Relation of Densification To Performance of Small-Scale Asphaltic Concrete Test Sections

C. T. METCALF, Research Laboratory, Shell Oil Co., Wood River, Ill.

Performance of small-scale road test sections containing dense-graded asphaltic concrete has shown the effect of several variables in mix design on resistance to pavement cracking, displacement and flushing. Core samples removed from the sections after 15 and 43 months service reveal the considerable densification that took place under traffic while the pavement was still relatively new. Substantially all densification occurred during the first 15 months.

The amount of densification during this period is shown to be related to the potential densification, or the change in aggregate voids that can occur before the theoretical zero-air-voids content is reached. Lean mixes, with a high potential change, show an average reduction of 8.0 percent, whereas rich mixes produce only a 4.0 percent change. Densification of the pavement tends to continue until the aggregate voids are within approximately 3.6 percent of the theoretical ultimate content for mixes of this type.

Performance ratings from "plastic" to "brittle" show the strong influence of asphalt content on performance. Lean mixes in these tests are predominantly satisfactory, their asphalt contents being below the amount necessary to fill the void space when the aggregate achieves its maximum density. When this limiting space is overfilled, the mix becomes susceptible to plastic deformation and flushing. Limiting asphalt contents of 4.1 percent to 4.5 percent (by weight) correspond to aggregate voids values of less than 14 percent (by volume) which have been measured in the pavement.

These findings should aid in describing the changes that can be expected in service. They also provide information relating to the important factors affecting satisfactory design.

● REALISTIC answers to problems involving the use of road asphalts often require field tests. Conclusions based on laboratory analysis ultimately must be confirmed by performance tests on the road. With this in mind, a series of road trials was constructed in November 1952, for the purpose of investigating several problems in the design and performance of asphaltic concrete, surface-course type pavements. A prominent objective of these tests was to investigate the effect of asphalt content and grade on pavement cracking, displacement, densification and flushing. For this purpose, several 85/100 penetration asphalts and representatives of other grades from 40/50 to 120/150 penetration grade were included in the study. Two "rubberized" asphalts containing small amounts (5 percent by weight) of natural rubber powder were also included. Because binder content is recognized as an important factor in pavement performance, mixes were prepared with asphalt contents varying over the range of common use.

The experimental surfaces were constructed on a single, uniform base course as part of a refinery entrance road at Wood River, Illinois. Traffic over this road consisted principally of heavy trucks (average about 150 per day) with axle loads averaging about 14,000 lb. The behavior of the test sections was studied for 43 months. A series of cores was taken from each section after construction, after 15 months service and at the conclusion of the test. This report describes the detection and classification of the service-established patterns and the conclusions in regard to mix behavior which can be derived from them.

CONSTRUCTION OF EXPERIMENTAL TEST SECTIONS

In order to provide the necessary uniform support for all the test sections, the debris in the area was replaced with selected base material. This consisted of a 10-in. sand subbase topped by two 3-in. courses of Type B-7 penetration macadam (1) as shown in Figure 1. The chosen thicknesses of base and subbase were determined on the basis of the bearing power of the silty subgrade. This soil classifies as an HRB Type A-4 (2) with plasticity index of eight and California Bearing Ratio (soaked) of four.

TABLE 1

GRADING OF TEST ROAD AGGREGATES

Aggregates	Cumulative Percent Passing Sieve Number						
	½ in.	⅜ in.	No. 4	No. 10	No. 40	No. 80	No. 200
Crushed Limestone (Apparent Spec Grav = 2.720)[1] (Bulk Spec Grav = 2.652)	100	90	38	5	3	2	2
Coarse River Sand (Apparent Spec Grav = 2.677)[2] (Bulk Spec Grav = 2.599)			100	98	35	2	0
Blending Sand (Apparent Spec Grav = 2.676)[2] (Bulk Spec Grav = 2.604)				100	94	65	11
Combined Gradation							
55 Percent Crushed Limestone							
22 Percent Coarse Sand							
17 Percent Blending Sand							
6 Percent Limestone Dust							
Combined Grading (Apparent spec grav = 2.703)	100	94	66	48	32	19	9

[1]ASTM Method C127-42.
[2]ASTM Method C128-42.

