-	60	64	67	0.0395	15	45	50	50
	15	27 Dec 61	40	48	46	44	0.0335	20	25	40	40
	15	19 Apr 62	-	94-91	99-89	100-84	0.054	0	60	215	65
	15	30 Aug 62	-	98-100	111	113-117	0.0425	70	30	415	175
	15	13 Apr 63	74	71-78	80-84	85-88	0.029	40	*	90	75
	16	26 Oct 61	-	74	78	81	0.033	*	*	100	150
8-0	18	25 Oct 61	-	79	-	80	*	170	280	115	100
	18	26 Dec 61	52	62	-	-	*	60	125	525	150
	18	27 Dec 61	-	45	-	40	*	80	230	25	15
	18	19 Apr 62	-	94	-	-	0.043	575	500	150	240
	18	30 Aug 62	-	114	-	120	0.037	365	420	450	150
	18	31 Aug 62	-	74-82	-	78-84	0.0255	220	285	100	75
	18	13 Apr 63	70-73	70	80-84	84-85	0.023	350	345	115	175
	19	24 Oct 61	-	86-88	-	-	0.036	*	*	150	140
	19	26 Dec 61	46	50-53	-	-	0.0265	*	*	40	200
	19	27 Dec 61	48-46	54-56	-	-	0.025	*	*	140	125
	19	27 Dec 61	39	45	-	-	0.0205	*	*	125	40

*Gage inoperative.

complete patterns of deflection and strain at the gage point. In addition to the series of measurements at creep speed, additional measurements of deflection and strain were obtained for velocities of up to 40 mph for passage of the center of the duals of the rear axle over the gage point.

The data in Table 12 were obtained from the creep speed measurements. Maximum measured values for deflection and tensile strain are listed to permit a comparison between sections. In general, the highest values of strain were recorded in Section 8-0. Comparing gage points 18 (Section 8-0) and 15 (Section 8-W) on April 13, 1963, for example, with about the same pavement temperatures in both instances, the observed tensile strains at gage point 18 are considerably larger than those at point 15.

Many different analyses can be made for the strain and deflection data. Some of the possibilities are included to give an idea of what can be done rather than to establish definite criteria.

Figure 18 shows the complete deflection pattern of the pavement at gage point 18 on April 19, 1962. The maximum deflection occurs under one of the tires in this instance. These data were developed from the recordings of deflections obtained by moving the loaded wheel with respect to the gage installation. Figures 19 and 20 illustrate the variation of longitudinal strain both at the top and underside of the pavement for the same conditions.

At the time these measurements were being obtained, a 5-axle truck with a gross load of approximately 75,000 lb also passed over the gage installation. The recording (Fig. 21) was made with the centerline of the dual tires passing over the point; therefore

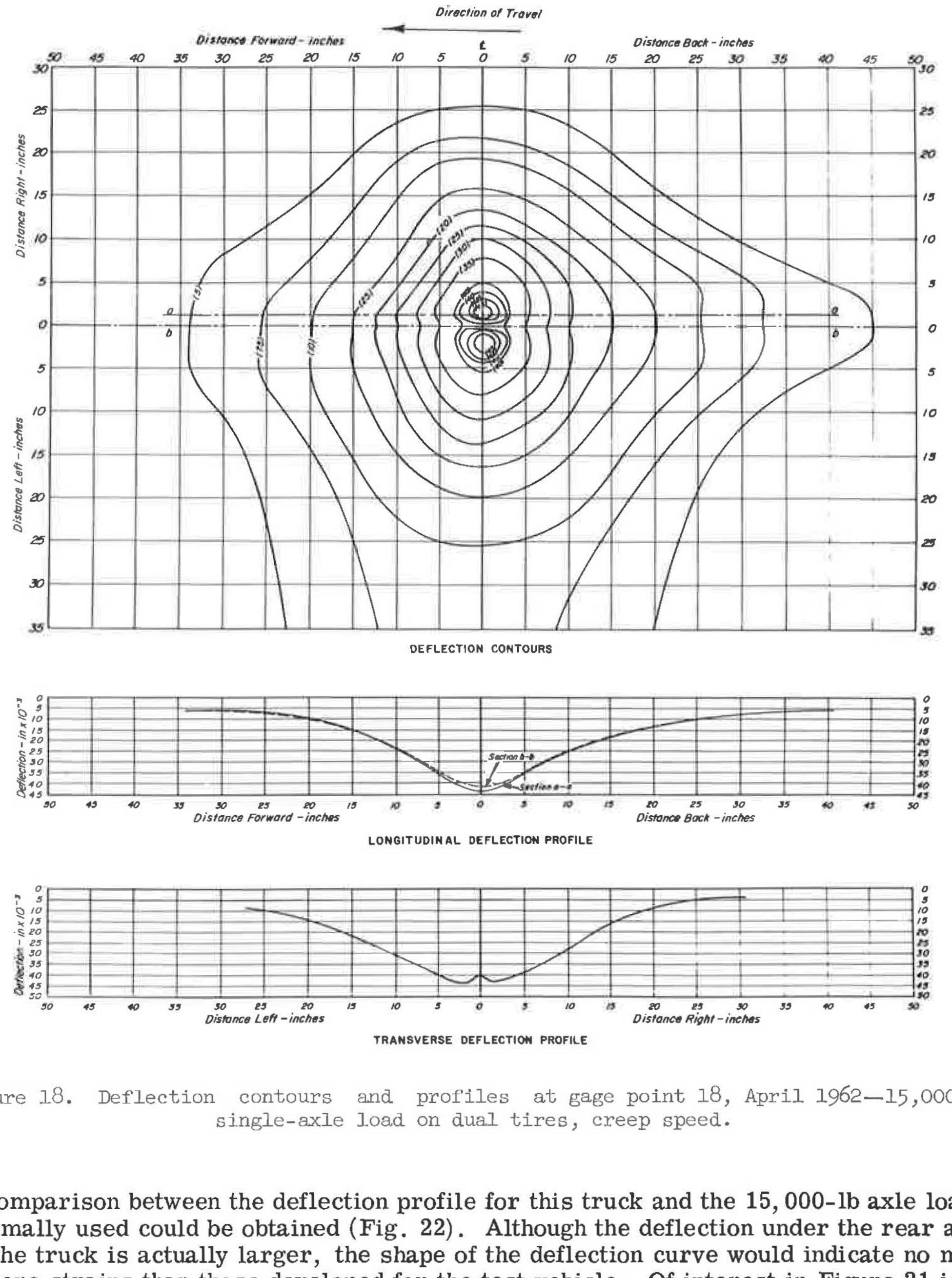

Figure 18. Deflection contours and profiles at gage point 18, April 1962—15,000-lb single-axle load on dual tires, creep speed.