The dense-graded mixes in the test sections, typical of many U.S. highway surfacings, (3) meet Asphalt Institute specifications for Type IV hot-mix asphaltic concrete (4). The hot mixes were hand-raked and compacted with a vibrating-type impactor, which delivered 1,750 to 2,000 blows per minute, each of 717 ft-lb impact. Gradings of the mixes are shown in Table 1 together with sieve analyses of the limestone and sands used in mix preparation.

Asphalt contents for the mixes were selected according to criteria recommended by the Corps of Engineers for the Marshall Stability test. Optimum binder content was determined from the average of the values below based on the use of regular 85/100 penetration asphalt (Asphalt A).

Test Property	Point of Selection	Selected Asphalt Content, Percent
Marshall Stability	Maximum	4.0
Bulk Specific Gravity	Maximum	5.0
Percent Air Voids	4%	5.5
Percent Voids Filled with Asphalt	80%	6.0
	Average	5.2

Optimum asphalt content was taken as 5 percent.

It is well known that the amount of asphalt, as well as its consistency, affects the flexibility of a pavement. Generally, the use of more asphalt increases resistance to cracking at the expense of resistance to plastic deformation. The test sections contained a range of asphalt contents from lean to rich to explore the effect of asphalt content. Lean, optimum, and rich mixes represented a range in asphalt content from 20 percent less than optimum to 20 percent greater. These amounts correspond to 4, 5 and 6 percent by weight, respectively.

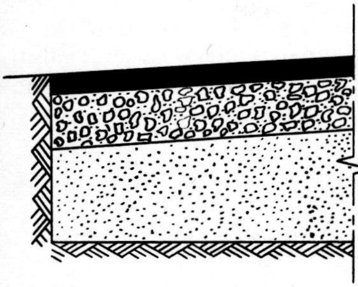

Figure 1. Typical section Wood River test road.

Figure 2. Extensive cracking, characteristic of "brittle" performance.

Figure 3. "Satisfactory" performance of dense-graded mix showing no cracking or flushing.

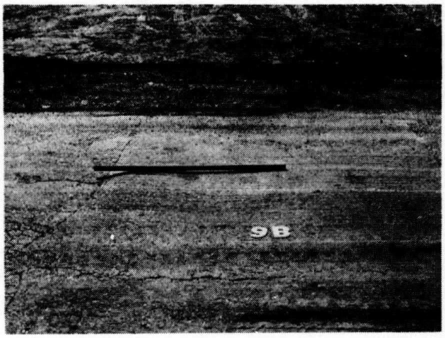

Figure 4. "Shoving", associated with "plastic" performance of a "rich" mix.

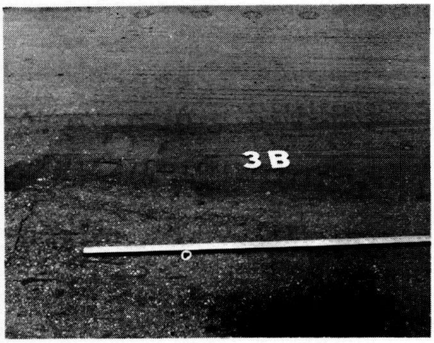

Figure 5. Severe "tire printing", which accompanies extensive "flushing" in hot weather.

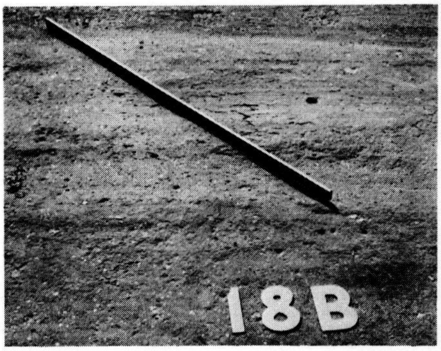

Figure 6. Pronounced "rutting" and a faint pattern of cracking associated with high-temperature deformations.

For adequate performance during both summer and winter, asphalt must fulfill a multigrade function. It must not become too brittle at low temperatures nor too fluid at high temperatures. Four grades of asphalt ranging from 40/50 to 120/150 were used to aid in defining the proper consistency balance for year around use.

Properties of the asphalts in the test sections are listed in Appendix I together with the properties of the mix after laying.

PERFORMANCE OF TEST SECTIONS

After construction of the sections, their performance was observed and rated visually at periodic intervals. Performance was classified by a descriptive rating system similar to one employed by the U.S. Engineers (2). Ratings below are based on characteristic marks of distress such as "surface cracking", "flushing", and "rutting or shoving".

1. Brittle: Excessive surface cracking in absence of flushing.
2. Semibrittle: Moderate surface cracking.
3. Satisfactory: Slight to no evidence of cracking, and slight to no evidence of flushing or rutting.
4. Semiplastic: Moderate flushing clearly visible to the eye giving the appearance of a slick surface.
5. Plastic: Excessive flushing usually combined with rutting or shoving.

Typical features of sections with various ratings are pictured in Figures 2 to 6.

At the conclusion of the test, after 43 months service, the sections were given a final evaluation and cores were taken. Performance ratings based on the above rating system are shown in Table 2.

The final ratings after 43 months service are no different from the ratings after 15 months when the first series of cores was taken. Thus, the significant changes in performance occurred approximately during the first year after construction. By this time the sections had achieved a relative equilibrium and performance patterns had become well established.

TABLE 2

PERFORMANCE RATINGS OF WOOD RIVER TEST SECTIONS
15 TO 43 MONTHS SERVICE

Asphalt	Dense-Graded Mixes: Lean (4% by weight)	Optimum (5% by weight)	Rich (6% by weight)
Regular Asphalts			
40/50	Brittle	Brittle	Semiplastic
60/70	Semibrittle[1]	Satisfactory[1]	Semiplastic[1]
85/100 Asphalt A	Satisfactory[1]	Satisfactory[1]	Semiplastic[1]
85/100 Asphalt B	Satisfactory	Semiplastic	Plastic
85/100 Asphalt C	Satisfactory	Semiplastic	Plastic
85/100 Asphalt D	Satisfactory	Semiplastic	Plastic
85/100 Asphalt E	Satisfactory	Satisfactory	Semiplastic
85/100 Asphalt F[2]	Brittle	Semibrittle	Semiplastic
85/100 Asphalt G[2]	Satisfactory	Semiplastic	Plastic
120/150	Satisfactory[1]	Semiplastic	Plastic

[1] Sections rated after 8 months service when edge failure forced removal of half the section. Sufficient material remained for a final series of cores to be taken.

[2] Asphalt A plus rubber additive. Asphalt F was prepared by adding the crumb rubber just before mixing asphalt and aggregate. Asphalt G was prepared by maintaining the asphalt-rubber mixture in heated storage for 3 days before mixing with aggregate.

The importance of asphalt content to performance is immediately indicated by the ratings. Only the 40/50 and Asphalt F fail to achieve satisfactory performance in at least one mix. Table 3 showing the number of sections in each performance rating, indicates that lean mixes are predominantely satisfactory. Optimum mixes tend to be semiplastic, and rich mixes are semiplastic to plastic. Deviations from the characteristic ratings can be attributed to asphalt consistency. In the lean mixes, the 40/50 and 60/70, in addition to Asphalt F, are too brittle for satisfactory performance under these conditions. At the optimum content, the 40/50 and Asphalt F are still in the brittle range, but 60/70 becomes satisfactory. Rich mixes with excessive asphalt contents, are all within the plastic groups.

TABLE 3

Performance Rating	Number of Dense-Graded Mixes		
	Lean (4%)	Optimum (5%)	Rich (6%)
Brittle	2	1	
Semibrittle	1	1	
Satisfactory	7	3	
Semiplastic		5	5
Plastic			5

The ratings emphasize again the delicate balance between summer and winter performance that must be accomplished in good design. The softer 120/150 and 85/100 grades are generally satisfactory in a lean mix. The 60/70 grade requires the higher asphalt content of an optimum mix to obtain necessary low-temperature flexibility. The hard 40/50 grade can provide adequate flexibility only at high asphalt contents with accompanying hot-weather instability.

PAVEMENT DENSIFICATION

During service the density of a paving mix tends to increase. This process is illustrated in Figure 7. Initially asphalt occupies only a portion of the space available to it between the particles of mineral aggregate. However, under the compressive action of traffic, the aggregate void space is reduced and eventually an equilibrium, uncompactible state is achieved.

Core samples were removed from the test sections immediately after construction, after 15 months service and, finally, after 43 months service. The aggregate void contents listed in Table 4 emphasize the considerable densification which took place under traffic while the pavement was still relatively new.