a comparison between the deflection profile for this truck and the 15,000-lb axle load normally used could be obtained (Fig. 22). Although the deflection under the rear axle of the truck is actually larger, the shape of the deflection curve would indicate no more severe strains than those developed for the test vehicle. Of interest in Figure 21 is the high tensile strain developed by the front axle in this instance. At times the front axle is neglected in pavement design evaluation. This measurement, along with analysis of many of the recordings from this project, indicate that often the front axle is at least as severe as the rear axle in terms of inducement of strain.

As noted earlier, the effect of vehicle speed on deflection and strain was obtained by passing the center of the rear duals of the vehicle over the gage point at speeds up to 40 mph. Figure 23 shows the reduction in deflection with increase in speed at point 18 on April 19, 1962. At this time, the temperature at the bottom of the overlay pavement was of the order of 97 F. A reduction of almost 0.008 in. was obtained over the

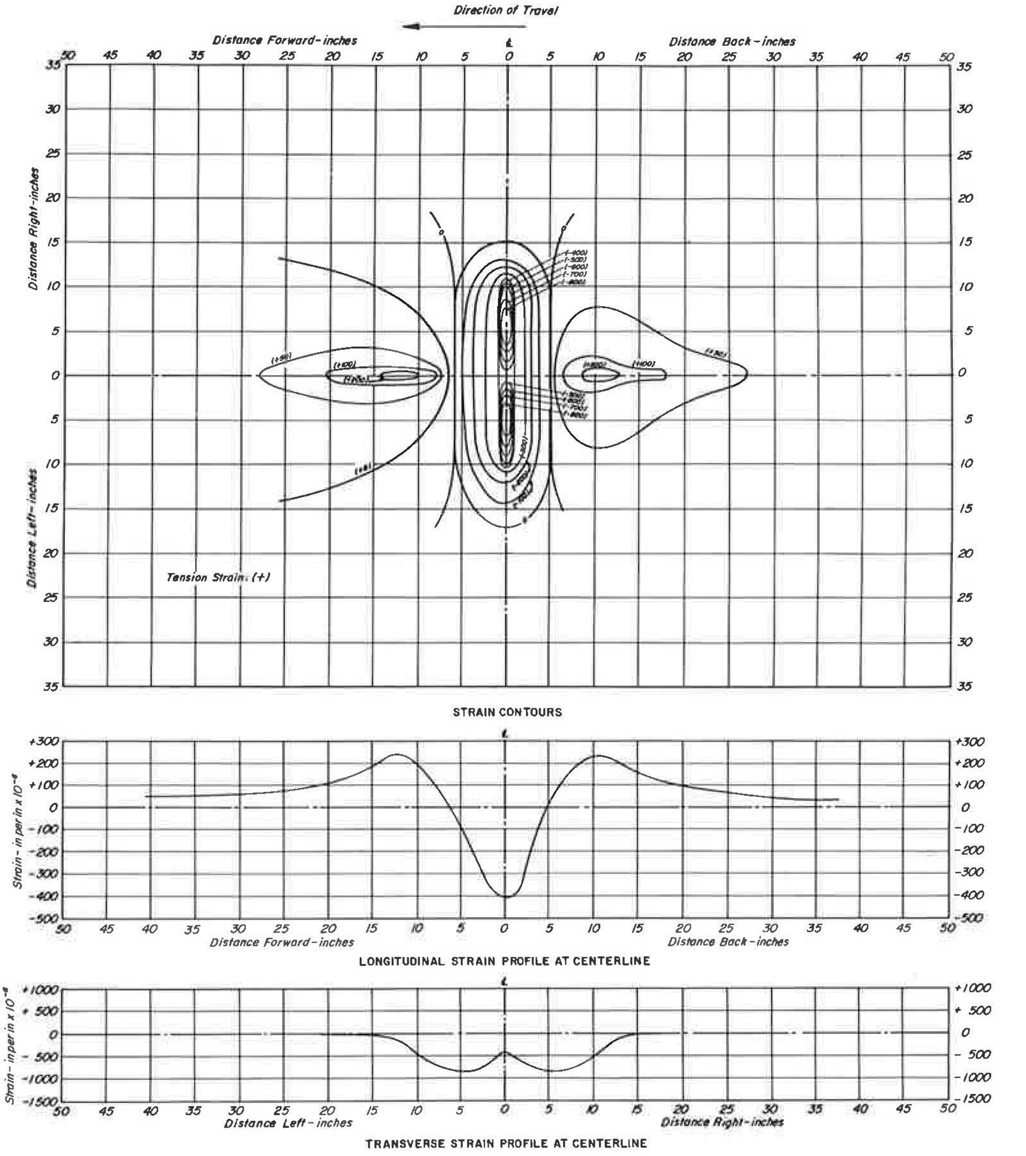

Figure 19. Contours and profiles of top longitudinal strain at gage point 18, April 1962—15,000-lb single-axle load on dual tires, creep speed.

range of vehicle speeds investigated. One cannot, of course, always guarantee this much change. Figure 24 shows, for the 4-0 section, the effects of both speed and temperature on deflection. At the lesser temperatures the reduction in deflection with increased speed is comparatively small.

The shape of the deflected surface is considerably affected by temperature. Figure 25 shows deflection profiles normal to the direction of travel at gage point 18 obtained in August 1962. The effect of mixture stiffness is quite apparent.

One of the interesting, and perhaps significant, measurements in the field investigation is the transverse strain. Figures 25 and 26 show the transverse deflection profile and variation of transverse strain both in the top and the underside of the overlay.

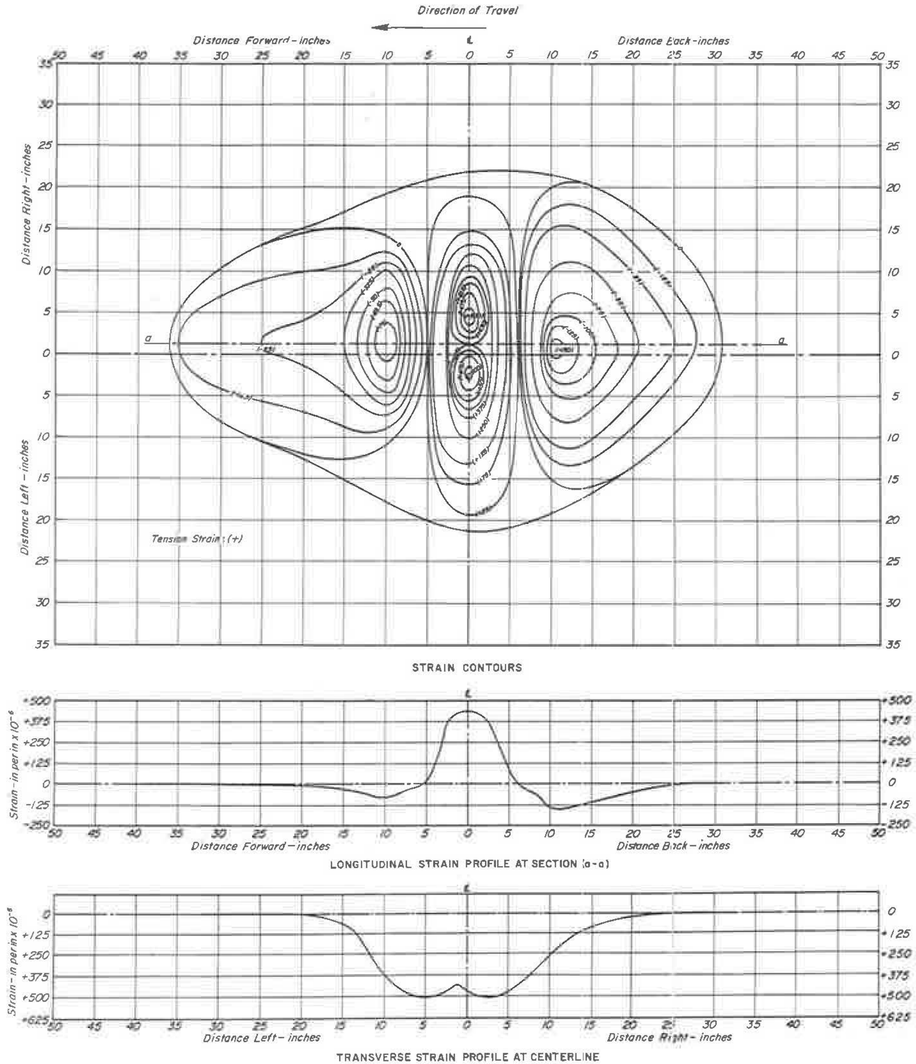

Figure 20. Contours and profiles of bottom longitudinal strain at gage point 18, April 1962-15,000-lb single-axle load on dual tires, creep speed.

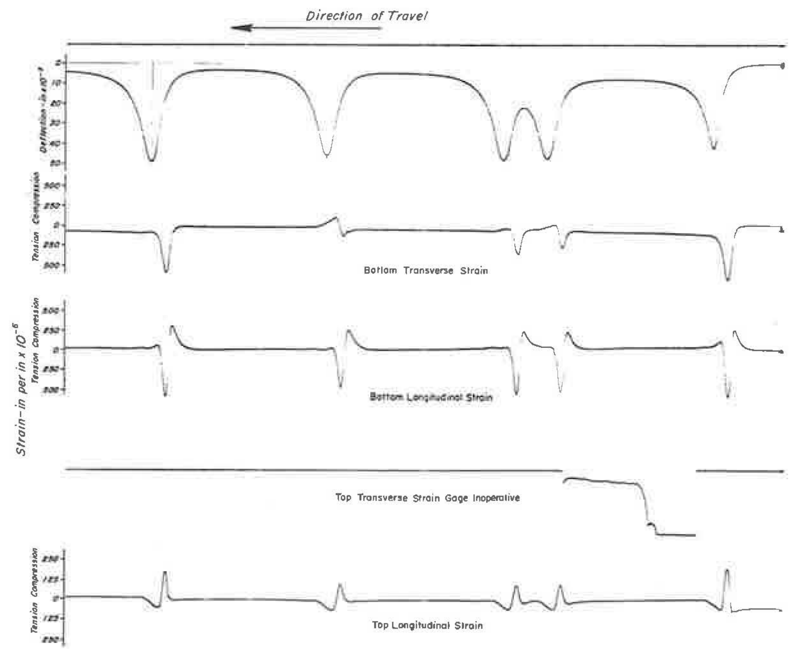

Figure 21. Oscillogram illustrating strain and deflection variation with passage of 5-axle truck with a gross load of 75,000 lb over gage point 18.

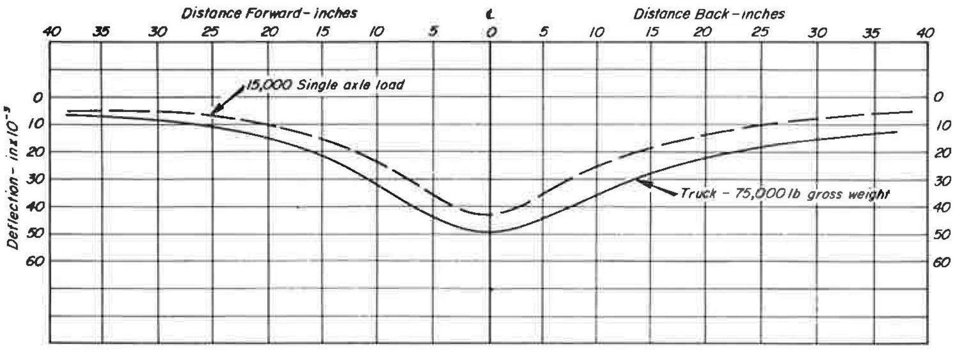

Figure 22. Comparison of centerline longitudinal deflection profiles for 15,000-lb single-axle load and rear axle of 5-axle 75,000-lb gross load truck at gage point 18.