Average VMA (voids in mineral aggregate) figures show that initial compaction was unaffected by asphalt content. Originally, lean, optimum, and rich mixes all contained approximately the same amount of aggregate voids (22.5 percent). After 15 months service, however, aggregate voids were lower in lean than in rich mixes. This shows that the greatest aggregate densities were produced by the closer packing of particles which is possible in the lean mixes. After 43 months service, this pattern still existed. Comparison of average VMA figures shows that there was no significant change in aggregate packing in the 15- to 43-month interval. Thus, as noted also in the trend of performance patterns, the greatest changes in physical properties were developed early in the life of the pavement. All types of mix contained approximately the same VMA originally and thus the subsequent densification under traffic was greater in lean (average 8.0 percent VMA reduction) than in rich mixes (average 4.0 percent VMA reduction).

Figure 8 illustrates that extent of densification can be related directly to the opportunity a mix has to densify. Opportunity, or potential densification, is the largest change in aggregate voids that can take place before densification is complete. It is the difference between the original VMA and the ultimate VMA of a fully compacted (zero air voids) mix. Ultimate VMA is the aggregate void content at which the asphalt completely fills the available void space.

Ultimate VMA's are calculated from the equation:

$$\frac{Pac/Gac}{Pac/Gac + Pag/Gag}$$ (symbols correspond to those in reference 4)

Ultimate VMA's corresponding to the three asphalt contents used are:

Lean (4% by weight) = 10.1% by volume
Optimum (5% by weight) = 12.5% by volume
Rich (6% by weight) = 14.7% by volume

TABLE 4

AGGREGATE VOID CONTENTS OF PAVEMENT CORES

	Voids in Mineral Aggregate (VMA, % by volume)[1]								
	Lean (4% by weight)			Optimum (5% by weight)			Rich (6% by weight)		
Asphalt	Orig	15 Mo	43 Mo	Orig	15 Mo	43 Mo	Orig	15 Mo	43 Mo
40/50	24.9	15.4	15.1	-	16.4	15.3	19.1	18.5	17.7
60/70	21.5	13.3	13.4	25.6	15.3	15.6	21.1	18.4	19.4
85/100 Asphalt A	20.8	13.8	14.3	18.6	14.8	16.3	18.7	18.0	-
85/100 Asphalt B	-	15.6	13.3	-	15.1	14.9	19.9	17.6	17.4
85/100 Asphalt C	20.6	13.6	14.3	18.4	15.2	15.0	20.8	17.3	17.5
85/100 Asphalt D	22.9	13.3	13.1	31.7	15.6	15.0	26.7	18.9	19.0
85/100 Asphalt E	23.1	13.2	14.9	23.8	15.3	16.3	22.5	19.8	19.5
85/100 Asphalt F	25.0	19.1	17.5	22.5	16.6	17.3	31.8	18.5	17.7
85/100 Asphalt G	24.8	14.4	16.1	21.7	16.0	15.9	24.5	19.3	18.5
120/150	18.4	13.2	14.0	17.7	16.2	15.7	18.9	18.6	18.0
Average	22.4	14.4	14.5	22.5	15.6	15.7	22.4	18.4	18.3

[1] Aggregate void contents based on apparent specific gravities listed in Table 1.

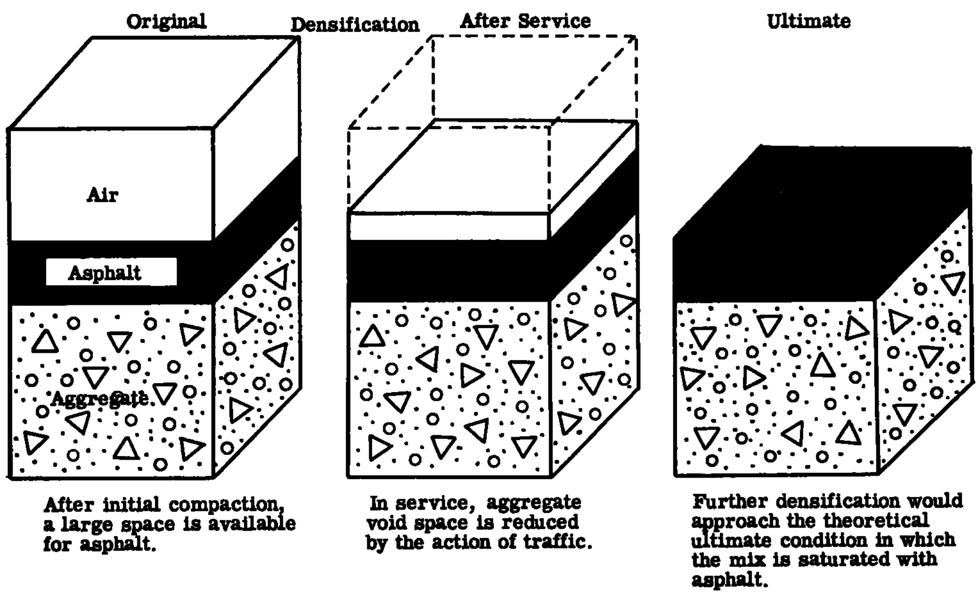

Figure 7. Volume relationships for asphaltic concrete (three-phase system).

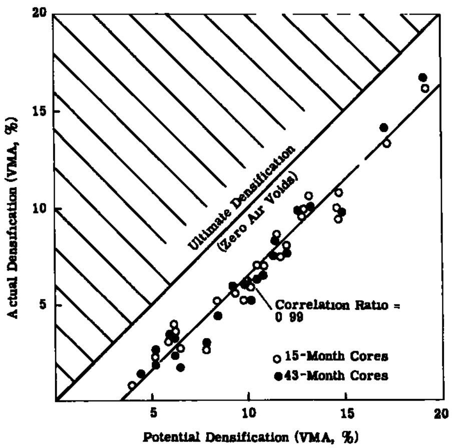

Figure 8. Relation of actual densification to potential densification.

The observation that densification is usually greater in lean than in rich mixes can be attributed to the fact that the former are compacted initially to a lesser degree relative to ultimate minimum void content. Aggregate voids can also be reduced to a smaller ultimate space in lean mixes and thus a greater change is possible before ultimate density is reached.

The excellent correlation between actual and potential densification is indicated by the correlation ratio of 0.99. Actual densification parallels the ultimate condition at an average VMA that is 3.6 percent less than ultimate. This indicates that dense-graded mixes of this type continued to densify until the aggregate voids were within 3.6 percent of ultimate. For these mixes this principle provides a convenient method for estimating the VMA that can be achieved under traffic.

Type of Mix	Estimated VMA (%) (Ultimate VMA + 3.6 %)	Actual VMA (%) (43 Months)
Lean (4% by weight)	(10.1 + 3.6) = 13.7	14.5
Optimum (5% by weight)	(12.5 + 3.6) = 16.1	15.7
Rich (6% by weight)	(14.7 + 3.6) = 18.3	18.3

LIMITING ASPHALT CONTENTS

The total volume of aggregate voids limits the amount of asphalt which can be incorporated in the mix. Thus, the space that remains between aggregate particles when the mix achieves its greatest density governs the amount of asphalt that the mix can safely hold. Increasing the asphalt content beyond this amount reduces aggregate density and leads to plastic instability.

The minimum VMA values developed during the test are listed in Table 5 for each asphalt. Asphalt contents corresponding to these VMA's, with allowance for 3 percent air voids, are listed as "limiting asphalt contents." They are similar to "optimum moisture contents" in soil compaction. Limiting contents of the 85/100 grades appear in the range of 4.1 to 4.5 percent by weight corresponding to VMA values of less than 14 percent by volume.

Comparison of "limiting asphalt contents" with performance ratings shows the strong influence of asphalt content on performance. Table 6 lists performance of the mixes with respect to their asphalt contents expressed as percentages of the "limiting contents." The "relative asphalt content" is a measure of the extent to which the potential capacity of a mix is filled or overfilled with asphalt.

By assigning numerical values (that is, 1, 2, 3, 4, 5) to the performance ratings, good correlation is obtained between performance and "relative asphalt content" as shown in Figure 9 (correlation ratio = 0.84). Satisfactory performance corresponds to a "relative content" that averages 100 percent. Thus, satisfactory performance can be expected at asphalt contents near the "limiting content". When the "limiting content" is exceeded (relative content >100 percent), the mix is overfilled and plastic distress in hot weather results. When the mix contains less than the limiting content (relative content <100 percent) there is a trend to brittleness.