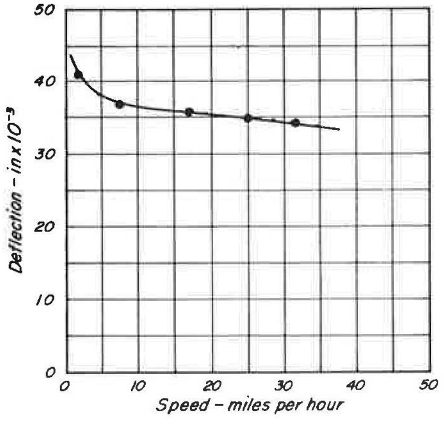

Figure 23. Effect of speed on centerline deflection for 15,000-lb single-axle load at gage point 18, April 1962.

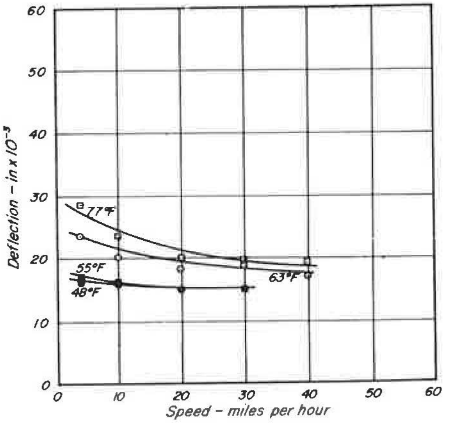

Figure 24. Effect of speed and temperature on centerline deflection for 15,000-lb single-axle load at gage point 9, Nov.-Dec. 1961.

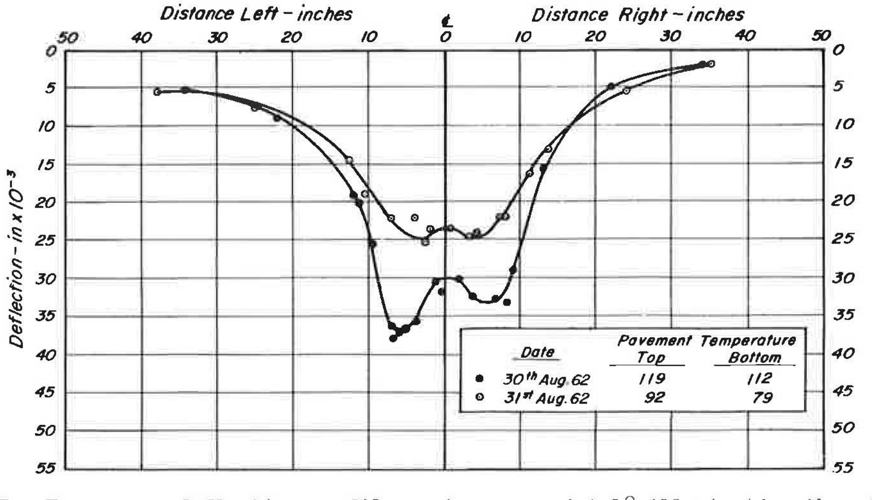

Figure 25. Transverse deflection profiles at gage point 18 illustrating the effect of temperature—15,000-lb single-axle load.

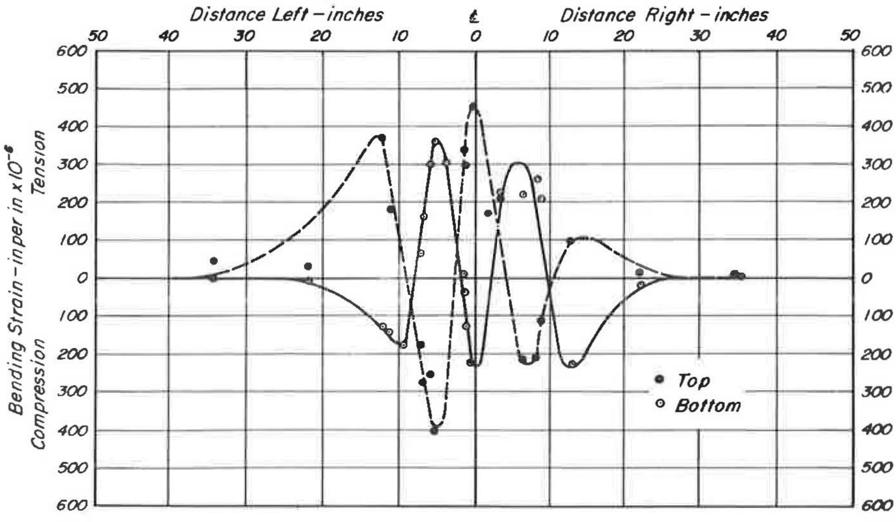

Figure 26. Comparison of transverse profiles of top and bottom transverse strain at gage point 18—15,000-lb single-axle load, 30 Aug. 1962.

Comparing the intensity of transverse strain at the surface, it is of at least the same order of magnitude as the tensile strain on the underside of the overlay in the longitudinal direction. The rate of change of this strain with distance is also interesting. Figure 25 emphasizes the importance of accurate measurement of the placement of the vehicle and also emphasizes why the strain data in Table 12 have been termed maximum observed values, since it is possible that higher values than those measured may have occurred.

DISCUSSION AND EVALUATION

From the data presented, we would conclude that the pavement is a well-constructed, dense, impervious overlay. Measurements of density and of water and air permeability substantiate this.

Road roughness measurements indicate little difference between northbound and southbound lanes, and, based on the level of roughness of 102 to 106 in./mi, the pavement is considered to be comparatively smooth. Skid resistance measurements made both in 1962 and 1963 indicate safe values of friction coefficient for all sections under wet, skidding conditions.

Since the riding quality of all sections is about the same and at a comparatively high level, it is difficult to judge performance on this basis. However, cracking has been observed in some sections of the overlay construction (Tables 10 and 11). While this cracking does not impair the riding qualities at this time, it is symptomatic of some undesirable condition, as yet undefined, and may be considered as a measure of performance. Thus the sections with the greatest amount of cracking could be considered to exhibit the poorest performance. On this basis the order of performance indicated by Tables 10 and 11 would be (1) Sections 4-0 and 8-0, (2) Section 8-W, (3) Section 4-W-2, and (4) Section 4-W-1.

However, it will be noted from Table 11 that a considerable part of the original pavement area in Section 4-W-1 (approximately 80 percent) and 4-W-2 (approximately 48 percent) was cracked. Thus, particularly in the case of 4-W-1, the cracking in the overlay probably was markedly influenced by the cracking in the original pavement. In addition (from Figs. 15 and 16), the deflections after the overlay in Section 4-W-1 are higher than the other four; thus, it cannot be compared with these sections.

Considering data in Table 11 and Figures 15 and 16 and excluding Section 4-W-1, a modified order of ranking for performance to date would be (1) Section 8-0, (2) Section 4-0, (3) Section 8-W, and (4) Section 4-W-2.

Beyond rating the pavement performance to date, it is important to attempt to explain present observed behavior and to predict future performance. The deflection measurements serve as a useful tool, particularly in the light of developments such as those presented by Hveem (8), the Canadian Good Roads Association (9), and in the AASHO Road Test report (10).

Hveem (8) has suggested that the safe limiting deflection value under a 15,000-lb single-axle load ranges from 0.020 in. for a 3-in. surfacing to the order of 0.012 in. for an 8-in. surfacing. In the actual pavement, the average thickness of the resurfacing, plus the existing pavement, is 4¾ in. On this basis a safe limiting deflection of the order of 0.016 in. would be indicated for heavy traffic conditions. From Figures 15 and 16, the average deflections for March 1962 in all sections (except 4-W-1) are in the range of 0.015 in. to 0.020 in. While the safe limiting deflection value noted above has been associated with heavy traffic, Sherman (11) has presented an analysis of the WASHO data which would indicate that this value is traffic-dependent. According to the results of the WASHO Road Test (12), critical deflections ranged from 0.045 in. to 0.030 in. for warm and cold weather, respectively. The traffic index on the 238,000 applications at WASHO, as determined by the California EWL_{57} procedure, ranged from 7.2 for the 18,000-lb single-axle load to 8.5 for the 40,000-lb tandem-axle load. According to the traffic index as of June 1964 for this project, the value was 6.9 for the southbound lane and 6.3 for the northbound lane. Since both the deflection values and traffic indexes are less than those for WASHO, no cracking should be indicated. This is the situation in the 4-0 and 8-0 sections.

Another method for evaluation of present and future performance is that developed by the Canadian Good Roads Association. According to the CGRA (9), pavement performance can be related to deflection, age and traffic. Deflections are analyzed not as average values but as an average plus 2 standard deviations ($\bar{x} + 2\sigma$). This technique recognizes not only the order of magnitude of deflection measurements but also the range intensity of the distribution of measurements. According to their analysis, a pavement whose deflection factor ($\bar{x} + 2\sigma$) does not exceed 0.025 in. should provide "good" performance for heavy traffic up to 14 yr. A pavement with a deflection factor of 0.050 in. should provide the same level of performance for about 6 yr, and a pavement with a deflection factor of 0.075 in. for about 2 yr. The average deflection data, together with standard deviations, for the various periods are summarized in Table 13. Table 14 summarizes the deflection data for each Shell Avenue section according to the CGRA method of evaluation. From this intepretation, Sections 4-W-1 and 4-W-2 would not be expected to maintain a high level of performance for longer than 3 to 5 yr. Sections 8-0 and 8-W should last about 8 yr and Section 4-0 about 10 yr.

Performance by the CGRA criteria was an overall subjective rating by a panel of 5 raters and, as such, is not directly comparable to the ratings used on the Shell Avenue Test Road. Also, environmental differences exist which would tend to make Canadian criteria conservative for conditions in California. Nevertheless, it does provide a means for comparing the efficiency of deflection testing for predicting performance.

One of the most comprehensive programs of field studies to relate deflection to pavement performance was the AASHO Road Test conducted from 1956 to 1960 in Ottawa, Ill. (10). Part of this research included a study of the deflection-performance relationship. It is pertinent here, as with the CGRA investigation, to point out that performance criteria were not based exclusively on cracking. In fact, examination of the formula used to predict performance on the AASHO project might lead to the conclusion that cracking played only a minor role in performance. However, a closer analysis will show that cracking may have a significant effect on the longitudinal profile as measured with the special Road Test profilometer and, therefore, cracking would influence this measurement, which is the prime factor in the performance term.

The general equation found from the Road Test data (10, p. 110) resulted in the following relationship for associating deflection with performance:

$$\log W_{2.5} = 7.98 + 1.72 \log L - 3.07 \log d$$

where

TABLE 13

SUMMARY OF DEFLECTION DATA

Section	Wheelpath	Date	Southbound Avg	Southbound σ	Northbound Avg	Northbound σ
4-W-1	Inner	9-61	56.58	19.86	47.22	15.21
		3-62	34.22	12.91	25.20	11.78
		3-63	35.32	13.23	25.24	3.21
		Avg	42.04	15.09	32.55	10.07
	Outer	9-61	55.74	15.30	49.05	15.75
		3-62	25.94	11.63	27.60	13.09
		3.63	29.26	8.48	32.94	12.93
		Avg	36.98	11.80	36.53	13.92
4-W-2	Inner	9-61	23.11	10.67	21.50	8.81
		3-62	19.06	8.66	18.08	6.79
		3-63	18.55	8.16	16.06	5.79
		Avg	20.24	9.16	18.55	7.13
	Outer	9-61	30.85	6.83	30.22	9.32
		3-62	20.65	5.45	21.50	6.14
		3-63	20.08	5.79	22.30	6.71
		Avg	23.86	6.02	24.67	7.39
4-0	Inner	9-61	13.48	6.28	16.75	7.00
		3-62	11.65	4.79	14.00	5.55
		3-63	11.70	4.55	10.19	4.01
		Avg	12.28	5.21	13.65	5.52
	Outer	9-61	25.26	7.79	24.68	9.19
		3-62	20.62	6.10	16.39	5.91
		3-63	18.19	6.23	15.15	4.76
		Avg	21.36	6.71	18.74	6.62
8-W	Inner	9-61	13.51	8.39	17.67	6.47
		3-62	13.58	7.11	14.41	5.10
		3-63	12.92	5.49	11.92	3.55
		Avg	13.34	7.00	14.67	5.04
	Outer	9-61	27.00	5.72	30.72	9.12
		3-62	20.18	4.73	21.79	5.23
		3-63	21.64	4.50	21.87	6.40
		Avg	22.94	4.98	24.79	6.92
8-0	Inner	9-61	19.82	6.17	20.26	5.37
		3-62	20.11	5.17	16.49	3.26
		3-63	20.55	5.17	14.71	4.27
		Avg	20.16	5.50	17.15	4.30
	Outer	9-61	29.95	7.01	27.77	5.17
		3-62	27.47	4.96	23.88	4.83
		3-63	24.63	5.30	21.62	5.43
		Avg	27.35	5.76	24.42	5.14

TABLE 14

SUMMARY OF DEFLECTION DATA BY CGRA METHOD

Section	Deflection in Southbound Lane (in. × 10^{-3}) Avg	2σ	Avg + 2σ	Adjusted to[1] 18,000-Lb Single-Axle Load
4-W-1	30	12	54	65
4-W-2	20	6	32	38
4-0	15	5	25	30
8-W	17	6	29	35
8-0	23	5	33	40

[1]Multiply by $\frac{18}{15}$ (Avg + 2σ).

$W_{2.5}$ = number of applications of axle L sustained by the pavement at the time serviceability was at level 2.5;

L = single-axle load in kips; and

d = normal fall deflection in 0.001 in., measured under a wheel load equal to L/2.

According to this equation, it should be possible to estimate the number of 10-kip single-axle load repetitions to a serviceability index of 2.5. Utilizing actual traffic and deflections on the Shell Avenue Test Road in the equation would then provide a tie to the Road Test data. Based on the results in Table 13 and the above equation, Table 15

TABLE 15

SUMMARY OF ESTIMATED ALLOWABLE TRAFFIC FOR VARIOUS SECTIONS ACCORDING TO AASHO ROAD TEST EQUATION

Section	Lane	Deflection (in. × 10^{-3})	Total No. 15,000-Lb Axle Loads	EWL_{57} × 1000	TI_{57}^{1}	EWL_{57} × 1000	TI_{63}
4-W-1	SB	30	298,000	2,235	6.7	1,640	7.0
	NB	26	445,000	3,340	7.1	2,450	7.5
4-W-2	SB	20	1,000,000	7,500	7.7	5,500	8.3
	NB	20	1,000,000	7,500	7.7	5,500	8.3
4-0	SB	16	1,990,000	14,980	8.3	10,940	8.9
	NB	15	2,450,000	18,400	8.5	13,480	9.1
8-W	SB	17	1,655,000	12,400	8.1	9,120	8.8
	NB	18	1,410,000	10,580	8.0	7,760	8.7
8-0	SB	24	574,000	4,300	7.3	3,160	7.7
	NB	20	1,000,000	7,500	7.7	5,500	8.3

[1] Traffic Index: $TI_{57} = 1.35\,(EWL_{57})^{0.11}$; $TI_{63} = 6.7\,\frac{(EWL_{63})^{0.119}}{10^6}$

summarizes the estimated number of 15,000-lb axle loads and the corresponding traffic index. Using average deflection values from the Shell Avenue Test Road, it is possible to estimate the EWL associated with the critical serviceability index of 2.5. By comparing the traffic indexes in Table 15 with those reported for the Shell Avenue project, it can be concluded that all the various test sections should be at a relatively high level of serviceability through June 1964, and this is the case.

The various analyses which utilized traffic and deflection data indicate that all sections should be performing at a high level of serviceability and, generally, that there should be no cracking.

This conclusion is also substantiated by the results of the strain measurements. When the average pavement temperature is 75 F or less, the maximum observed tensile strain does not exceed 150 × 10^{-6} in./in. (Table 12). Further, if we assume that the constant strain amplitude fatigue tests are representative of field performance, this level of strain corresponds to more than 1,000,000 load applications for 40 F and 75 F data. According to California EWL_{63} procedure, this corresponds to a traffic index of greater than 8.0, since the level of strain was associated with a 15,000-lb axle load. Thus, the strain data indicate, at least in a qualitative way, the same trends shown by the deflection data. Moreover, the observed strain data do not show any major differences between the various sections when comparisons are made at the same temperature.

Thus, on the basis of the deflection and strain data, one cannot find an explanation in terms of load application for the development of cracking in the 8-W and 4-W-2 sections and essentially no cracking in the 4-0 and 8-0 sections. However, part of the cracking in the 4-W-2 section might be related to preconstruction cracking. In spite of this, one must conclude that the cracking observed in the 8-W and 4-W sections is not completely load-associated, and some other factors may be contributing. No data are available at present to indicate what these factors may be.

Finally, it should again be emphasized that the cracking which has developed does not detract from the riding qualities of the pavement; that is, the present serviceability of all sections is high.

SUMMARY AND CONCLUSIONS

On the basis of the data presented, particularly those relating to deflection, strain and cracking, a few general conclusions are presented.

1. The deflection and strain data in themselves appear to offer no explanation for the cracking observed in sections 4-W-2 and 8-W.
2. The addition of asbestos appears to offer no advantage for this project, particularly when viewed in the light of appreciable differences in costs of mixes with and without asbestos. (Appendix B of this report includes an estimate of complete mix costs.)