Limiting asphalt contents in Table 5 indicate that the use of a high-viscosity binder will provide only a temporary benefit as regards densification. Such a binder will only be of long-term value in preventing overfilling of the voids if it is sufficiently viscous to delay attainment of ultimate density for a period of at least several years. Even the

TABLE 5

LIMITING ASPHALT CONTENTS

Asphalt	Minimum VMA During Road Test	Limiting[1] Asphalt Content (% by weight)
40/50	15.1	5.0
60/70	13.4	4.25
85/100 Asphalt A	13.8	4.45
85/100 Asphalt B	13.3	4.2
85/100 Asphalt C	13.6	4.35
85/100 Asphalt D	13.1	4.1
85/100 Asphalt E	13.2	4.2
85/100 Asphalt F	16.6	5.7
85/100 Asphalt G	14.1	4.5
120/150	13.2	4.2

[1] Limiting Content = $\frac{\text{Wac}}{\text{Wac + Wag}}$

Where: Wac = (Min VMA (%) - 3% air voids) x Asphalt specific gravity.
Wag = (100 - Min VMA (%)) x Aggregate specific gravity.

TABLE 6

COMPARISON OF PERFORMANCE WITH RELATIVE ASPHALT CONTENT

Performance Rating	Relative Asph Cont $\left(\frac{\text{Asphalt Content}}{\text{Limiting Asph Cont}}\right)$ x 100					Average Relative Content (%)
	<80%	80-100%	100-120%	120-140%	>140%	
Brittle	1	2				83
Semibrittle		2				89
Satisfactory		7	3			100
Semiplastic			7	3		121
Plastic				4	1	137

TABLE 7

AGGREGATE VOID CONTENTS OF SPECIMENS COMPACTED BY THE MARSHALL HAMMER

Asphalt	Lean (4% by weight)	Optimum (5% by weight)	Rich (6% by weight)
40/50	15.2	15.4	17.1
60/70	14.3	15.1	17.3
85/100 Asphalt A	13.3	14.5	16.8
85/100 Asphalt B	14.4	14.8	17.1
85/100 Asphalt C	15.3	15.2	17.0
85/100 Asphalt D	14.3	15.2	17.3
85/100 Asphalt E	14.6	15.2	17.3
85/100 Asphalt F	15.1	16.6	17.6
85/100 Asphalt G	14.6	15.6	17.0
120/150	13.5	14.6	17.0

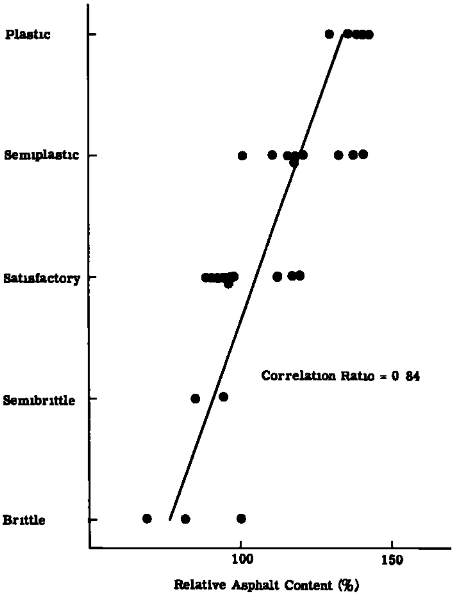

Figure 9. Correlation between performance and relative asphalt content.

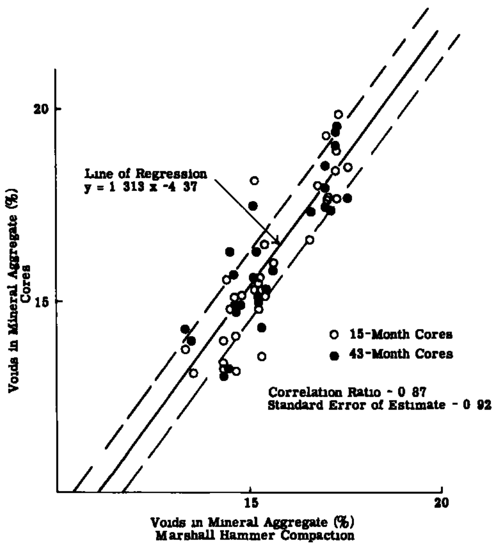

Figure 10. Correlation between aggregate voids of marshall hammer compacted specimens and pavement cores.