It should be emphasized, however, that these conclusions apply only to the materials and specific combinations of asphalt, aggregate and asbestos used in this project, and only to the traffic and environment to which the pavement sections were subjected during the period of test.

ACKNOWLEDGMENTS

The Committee wishes to acknowledge gratefully the assistance of all those who participated in this project. Particular recognition should be given to the contributions of: F. N. Finn, Materials Research and Development, for analysis of pavement deflection measurements; T. W. Pickrell, ITTE, for electronic field measurements; and G. B. Dierking, ITTE, for preparation of figures.

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Appendix A

SHELL AVENUE TEST ROAD COMMITTEE MEMBERSHIP AND FUNCTIONS

As a result of informal discussions regarding the desirability and feasibility of constructing a test road for the purpose of conducting full-size, in-service comparative performance studies of asphalt-concrete mixtures with and without asbestos filler, a planning committee was organized from representatives of agencies interested in participating in such a study. This committee held numerous meetings to plan the experiment, supervise the construction, and to gather and analyze the numerous data obtained.

It is important to note that the size and scope of the investigative program developed by this cooperative effort would have been beyond the reasonable capabilities of any one of the individual participating organizations. Any success that this project may have had is directly attributable to the combined efforts of the individual committee members.

The members of the Committee are:

1. W. A. Garrison, Materials Engineer, Contra Costa County (Committee Chairman);
2. W. J. Kari, Technical Supervisor, American Bitumuls and Asphalt Company, Emeryville;
3. R. S. Latchaw, Construction Engineer, Contra Costa County;
4. J. A. Lettier, Products Application Engineer, Shell Oil Company, San Francisco (now D. F. Fink)
5. Vaughn Marker, Managing Engineer, Pacific Coast Division, The Asphalt Institute, Berkeley;
6. C. L. Monismith, Associate Professor of Civil Engineering, University of California, Berkeley;
7. C. J. Van Til, Staff Engineer, Pacific Coast Division, The Asphalt Institute, Berkeley (Committee Secretary);
8. C. W. Weitzel, Special Representative, Asbestos Fiber Division, Canadian Johns-Manville, Ltd., San Francisco (now Los Angeles);
9. Lew Wulff, Materials Engineer, District IV, California State Division of Highways, San Francisco.

Appendix B

ESTIMATED COMPARATIVE COSTS OF ASPHALT CONCRETE WITH AND WITHOUT ASBESTOS FILLER

A. Without Filler

¾-in. max, with 85-100 or 40-50 penetration asphalt	$4.75 per ton

B. With Asbestos Filler

1. Base price	$4.75 per ton
2. Additional asphalt—20 lb at $25.00 per ton	0.25
3. Asbestos—50 lb at $70.00 per ton	1.75
4. Add asbestos to mix at plant	0.15
5. Additional mixing time	0.20
Total	$7.10 per ton
	$7.10
	- 4.75
Additional Cost	$2.35 per ton
Additional Cost	49.5 percent

Discussion

J. H. KIETZMAN and J. W. AXELSON, Johns-Manville Research Center, Manville, New Jersey—One of the items of interest which was not included in this report is the

TABLE 16

DATA ON CORES TAKEN FROM SHELL AVENUE IN 1961
(Chicago Testing Laboratory Report No. 07705-12, Nov. 7, 1961)

Section	Identif.	Asphalt Content (%)	Properties of Recovered Asphalt	
			Penetration, 77 F (100/5)	Ductility, 77 F (5/60)
4-0	E-1, east	5.7	36	100+
4-0	W-2, west	4.6	35	100+
Avg.		5.2	36	
4-W	E-1, east	5.8	37	100
4-W	W-1, west	6.0	30	100+
Avg.		5.9	34	
8-0	E-3, east	5.0	51	100+
8-0	W-2, west	5.0	49	100+
Avg.		5.0	50	
8-W	E-1, east	5.3	70	100+
8-W	W-1, west	5.2	57	100+
Avg.		5.3	64	

TABLE 17

DATA ON CORES TAKEN FROM SHELL AVENUE IN 1964
(Chicago Testing Laboratory Report No. 20184, Nov. 11, 1964)

No.	Core No.	Air Voids (%)	Asphalt Content (%)	Asbestos Present	Properties of Recovered Asphalt		
					Penetration, 77 F	Ductility	
						77 F	45 F
I	1320	4.4	6.2	yes	12	11	0.0
I	1321	4.0	6.2	yes	16	23	0.0
I	1325	5.1	5.4	yes	12	12	0.0
I	1326	5.1	5.7	yes	13	16	0.3
II	1328	5.0	5.2	no	16	25	0.3
II	1329	6.7	5.7	yes	13	12	0.5
II	1331	4.7	5.7	yes	17	24	0.3
III	1323	3.5	5.2	no	15	22	0.3
III	1324	3.9	4.9	no	18	32	0.0
III	1330	4.3	5.2	no	18	41	0.3
IV	1322	4.7	5.9	yes	27	72	5.5
V	1327	4.7	5.6	yes	27	44	0.5

properties of the recovered asphalt. Cores were taken in 1961 and again in 1964 and were tested by the Chicago Testing Laboratory for aggregate gradation, amount of bitumen and properties of the recovered asphalt. Pertinent data are given in Tables 16 and 17.

Table 16 indicates that the asphalt content in all sections was considerably below the design values. The values for sections 4-0 and 8-0 check with those for the plant mixes as given in Table 9 of the report but the core values for sections 4-W and 8-W are appreciably below the plant mix values. Unfortunately, the cores in Table 17 have not been identified by section and it is not possible to check these asphalt contents. Table 16 also shows that there was not any abnormal hardening of the asphalt during the mixing and placing although the penetrations on the asphalts from the section 8 cores was somewhat lower than normally expected. To quote the CTL report "the properties of the recovered asphalts are all satisfactory for the respective penetration grades which were used in this project."