40/50 penetration asphalt included in this study was too fluid to satisfy this requirement. Only Asphalt F prolonged densification beyond 3½ years. This ability to retard densification has been attributed to the large increase in viscosity (6) that results when rubber is added to the asphalt during the mixing operation. The most satisfactory solution to the problem of plastic instability appears, on the basis of the present work, to be the use of lower asphalt contents.

It should be remembered that specific findings of this investigation apply only to mixes and materials involved. Although the mix is typical of many dense-graded types in common use, only one grading and one series of aggregate types is represented in the test sections.

COMPARISON OF LABORATORY AND FIELD COMPACTION

An interesting comparison of the aggregate void contents in laboratory compacted specimens and those in pavement cores is shown in Figure 10. Laboratory compaction was obtained by the standard 50-blow Marshall hammer compaction (4). These results provide a measure of the ability of Marshall hammer compaction to reproduce or predict the extent of pavement densification that will occur under traffic. Comparison of aggregate voids in Marshall hammer compacted specimens shown in Table 7 with corresponding values for pavement cores from Table 4 reveals a fairly good agreement between laboratory and field properties.

Average values for lean, optimum and rich mixes in Table 7 show that only in the rich mixes is there a large difference between compacted densities produced in the laboratory and in the field. The more effective compaction of laboratory specimens of rich mixes probably results from the confinement provided by the compaction mold. No significant difference is shown by the figures representing lean and optimum mixes.

AVERAGE AGGREGATE VOID CONTENTS, (%)

	Marshall Hammer Compaction	Cores	
		15 Mo	43 Mo
Lean	14.5	14.4	14.5
Optimum	15.6	15.6	15.7
Rich	17.2	18.4	18.3

CONCLUSIONS

The conclusions to be drawn from observation of the test sections are limited to the materials represented in the sections and can be summarized as follows:

1. Asphalt content is a major factor controlling performance of a dense-graded mix. Lean mixes (asphalt content ca. 20 percent less than Marshall optimum) generally provide the best performance when they contain asphalts that are soft enough to avoid brittleness.
2. An increase in asphalt content improves resistance to flexural cracking but lowers resistance to plastic displacement.
3. When an asphaltic surface is exposed to traffic, considerable densification, or reduction in aggregate void space, takes place. Densification is related to theoretical ultimate density and is a function of asphalt content.
4. Traffic compaction can reduce the aggregate void space in this type of dense-graded mix to an amount less than 14 percent of the bulk volume. For these aggregates, this space can contain a maximum "limiting content" of 4.5 percent by weight asphalt.
5. Performance reveals that asphalt contents greater than the "limiting contents" tend to cause plastic distress, and those less than the "limiting contents" tend to cause brittleness.
6. Densification is retarded when the mix contains a more viscous binder. This effect is particularly noticeable when rubber has been incorporated into the asphalt in such a way as to cause a large increase in viscosity.
7. Reasonably good agreement has been found between compaction that is produced in the laboratory by the Marshall hammer and that which occurs in the field under traffic.

APPENDIX A

PROPERTIES OF PAVEMENT SECTIONS AFTER MIXING AND LAYING

			Asphalt Original Properties			Properties After Mixing and Laying: Asphalt				Properties After Mixing and Laying: Mix		
Asphalt	Cont (% by weight)	Grading	Pen. at 77°F	SP (°F)	PI[1]	Pen. at 77°F	SP (°F)	PI[1]	% Pen Ret[2]	Air Voids (%)	Spec Grav of Mix	Spec Grav, % of Lab Compacted Specimen
40/50	4.0	Dense	39	125	-1.3	34	141	+0.2	87	16.4	2.117	88.8
"	5.0	"				21	140	-0.8	54	-	-	-
"	6.0	"				37	130	-0.8	95	5.1	2.323	97.6
60/70	4.0	"	65	117	-1.3	40	133	-0.3	62	12.7	2.208	91.7
"	5.0	"				40	130	-0.7	62	15.0	2.118	87.7
"	6.0	"				55	127	-0.3	85	7.5	2.267	95.5
85/100 Asphalt A	4.0	"	89	113	-1.1	60	125	-0 3	67	11.9	2.228	91.5
"	5.0	"				55	123	-0.8	62	7.0	2.317	95.4
"	6.0	"				68	122	-0.5	76	4.7	2.337	97.7
85/100 Asphalt B	4.0	"	78	110.5	-1.9	45	122	-1.5	58	-	-	-
"	5.0	"				54	123	-0.9	69	-	-	-
"	6.0	"				66	117	-1.3	85	6.1	2.301	96.5
85/100 Asphalt C	4.0	"	87	112.5	-1.3	-	-	-	-	11.7	2.231	93.7
"	5.0	"				66	118	-1.1	76	6.8	2.319	96.2
"	6.0	"				-	-	-	-	7.2	2.273	95.4
85/100 Asphalt D	4.0	"	86	116	-0.7	57	124	-0.6	66	14.2	2.167	89.9
"	5.0	"				48	128	-0.5	56	22.0	1.941	80.6
"	6.0	"				68	122	-0.5	79	14.1	2.103	88.5
85/100 Asphalt E	4.0	"	90	117	-0.5	55	132	+0.3	61	14.4	2.163	90.1
"	5.0	"				54	124	-0.7	60	13.0	2.169	90.0
"	6.0	"				64	122	-0.6	71	9.1	2.225	93.6
120/150	4.0	"	138	108	-0.8	69	124	-0.1	50	9.2	2.297	94.4
"	5.0	"				-	-	-	-	7.1	2.314	97.7
"	6.0	"				82	121	-0.1	59	4.9	2.330	97.7
85/100 Asphalt F	4.0	"				55	129	0.0	62	16.6	2.109	88.3
"	5.0	"				53	127	-0.3	60	11.5	2.202	82.8
"	6.0	"				60	124	-0.5	67	20.0	1.960	82.8
85/100 Asphalt G	4.0	"	74	120	-0.6	56	130	+0.1	63	16.3	2.119	88.3
"	5.0	"				56	128	-0.1	63	10.5	2.230	93.0
"	6.0	"				65	130	+0.6	73	11.5	2.165	90.7

[1] Penetration Index (see Pfeiffer, J. Ph. and van Doormool, P.M., National Petroleum News, Refinery Technology Edition, Feb. 23, 1938).
[2] Percent Penetration Retained = (penetration of asphalt recovered from mix x 100)/Penetration of original asphalt.

APPENDIX B

Asphalt	Specific Gravity Asphalt	Asphalt Content	Properties of Cores: After 15 Months Specific Gravity	After 15 Months Air Voids (%)	After 43 Months Specific Gravity	After 43 Months Air Voids (%)
40/50	1.028	4	2.380	6.1	2.387	5.8
"		5	2.378	4.8	2.408	3.6
"		6	2.343	4.8	2.363	3.9
60/70	1.023	4	2.442	3.8	2.436	3.9
"		5	2.406	3.5	2.398	3.9
"		6	2.347	4.6	2.315	5.8
Asphalt A	1.022	4	2.425	4.3	2.411	4.9
		5	2.422	2.9	2.380	4.7
		6	2.355	4.2	-	-
Asphalt B	1.021	4	2.372	6.3	2.439	3.8
		5	2.415	3.3	2.418	3.1
		6	2.367	3.7	2.372	3.5
Asphalt C	1.008	4	2.429	4.0	2.411	4.7
		5	2.412	3.2	2.415	3.0
		6	2.377	3.1	2.370	3.4
Asphalt D	1.008	4	2.436	3.7	2.444	3.4
		5	2.399	3.7	2.416	3.0
		6	2.332	5.0	2.326	5.1
Asphalt E	1.001	4	2.438	3.5	2.394	5.4
		5	2.410	3.3	2.379	4.4
		6	2.305	6.0	2.313	5.6
Asphalt F	1.018	4	2.279	10.1	2.320	8.3
		5	2.373	4.8	2.349	5.6
		6	2.344	4.5	2.363	3.6
Asphalt G	1.018	4	2.411	4.8	2.364	6.7
		5	2.392	4.1	2.391	4.0
		6	2.319	5.5	2.341	4.5
120/150	1.009	4	2.443	3.5	2.419	4.4
		5	2.382	4.4	2.396	3.8
		6	2.338	4.7	2.356	3.9

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