Table 17, however, shows that there has been excessive hardening of the asphalt from all sections in the three years between corings. This hardening is considerably greater than we have ever experienced in our work in the east and from all correlations that have been made between ductility or penetration of recovered asphalt and excessive cracking, it would be expected that all sections would show degeneration through cracking. In order to understand these data more fully, it is requested that the Committee supply core identification by section and give a possible explanation for the excessive hardening that took place.

The addendum is a duplicate of an inspection report made on February 21, 1963. In general, this report agrees reasonably well with the other inspection reports given. However, it is noted that we observed some 20 ft of alligator cracking in the loaded lane of the 4-0 section whereas none was reported by the Committee. Presumably, this cracking was in the 100-ft transition zone between sections.

It is hoped that this additional information will be of value in further analyses of this test road and that the study will continue. It would be desirable to know what the effect of the present cracking will be on future serviceability and whether or not cracking becomes more extensive throughout the entire project as would be expected with the low asphalt ductilities now present.

Addendum

Inspection Report*
Appearance of Test Pavement on
Shell Avenue, Martinez, California (Contra Costa County)
February 21, 1963

I. Section 8-0 Standard Mix with 85-100 Pen. Asphalt

Both loaded and unloaded lanes appear free of cracks.

II. Section 8-W 2½ Percent Asbestos, 85-100 Pen. Asphalt

A. Unloaded lane

1. Inner wheelpath—incipient alligator cracking at 2 locations, totaling about 25 ft.
2. Outer wheelpath—one longitudinal crack, 2-ft length. Generally good condition structurally.

B. Loaded lane

Center crack attributed to paver.

1. Inner wheelpath—intermittent alligator cracking evident at three locations for a total length of about 50 ft. One longitudinal crack near LVDT gage No. 15.
2. Outer wheelpath—generally good appearance.

*Inspection by J. H. Kietzman with C. W. Weitzel and W. A. Garrison, Materials Engineer of the Contra Costa Department of Public Works.

III. Section 4-0 Standard Mix with 40-50 Pen. Asphalt

Few intermittent longitudinal joint cracks.

A. Unloaded lane

Very good condition. No cracking evident.

B. Loaded lane

Alligator cracking 20-ft total length starting 10 ft inside transition zone at south end. Center crack intermittent but extensive, attributed to paver.

IV. Section 4-W Asbestos Mix with 40-50 Pen. Asphalt

Intermittent longitudinal joint cracks.

A. Loaded lane

Center cracking (due to paver) continuous.
1. Inner wheelpath—Alligator cracks at a few locations near middle of section.
2. Outer wheelpath—generally good.

B. Loaded lane

1. Inner wheelpath—"wet" spots 25-ft length starting at south end. Few longitudinal cracks and intermittent alligator cracking for 50-ft length.
2. Outer wheelpath—generally good appearance.

V. The 5th section with 40-50 pen. asphalt and asbestos was not officially part of the test because of the extremely poor condition of the old pavement and base. Just about every type of cracking is evident in this section.

General Comments

1. Surface texture of all of the standard mixes was tight, but considerably more open than the asbestos section. Extensive but very slight surface checking is evident, apparently still remaining from placement. Wet spots on the surface were observed with the alligator cracking and in the wheelpaths at places where cracking appears to be just beginning. The impression is that cracking is starting at the bottom of the resurfacing layer and working its way upward.

2. The inferior appearance of the asbestos mix with 40-50 pen. asphalt may be attributed to the deliberate location on the worst part of the old pavement.

3. To date, cracking has had negligible effect on ridability (serviceability) of the pavement.

4. The location of cracking almost exclusively in the inner wheelpaths is reportedly due to the thinness of the overlay pavement near the centerline of the road where the original pavement grade was high.

AUTHOR'S CLOSURE

The committee wishes to thank Messrs. Kietzman and Axelson for their discussion of the paper.

Core identification for the data presented in Table 17 of their discussion is given in Table 18.

A number of other points raised by Kietzman and Axelson deserve some comment.

In the inspection report listed as an Addendum to their discussion, the general comment "The inferior performance of the asbestos mix with 40-50 penetration asphalt may be attributed to the deliberate location in the worst part of the old pavement" was made. Out of context this could be somewhat misleading. In the report it was noted

TABLE 18

CORE LOCATIONS—1964 SAMPLES

Core No.	Location
1320	4-W uncracked area
1321	4-W uncracked area
1325	4-W cracked area
1326	4-W uncracked area
1328	8-0 uncracked area
1329	4-W cracked area
1331	4-W uncracked area
1323	4-0 uncracked area
1324	4-0 uncracked area
1330	4-0 uncracked area
1322	8-W uncracked area
1327	8-W uncracked area

in considerable detail that the first 640 ft of pavement, that containing 40-50 penetration asphalt with asbestos, was not considered a part of the test since the underlying conditions were not comparable to the remaining approximately 2, 500 ft of pavement. Thus, while cracking data are reported for this section (4-W-1), no comparisons are made with the other sections.

With regard to the comment on alligator cracking in the 4-0 section, this cracking occurred in the transition section between 4-0 and 4-W and thus was not reported.

The discussors call attention to the fact that the asphalts have hardened excessively in the sections covered by the core data presented in Table 17.

Whether or not this is excessive hardening is not pertinent at this point since comparisons were made between sections and, as seen in Tables 17 and 18, this hardening is about the same in all sections. As noted in the report, the sections containing asbestos exhibit cracks while those without have no cracking. Furthermore, the cracking which appears cannot be explained in terms of the analyses presented. Thus, to present conjecture as to the cause of cracking in the asbestos sections would be difficult since there is no unanimity of opinion among the members of the committee. Some feel that the cracking may be due to load stresses, others to stresses resulting from volume change (13) and still others to a combination of load stresses and those associated with volume changes.

In conclusion it should be noted that observations, though not as extensive as those made during the first 3 years, will be continued on the project through at least 5 years. Thus, some measure of the influence of existing cracking on future serviceability will be obtained.

Reference

13. Zube, Ernest and Cechetini, James. Expansion and Contraction of Asphalt Concrete mixes. Highway Research Record 104, pp. 141-163, 1965.